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Land subsidence

Proceedings of the Tokyo Symposium
September 1969

Affaissement du sol

Actes du colloque de Tokyo
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Volume 1

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A contribution to the International Hydrological Decade
Une contribution à la Décennie hydrologique internationale

IASH / AIHS - Unesco

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Preface

The International Hydrological Decade (IHD) 1965-1967 was launched by the thirteenth session of the General Conference of Unesco to promote international co-operation in research and studies and the training of specialists and technicians in scientific hydrology. Its purpose is to enable all countries to make a fuller assessment of their water resources and a more rational use of them as man's demands for water constantly increase in face of developments in population, industry and agriculture. In 1970 National Committees for the Decade had been formed in 104 of Unesco's 125 Member States to carry out national activities and to contribute to regional and international activities within the programme of the Decade. The implementation of the programme is supervised by a Co-ordinating Council, composed of twenty-one Member States selected by the General Conference of Unesco, which studies proposals for developments of the programme, recommends projects of interest to all or a large number of countries, assists in the development of national and regional projects and co-ordinates international co-operation.

Promotion of collaboration in developing hydrological research techniques, diffusing hydrological data and planning hydrological installations is a major feature of the programme of the IHD which encompasses all aspects of hydrological studies and research. Hydrological investigations are encouraged at the national, regional and international level to strengthen and to improve the use of natural resources from a local and a global perspective. The programme provides a means for countries well advanced in hydrological research to exchange scientific views and for developing countries to benefit from this exchange of information in elaborating research projects and in implementing recent developments in the planning of hydrological installations.

As part of Unesco's contribution to the achievement of the objectives of the IHD the General Conference authorized the Director-General to collect, exchange and disseminate information concerning research on scientific hydrology and to facilitate contacts between research workers in this field. To this end Unesco has initiated two collections of publications "Studies and Reports in Hydrology" and "Technical Papers in Hydrology".

The collection "Studies and Reports in Hydrology" is aimed at recording data collected and the main results of hydrological studies undertaken within the framework of the Decade as well as providing information on research techniques. Also included in the collection will be proceedings of symposia. Thus, the collection will comprise the compilation of data, discussion of hydrological research techniques and findings, and guidance material for future scientific investigations. It is hoped that the volumes will furnish material of both practical and theoretical interest to hydrologists and governments participating in the IHD and respond to the needs of technicians and scientists concerned with problems of water in all countries.

Unesco and the IASH have together undertaken the implementation of several important projects of the IHD of interest to both organizations, and in this spirit a number of joint Unesco-IASH publications are envisaged.

Préface

La Décennie hydrologique internationale (DHI) 1965-1975 a été ouverte par la Conférence générale de l'Unesco à sa treizième session pour favoriser la coopération internationale en matière de recherches et d'études et la formation de spécialistes et de techniciens de l'hydrologie scientifique. Son but est de permettre à tous les pays d'évaluer plus complètement leurs ressources en eau et de les exploiter plus rationnellement, les besoins en eau augmentant constamment par suite de l'expansion démographique, industrielle et agricole. En 1970 des comités nationaux pour la Décennie ont été constitués dans 104 des 125 États membres de l'Unesco en vue de mener à bien les activités nationales et de participer aux activités régionales et internationales dans le cadre du programme de la Décennie. Ce programme est exécuté sous la direction d'un Conseil de coordination composé de vingt et un États membres désignés par la Conférence générale de l'Unesco, qui étudie les propositions d'extension du programme, recommande l'adoption de projets intéressant tous les pays ou un grand nombre d'entre eux, aide à la mise sur pied de projets nationaux et régionaux et coordonne la coopération à l'échelon international.

Le programme de la DHI, qui porte sur tous les aspects des études et des recherches hydrologiques, vise essentiellement à developper la collaboration dans les domaines de la mise au point de techniques de recherches hydrologiques, de la diffusion des données hydrologiques et de l'organisation des installations hydrologiques. Il encourage les enquêtes nationales, régionales et internationales visant à accroître et à améliorer l'utilisation des ressources naturelles, dans une perspective locale et générale. Il offre la possibilité aux pays avancés en matière de recherches hydrologiques d'échanger des idées, et aux pays en voie de développement de profiter de ces échanges d'informations pour l'élaboration de leurs projets de recherche et pour la planification de leurs installations hydrologiques selon les derniers progrès réalisés.

Pour permettre à l'Unesco de contribuer au succès de la DHI, la Conférence générale a autorisé le Directeur général à rassembler, échanger et diffuser des renseignements sur les recherches d'hydrologie scientifique et à faciliter les contacts entre chercheurs de ce domaine. A cette fin, l'Unesco publie deux nouvelles collections: « Études et rapports d'hydrologie » et « Documents techniques d'hydrologie ».

La collection « Études et rapports d'hydrologie » a pour but de présenter les données recueillies et les principaux résultats des études hydrologiques effectuées dans le cadre de la Décennie, et de fournir des renseignements sur les techniques de recherche. On y trouvera aussi les Actes de colloques. Cette collection comprendra donc des données, l'exposé de techniques de recherches hydrologiques et des résultats de ces recherches, et une documentation pour des travaux scientifiques futurs. On espère que ces volumes fourniront aux hydrologues et aux gouvernements qui participent à la DHI des matériaux d'un intérêt tant pratique que théorique et qu'ils répondront aux besoins des techniciens et des hommes de science qui s'occupent, dans tous les pays, des problèmes de l'eau.

L'Unesco et l'AIHS ont entrepris de réaliser conjointement plusieurs projets importants de la DHI qui les intéressent l'une et l'autre; dans cette perspective, elles ont prévu un certain nombre de publications Unesco-AIHS.

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Foreword

The problems of land subsidence have not perhaps up till now attracted sufficient attention. Yet, from the point of view of hydrology, these phenomena have had what may be called catastrophic results. The Tokyo symposium, the fruit of perseverance by Japanese and American scientists, has now shown that a handful of specialists has enabled this question to be studied very thoroughly.

Although no limit had been placed upon the type of subsidence to be studied, most of the Japanese and American authors paid particular attention to subsidence caused by the lowering of water-tables or by the exploitation of oil fields. Theory has advanced very far in these spheres, thanks to study of the compaction of strata, considered as a mechanical problem. Some of the papers, however, attributed land subsidence to other causes, such as the dissolution of limestone deposits, the working of mines, or purely tectonic movements.

The influence of soil components (sand, clay, loess, peat, etc.) interested certain researchers. The action of hurricanes and tides was also considered, particularly by Japanese specialists.

A number of contributions were confined to the effects of subsidence upon surface and sub-soil hydrology, irrespective of its causes. Finally, appropriate means of reducing subsidence were considered.

The 70 or so papers submitted do not however supply a full picture of the problem. As already stated, the symposium was primarily the work of Japanese and American researchers. Europe might have been more largely represented, but, all the same the results of the meeting clearly indicate the present state of our knowledge; more especially they help to give an idea of the dangers caused by subsidence in areas which are among the most advanced, both industrially and agriculturally.

L.J. Tison

Avant-propos

Les problèmes que posent les affaissements du sol n'ont peut-être pas, jusqu'ici, retenu suffisamment l'attention. Pourtant ces phénomènes ont eu, du point de vue hydrologique, des résultats que l'on peut qualifier de catastrophiques. Le colloque de Tokyo, réuni grâce à la persévérance de savants japonais et américains, vient de montrer qu'un petit nombre de spécialistes ont permis de pousser très loin l'étude de ce problème.

Alors qu'aucune limitation n'avait été imposée quant à la nature des affaissements à étudier, la plupart des auteurs japonais et américains ont particulièrement étudié les affaissements dus au rabattement des nappes aquifères ou à l'exploitation des champs pétrolifères. Dans ces domaines, les considérations théoriques ont été poussées très loin, grâce à l'étude de la compaction des couches, considérée comme un problème mécanique. Certaines des études présentées ont cependant envisagé d'autres causes d'affaissement des sols, comme celles dues à la dissolution des calcaires, à l'exploitation des mines, à de purs mouvements tectoniques.

L'influence de la composition des sols (sables, argiles, lœss, tourbes, etc.) a retenu l'attention de certains chercheurs. L'action des ouragans et des marées n'a pas été négligée, surtout par les spécialistes japonais.

Quelques communications se sont limitées à l'action des affaissements, sur l'hydrologie de surface et du sous-sol, indépendamment de leur origine.

Enfin l'étude des mesures propres à réduire les affaissements n'a pas été négligée.

Les quelque 70 rapports présentés ne donnent cependant pas une vue complète de la question. Comme on l'a dit, le colloque fut avant tout l'œuvre des chercheurs japonais et américains. L'Europe aurait pu être mieux représentée, mais, tels quels, les résultats présentés font bien le point de l'état actuel de nos connaissances; ils aident surtout à donner une idée des situations dangereuses créées par les affaissements dans des régions qui sont parmi les plus avancées tant sur le plan industriel que sur le plan agricole.

L. J. TISON

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INTERNATIONAL ASSOCIATION OF SCIENTIFIC HYDROLOGY

TISON, Léon. J.

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND FOUNDATION ENGINEERING

JOHNSON, Arnold J.

Opening Session

Chairman: Dr. Koichi Aki

Chairman:

Your Excellency, Minister of Education, Ladies and Gentlemen, and distinguished participants of this meeting.

On behalf of the Japanese National Committee for IHD and my own self, I heartily extend my sincere thanks of your participation to the International Symposium on Land Subsidence.

As you are aware, we Japanese have long been troubled with the land subsidence in many places in Japan and some of them were due to overdrawing of groundwater for domestic and industrial use and some were due to the withdrawing of natural gas and so on. And, according to my feeling this is directly connected to the promotion of economic activities of my country. This is really a key problem of extension of economic activities of every country of the world. We have to try better use of natural resources available and for the conservation of our natural environment to improve these circumstances is a key problem among us. And I think this is the first occasion that leading geophysicists, geologists and engineers of the world gathered here in Tokyo and have chance to discuss important problem of mutual interest. I am honestly wishing fruitful result through our discussion of the coming four days.

Now, I declare the meeting is open.

WELCOME ADDRESS OF Dr. KIYOO WADATI, Chairman of the Organizing Committee

Mr. Chairman, Your Excellency, Ladies and Gentlemen

It gives me a great pleasure to say a few words on behalf of the Organizing Committee to welcome you at the opening of this International Symposium on Land Subsidence.

It goes without saying that the importance of the problem of water, which is common to all over the world today, cannot be overemphasized. Above all with respect to the problem of groundwater, the understanding of the situation and proper development of this valuable resources for utilization remains as an important subject for the future study. If there is not proper development and utilization of groundwater resources, it may cause such phenomena as land subsidence and exert a bad influence on irrigation and drainage, enhancing the possiblity of flood and high tide dangers. Land subsidence is one of the hazards of modern age which prevails in all the countries in its particular forms as a result of the progress of human activity. The countermeasure to this problem is closely related to the problem of water resources which is an important issue of today and tomorrow. Therefore I feel the great significance of this meeting to be held here from today, receiving such eminent experts who are concerned to this field to study the mechanisms of this particular phenomeon and discuss its countermeasures.

It is indeed a great opportunity and we are much pleased that this symposium is being held in Japan where we are working hard to cope with a considerable land subsidenc

I would like to extend my heartfelt welcome to all of you who have come from the four corners of the globe to attend this meeting and I earnestly wish the success of the symposium.

Thank you.

ADRESS OF MR. MICHITA SAKATA
Minister of Education

GREETINGS OF EDUCATION MINISTER

I am very pleased to have been given the opportunity today of extending greetings at the opening of the International Symposium on Land Subsidence.

Recent progress of science and technology in the world is eye-opening, and it is a matter for congratulation that a long stride has been taken also in the field of hydrology.

UNESCO attached particular importance to the problems of water which arise with the socio-economic development of countries and in 1964 initiated the International Hydrological Decade for the international cooperative study on the problems of water. This year marks mid-point in the implementation of the Decade. Japan is also taking part in this project and has cooperated with other nations and international academic organizations in promoting the hydrological studies. We will renew our effort in the promotion of research on hydrology.

In this International Hydrological Decade, the problem of ground subsidence which accompanied social development in particular has been taken up as an important task. Elucidation of its cause and the development of countermeasure for it is awaited with much expectations. I am firmly convinced that the result of this symposium will not only accelerate the progress of scientific research but also will, through its application to practice, make a great contribution to the stability and elevation of people's social life.

In order to achieve progress of science and technology, further promotion of international exchange and cooperation is desirable, and I hope the participants in this symposium will avail of this opportunity to achieve substantial cooperation in their conduct of research with deeper mutual understanding.

At this opening of the symposium, I wish to express our deep gratitude to the International Association of Scientific Hydrology, UNESCO and other international organizations, Government Agencies and local public bodies concerned for the enormous assistance and cooperation they have rendered to it. At the same time I wish that the participants fiom overseas will enjoy your short sejourn in Japan and avail of this opportunity to come into contact with Japanese culture and deepen your understanding about this country.

Lastly I wish to express our heartfelt gratitude to the members of the Organizing Committee and others concerned for their efforts made in organizing this symposium.

I hope the symposium will achieve every success.

September 17, 1969

Mr. Michita SAKATA
Minister of Education

ADRESS OF PROF. HARUSADA SUGINOME
Chairman of the Japanese National Commission for Unesco

GREETING OF COMMISSION CHAIRMAN

I deem it a great honor to have the opportunity today of representing the Japanese National Commission for UNESCO to say a few words of greeting at the opening of the International Symposiums on Land Subsidence. I would like to express our heartly welcom to all the participants in the symposium who have come a long way from various countries of the world.

This symposium has been co-sponsored by the Japanese National Commission for UNESCO, the International Association of Scientific Hydrology and the Tokyo Metropolitan Government by enlisting co-operation from the UNESCO, the Science Council of Japan and the International Society of Soil Mechanics and Foundation Engineering. But I wish to express our heartfelt gratitude not only to these organizations but also to all other people directly or indirectly concerned for the profound understanding and zealous support they have extended to this symposium.

Needless to say, the elucidation of the phenomenon of ground subsidence has now become an important task for many nations of the world in parallel with the progress of their socio-economic development; and so the application to practice of the result of studies in this field is awaited with much expectations in all quarters of society.

The UNESCO attaches particular importance to the problem of water, and the International Hydrological Decade, which it started in 1964 has already reached its middle point this year with the result that the study of water phenomena has accelerated its pace. Japan also joined this project and has since been cooperating in the effort to develop water science. In this project, too, a sizable part is occupied by the problem of elucidating land subsidence phenomena. Japan being one of the countries witnessing remarkable subsidence, she always has taken a keen interest in it. In this sense, it is quite significant that it has been decided to hold this symposium in Japan.

Progress of science and technology cannot be expected today without international exchange and cooperation, and an international congress can provide the best opportunity for realizing such exchange and cooperation.

It is my sincere wish that you participants will avail of this opportunity to further advance your international cooperation in promotion of studies on this problem of land subsidence.

I also wish that the participants from overseas will avail of this chance to further deepen their understanding about Japanese culture and mode of living. In concluding my short speech of welcome, I wish the symposium will fully achieve its successful result.
September 17, 1969.

Prof. Harusada SUGINOME
Chairman,
Japanese National Commission for Unesco

ADRESS BY PROF. LÉON J. TISON
Secretary-General and Representative of IASH

Mr. Chairman, Ladies and Gentlemen.

It is my duty and my honour to represent International Association of Scientific Hydrology at this symposium.

You know that the organization of this meeting is a result of the proposal of the Japanese National Commission for UNESCO to the Coordinating Council of the International Hydrological Decade and it was a great pleasure for me first to collaborate to the organization and in latter, last to accept with Japanese Organizing Committee the full responsibility of the symposium.

May I tell you that at that time, I was a little anxious to see the development of the organization. The subject was, in a certain sense, a restricted one, when compared to with often more general themes of meetings. On the other hand, the old French proverb could be reconsidered.

"It is not good to change horses of a car when passing a river."

And it is what happened when we took the place of Unesco last March. Different elements helped to give an excellent solution to the problem: first of all the activity, the knowledge of the question and the continued action of the Japanese National Organizing Committee which realized wonders presenting almost fifty papers of first quality. Another element was also very favourable; the personal action of Mr. Arnold Ivan Johnson, representing both Association of Soil Mechanics and the National U.S.A. Committee who arrived with another twenty papers after very strict screening. The result in Europe, was not so satisfactory: only ten papers. It is certainly the consequence of the already mentioned change in the international part or the organization. We shall do better another time. When we say another time you will immediatelyunderstand that we are already planning to continue our action: in two remarkable papers Dr. Miyabe and Mr. Poland will give you the state of this important question and with the presentation of all the other papers, we shall have more accurate ideas on the problem.

Well, it seems to me that a good method for this continuation would be an international survey of existing areas of land subsidence with their characteristics. Such a survey would certainly receive support of UNESCO and on the other side the International Council of Scientific Unions recently expressed their worry of the deep degradation of our environment under human influence. With the help of the biological scientists, the International Council of Scientific Unions is busy to organize its action in this direction. Land subsidence is certainly to be considered in this field as it was said by our President, Mr. Szesztay. Further actions have to be considered but, I think, it would be better to postpone the details until the end of the symposium.

Mesdames, Messieurs,

Notre Association possède une seconde langue officielle qui est d'ailleurs ma langue maternelle. J'ai le devoir de dire quelques mots dans cette langue qui a d'ailleurs été adoptée par quelques auteurs. Je leur dirai, ainsi d'ailleurs qu'aux autres auteurs, nos félicitations et nos remerciements pour le travail accompli.

A last word, let me say our appreciation and our thanks for the assistance of the Honourable Minister of Education and also the Honourable Governor of Tokyo. Let me also say my admiration for the work accomplished by the Japanese Organizing Committee and namely the Japanese National Committee of IUGG. I also thank UNESCO for its support and let us finally say how much I am impressed by the number of participants. It is an excellent sign for the future.

Mr. Chairman, I shall ask to say a few words on behalf of UNESCO. Mr. da Costa, Acting Director of the Unesco Division of Hydrology and Secretary of the Coordinating Council of IHD, told me two weeks ago that he would have to stay in Paris during our symposium in order to assist the meeting of UNESCO executive committee. And he asked me to say some words in his name. I am therefore, very glad to present you the greetings and wishes of UNESCO for the succes of this symposium.

Thank you.

ADRESS OF DR. RYOKICHI MINOBE
Governor of Tokyo

It is a great pleasure for me today to have this opportunity to address you at this International Symposium on Land Subsidence. This feeling of mine is by no means a simple verbal expression; I really feel so from the bottom of my heart, for one of the greatest problems confronting our Tokyo Metropolis is, in fact, land subsidence. As you are aware, Tokyo is a tremendous city with 11 million inhabitants. It is said that Tokyo, in recent years, has come to worldwide fame for the economic growth it has attained. But I, in the capacity of the head of the local public entity of Tokyo, feel much embarrassed to have such a reputation. Our Tokyo is suffering from imbalance, or distortion stemming from that high-degree development—the distortion which brings about a number of problems—knotty and yet vital problems affecting the live of its citizens. And all the responsibilities for these problems rest solely on my shoulders.

The present situation of land subsidence in Tokyo has become more critical with every passing day. When the Metropolitan authorities controlled the pumping-up of underground water some time ago, the question of land subsidence seemed to be turning for the better. But again the question has become more serious recently.

To cite an extreme case, a subsidence of 24 cm a year has been noticed in the area at the mouth of the river Edo. This area, called the zero-meter zone, is the lowest land in Tokyo and is 4 to 5 meters below the sea-level. We made every effort to provide against floods and other calamities by constructing an embankment along the subsiding zone. But as we hardly reckoned that the land there would subside 24 cm a year, we have not had sufficient preparation for the construction of an embankment answering to the 24 cm-deep subsidence. Every year we must add to the embankment little by little. The added parts are weak as a matter of course and have little resistibility to floods. Furthermore, Japan is subject to earthquakes and what is worse, grave apprehensions are felt nowadays that another great earthquake might hit our densely-populated Metropolis in the near future—a quake which may bear comparison with the Great Earthquake of 1923.

Under these circumstances, it has become most important to construct an embankment along the subsidence-ridden zero-meter zone. The mere thought of this will be enough to make you understand how great a weight the question of land subsidesnce carries among other urban problems in Tokyo. The trouble is that we do not have any effective measure to prevent land subsidence for the moment. It is believed that land subsidence takes place by a contraction that takes place in the deep strata of the earth.

I think we can grapple with the question of land subsidence from two angles—administrative and scientific. The distribution of underground water covers not only the Tokyo district, but also neighboring Saitama Prefecture and Chiba Prefecture. So measures for Tokyo alone will be of no use. We do not yet have a joint administration under the control of which Tokyo, in cooperation with Saitama Prefecture and Chiba Prefecture, can organize counter-measures against land subsidence. This is also a pressing necessity if we are to cope with the problem in wide range. But supposing such an administration is organized, how could we then prevent subsidence effectively? Aren't there any other proper steps than the controlled pumping-up of underground water? A number of outstanding questions lie before us. I, on behalf of the citizens of Tokyo much annoyed with land subsidence, sincerely hope that all of you present here at this symposium will reach a satisfactory conclusion and will give proper advice to our policy. Tokyo is a typical city annoyed with urban problems. All kinds of problems confronting present-day cities find expression in Tokyo.

We heartily hope you will inspect not only the newly-built highways that run through the center of Tokyo, but also many other problems including land subsidence—traffic congestion, housing questions, rise in prices, etc.

We should be much obliged if you could offer us helpful suggestions with regard not only to land subsidence, but also to the whole range of urban problems.

Thank you.

WELCOMING REMARKS ON BEHALF OF THE SCIENCE COUNCIL OF JAPAN

Mr. Chairman, Distinguished Guests, Ladies and Gentlemen,

It is a great pleasure and honour for me to say a few words, on behalf of the Science Council of Japan, on the occasion of the Opening Ceremony of the International Symposium on Land Subsidence.

First of all, I wish to express my hearty welcome to our fellow scientists who are assembled here from different parts of the world including this country.

The Science Council of Japan, as the organization representing the scientists of Japan, is deeply concerned with the promotion of international scientific cooperation and exchange as one of its most important functions. For this purpose, the Council has sponsored international scientific meetings in our country and dispatched scientists to international meetings in other countries every year since its establishement in 1949.

It is a great honour for the Science Council of Japan to act as sponsor to the International Symposium on Land Subsidence with the attendance of no less than 250 distinguished scholars inclusive of 50 foreign delegates.

The use and control of water has a close relation to the life of human beings, who have from oldest times great interest in water problem. The problem of land subsidence resulting from water resources development is, with the advance of industrialization, a new problem of great importance, pressed for urgent solution. The decision of the Co-ordinating Council of the International Hydrological Decade that this problem should be included in the group of hydrological research problems with world-wide interest, requiring international co-operation was, I take it, most pertinent to the occasion.

Japan is one of the countries that have great abundance of water, but she is still confronted with this problem of land subsidence. It would have a great significance, I believe, that this symposium is held in our country. It is to be hoped that, as a result of presentation of excellent papers and lively discussions, the symposium achieve a brilliant success in order to answer the expectations from all quarters.

Through personal contacts among participants, the symposium will provide an excellent opportunity for furthering international cooperation and mutual understanding among scientists all over the world. I am sure that, in spite of the difference in languages and customs, the symposium will achieve success in every way through the goodwill and active cooperation of the participants.

Finally I hope that the participants from abroad will have enough time to see various things in this country and to get better acquainted with our people and culture.

Thank you.

Fujio EGAMI
President
Science Council of Japan

ADRESS OF DR. ARNOLD I. JOHNSON

Mr. Chairman, Distinguished Guests, Ladies and Gentlemen: this symposium represents the fulfillment of a long dream. It was about four years ago that Mr. Joseph Callahan and I started corresponding from the United States to Drs. Naomi Miyabe and Soki

Yamamoto, regarding plans for an international symposium on land subsidence. Thus, I receive double pleasure in giving a few opening remarks as a representative of the International Society for Soil Mechanics and Foundation Engineering.

As you may know, this international society held its Seventh International Conference in Mexico City the last week of August. The attendance at that meeting was between two and three thousand—the largest conference in the history of the Society. You will be interested in learning that Dr. T. Mogami of the University of Tokyo was elected Vice-President for Asia for the International Society.

Now, I take pleasure in saying that the International Society for Soil Mechanics and Foundation Engineering is proud to be one of the organizations sponsoring this International Symposium on Land Subsidence. We congratulate the Japanese Organizing Committee for their excellent arrangements and fine program.

Thank you.

DIRECTIONS OF RESEARCH ON LAND SUBSIDENCE

Naomi MIYABE

ABSTRACT

Considering the results of investigations carried out until now with regard to the land subsidence in various regions, further research on this subject should be advanced in directions;

(1) to develop laboratory experiments on models and analogs;
(2) to extend chemical and mineralogical analyses of soil that constitute the beds in the area under consideration;
(3) to extend hydrological studies on aquifers underneath wider areas including land subsidence areas, and
(4) to extend studies on the compaction of deep seated soil layers.

The results of advanced studies will be more help in placing into effect preventive measures against disasters due to the land subsidence.

RÉSUMÉ

Considérant le résultat des investigations effectuées jusqu'à ce jour au sujet de l'affaissement des terrains dans les diverses régions, on devrait développer une recherche plus avancée sur ce sujet dans les directions suivantes :

(1) développer les expériences sur les modèles et les analogues en laboratoire ;
(2) étendre les analyses chimiques et minéralogiques du sol constituant les couches dans la région considérée ;
(3) étendre les études hydrologiques sur les aquifères des régions souterraines plus vastes comprenant la zone d'affaissement, et
(4) étendre les études sur la compaction des strates les plus profondes.

Les résultats des études avancées faciliteront la mise en œuvre de mesures préventives contre les désastres dus à l'affaissement des terrains.

1. The phenomenon known as land subsidence has been found in the industrial areas as the results of abnormal lowering of the ground-water level, which might have been caused chiefly by excessive withdrawal of ground water. This land subsidence has brought about serious influences on public properties, through lowering and deformation of the land surface. In order to prevent the occurrences of disasters caused by the land subsidence, various devices have been furnished and fundamental research on the nature of this phenomenon has been developed, based on the results of observations obtained by means of precise levelings and compaction recorders at various places, and those of experimental investigations with regard to the soil samples taken from core borings, as well those of hydrological studies with regard to the aquifer systems concerned.

This fundamental research aims at finding proper methods to minimize the harmful effects of land subsidence. The scope of land subsidence may be represented by a simple diagram as given in figure 1, in which the balance of withdrawal and natural supply of ground water is shown, expressed as the input to a system of aquifer and soil layer, which may yield the output of land subsidence. It is quite natural that our research has been concentrated on the explanation of the mechanism of the action of the system of aquifer and soil layer which transforms the unbalance in the ground-water pressure into land subsidence.

2. An important finding in the main direction of our research was the linear relation between the rate of subsidence of clayey soil layer and the ground-water pressure, as given

by Wadati and Hirono [1], in the form

$$u = k(p - p_0), \tag{1}$$

in which k and p_0 are assumed as constants. In finding this relation, every five days' amount of subsidence was taken for the rate of subsidence, and the mean value of the heights of the ground-water level during the same interval of time was taken for p. This relation was a clue to the application of Terzaghi's theory of consolidation in explaining the phenomenon of the land subsidence [2].

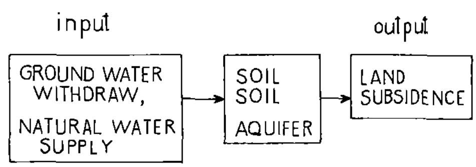

FIGURE 1. *Diagrammatic illustration of scope of land subsidence*

A question may then arise whether the same relation applies to the case where more than five day's amount of subsidence is taken for the rate of subsidence. Trial plots of five days', ten days', fifteen days' and thirty days' are made with regard to the compaction records, against the heights of the ground-water level for the same time interval obtained at the Kameido observation station, Tokyo, and those obtained at Yamanosita observation station, Niigata. The results are shown in figures 2 and 3.

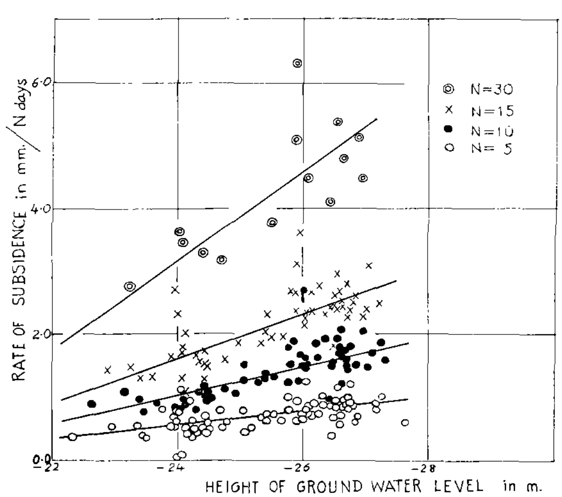

FIGURE 2. *Relation between rate of subsidence and the height of ground-water level; at Kameido station, Tokyo*

In figure 2, the straight lines designating respectively the relations between the five days', ten days', fifteen days' and thirty day's rates of subsidence and the heights of the ground-water level for corresponding time intervals are seen to have slopes in the ratio of 5:10:15:30, which are just the same as the ratio of the time intervals to which the respective rates of subsidence are due. Of course, the relations designated by the straight lines are accompanied by certain amounts of fluctuations. In contrast, in the case of the compaction obtained at Yamanosita (Niigata), as given in figure 3, the straight lines designating the relations between the rates of subsidence and the heights of the ground-water level have slopes in the ratio of 5:8:10:19, with wider ranges of fluctuations. These facts suggest, on the one hand, that there may be no objection to take thirty days' rate of subsidence instead of five days' rate, in discussing the relation between the rate of subsidence and the height of the ground-water level, with regard to the compaction obtained at the Kameido station, Tokyo, but, on the other hand, a similar procedure would not apply to the case of compaction obtained at the Yamanosita station, Niigata.

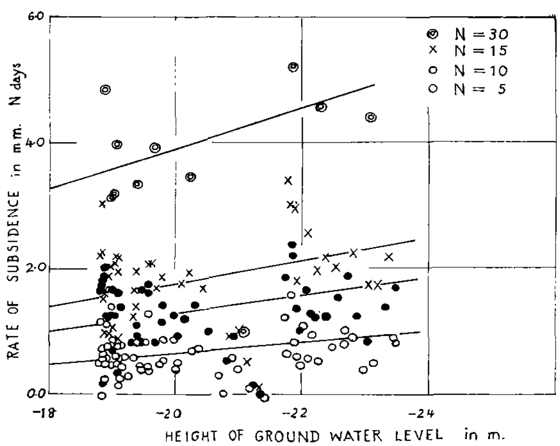

FIGURE 3. *Relation between rate of subsidence and the height of ground-water level; at Yamanosita, Niigata*

From the relation between the rate of subsidence and the height of the ground-water level with regard to the observations at Yamanosita, Niigata, it is inferable that the actual subsidence may be the integration of various influences yielded by the changes in the heights of the ground-water levels in a number of aquifers concerned.

3. As seen in figure 2, the thirty days' or montly rate of subsidence is equally meritorious in applying to deduce the above mentioned relation, with regard to the compaction observed at Kameido station, Tokyo. The values of montly subsidences are then plotted against the monthly mean heights of the ground-water level, observed at this station, and the results are shown in figure 4. As seen in this figure 4, the constant k in the equation does not hold for extremely low ground-water level. This fall of the constant k will make the explanation of the mechanism of land subsidence more complicated.

On the other hand, it is evident that the land subsidence, once caused as the results of lowering in the heights of the ground-water level, does not recover even when the ground water has recovered as before; that is, the land subsidence is said to be an irreversible phenomenon. In this connection, it may be conceivable that the compaction of soil layer is attributed to the rearrangements of soil particles which constitute the soil mass structure. In this concept, the stress applied to the soil mass as the results of decay in the pressure of the confined aquifer may cause the rearrangement of soil particles and yield strain within the soil mass. The constant which links the stress and the strain thus yielded may be dependent on the modes of arrangements of soil particles [3].

The above-mentioned concept is, of course, introduced for one of the possible solutions to the question of irreversible land subsidence, in connection with the fact that the monthly rate of subsidence decreases with further lowering in the ground-water level when it has become extremely low.

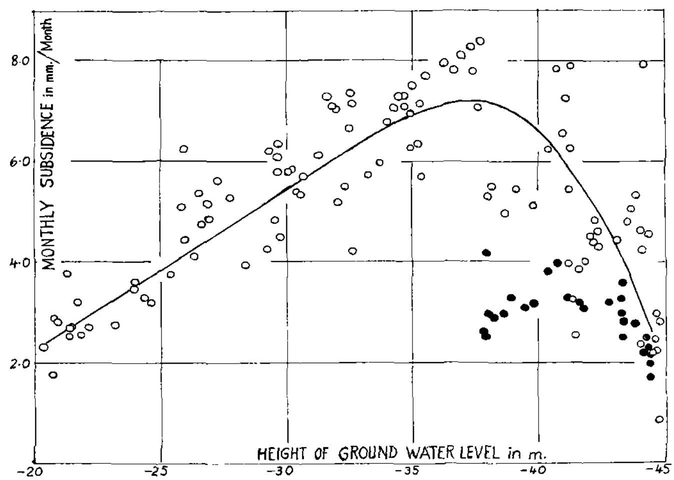

FIGURE 4. *Variation of monthly subsidence with descending ground-water level*

4. In going on with the quantitative analyses of the land subsidence, it is thought desirable to take into consideration the fact that the soil layer, being compacted as the result of lowering in the ground-water level, is composed of various components with various thicknesses. This was suggested by the results of studies on partial compaction of soil layers in Tokyo [4] and in Niigata [5].

The soil layer in the northern and eastern parts of Tokyo is composed of layers with various constituent materials. It may roughly be divided into three parts; that is, the upper part, composed in main of loose clay and silt, is regarded as alluvial deposits; the middle part, composed of rather fine sand with insertions of clayey sheets, is regarded as the upper part of diluvial deposits and the lower part of the diluvial deposits is generally composed of alternate coarse sand and gravel-containing sand layers, underneath of which appears the tertiary bed of harder rocks.

The observed amount of land subsidence owes its principal part to the compaction of alluvial and diluvial deposits on lowering in the ground-water level. There is of course some indication of the compaction of tertiary layer, situated underneath the diluvial deposits, as may be referred to shortly later.

Under these circumstances, it would be desirable to partition the contributions to the land subsidence of each soil layer which constitute the alluvial and diluvial layers. Studies in this direction were first tried by Miki [6], who assumed that the observed amount of land subsidence u is a linear combination of partial compactions of three composite soil layers, each being proportional to the thickness of the corresponding layer; that is, u is given by

$$u = c_1 h_1 + c_2 h_2 + c_3 h_3, \tag{2}$$

where h_1, h_2, h_3 are respectively the thickness of the composite layers and c_1, c_2, c_3 are characteristic constants of respective layers for the compaction or the subsidence.

Similar investigation was also developed by Aoki and the present author [7]. In our study, the rate of land subsidence (per year) is assumed as a function of the mean height of the ground-water level, p, and the change in the height of the ground-water level, Δp, during the period concerned; that is, the rate of compaction u is given by

$$u = \sum a_i(p-p_0) + \sum b_i \Delta p, \tag{3}$$

where a_i and b_i are constants which characterize the contribution of the i-th constituent soil layer.

For an example, the results of analysis with regard to the records of compaction obtained at Kameido station are herewith referred to. At this station, the reference tube of the compaction recorder is piled down to the depth of 64 metres and fixed there. As the results of core boring, the underground structure at this spot is known to be such that, except for surface reclamation earth 2 or 3 metres thick, the alluvial clay deposit exists to the depth of 33.0 metres, and then comes a sandy layer of diluvial deposits to the depth of 74.0 metres. The observed land subsidence will then be the sum of compactions of these two kinds of soil layers in response to the decline in the ground-water pressure in the second layer. It is assumed that, if the ground-water level designated by the height, measured from the bottom of the second layer, stands at higher level than the boundary of the first and the second layer, but lower than the ground surface, and, if the depression in the ground-water level occurs as a linear function of time, that is $q = at$, the expression (3) is to be replaced by

$$u = k_1 a h_1' + k_2 a h_2' + k_1' \Delta p h_1' + k_2' \Delta p h, \tag{4}$$

where h'_1 and h'_2 are effective thickness of the first and the second layers, Δp is the depression in the ground-water pressure due to the time interval concerned. For the Kameido station, the depth from the ground surface to the bottom of the first layer is $h_1 = 33.0$ m., and the thickness of the second layer is $h_2 = 41.0$ m, and the depression of the ground-water level amounted to 19 metres during the time interval from 1956 to 1967.

The results of analyses mentioned above show that the clayey layer plays a leading part during the period in which the subsidence is taking place in response to relatively rapid decline in the ground-water pressure, while the sandy layer is important during the period in which the ground-water level becomes lower than the bottom level of the first layer and the subsidence advances in response to slower decline in the ground-water pressure.

5. Since we first noticed the land subsidence as the results of repeated precise levelings, this method of survey has been employed as the most useful means in finding the areal distribution of the land subsidence. As the results of repeated precise levelings, the gradual changes in the patterns of the distributions of the land subsidence in Tokyo, Osaka and Niigata are obtained. Thus it has been made possible to discuss the development of subsidence with increasing withdrawal of the ground water, as may be expected from increasing industrial activity, and its decay under some remedial measures taken for control of the declining ground-water level.

Since it is very difficult to collect reliable information on the amounts of ground-water withdrawal for a certain period, notwithstanding it is quite indispensable in the investigation of water balance, we tried to study the gradual changes in the distribution patterns of subsidence on the basis of theory of diffusion [8], substitutive of studies in slow migratory changes in the heights of the ground-water level, expected from withdrawal of the ground water.

Considering the field of vertical displacements of subsidence, the amount of annual subsidence at any point in the field at any time is assumed to satisfy the equation

$$\frac{\partial U}{\partial t} = D\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2}\right), \tag{5}$$

where U designates the amount of annual subsidence, D is a constant analogous to the diffusion constant, x and y are taken eastward and northward respectively, referring to a certain point in the field as an origin. An example of the distribution of D thus obtained is shown in figure 5, which may be some indication of subsurface structures, as compared with figure 6, in which the diluvium surface topography in the area under consideration is given. Thus we may imagine that the lowering in the ground-water level due to excessive withdrawal of the ground water at a certain place will spread about, though very slowly, influenced by the subterranean structures of soil layers with various hydrogeological properties, and so does the land subsidence also.

Hydrology of ground-water balance would then be an important subject of our concern in connection with the land subsidence.

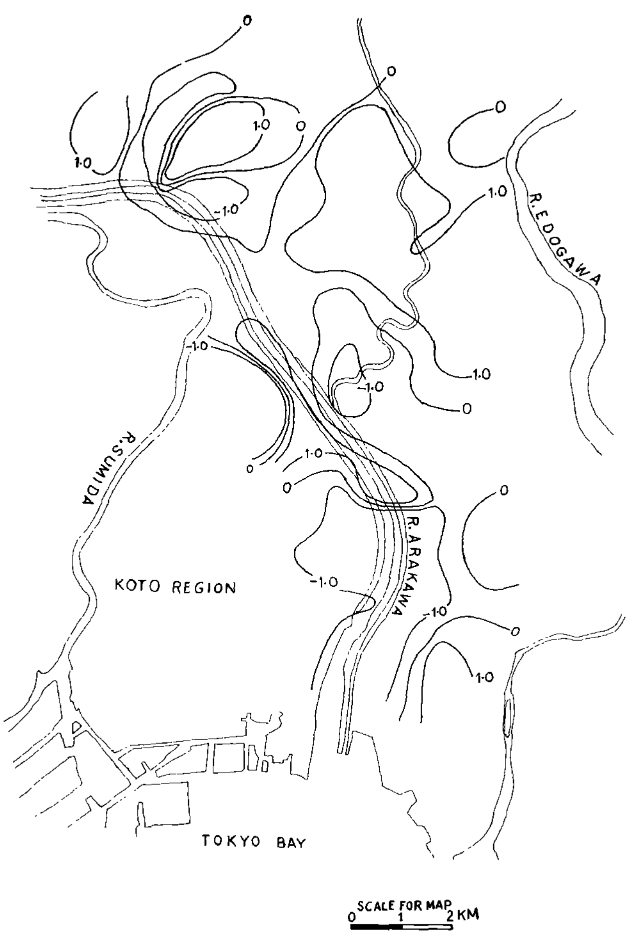

FIGURE 5. *Example of distribution of "D", in Koto Region of Tokyo*

6. By the diagram in figure 1, it was shown that the balance of the positive (natural water supply) and the negative (leakage and withdrawal of ground water) inputs is turned out to output (subsidence) through the system of soil layer and aquifer. Now this analog model would be supplemented in some respects; that is, some external disturbance is operative, either directly or indirectly, on this system of soil layer and aquifer.

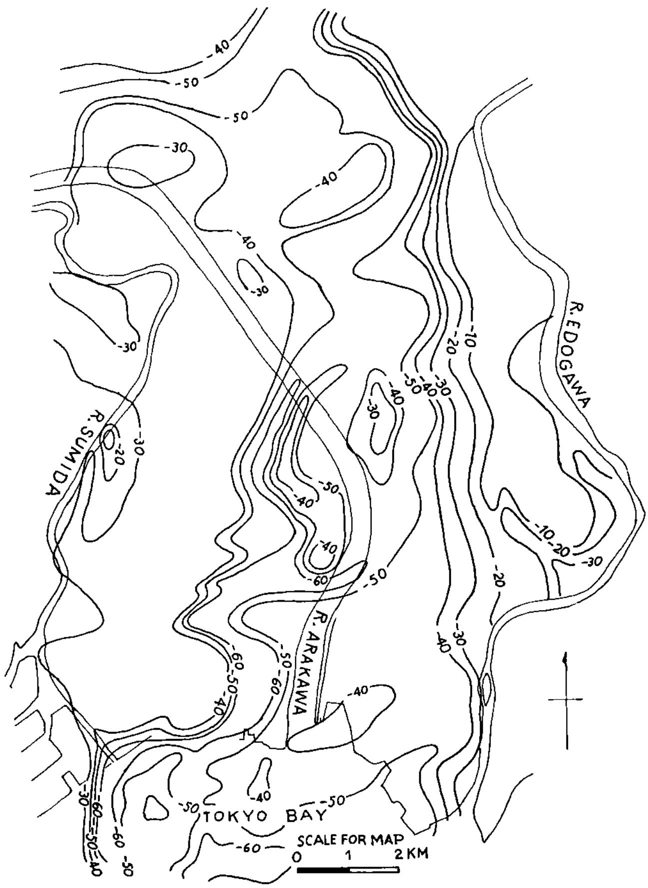

FIGURE 6. *Diluvium topographic features in Koto Region of Tokyo, (reproduced from geological map of Tokyo, published by Tokyo Institute of Civil Engineering)*

Since these external disturbances will modify the subsidence through the system of soil layer and aquifer, the system may be said to behave in response to these disturbances as it does in response to the negative input of lowering in the ground-water level.

The compaction records generally contain secular change superposed with periodic or quasi-periodic changes, which may be separated by taking the deviations of the values of hourly readings from their twenty five hours' moving averages. This procedure enables us to remove the terms of secular subsidence, and the residuals are mostly of the disturbances with diurnal, semi-diurnal and like periods.

Among the periodic fluctuations obtainable as the results of the present analysis, the term of diurnal change looks most remarkable. This diurnal change is characterized with a remarkable daytime swelling, which is just in phase with the diurnal change in the atmospheric pressure [9]. In this case, the land surface yields apparent expansion of approximately 0.2×10^{-2}mm per mili-bar pressure decline (see figure 7). We also notice slower change in the ground surface, superposed on the secular subsidence. This quasi-periodic slow change seems to be associated with the fluctuations in the atmospheric pressure as in the case when a cyclone passed by the observation station. In this case, however, the movement of the ground surface is not found in phase with the slow change in the atmospheric pressure. The ground surface movement in this case is rather parallel to the changes in the height of the ground-water level, which may be moved under the influence of the slower change in the atmospheric pressure, though with considerable phase lag. It may thus be surmised that the changes in the atmospheric pressures influence the up and down movements of the ground surface directly, on the one hand, and indirectly, on the other hand, through the movements of the ground water.

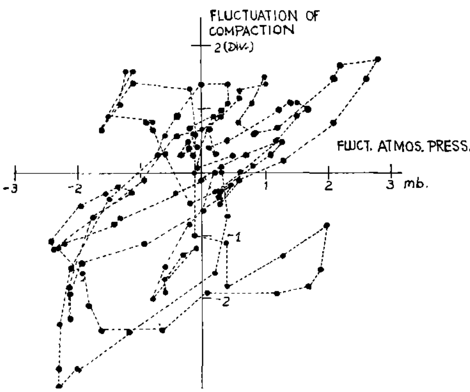

FIGURE 7. *Relation between fluctuations of compaction and those of barometric pressure.*

In order to delineate a model for concrete explanation of the above-mentioned phenomenon, and, consequently, to contribute in explaining the mechanism in detail of the response of the soil layer to the external disturbances including secular changes in the heights of the ground-water level, it is thought necessary to develop further research in this direction.

It is also notable that the ground surface undergoes acute subsidence or abrupt settling in association with the occurrence of the earthquake of considerable intensity, as has already been discussed by the author in collaboration with Inaba [10]. An example of the relation between the acute subsidence and the earthquake intensity is shown in figure 8, which shows a general tendency for the acute subsidence to be larger when the earthquake shock is stronger. Although the detailed discussion on this subject is herewith omitted, this fact is thought to be derived from the rheological nature of the materials which compose the soil layer concerned. It may also be remarked that this acute subsidence could be related in some respects with the periodic changes in the heights of the ground surface mentioned above.

7. It may be worthy of remark that there are some indications of the compaction of soil layers of the tertiary deposits.

As has been referred to, there are a number of reference tubes of the compaction recorders, of which two tubes at least are evidently fixed to the tertiary deposits. One of these tubes, situated at the Todabasi observation station, which is piled down to the depth of 290 metres from the ground surface and fixed there, underwent subsidence of 45.9 mm during 1967, and the other tube, situated at the Sin-edogawa observation station, which is piled down to the depth of 450 metres from the ground surface and fixed there, underwent subsidence of 24.9 mm during the same interval of time. Although these amounts of subsidence are only a fraction of the amount of subsidence measured at the benchmarks near the respective stations for the same interval of time, the amount of subsidence of several centimeter per year should not be regarded as negligible.

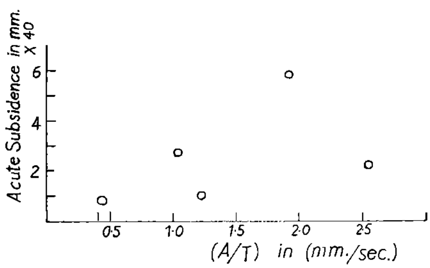

FIGURE 8. *Relation between the intensity of earthquake (represented by the quantity corresponding to velocity) and the amount of associated acute subsidence*

However, the fact that these soil or rock layers are situated at the depth of several hundred metres or so, is a mortal difficulty in explaining the compaction of probably hardened soil there, until som new device to overcome the difficulty is developed.

8. Taking into consideration the results of studies which have been developed until now, it may be thought desirable, besides the routine surveys and observations to be carried on, to develop further research in the following directions:

(a) Model experiments such as executed by Murayama [11] would be extended under various conditions. Special attention would be concentrated on the thermal conditions on executing these experiments; because thermodynamical consideration would be necessary in understanding the results of the experimental studies and in applying them to practical cases. The experiments on models and analogs are recommendable, because their results may provide the basic concept required for the explanation of the phenomenon of land subsidence.

(*b*) Chemical and mineralogical analyses of the materials which constitute the soil layer of the subsidence area would be requirable; because the chemico-physical properties of soil particles are largely dependent on the composition and are important factors for the constitution of microscopic structure of soil mass. In our opinion, the change or the rearrangement in this microscopic structure of soil mass willl be the main source of the compaction. Consequently, the results of analyses will be an approach to complete understanding of the phenomenon of land subsidence, particularly its irreversible character.

(*c*) The hydrological studies on the states of aquifer systems with their changes in the area concerned would be developed in more detail and for a widely extended area. Main difficulties in this field of study may consist in obtaining reliable information on amounts of ground-water withdrawal, and also in investigating the chronic changes in the heights of ground-water level which may be caused by migratory movement of the ground water through heterogeneous aquifers. These difficulties may be overcome by tenacious efforts in field observations and simulation studies.

(*d*) Studies on physical properties of hardened soil or rock situated at deeper level would be developed. The land subsidence was once understood as the results of compaction of loose soil mass deposited over the low land. But, as already referred to, we have some information indicating the compaction of deep-seated hard soil, or hard soil mass of tertiary formation. Such subsidence as is occurring in the area of hard soil is particularly noteworthy in the area where the underground gas or some buried resources, inclusive of ground water, are being withdrawn. It is therefore desirable to develop studies on the physical and chemical properties of these hard deposits, as we have done with the loose soil of the upper layer in the subsidence area; as well as the investigation on actual movements (subsidence) of these deep-seated layers discriminated from the compaction of the overlaying layers.

The developments of scientific research in directions mentioned above may not only contribute to the advancement of science, but also develop a way of approach to the solution of the problem how to make possible the prediction of future land subsidence which is required on planning the preventive measures against disasters expected as the result of advancement of land subsidence.

In conclusion, the author would like to express his sincere thanks to the colleagues of Tokyo Institute of Civil Engineering for their kind cooperation on preparation of the present report.

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DISCUSSION

Intervention of Mr. Arnold, A. JOHNSON, in answering Dr. Miyabe's suggestion on organization or machinery for continuing international exchange of information on land subsidence.

Comments of Mr. JOHNSON:

Dr. Miyable, I believe that land subsidence definitely is an interdisciplinary problem and that this problem will be with the world for many years to come. Furthermore, land subsidence problems will increase in number and complexity. Thus, some mechanism must be found by which scientists and engineers of all disciplines may easily coordinate studies of land subsidence and may communicate to each other the results of their researcht, both within a country and internationnally.

A large percentage of land subsidence appears to be due to hydrologic causes and most of the studies are being made by hydrologists at present. This cause and the interest of hydrologists in land subsidence probably will increase in the future.

Thus, some international hydrologically oriented scientific society probably would be the most effective sponsor of international communication and coordination for studies of land subsidence. The International Association of Scientific Hydrology would be such an organization. ISAH could cooperate in such activities with other international societies related to other disciplines also interested in land subsidence such as the International Society for Soil Mechanics and Foundation Engineering.

As a first step, I would propose that a questionnaire, sponsored by IASH, be given world-wide circulation to determine the interest and progress in studies of land subsidence. The assistance of ISSMFE also could be solicited to obtain the widest possible circulation of the questionnaire. The resulting information not only would provide back ground for planning the most efficient arrangement for handling cooperation and communication on land subsidence between scientists of many disciplines, but also could be published and thus provide another means of communication on the subject.

The information received from the questionnaires also could indicate the frequency for holding symposia on land subsidence in order to provide optimum communication amoung interested scientists. In my opinion national symposia on this subject could be held in the most concerned countries every 2 or 3 years, but international symposia should not be held more frequently than 5 or 6 years minimum. Speaking unofficially for the U. S. National Commitees for IASH and ISSMFE, I believe that these organizations would be interested in cooperating in another symposium on land subsidence if it were to be held approximately 6 years from now.

STATUS OF PRESENT KNOWLEDGE AND NEEDS FOR ADDITIONAL RESEARCH ON COMPACTION OF AQUIFER SYSTEMS[1]

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ABSTRACT

The deposits that are compacting in areas of major subsidence are relatively unconsolidated, chiefly alluvial and lacustrine sediments, of late Cenozoic age, in confined aquifer systems of heterogeneous texture.

Parameters needed to predict compaction include compressibility; initial stress and change in stress; number and thickness of compressible beds, and vertical hydraulic conductivity. Methods used for estimating compaction involve computing it from laboratory test results on cores, or deriving parameters for the entire aquifer system from field stress-strain relations. Methods used to measure compaction (or expansion) of sediments include depth-bench-marks and counterweighted-cable or free-pipe extensometers, periodic casing-collar logging, and periodic measurement of emplaced radioactive bullets.

1. Publication authorized by the Director, U.S. Geological Survey.

Additional research is needed on the effect of multi-cyclic loading; improvement of field methods for determining compressibility of aquifer systems and measuring compaction and pore pressures; estimating preconsolidation loads; development of mathematical and electronic models capable of simulating compaction of confined aquifer systems; and several other topics.

RÉSUMÉ

Les alluvions qui s'accumulent dans des régions de considérable affaissement sont relativement inconsistantes, surtout les sédiments alluviaux et lacustres, du Cénozoïque récent, dans les nappes aquifères artésiennes de texture hétérogène.

Les paramètres nécessaires pour prédire la compaction incluent la compressibilité, la pression initiale et le changement dans la pression, le nombre et l'épaisseur des couches compressibles et la perméabilité verticale. Les méthodes utilisées pour évaluer la compaction comprennent le calcul de celle-ci d'après les résultats d'essais de laboratoire sur carottes de sondage ou d'après les paramètres qui dérivent des relations pression-tension relevées sur le terrain, pour la nappe aquifère entière. Les méthodes utilisées pour mesurer la compaction (ou l'expansion) des sédiments incluent des repères indicateurs de profondeur et des câbles à contrepoids ou des extensomètres à tuyau libre, des inspections périodiques de "casing-collar logging" et des mesures périodiques de balles radioactives placées dans le sol.

Des recherches supplémentaires sont nécessaires pour déterminer l'effet de charges multicycliques ainsi que pour l'amélioration des méthodes sur le terrain afin de déterminer la compressibilité des nappes aquifères et mesurer la compaction et les pressions des pores des couches, de même que pour évaluer les poids de préconsolidation et pour développer des maquettes mathématiques et électroniques capables de simuler la compaction des nappes aquifères ainsi que pour plusieurs autres sujets encore.

INTRODUCTION

The worldwide exploitation of ground-water resources and the consequent water-level declines are creating many areas of land subsidence and associated problems. Poland and Davis (1969) summarized available pertinent information on areas of substantial known subsidence due to fluid withdrawal as of 1963. These included oil fields in Goose Creek, Texas, and Wilmington, California, USA, and Lake Maracaibo, Venezuela; gas fields in Niigata, Japan, and the Po Delta, Italy; and ground-water reservoirs in Japan, London, England, Mexico City, Mexico, and in USA–Savannah, Georgia; the Houston-Galveston area, Texas; Denver, Colorado; the Eloy-Picacho area, Arizona; Las Vegas, Nevada; and the San Joaquin and Santa Clara Valleys in California. Since 1963, subsidence of 30 cm (1 foot) or more over confined aquifer systems has been reported in Baton Rouge, Louisiana, USA (Davis and Rollo, 1970), and in the Taipei Basin, Taiwan (Tang, Min-1967). Van der Knaap and van der Vlis (1967) have contributed substantially to know, ledge on the characteristics of the compacting sediments in oil fields at Lake Maracaibo, Venezuela.

We can anticipate that in the next few decades areas of land subsidence will multiply and hence that problems will become more widespread. The problems caused to date by land subsidence due to the withdrawal of fluids are a principal reason for this International Symposium at Tokyo. Therefore, it is appropriate at this time and place to assess the state of knowledge concerning subsidence and aquifer-system compaction, the deficiencies in knowledge, and the needs for additional research.

TYPICAL SUBSIDENCE ENVIRONMENT

The deposits that are compacting in areas of major subsidence due to groundwater-extraction are unconsolidated to semiconsolidated clastic deposits of late Cenozoic age (see table 1). Most are alluvial and lacustrine deposits. The compacting sediments on the

Texas Gulf Coast and at Baton Rouge, Louisiana, were laid down in a fluviatile and shallow-marine environment. All areas are characterized by confined aquifer systems containing permeable aquifers of sand and (or) gravel of low compressibility, interbedded with clayey aquitards of low permeability, high compressibility, and variable thickness.

TABLE 1. Description of areas of major land subsidence due to ground-water extraction

Location	Depositional environment and age	Depth range of compacting beds below land surface (meters)	Maximum subsidence (meters)	Area of subsidence (sq km)	Time of principal occurrence
Japan, Osaka, and Tokyo	Alluvial (?); Quaternary (?)	10-200 (?)	3-4	?	1928-1943 1948-1965+
Mexico, Mexico City	Alluvial and lacustrine; late Cenozoic	Chiefly 10-50	8	25+	1938-1968+
Taiwan, Taipei Basin	Alluvial, late Cenozoic	30-200 (?)	1	100±	? -1966+
Arizona, central	Alluvial and lacustrine (?); late Cenozoic	100-300+	2.3	?	1952-1967+
California, Santa Clara Valley	Alluvial; late Cenozoic	50-300	4	600	1920-1967+
California, San Joaquin Valley (three areas)	Alluvial and lacustrine; late Cenozoic	90-900	8	9,000	1935-1966+
Nevada, Las Vegas	Alluvial; late Cenozoic	60-300 (?)	1	500	1935-1963+
Texas, Houston-Galveston area	Fluviatile and shallow marine; late Cenozoic	50-600+	1-2	10,000	1943-1964+
Louisiana, Baton Rouge	Fluviatile and shallow marine; Miocene to Holocene	40-900 (?)	0.3	500	1934-1965+

All the compacting deposits are presumed to be normally consolidated. All are tapped by water wells, to depths ranging from 200-900 m. Porosity of the cored, primarily fine-grained, deposits in the central California areas averages about 40 percent and specific gravity of the grains about 2.7 (Johnson, Moston, and Morris, 1968); montmorillonite is the predominant clay mineral in the subsiding areas in Texas (Corliss and Meade, 1964), central Arizona (Poland, 1968), central California (Meade, 1967), and in Mexico City (Marsal and Mazari, 1959), comprising 60-80 percent of the clay-mineral assemblage. Clays rich in montmorillonite are more porous and more compressible under a given load than clays consisting mainly of illite or kaolinite.

Head decline in the confined aquifer systems has ranged from about 30 m at Mexico City and Las Vegas, Nevada, to 150 m on the west side of the San Joaquin Valley, California.

CAUSE OF LAND SUBSIDENCE DUE TO EXTRACTION OF WATER

The withdrawal of water from wells reduces the head in the aquifers tapped and increases the effective stress (grain-to-grain load) borne by the aquifer matrix .As first stated by Terzaghi (1925)

$$p = p' + u_w$$

where:

p total stress (total overburden load or geostatic pressure);
p' effective stress (effective overburden pressure or grain-to-grain load), and
u_w neutral stress (fluid or pore pressure).

As shown by figure 1, the lowering of head in a confined aquifer system does not change the geostatic pressure (except for the water removed by expansion of the fluid and by compaction of the aquifer system, which is small and is disregarded in this diagram).

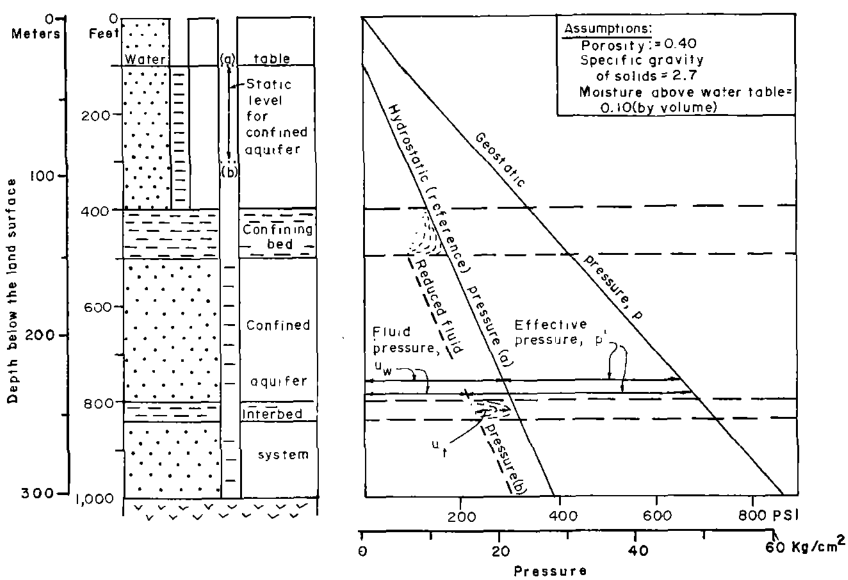

FIGURE 1. *Pressure diagram for an unconfined and a confined aquifer, with head reduction in confined aquifer only*

Therefore, the effective stress increase in the confined aquifers is equal to the decrease in fluid pressure. The aquifers respond essentially as elastic bodies. Hence the compaction in these is immediate and is chiefly recoverable if fluid pressure is restored, but usually is very small.

On the other hand, in the aquitards and aquicludes which have low hydraulic conductivity and high specific storage, the vertical escape of water and adjustment of pore pressure is slow and time-dependent. Hence, in these fine-grained beds the stress applied by the head decline becomes effective only as rapidly as pore pressures decay toward equilibrium with pressures in adjacent aquifers. The pattern of excess pore pressure decay is illustrated diagrammatically in figure 1. It is the time-dependent nature of the pore-pressure decay in the aquitards and aquicludes that complicates the problem of estimating or predicting compaction of heterogeneous aquifer systems.

CONSOLIDATION THEORY VERSUS AQUIFER-SYSTEM COMPACTION

The theory of time-dependent soil consolidation pioneered by Karl Terzaghi (1925) for the solution of soil settlement problems was developed primarily for appraisal of the effects of load applied at the land surface. It supplies a useful approach to quantitative analysis of compaction due to head decline. However, certain basic differences should be recognized because they affect the application of the theory to estimates of compaction and subsidence due to ground-water extraction from depths as great as 900 m.

LOAD APPLIED AT THE LAND SURFACE

For a load applied at the land surface, the following conditions usually exist:

1. The area loaded is small, usually not more than a few acres, and the applied stress decreases with depth.
2. The load normally is applied in a period of months and is constant thereafter.
3. Total geostatic load is increased, usually in the range of 0.2 to 2 kg/cm^2.
4. In the fine-grained compressible layers of low permeability, increase in stress is borne initially by an increase in pore pressure, similar to the response to loading in the standard consolidometer test.
5. The time-duration of the load usually is sufficient to permit excess pore pressures to decay to equilibrium and hence for effective stress and consolidation to reach ultimate values (neglecting secondary consolidation).
6. Significantly compacting layers are at shallow depth and few in number. Hence, the collection and laboratory testing of an adequate number of "undisturbed" cores and the installation of piezometers for observing pore-pressure change usually is economically feasible.

STRESSES APPLIED BY HEAD REDUCTION IN A CONFINED AQUIFER SYSTEM

For the areas of major land subsidence described in table 1, the following conditions generally prevail:

1. The area of substantial stress change (and appreciable subsidence) usually is laterally extensive, from 10^2 to 10^4 km^2.
2. The long-term increase in applied stress is gradual (10-50 years or more) but its magnitude is constantly changing; overall increase may be as much as 10 kg/cm^2; the annual variation (seasonal head change) in applied stress may be 5-20 times as great as the average annual increase; also, applied stress may be decreased by appropriate water-level changes.
3. Total geostatic stress is unchanged, except for the water squeezed out by compaction; but if the water table changes, geostatic stress also changes.
4. Stress changes induced by the artesian-head decline are seepage stresses. Change in the position of an overlying water table changes both gravitational and seepage stresses.
5. Because head in the aquifers fluctuates with time, pore pressures in the aquitards seldom reach equilibrium with the head in adjacent aquifers/
6. Compacting zones span large depth intervals (50-800 m) and aquitards are numerous and usually highly variable in thickness and vertical permeability, both vertically and laterally. Hence, the collection and laboratory testing of "undisturbed" cores, and the installation of multiple piezometers for the observation of pore-pressure changes are expensive, and usually funds are not available for obtaining adequate vertical and areal control.

METHODS OF OBTAINING ESSENTIAL PARAMETERS

The essential parameters and criteria in evaluating present or potential subsidence are compressibility (or a related measure, such as C_c, the compression index, derived from time-consolidation tests); initial stress and change in stress with respect to depth and time; appraisal of whether fine-grained beds are normally loaded or are overconsolidated [estimated from plots of void ratio vs. the logarithm of load (e-log p' plot) from one-dimensional consolidation tests]; the number and thickness of the fine-grained compressible beds; and the coefficient of consolidation, c_v, which is obtained from the time-consolidation test.

According to soil-consolidation theory, the time, t, required for a homogeneous consolidating clay bed draining from both faces (aquitard) to reach a specified percentage of ultimate compaction can be calculated from

$$t = \frac{T(b'/2)^2}{c_v} \tag{1}$$

The parameter $c_v = K'/S'_s$, the hydraulic diffusivity (Domenico and Mifflin, 1965). Therefore, in hydrologic terms,

$$t = \frac{T S'_s(b'/2)^2}{K'} \tag{2}$$

where

T a dimensionless time factor obtained from type curves for percent consolidation vs time;

S'_s the specific storage;

b' the bed thickness, and

K' the vertical hydraulic conductivity of the clay bed.

Thus, the time required to achieve a specified percentage of ultimate compaction varies directly as the square of the draining thickness and the specific storage, and inversely as the vertical hydraulic conductivity.

The problem of time-dependent aquifer-system compaction can be investigated by one or both of two methods, which I have termed the laboratory approach and the field approach.

Laboratory approach

This approach includes drilling core holes, running geophysical logs, and making laboratory tests of the core samples. These tests should include particle-size distribution, dry unit weight, specific gravity of solids, porosity, Atterberg limits, and one-dimensional consolidation and rebound tests to give compressibility (m_v) and hydraulic diffusivity, from which K' and S'_s can be computed. Direct permeameter tests for hydraulic conductivity (horizontal and vertical) also are useful but techniques for testing clayey cores are critical and subject to further research and verification (Johnson and others, 1968, p. A26; Olsen, 1966).

The antecedent records of head change in the confined aquifer system and of the water table are needed to determine the history of stress changes. We have found it quantitatively convenient in treating complex aquifer systems to compute effective stresses and stress changes in terms of gravitational and seepage stresses (Lofgren, 1968). The number, thickness, and lithologic character of the aquifers, aquitards, and aquicludes can be determined from the sample log and the electric log.

Ultimate compaction and subsidence due to known or assumed changes in applied stress can be computed (Gibbs, 1960) by utilizing representative e-log p' plots and the equation

$$\Delta m = \frac{e-e_1}{1+e} m \tag{3}$$

in which

Δm compaction;
e initial void ratio;
e_1 void ratio after loading, and
m thickness of the aquifer-system segment.

Equations 1 or 2 can be utilized to estimate the percentage of ultimate compaction (the magnitude of transient compaction) accomplished at any time in response to a specified stress increase. This method was used by Miller (1961) in computing subsidence at core holes in the San Joaquin Valley. Although this approach supplies useful predictions, estimates made for thick heterogeneous aquifer systems by this method are subject to considerable latitude. The numerical result depends in part on the number and distribution of cores tested, but especially on judgment as to what tests from samples about 2 cm thick are representative of lithologic intervals of variable texture.

Field approach

In problems of evaluating potential compaction of heterogeneous aquifer systems, a field approach to the determination of response of the aquifer system to change in applied stress, and derivation of the controlling parameters from the measured stress-strain relations, is to be preferred to the procedure and the problems of summing up characteristics of a large number of individual beds or zones. For the same reason, the advantages of utilizing the overall response of the system for estimating transmissivity and storage coefficient of aquifer systems have led to the widespread use of aquifer pumping tests.

Several methods have been used to measure the change in thickness of compacting sediments in response to increase in stress. These include (1) the depth-bench-mark and counterweighted-cable extensometers and recording and amplifying equipment developed and used by the US Geological Survey in California, Arizona, and Texas, and described by Lofgren (1961, 1970). (2) Somewhat similar extensometers utilizing "free" pipes with centering guides within an outer casing and employing an amplifying lever-system at land surface have been developed and used in Tokyo (Miyabe, 1962, 1967) and Niigata (Committee for Investigation of Earth Level Subsidence in Niigata, 1958), Japan. (3) At Long Beach, California, in the Wilmington oil field, the depth of compacting zones and magnitude of compaction have been measured successfully by running casing-collar logs periodically in a single well and comparing the distance between casing collars with the distance shown by the original casing tally when the well was drilled. As much as 17 feet of casing shortening (and compaction) has been measured in a single well by repeated surveys (Poland and Davis, 1969, fig. 9). (4) Also at Wilmington and in Venezuela, radioactive bullets have been emplaced in the formation behind the casings at known depths and their position has been resurveyed by radioactive detector systems at later time. At Wilmington, the results reportedly have not been very satisfactory, partly due to cable-stretch and equipment problems and partly to the difficulty of accurate resolution of the radioactivity pickup curves. (5) At Niigata, Japan, the radioactive bullet technique has been refined by use of observation wells (up to 950 m deep) in which radioactive bullets were shot into the strata every 40 m, and radioactive reference pellets were attached to the casing every 20 m. Special logging equipment was developed to eliminate some of the mechanical causes of error. Observations have been made at about 1-year intervals since 1961 and apparently have been reasonably successful in determining the depth location and general magnitude of compaction (Sano and Kanaya, 1966).

Significant horizontal strain at the land surface in the vicinity of a pumped well has been measured by Davis, Peterson, and Halderman (1969). They observed a zone of compression near the well and a zone of tension farther away. They also measured vertical strain simultaneously. Additional research is needed to find whether measured displacements can be related directly to the hydrologic and mechanical properties of the producing aquifer system.

The utility of stress-strain curves derived from measurements of compaction and change in artesian head (stress) in confined aquifer systems to obtain compressibility parameters in both the plastic and the elastic range of stress application is discussed in a separate paper being presented at this conference (Riley, 1970). One example in the elastic range, for a well 176 m (577 ft) deep, on the west side of the San Joaquin Valley, is presented here (fig. 2). The plot extends from March 1967 to July 1968. The obviously purely elastic response between November 26, 1967, and February 26,1968, indicates that the component of the storage coefficient due to deformation of the aquifer system skeleton is 1.2×10^{-3}. The quifer system is 85 m (280 ft) thick. Therefore, the component of specific storage contributed by deformation of the system skeleton is $1.4 \times 10^{-5} m^{-1}$ ($4.3 \times 10^{-6} ft^{-1}$) and the compressibility of the skeleton is $1.4 \times 10^{-4} cm^2/kg$ ($1 \times 10^{-5} psi^{-1}$).

If the compaction and water-level measurements are adequate to yield stress-strain plots that define compressibility in the plastic-plus-elastic range (stress exceeds preconsolidation stress) for the full compacting interval, approximate ultimate compaction (and subsidence) for a specified stress increase can be computed by use of the equation

$$\Delta m = M_v m \Delta p'$$

where:

M_v gross coefficient of volume compressibility for the system;
m thickness of system, and
$\Delta p'$ change in effective stress.

In the elastic range of deformation, the compaction or expansion of the system can be computed from a similar equation in which M_v is replaced by a, the gross compressibility in the elastic range.

A second field approach to direct determination of pertinent parameters is by stressing aquifer systems through pumping tests. Recent advances in interpretive procedures that recognize and include the storage yielded by semipervious compressible aquitards and aquicludes adjacent to pumped leaky aquifers were pioneered by Hantush (1960). Recent papers by Witherspoon and Neuman (1967) and Neuman and Witherspoon (1968) develop simplified methods of evaluating aquiclude permeabilities under field conditions (1967) and analytical expressions and type curves for drawdown in aquicludes adjacent to pumped slightly leaky aquifers (1968).

NEEDS FOR ADDITIONAL RESEARCH

The needs for additional research on the subject of aquifer-system compaction include but are not necessarily limited to the following topics.

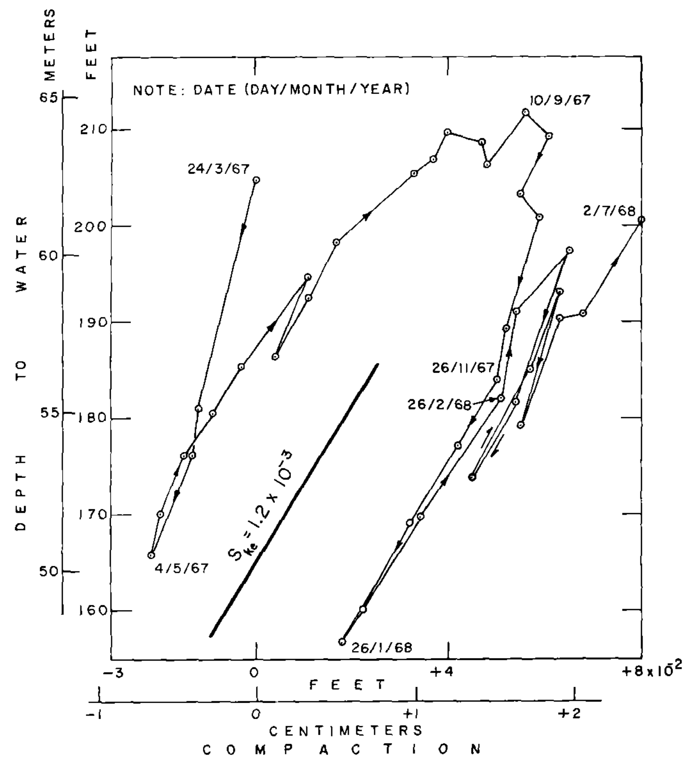

FIGURE 2. *Stress-strain-plot for well 18/19-20P2, western Fresno County California*

1. The effect of multi-cyclic loading of clays and sands within the same stress range, as compared to the usual loading technique in time-consolidation tests.
2. Research on development of field methods for determining approximate gross compressibility of an aquifer system from aquifer pumping tests, in both the elastic and plastic-plus-elastic ranges.
3. Research on improved methods for measuring compaction or expansion of aquifer systems under changing stress. Use of free pipes, as practiced in Japan, decreases the friction problem encountered with cables, but is expensive in deep wells. Potential frictionless methods include use of energy sources to measure changes in aquifer-system thickness by measuring reflection from a depth bench mark, by elapsed time, phase displacement, or other methods.
4. Additional research on vertical and horizontal strain in the vicinity of a pumped well, combined with measurements in observation wells, to explore whether direct determinations of the hydrologic or mechanical properties of the aquifer system can be derived from such data.
5. Improved methods for determining preconsolidation loads of compressible beds (aquitards) at substantial depth.
6. Improved geophysical methods for logging density, porosity, close-up and other physical properties in bore holes in unconsolidadet deposits.
7. Further development of numerical and electrical simulation methods capable of predicting compaction in heterogeneous compressible deposits.

8. Research on the importance of secondary consolidation in the ultimate compaction of fine-grained sediments. In Mexico City, secondary consolidation is of substantial importance (Zeevaert, 1957).
9. Research on the influence and importance of other internal or external stresses on aquifer systems, such as electrochemical stresses, the influence of physico-chemical factors, and the effect of bonded or adsorbed water.
10. Additional research on the relation between liquid limit and the compression index; in some places, a good correlation exists (Terzaghi and Peck, 1948, p. 66), but in others there is little correlation (Johnson and Moston, 1970). Obtaining cores and making one-dimensional consolidation tests are expensive. Samples for determining liquid limit and clay content can be obtained from water wells during drilling, and the tests are inexpensive. Therefore, in areas where these tests do furnish approximate compression characteristics of aquitards, they can be used to advantage in extending compressibility estimates areally from cored holes.
11. Research on compressibility and elasticity of sands, including effect of shape, size, sorting, mica content, and multi-cyclic loading.
12. Development of economic methods of determining pore pressures in fine-grained clayey beds *in situ*.
13. Improved methods of obtaining "undisturbed" cores.

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LAND SUBSIDENCE PROBLEMS IN TAIPEI BASIN

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ABSTRACT

Releveling of bench marks in 1968 and 1969 indicates that subsidence of the land surface in the Taipei Basin has now exceeded 1.3 meters. In the sharp subsidence area, subsidence which was as much as 0.6 meter in 1966 now has about doubled. The maximum

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rate of subsidence in recent years has been about 25 cm a year. Plots of subsidence against decline in artesian pressure suggest that pressure decline is a major cause of the subsidence. Consolidation data are used to verify the relationship between the pressure drop and the subsidence. However, other causes such as compaction of soil, tectonic adjustment and loading at land surface may have contributed to the subsidence.

RÉSUMÉ

Le renivellement de repères en 1968 et 1969 indique que l'affaissement de la surface du sol dans le Bassin de Taipei dépasse maintenant 1,30 m. Dans la zone des forts affaissements, la descente du sol qui était de 0,6 m. en 1966, a maintenant sensiblement doublé. Le taux maximum de l'affaissement dans les dernières années a été d'environ 25 cm par an. Des diagrammes de l'affaissement en fonction du déclin de la pression artésienne suggèrent que la diminution de pression est une cause majeure de l'affaissement. Des données sur la consolidation sont utilisées pour vérifier la relation entre la chute de pression et l'affaissement. Cependant d'autres causes comme la compaction du sol, des ajustements tectoniques et la surcharge de la surface du sol peuvent avoir contribué à la production des affaissements.

INTRODUCTION

For many years substantial subsidence of the land surface has been occurring at several places in Taipei Basin, figure 1. However, little attention was received until 1964 when the large scale groundwater development resulted in rapid decline of ground water level.

Information on the magnitude of the subsidence is available chiefly through first- order leveling and releveling of the Provincial Water Conservancy Bureau, with supplementary leveling available from several other agencies. The subsidence area is about 125 square kilometers, nearly half of the total basin area. Subsidence was as much as 30 centimeters in 1961 and is now tripled. The maximum rate of subsidence is recent years has been about 25 centimeters per year.

This paper presents information on the status of subsidence in the Taipei Basin, describes what is being done at the present time to determine the causes and to predict the extent, magnitude, and rate of future subsidence, and suggests additional work needed as a basis for making plans to alleviate or minimize the various problems caused by land subsidence.

GENERAL FEATURES

The Taipei Basin is located in the northern corner of Taiwan, Republic of China, bordered by Hseuling on Tahan Creek, Hsintien on Hsintien Creek and Sungshan on Keelung River, having an area of 243 square kilometers and with a general elevation of less than 9 meters above sea level (fig. 1). This basin is roughly triangular in shape, defined by the surface geology at the foothill line of the surrounding hills and in the southwest, at the line formed by the uplifted older sediments.

Taipei City is located at the eastern edge of the basin. With a population of 1,200,000 (1969), most of the basin area is extensively developed. Tamshui River drains an area of 2,726 square kilometers along a length of 195 kilometers and is the main stream system in the basin.

Wells have been used for domestic water supply in Taipei Basin since about 1895. In a field examination in 1894-1896, W.K. Bardon and others found that the area of flowing artesian wells extended south from Chinmei to almost all of the basin area. It was also known that an extensive area of free water underlaid the whole basin. However, as a result of the accelerated urbanization of the Basin, most of the modern ground water development has occurred since 1957, with pumpage tripling from 1957 to 1964. In 1964 it was about 329.1×10^6 cubic meters.

The strata exposed and the deposits containing fresh water in this area are all of unconsolidated Quaternary sediments, extending to depths from less than 1 meter to at least 240 meters below the land surface. They can be divided into three main formations: (1) an upper unit of Recent alluvium and soil, chiefly consisting of sand and gravel, that extends from the land surface to depths of about 3 meters; disseminated peat lentils are

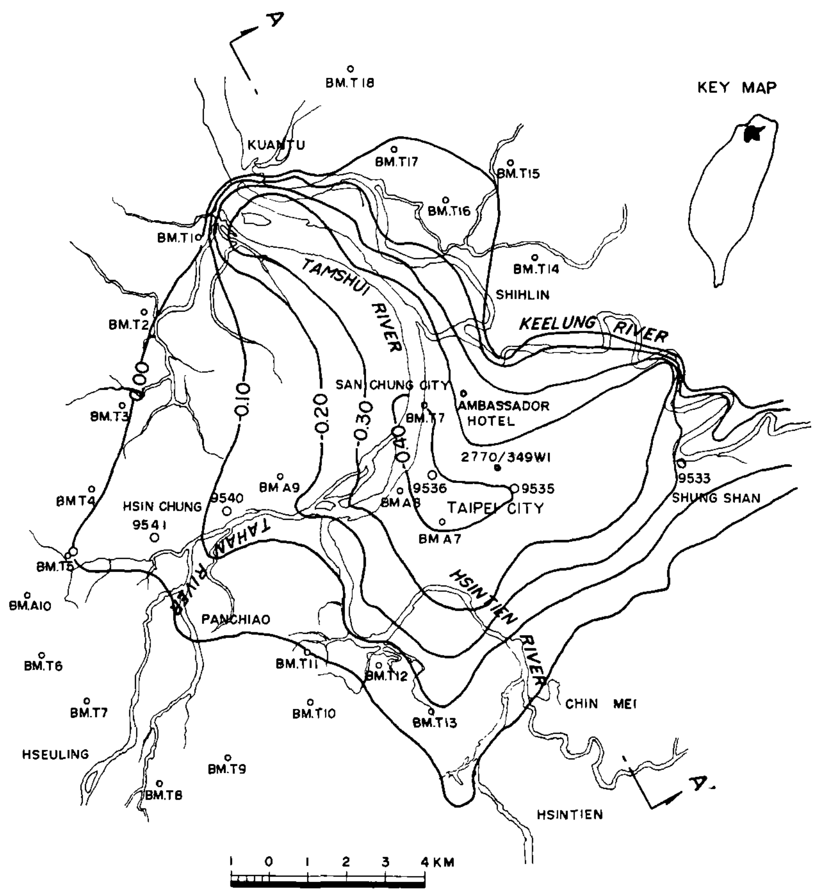

FIGURE 1. *Location map and land subsidence in the Taipei City, 1963-1967 (lines of equal subsidence in meter)*

found in its lower part in the southeastern part of the basin, (2) a middle unit, named Sungshan formation, composed mainly of gray mud and sandy mud with intercalated sand and gravel, commonly 40 to 60 meters thick, and (3) a lower unit, named Linkou formation, composed essentially of gravel and subordinately of sand and clay, with or without impervious reddish lateritic cover, 90 meters to 130 meters thick. The Linkou formation unconformably overlies the Tertiary formations. With the exception of the western part of the basin, this formation often pinches out toward the margin forming the intake area of the groundwater basin.

A body of semiconfined to free water occupies most of the middle unit (Sungshan formation); the water table is 1 meter to 10 meters below the land surface. The groundwater in the lower unit (Linkou formation), the lower water-bearing zone, is confined in

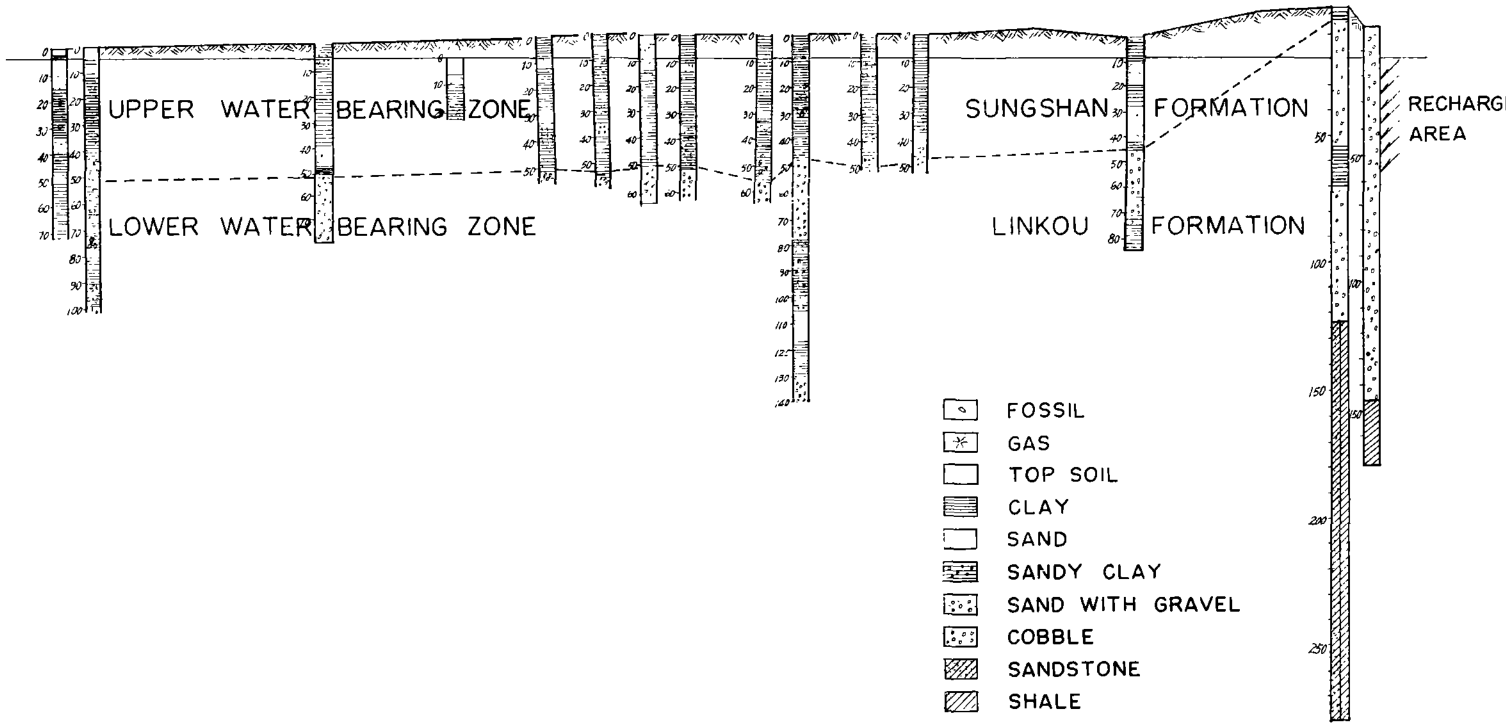

FIGURE 2. *Geological columnar section A-A′ in Taipei Basin*

most of the area by the Sungshan formation and the impervious reddish lateritic cover. Most of the pumping draft, probably 99 percent or more, is from the lower unit. The general position of the middle and lower water-bearing zones and the impervious lateritic clay that separates them, as encountered along line A-A′ (fig. 1), are shown in figure 2.

The water table in the middle water bearing zone is controlled by the fluctuation of the stream flow, but has not changed much from its initial position. On the contrary, the heavy pumping draft has decreased the pressure head in the lower water-bearing zone as much as 32 meters in the past 11 years. The decline has ranged from 25 meters near Shungshan on the northeast to roughly 32 meters near Taipei City and 5 meters on the south. Since 1964 the average rate of yearly decline has been 1 to 2 meters in the northern part and 2 to 3 meters in the southern part of the area. With this increasing rate, it is expected that in 1972 the piezometric head would probably drop below the top of the pressure aquifer.

SUBSIDENCE

Subsidence of the land surface in the Taipei City area was noted as early as 1961, when releveling of the first order bench marks established in 1950 indicated changes in altitude of several centimeters to 3 decimeters in some areas. Subsequent levelings in 1963, 1966, 1967, and 1969 indicate the maximum subsidence along the line has been 1.35 meters since 1950 at bench mark 9536, figure 1, indicating an average rate of about 7.4 centimeter per year.

Levelings by the Provincial Water Conservancy Bureau (PWCB) in 1963, 1966, and 1967 have supplied the data for the map showing subsidence from 1963 to 1967 (fig. 1). Approximately 125 sq. km is enclosed within the subsidence line. The sharp subsidence area, in which the decline exceeded 40 centimeters during 4-year period of 1963 to 1967 (fig. 1), is located in the central part of the Taipei City where sharp decline in artesian head also is noted (fig. 3).

Profiles of land subsidence and decline in artesian head of the lower water-bearing zone along the first-order leveling line of figure 1, presented on figures 4 and 5, respectively, show the relationship of subsidence to decline in head in the years 1961 to 1969. Although the ratio of subsidence to decline change along the profile, the profiles along this line indicate a rude correlation.

Figure 5 shows changes in altitude at bench marks 9536 and 9537 and change in pressure head in well 2770/349 NW 1, as well as the maximum and minimum pressure heads in nearby wells. Both the hydrograph and the maximum and minimum pressure heads show seasonal fluctuations and the long-term water-level trend since 1957 in typical wells tapping the lower confined aquifers. They show a continuing decline from 1957 to date, evidently a result of overdraft on local supplies of groundwater.

The plot of change in altitude of bench marks 9536 and 9537 (fig. 5) was included merely to show conditions in the area of greatest 18-year subsidence, but leveling to other bench marks in the vicinity suggests that the rate has been increasing in recent years, especially in the south-eastern part of the basin (fig. 1) where the development of industry has accelerated in recent years. The ratio of subsidence to pressure decline as shown for the bench marks in figure 5 for the 5-year 1962 to 1967 was 1/27. However, the corresponding ratio in the vicinity of the bench marks was 1/20 to 1/30.

CAUSES OF SUBSIDENCE

Subsidence of the land surface has been noted in many areas and ascribed to various causes. However, as was stated above, comparison of change in artesian head in confined aquifers and subsidence of the land surface shows fair correlation, suggesting that decline in artesian head has been a primary cause, if not the major cause, of subsidence in the Taipei Basin. There may be other causes, however, such as tectonic adjustment, loading at the land surface, and compaction due to irrigation. No specific measurements of subsidence of this nature are yet available to assist in evaluating its relative importance. In an effort to evaluate its importance, samples from each stratum were obtained and the rate of consolidation was computed to verify the result.

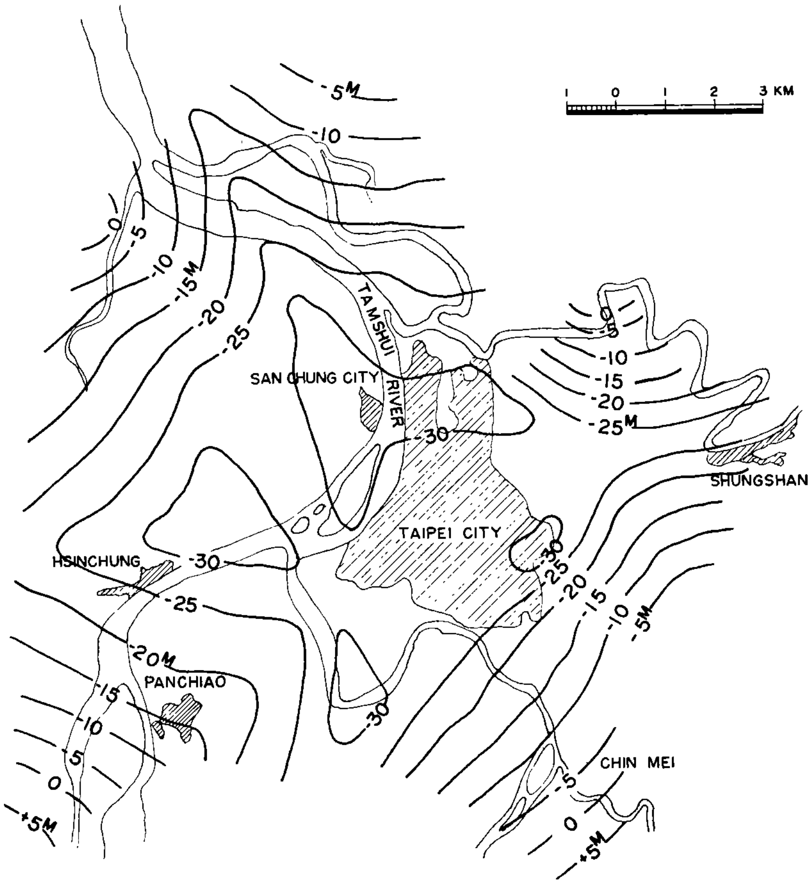

FIGURE 3. *Piezometric head in Taipei Basin, 1968*

STRATIGRAPHY AND SOIL CHARACTERISTICS

According to geohistorical study, Taipei Basin is a tectonic basin formed by the sinking of huge crust blocks by faulting [1]. The sea water and its marine organismes transgressed into the basin probably through the original water gap at Kuantu, resulting in two cycles of deposition of the Sungshan formation. As the deposition of the Sungshan formation was under a lacustrine environment characterized by shallow depth, weak current and quite water, and took place in less than 350 years, most of the subgeology is still muddy in nature. Thus, the stratigraphic study of the subgeology plays a great roll in the evaluation of the potential subsidence in this area. As the depositional environment of the lower unit, Linkou gravel, was interpreted as a transitional coastal environment, relatively consolidated in nature and comparatively deep in stratum, detailed stratigraphic study was made on the Sungshan Formation. The large amount of subsurface data and adequate well logs from more than 50 drill holes serve as a basis for stratigraphic study and subdivision of the Sungshan formation—which has two cycles of deposition and can be divided into six main members. In ascending order, the six members and their soil characteristics are described in table 1.

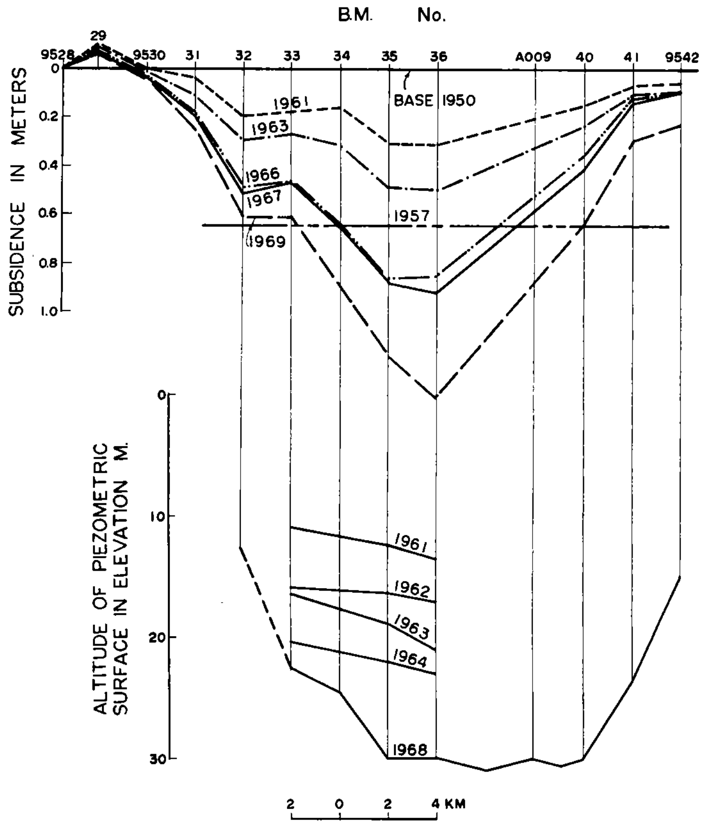

FIGURE 4. *Profiles showing land subsidence and change in artesian head along 1st order leveling line, Taipei Basin*

PREDICTION OF LAND SUBSIDENCE

It is well known that lowering of the water table within or above a stratum ultimately increases the intergranular pressure accompanied by strains in accordance with the stress-strain relationships for the material in question. The resulting displacements produce a settlement of the ground surface. The analytical methods for dealing with settlement due to this pressure must be chosen in accordance with the properties of the subsoil and the nature of stratification. The intergranular pressure, p, in a typical section bounded by a free and confined aquifer (fig. 6,) is computed as shown in table 2.

With the rate of change of piezometric head as shown on figure 4, increases in intergranular pressures in each clay stratum are computed.

Displacements due to those intergranular pressure changes are computed by using the Terzaghi theory on the one-dimensional consolidation of clay as a first approximation. Then, Mikasa's modified consolidation theory was applied for further simulation [2]. As the characteristics of clay strata in Taipei Basin are comparatively harder than those of described by Mikasa, no significant difference was noted.

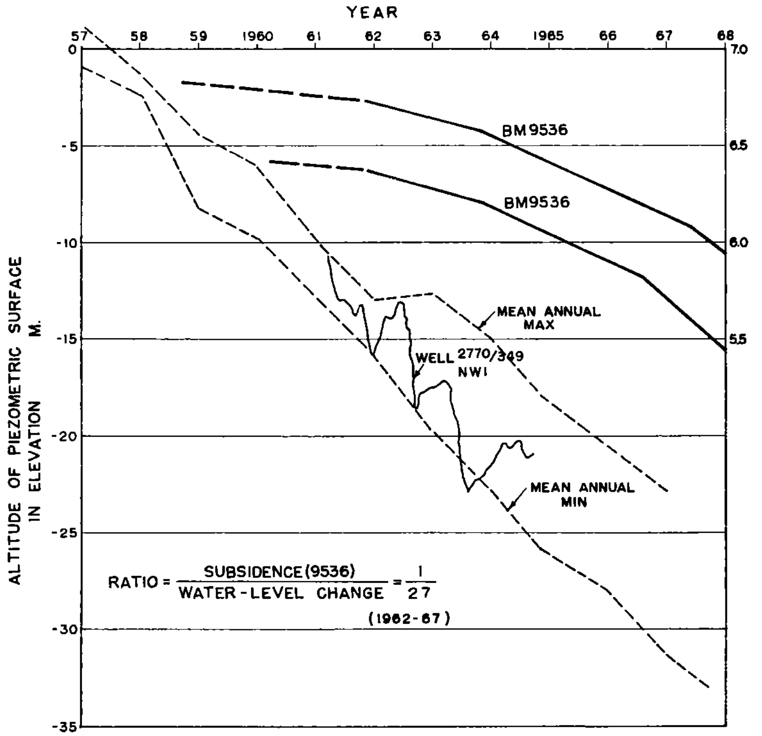

FIGURE 5. *Change in altitude at BM9536,-37 and change in artesian head in nearby wells*

Figure 7 shows the results of settlement computed by Terzaghi theory using subsurface log data at Ambassador Hotel, about 1 km north of BM 9536, as well as actual changes in altitude at BM 9536. The graphs of computed subsidence is about 75 pct less. This is expected because subsidences in the cohesiveless strata are not taken into consideration. Again, other causes such as loading at land surface, vibrations at or near land surface, compacting due to seepage water and tectonic movement might contribute appreciably to the overall land-subsidence. Computation results show that the second clay layer contributes the largest part of the soil settlement, and also show that even if the drawdown of the confined aquifer were to cease, the land surface would continue to settle for several years. Hence, on the basis of presently available data it is concluded that the extensive drawdown of the confined water and the softness of the muddy strata have been the major cause of subsidence in the Taipei Basin. Application of the laboratory physical property and field drawdown data might be used to estimate with fair accuracy the location, rate, and magnitude of future subsidence of the land surface.

TABLE 1. Stratigraphic Sequence & Soil Characteristics In Taipei Basin

Rock Unit	Lithologie & Unified Classification	Thickness m,	Size Analysis mm					Atterberg Limits		Specific Weight
			0.005	0.005~0.074	0.074 #4	# 4"~3"	3"~5"	Liquid Limit	Plastic index	
Alluvium	Sand and Gravel. CL to CL-ML, ML-OL	3								
	Unconformity ?	40~60	28~52	45~47	20~ 0	—	—	30~45	10~20	2.68~2.74
Sungshan Formation										
Upper Part:										
Sixth member	Mud; ML-OL, CL		15~45	55~45	30~10	—	—	20~50	5~20	2.60~2.67
Fifth member	Sandy Silt; SM		5~10	10~40	85~50	—	—	—	—	2.67~2.71
Fourth member	Sandy Mud; CL to CL-ML, ML-OL		10~14	35~60	35~0	—	—	20~45	5~20	2.67~2.73
Lower Part:										
Third member	Sand; SM		0~10	10~50	75~40	15~0	—	—	—	2.61~2.72
Second member	Sandy Mud; CL-CL-ML		20~45	40±	40~15	—	—	20~40	7~19	2.67~2.73
First member	Gravelly Sand; GM		0~10	0~40	60~45	4~05	—	—	—	2.71±
	Unconformity									
Linkou Formation	Gravel, GW With or Without Lateritic mantle	100~130	0	0	30~20	70~30	20~10			2.71±
	Unconformity									
Tertiary System										

TABLE 1. Stratigraphic Sequence & Soil Characteristics In Taipei Basin

Rock Unit	Lithologie & Unified Classification	Thickness m.	Field Wet Density g/cml	Field Water Content, %	Void ratio	Unconfined Compressive T/m²	Shear Strength		Bearing T/m² Stress
							C° T/m²	∮°	
Alluvium	Sand and Gravel. CL to CL-ML, ML-OL	3							
	Unconformity ?	40∼60	1.78∼1.96	25∼40	0.7∼0.8	4∼10	1.5∼2.5	18∼22	5∼15
Sungshan Formation									
Upper Part:									
Sixth member	Mud; ML-OL, CL		1.52∼1.90	20∼60	0.5∼0.7	0∼6	0.5∼2.0	15∼30	1∼5
Fifth member	Sandy Silt; SM		1.91∼2.25	15∼30	0.65		0∼1.8	25∼45	5∼20
Fourth member	Sandy Mud; CL to CL-ML, ML-OL		1.80∼2.0	20∼40	1.0±	5∼20	1.1∼3.0	15∼30	5∼10
Lower Part:									
Third member	Sand; SM		1.91∼2.25	15∼30	0.5∼0.65		0.2∼2.4	10∼40	15∼30
Second member	Sandy Mud; CL-CL-ML		1.80∼2.0	20∼40	1.0∼0.70	5∼20	1.1∼3.0	15∼30	5∼20
First member	Gravelly Sand; GM		1.91∼2.25	15∼30	—				20+
	Unconformity								
Linkou Formation	Gravel, GW With or Without Lateritic mantle	100∼130	—						50+
	Unconformity								
Tertiary System			—						

TABLE 1. Stratigraphic Sequence & Soil Characteristics In Taipei *Basin*

Rock Unit	Lithologie & Unified Classification	Thicknes m,	Consolidation Cv cm^2/sec	Pc kg/cm^2	Permea-bility $m^3/m^2/min$	Transmis-sibility $m^3/m/min$	Specific Capacity $m^3/m/min$	Clay Minerals
Alluvium	Sand and Gravel. CL to CL-ML, ML-OL	3						Well-crystallized
	Unconformity ?	40 60						ILLITE and
Sungshan Formation								CHLORITE
Upper Part:								
Sixth member	Mud; ML-OL, CL		$120\sim30$	$0.2\sim4.8$				
Fifth member	Sandy Silt; SM							〃
Fourth member	Sandy Mud; CL to CL-ML, ML-OL		$20\sim80$	$0.3\sim4.8$				〃
Lower Part:								
Third member	Sand; SM							〃
Second member	Sandy Mud; CL-CL-ML		$20\sim80$	$0.5\sim0.7$				〃
First member	Gravelly Sand; GM							〃
	Unconformity							
Linkou Formation	Gravel, GW With or Without Lateritic mantle	$100\sim130$			0.293 ~0.094	2.53 ~5.85	2.36 ~1.58	ILLITE, Well-crystallized; CHLORITE, Poorly-crystallized; mixed-layer clay
	Unconformity							
Tertiary System								

TABLE 2. Computation of intergranular pressure

Position	Total vertical pressure, p	Porewater pressure, u_w	Intergranular pressure, p
Top			
$z_0 = 0$	$h_1 r_w$	$h_1 r_w$	0
Bottom			
$z_0 = H_0$	$H_0 r_0'' + (h_1 + H_0) r_w$	$(H_0 - h_2) r_w$	$H_0 r_0 + (h_1 + h_2) r_w$

where:

r_w = unit weight of water, $r_0' = \dfrac{G_s - G_w}{f_0} r_w$

G_s = specific gravity of dry particle,
G_w = specific gravity of water,
$f_0 = 1 + e$. e = void ratio

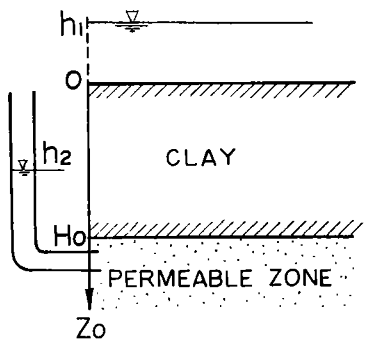

FIGURE 6. *Definition sketch*

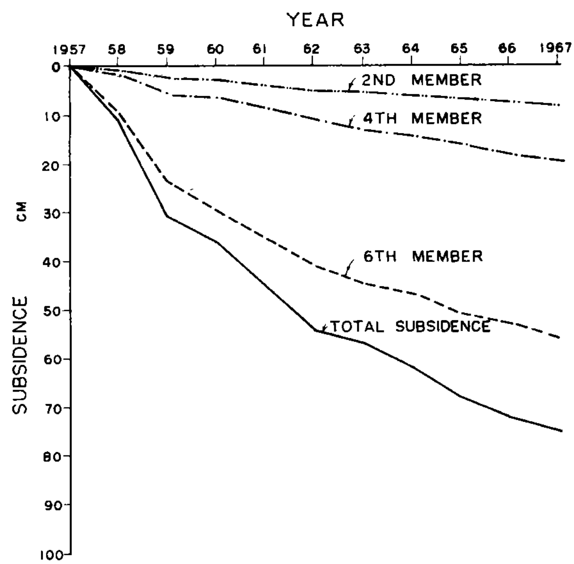

FIGURE 7. *Computed subsidence, Ambassador Hotel, Taipei Basin*

PROBLEM CAUSED BY THE SUBSIDENCE

The subsidence of the land surface in Taipei Basin poses serious problems in connection with maintenance of limited flood control works and construction of proposed additional levee works, as well as in construction and maintenance of other engineering structures such as pipeline, drainage, sewerage, power, and water-supply systems, highways, railroads buildings, and in various aspects of land development and use. Again, because the deformation and failure of water-well casings and the resulting loss of wells are a common occurrence in the Taipei Basin, continued ground water withdrawal or expanded development becomes critical in the region.

SOLUTION AND STUDIES NOW BEING MADE

As an immediate action to alleviate the problems caused by land subsidence and groundwater over-draft, a groundwater development management Code has been adopted. However, detailed knowledge concerning the extent, magnitude, rate, and causes of land surface subsidence is essential to proper planning for repair, maintenance, and construction of structures, as well as for continued ground water withdrawal and development, and for most economical land use. This can be done only by adequate investigation of the area in which subsidence has occurred to date.

Basic data are necessary for appraisal of a groundwater basin. One of the most difficult problems in basin appraisal in Taipei Basin will be the establishment of a criteria for safe yield. Two particular dangers exist for most Taiwan groundwater basins: (1) intrusion of salt water and (2) consolidation due to depletion of hydrostatic pressures in both free and confined aquifers. Both of these factors required a minimum flow through and disposal of water to sea. Enough data are not available on the magnitude of this required minimum flow. Studies of the mean hydraulic gradient of the ground water and the amount of electrical energy consumed during the year lead to an estimate of safe yield. Figure 8 shows the result of the safe yield study. As can be seen, after a period of significant drawdown, the linear relation between average annual draft and average annual change in ground water level was no longer in existence even though the moving average method was adopted. The safe yield thus obtained is 150×10^6cubic meters per year. Likewise, data on recharge are meager but they may be estimated by the hydrologic equation. The mean annual rainfall in this area is 2810 mm, equivalent to 7530×10^6 m^3 per year. Analysis of base flow data shows abundant water is available for recharging the ground water basin. The estimated base flow amounted to $1{,}410 \times 10^6$ m^3 per year. Though 170×10^6 m^3 is considered to be contributed to ground water, its contribution to the free aquifer and confined aquifer is not well known. Again, other times items included in the hydrologic equation are often more difficult to evaluate, even qualitatively, than the full hydrologic equation. Evapotranspiration, for example, is not measured. The average rate of evapotranspiration as computed by the Thornthwaite formula is 587 mm ($1{,}600 \times 10^6$ m^3); whereas by the Blaney-Criddle method it ranges from 350 mm to 400mm per year (940-1,080 $\times 10^6$ m^3 per year). Hence, the probable error of the computed ground water inflow and outflow is affected by the evapotranspiration. A continuing inventory program is now being carried on, so as to bridge the gap and to achieve an optimum development of the ground water resources in this basin.

Recently, stabilization of the subsoil by quicklime piles has been one of the most successful measures in the Taipei Basin. Test results showed that use of quicklime piles induced reduction in water content and void ratio of the soil, and consequently increased the apparent angles of shearing resistence and improved the bearing capacity of the soft material [3].

ADDITIONAL WORK NEEDED

Though some study has been made of the land subsidence in Taipei Basin, the results are not extensive enough to be conclusive. Additional work, such as (1) the establishment and maintenance of adequate verticals, both at surface and at selected depth intervals in wells, (2) adequate programs of water level measurement and inventory of groundwater draft, and (3) proper sampling and testing of subsoil, are badly needed.

Recently, land subsidence has been of widespread interest in the field of engineering, agriculture, geology, soil science, meteorology, hydrology and economics. Cooperation of the various professional fields is badly needed for the further investigation of the problem.

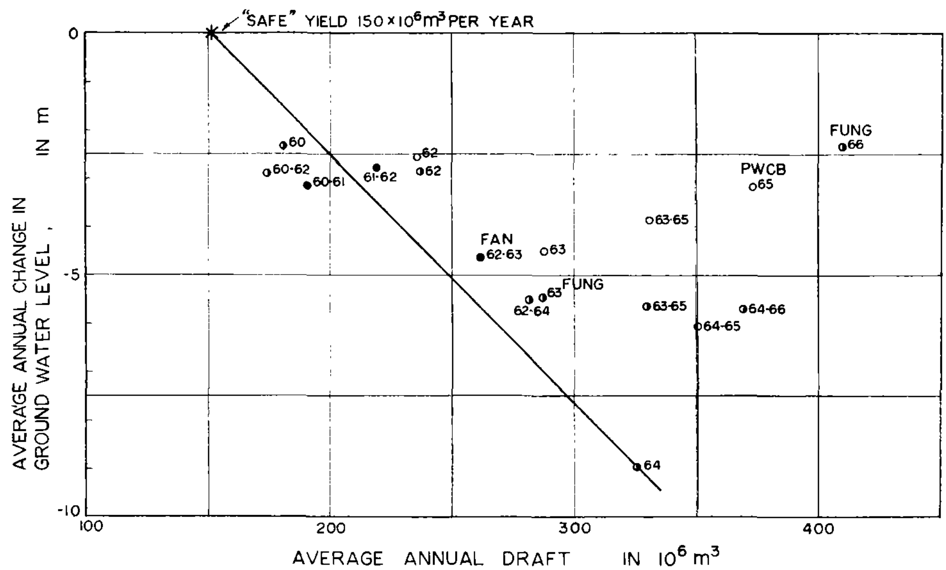

FIGURE 8. *"Direct" determination of ground water basin yield, Taipei Basin*

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SUBSIDENCE IN THE NORTH GERMAN COASTAL REGION

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ABSTRACT

We distinguish wide subsidences, settlements and lowerings.

Subsidences are tectonic processes, witk could not yet be shown in the north coastal region by repeated "Feinnivellements". The noted changes of niveau can now only be explained by rising of the water level.

Settlement are found in limited regions, in which the relation between water and soil has been disturbed by the withdrawal of water.

Lowerings originate from soil, regional limited, which is compressed and condensed by pressure.

Our examination is rectricted to settlements in "holozäne" sea and river marshes, which are blocked by dikes against floods with salty sea and brakish river water. The soil of the sea marshes mainly consists of pure sand with more or less organic components (silt). In river marshes organic components are dominate (moor).

These days the demands of intensive utilization of the lower grounds has caused a deeper withdrawal of water, that is obtained by the sum of withdrawing water (bigger ditches, drainage sluice, and pumping stations). The waterlevel will be held lower in relation to the land surface and absolutely lower than necessary for extensive utilization.

Fixed pipe supports and lowering plates are used to measure settlements. Fixed pipe supports are built through the stratum of "holozäne" just into the "pleistozäne" by taking care of certain demands for the purpose of avoiding them to take part in the settlement.

Plates are located 25 cm under the surface.

Each year the height of the fixed pipe supports and plates is controlled and determinated by "Feinnivellements". In diagrams of time and position the change at height will be computed. These measurements are completed by observing the ground water.

RÉSUMÉ

Nous distinguerons les larges affaissements, les tassements et les dépressions.

Les affaissements sont dus à des processus tectoniques qui ne peuvent pas être montrés maintenant dans la région côtière allemande par des nivellements répétés. Les changements de niveau considérés ne peuvent être expliqués actuellement que par la montée du niveau d'eau.

Les tassements se rencontrent dans des régions limitées dans lesquelles les relations entre l'eau et le sol ont été perturbées par les pompages.

Les dépressions ont leur origine dans le sol régionalement limité, qui est comprimé et condensé par la pression.

Notre étude est réduite aux tassements dans les marais fluviaux et maritimes qui sont protégés par des digues contre les crues d'eau de mer salée ou d'eau de rivière saumâtre. Le sol des marais maritimes consiste surtout en sable pur avec plus ou moins de matières organiques (vase). Dans les marais fluviaux les matières organiques ont la prépondérance.

A l'heure présente, l'utilisation intensive des sols bas a provoqué un retrait considérable de l'eau obtenu par des dispositifs d'évacuation des eaux (fossés plus larges, écluses de drainage, stations de pompage).

Des supports fixes de canalisation et des plaques sont utilisés pour la mesure des tassements. Les premiers traversent l'holocène jusque dans le pleistocène en prenant certaines précautions pour qu'ils ne prennent pas part au tassement.

Les plaques sont placées 25 cm sous la surface.

Des nivellements de précision déterminent chaque année la cote des repères. Des diagrammes présentent les résultats des mesures qui sont complétés par des observations sur l'eau souterraine.

GENERAL REMARKS

We distinguish between subsidences, settlements and lowerings.

Subsidence is a tectonic phenomenon involving comparatively large areas. The possibility of such a subsidence in this region was investigated by comparing two North Sea coastal levellings, the first made in the years 1928-1931 [9] and the second between 1948-1955. The base lines were arranged on high and dry land upon the pleistocene "Geest". Five subsoil points each about 2½ meters below the German land level (N.N.) were considered and the levels related to a bench mark at a point on a large diluvial sand area nearly 150 kilometers distant from the coast.

Comparisons of these two surveys indicated that the observed changes in the relative elevation of water and land could not be explained by subsidence of the coastal region and must therefore be explained by a rise in water level.

Settlements are of a more limited extent than subsidences and are due to a disturbance of the relationship between the soil and water [6] due to de-watering. On moorland, bor or peat subsoil the settlement may be very great. Such strata compressed by the dead weight of the soil above may lose substance by chemical means [3].

In the years immediately following poldering the volume of fresh marsh clay will be reduced by de-watering, de-salination, de-liming and by the deprivation of nutritive material. In comparison with this loss the loss of mass in old marshes is slight. The groundwater level is high and the loss of soluble salts occurs only in the upper two meters. Borings have shown that deeper clay strata are firm and tight.
Thus large settlements in these circumstances are not possible. Thus tectonic subsidence and regional settlements can be measured and explained [10] [11].

Recent intensive cultivation of the lowlands involving bigger ditches, drains, sluices and pumping works have caused a deeper de-watering in these areas. Currently the water level is held lower in relation to the land surface and absolutely lower than formerly.

Lowerings are of even more limited extent. The soil is compressed and condensed by artifical loads such as buildings, streets, dams, dykes, sluices etc. [2] [5]

SETTLEMENT AREAS AND MEASURING METHODS

Our examination was restricted to settlements in "holozäne" seamarches and river marches protected by dykes against floods of sea water and brackish river water.

Figure 1 shows the regions involved between the North Sea, the Baltic Sea and the mouth of the river Elbe. The surface of these lowlands is generally below NN + 2.50 m but goes down to −2.50 m in the river marshes [8].

The following two representative examples serve to explain the investigations carried out.

Seamarsh [1] with relatively young sandy-clay soil, poldered since 1925 and currently de-watered by sluices.

River Marsh [2] with relatively high parts on bog, moor, peat and clay subsoil,poldered since the middle ages and because of the low position, de-watered by pumping works [4].

Depending on different soil structures there are many different methods suitable for observing settlement. In sector 1, fixed pipes were driven through the stratum of holozäne and were surrounded by a group of three plate measuring stations located in the highest strata 25 centimeters below the land surface.

In the second sector (river marsh) the geological structure is much less regular. Therefore, an extensive system of sixteen settlement measuring stations was chosen. Three of these (A, B and C) are discussed in this paper.

The soil profiles belonging to the fixed pipe supports shows the geological section (Wilster march 2).

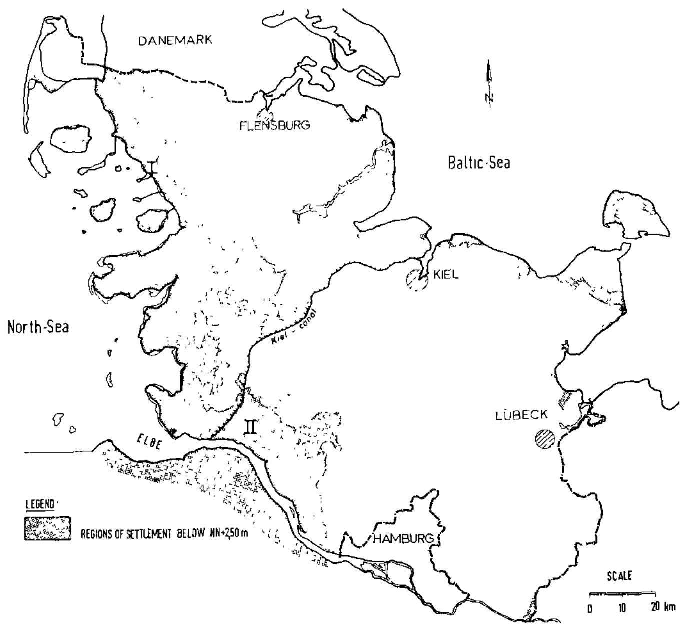

FIGURE 1.

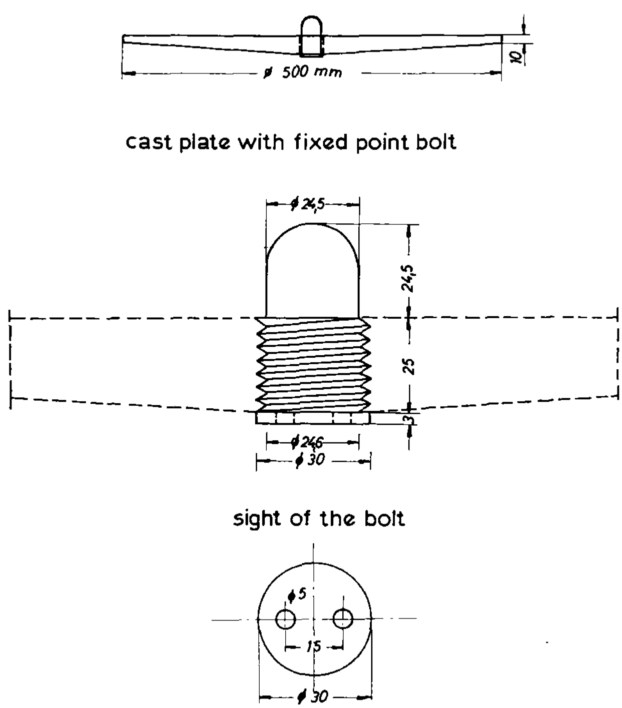

FIGURE 2. *Fixed pipe support plate for level of settlement*

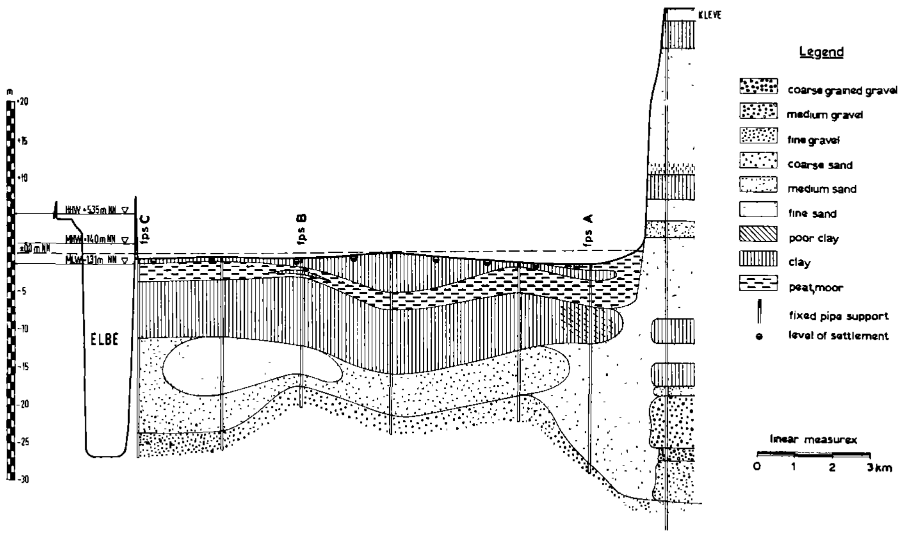

FIGURE 3. *Geology section Wilster marsh II*

FIXED PIPE SUPPORTS

Settlement is indicated by a change in the relative position of the pipe and plate [7]. After the Oldenburg and Hamburg tests the construction of these instruments was improved as follows.

1. The pipes now penetrate 5 m into the pleistocene.
2. After reaching a certain depth the pipe is rammed until the resulting penetration becomes nearly zero.

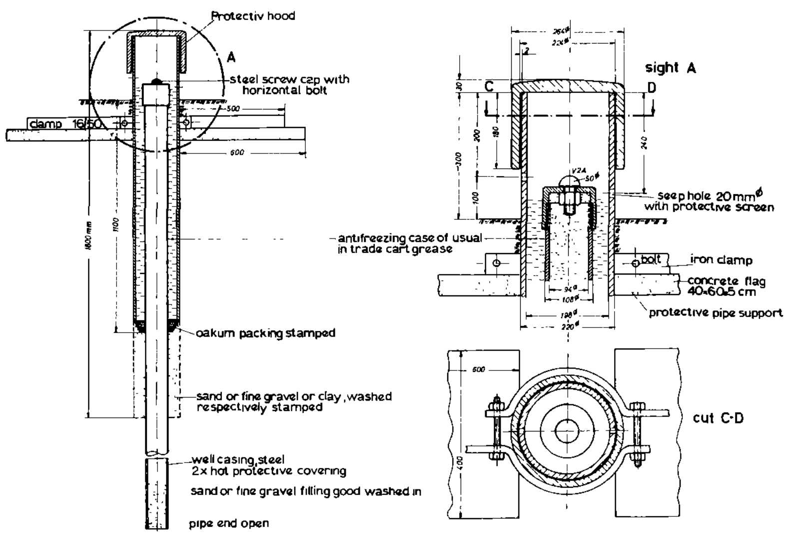

FIGURE 4. *Fixed pipe support*

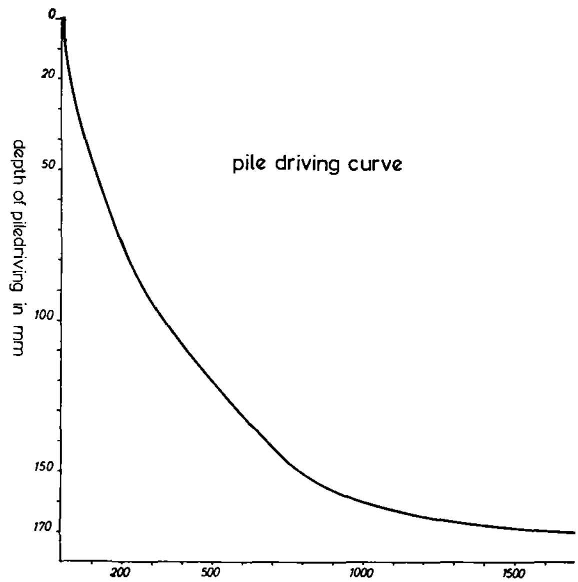

FIGURE 5. *Fixed pipe support A*

Figure 5 shows the relationship between penetration and ramming for the point A. After 1,000 ram-beats the pile penetrated only 160 mm's. On the other hand the pile in point B penetrated 2,000 mm's after 1,000 ram-beats (fig. 6).

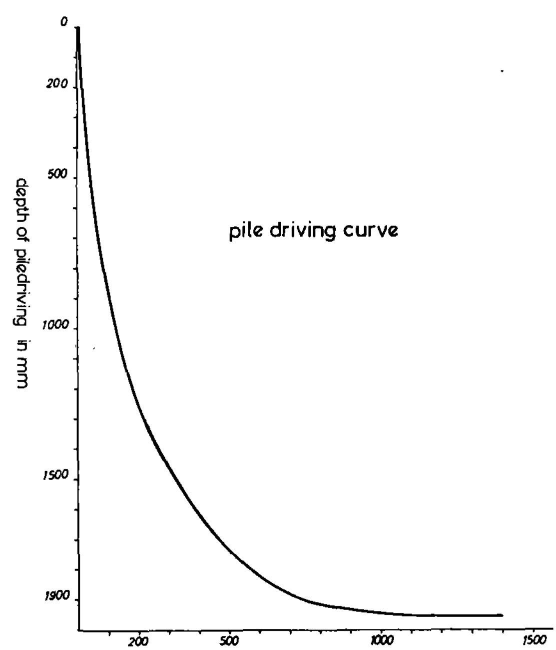

FIGURE 6. *Fixed pipe support B*

In both cases the firmness of the fixed pipe has been assured.

3. A winter proof mantle has been installed against the elements

GROUND WATER LEVELS

The region River marsh 2 may be used to explain the different behaviours of the ground water levels.

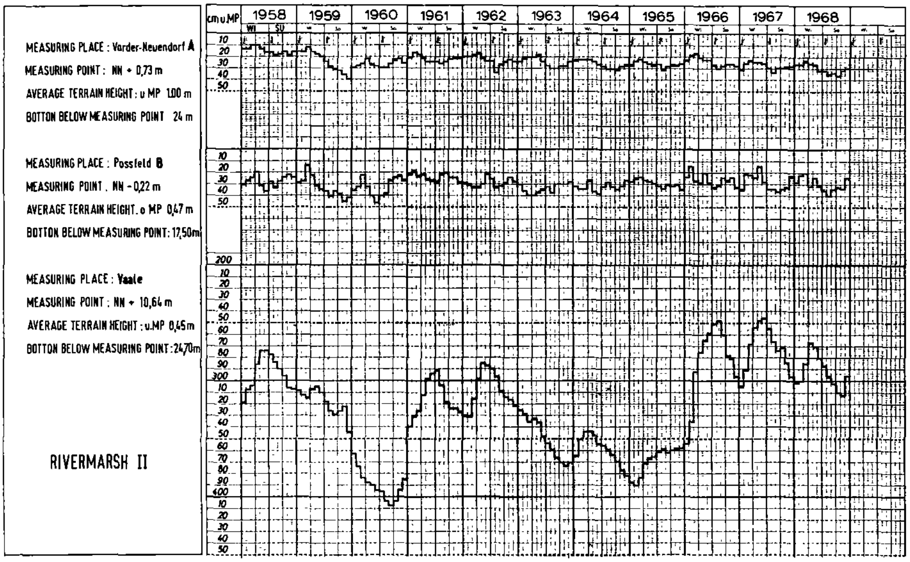

FIGURE 7. *Ground water hydrograph*

The measuring places A (Vorder-Neuendorf) and B (Possfeld) can be represented by the settlement measuring stations A and B. Only the ground water measuring place, Vaale, is located at the limit of the marsh—" Geest ", nearly 4.5 km north-east from the measuring station C (Vaalermoor).

The ground water hydrograph indicates the climatic changes during the observation period: falling water level during the dry years 1955-1960, rising water levels in humid years when the soil is filling with water.

EVALUATION

Figure 8 indicates the process of settlement in the seamarsh.

In the young polder settlement is active. Later the curves flatten to indicate the slower settlement rate in the older polders. In the period immediately following the construction of a new dyke influencing the soil water relationship the newly poldered seamarsh will quickly settle.

On the other hand in the very young and as yet unpoldered land in front of the dyke the plates move relative to the fixed pipe to the rythm of the tides, but without any general trend.

The river marsh polder, de-watered centuries ago, shows little change of settlement rate (A B).

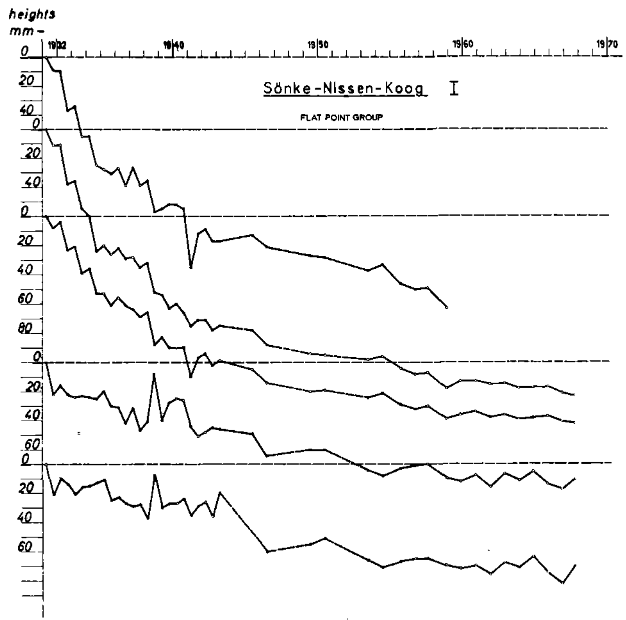

FIGURE 8. *Time-settlement-curves*

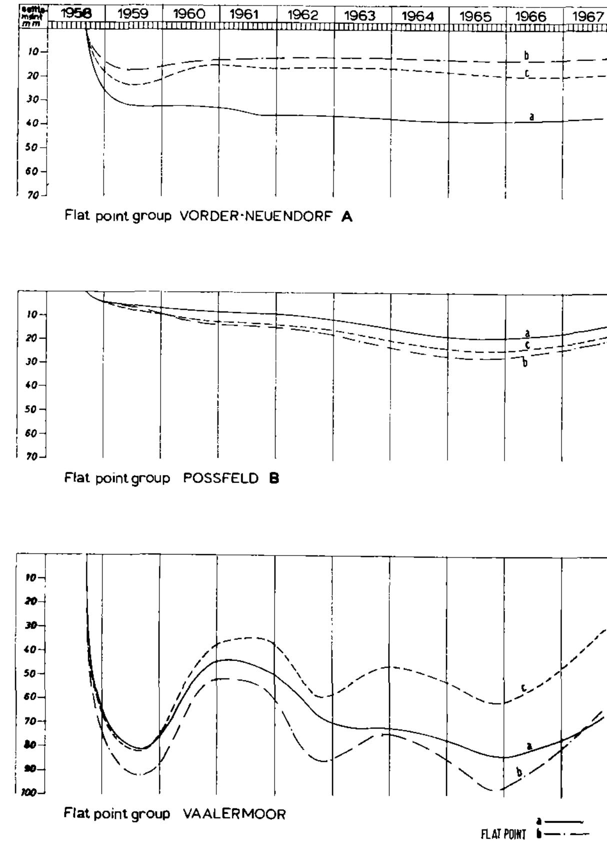

FIGURE 9. *Time-settlement river marsh II*

At all measuring places a large settlement of the plates relative to the pipe was observed immediately after installation. This phenomenon is no doubt connected with extreme dry year 1959.

How far the dead weight of the plates installed on soil more or less broken up, influences the shape of the time settlement curves cannot be assessed at this time.

SUMMARY

Our examination was restricted to settlements in holozäne sea-and river-marshes blocked by dykes against floods of sea water and brackish river water. The soils of the sea marshes consist mainly of pure sand and more or less organic components (silt and clay). In the river marshes organic components (bog and peat) predominate.

Fixed pipe supports and plates free to move with the subsidence are used to measure settlements. The former are inserted through the holozäne stratum into the pleistocene, care being taken to avoid the pipes undergoing settlement. Near the fixed pipes the plates are located 25 centimeters below the land surface.

Each year the height of the fixed pipe and the relative level of the plates is determined by "Feinnivellements". Graphs of the Relative height with time indicate the progress of the settlement. These measurements are completed by observing the ground water level.

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LAND-SURFACE SUBSIDENCE IN THE HOUSTON-GALVESTON REGION, TEXAS[1]

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ABSTRACT

In the Houston-Galveston region of Texas, the principal cause of land-surface subsidence is the lowering of pressure heads due to the removal of water and oil from subsurface strata. This paper emphasizes the effects of removal of water.

The region is underlain by a thick section of unconsolidated lenticular deposits of sand and clay. Clays separating beds of sand retard the vertical movement of water, thus creating artesian conditions within the aquifers. The ratio of sand to clay, which is a major factor controlling the degree of compaction, varies from place to place in the aquifers. Reduction of pressure due to withdrawal of water causes additional load to be transferred to the skeleton of the aquifer system, thus causing compaction.

As much as 5 feet of subsidence has occurred in the Houston-Galveston region between 1943 and 1964, and as much as 200 feet of water-level decline has occurred during the same period. The rate of subsidence increased from about 0.2 foot per year during the 1954-59 period to about 0.24 foot per year during the 1959-64 period. The decline in water levels increased from about 4 feet per year to about 7 feet per year in those same periods.

RÉSUMÉ

Dans la région de Houston-Galveston, Texas, la raison principale de l'affaissement du terrain naturel est la réduction de la hauteur piézométrique dans les couches souterraines du fait du pompage de puits à pétrole, et de puits à eau. On examine en particulier ici les résultats de l'extraction de l'eau souterraine.

A partir du terrain naturel on rencontre successivement d'innombrables couches ou des lentilles de sable et d'argile. Les couches sont discordantes, d'épaisseur variable, et plus ou moins compactes. L'inconvénient majeur pour le débit de l'eau et pour la perméabilité en direction verticale est constitué par les couches d'argile. Dans les couches de sable se trouvent des nappes d'eau captives. La distribution quantitative des sables et des sables argileux est un des facteurs très importants, qui peut influencer sérieusement le degré de compactage des couches. La réduction de la pression hydrostatique dans les couches sablonneuses par le pompage de l'eau souterraine, est telle que la charge et la contrainte à la compression sur les couches est cause de compaction et d'affaissement sensibles.

INTRODUCTION

The phenomenon of land-surface subsidence has been attributed(Poland and Davis, 1956, pp. 294-295) to: (1) Loading of the land surface, (2) vibrations at or near the land surface, (3) compaction due to irrigation, (4) solution due to irrigation, (5) drying and shrinkage of deposits, (6) oxidation of organic materials, (7) decline of the water table, (8) decline of artesian pressure in water sands, (9) decline of pressure in oil zones due to the removal of oil and gas, and (10) tectonic movements.

In the Houston-Galveston region, the principal cause of subsidence is the lowering of pressure heads due to the removal of water and oil. The other possible causes are either insignificant or ineffective.

The Houston-Galveston region, as described in this report, includes all of Harris and Galveston Counties, and parts of Brazoria, Fort Bend, Waller, Montgomery, Liberty and Chambers Counties.

1. Publication authorized by the Director, U.S. Geological Survey.
2. Hydrologist; Houston, Texas.

GEOLOGIC AND HYDROLOGIC CONDITIONS

The Houston-Galveston region is underlain by a thick section of unconsolidated lenticular deposits of sand and clay. Pertinent geologic formations in this section are, from oldest to youngest: Fleming Formation of Miocene age, Goliad Sand of Pliocene age, Willis Sand of Pliocene (?) age, and Lissie Formation and Beaumont Clay of Pleistocene age. The formations crop out in bands roughly parallel to the coast and dip toward the coast at an angle greater than the slope of the land surface.

The geologic formations compose the principal aquifers of the region, the Evangeline aquifer and the Chicot aquifer. Within these aquifers, the interbedded sands and clays are saturated with water almost to the land surface, but the clays retard the vertical movement of water, creating artesian conditions within the aquifers.

Withdrawal of water from the artesian aquifers results in an immediate decrease in hydraulic pressure, which partially supports the weight of the overburden. With reduction in pressure, an additional load is transferred to the skeleton of the aquifers, and a pressure difference between the sands and clays causes water to move from the clays to the sands. Most of this process of the sediment compaction takes place in the clays. Because the clays are mostly inelastic, the compaction is permanent.

The ratio of sand to clay, which is a major factor controlling the degree of compaction, varies from place to place in the aquifers. At the western edge of the region, the aquifers contain from 60 to 70 percent sand and from 30 to 40 percent clay. The percentage of clay increases downdip. Near the southern part of Harris County and the northern part of Galveston County, from 50 to 60 percent of the sediment is clay.

The clay-mineral assemblage – montmorillonite, illite, and kaolinite – in the Houston-Galveston region, is similar to the assemblages found in the two major areas of subsidence in the San Joaquin Valley of California. Montmorillonite is the major clay-mineral constituent of the material finer than 2 microns, making up at least half of the assemblages in all samples examined.

PUMPAGE AND DECLINE OF WATER LEVELS

Prio to 1954, nearly all water supplies were obtained from ground water. In 1954, water from Lake Houston on the San Jacinto River became available fort part of the industrial and municipal needs. The use of surface water temporarily lessened the ground-water draft, but the increase in water use and greater demands on the ground-water supply have required the construction of additional wells.

The ground-water draft in 1964 was 411 mgd (million gallons per day) compared to 354 mgd in 1953. Thus the Houston-Galveston region used 57 mgd more ground water in 1964 than in 1953 even with tha added Lake Houston supply of about 100 mgd. Pumpage by the city of Houston for public supply alone increases at the rate of about 5 mgd per year.

Ground-water is being extracted in the six major areas shown in figure 1. The average daily ground-water pumpage in 1964 in each area was as follows:

Pumpage area	*Average daily pumpage (million gallons per day)*
Houston	132
Pasadena	90
Baytown–La Porte	26
Texas City	11
Alta Loma	11
Katy	141
Total	411

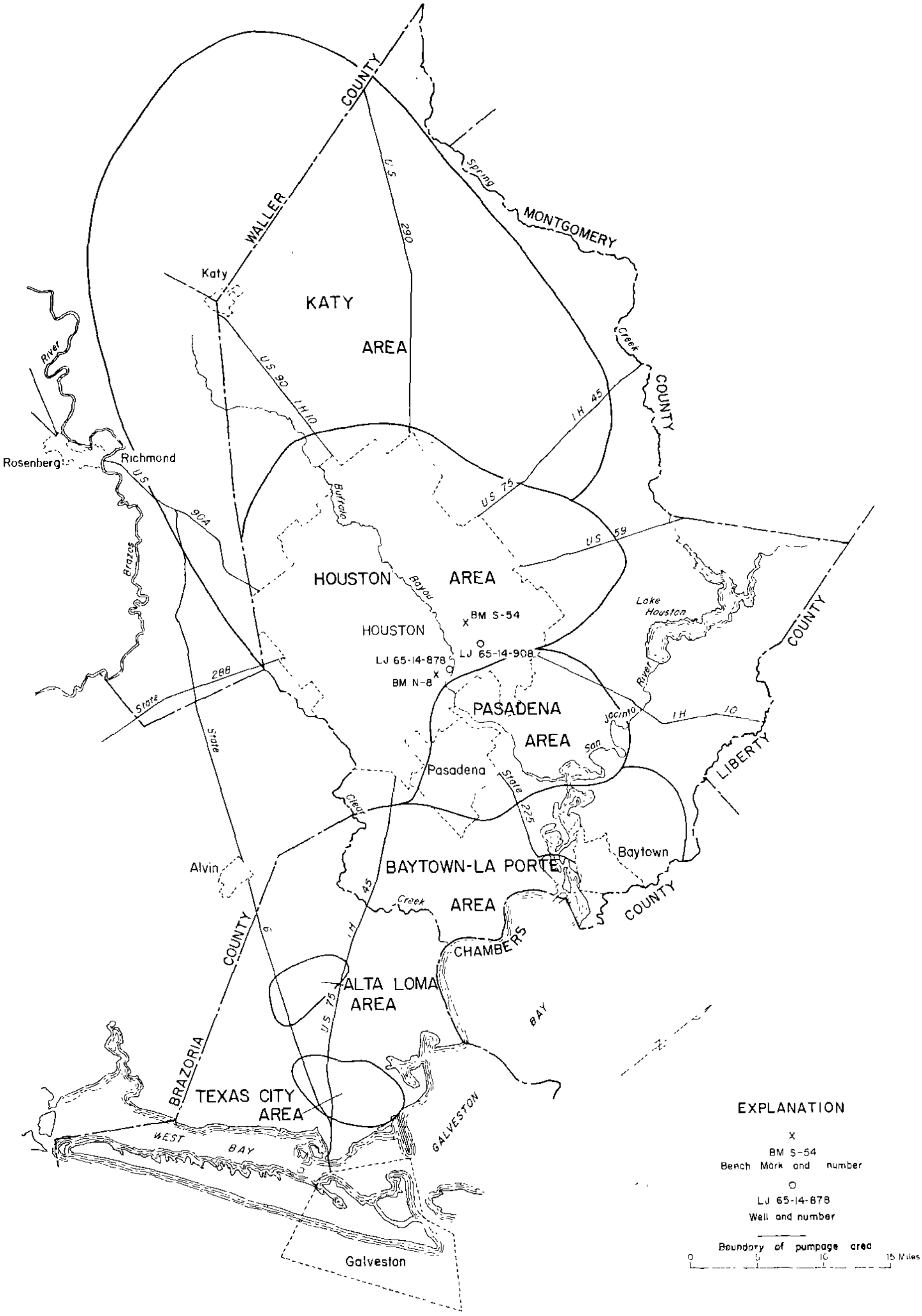

FIGURE 1. *Location of major pumpage areas*

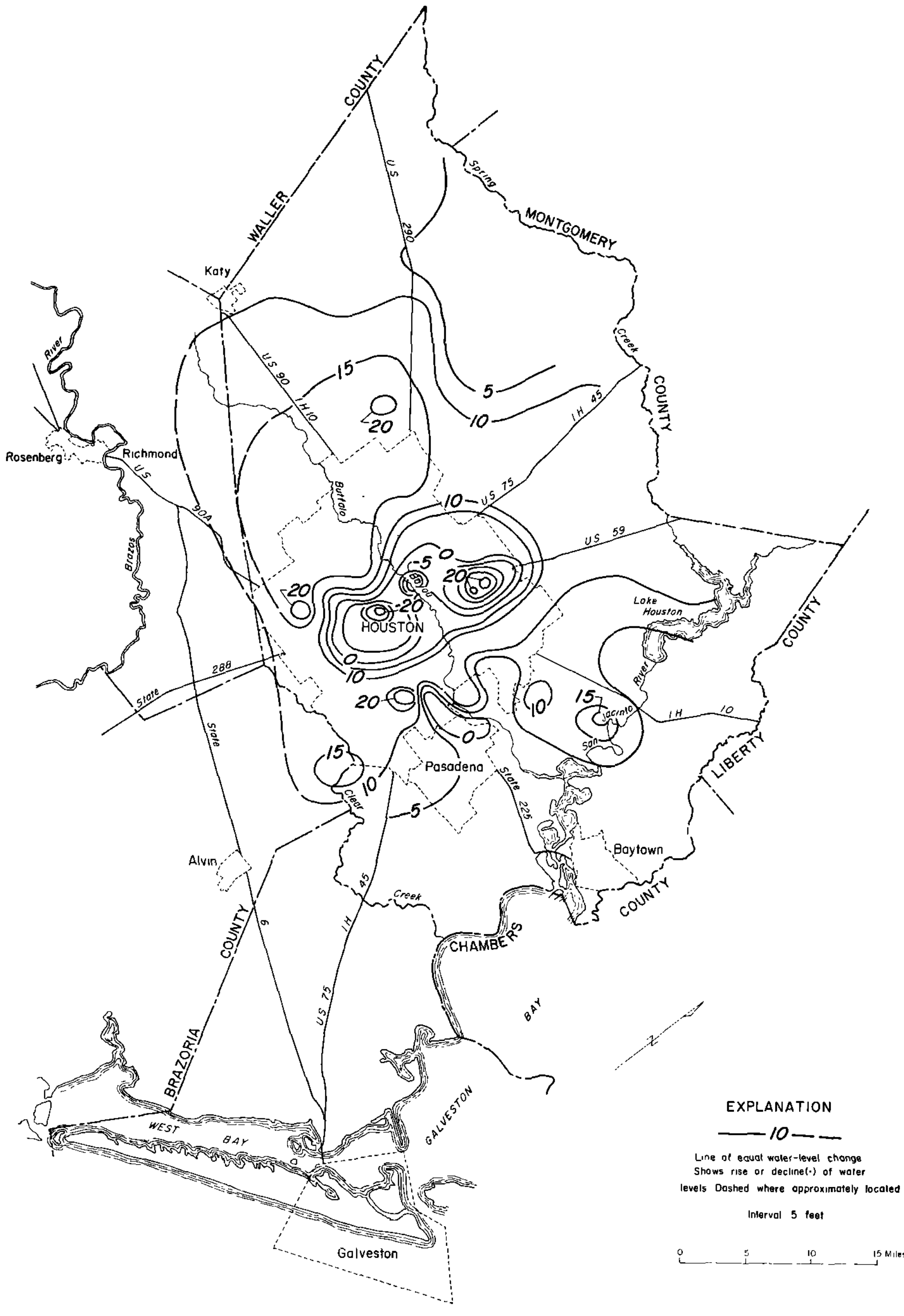

FIGURE 2. *Change of water levels, 1954-59*

Figures 2, 3, and 4 show the changes in water levels in wells tapping the heavily pumped sands in the Houston-Galveston region for the periods 1954-59, 1959-64, and 1963-64, respectively. These periods correspond to the periods of relevelling of lines of bench marks by the U.S. Coast and Geodetic Survey. In the Pasadena area, where ground-water withdrawals are heavily concentrated, the decline of water levels in wells has been about 200 feet during the period 1943-64.

RATE AND EXTENT OF SUBSIDENCE

Figures 5, 6, and 7, show the amount of subsidence in the Houston-Galveston region for the period 1954-59, 1959-64, and 1943-64. The maps are based on the results of the U.S. Coast and Geodetic Survey levelling program, supplemented by data from local industries.

A comparison of the water-level and subsidence maps shows a close correlation between water-level or pressure-head declines and land-surface subsidence. The maximum rate of subsidence increased from about 0.2 foot per year during the period 1954-59 to about 0.24 foot per year during the period 1959-64. The decline map (fig. 2) for the period 1954-59 shows much smaller declines (and recovery) than does the decline map for the succeeding five years (fig. 3).

Because of a lag between lowering of the piezometric head and compaction, correlation between subsidence and pressure-head decline may not be obvious for short periods of time. This lag is illustrated by the graphs in figure 8 which show declines in water levels in wells compared to subsidence of the land surface. The graphs show that the rate of decline in water levels decreased from 1954 to 1959, but the rate of subsidence did not decrease proportionately during the same period.

Figure 8 shows, since 1943, about 1.3 feet of subsidence for bench mark N-8 per 100 feet of water-level decline in well LJ-65-14-878 nearest the city of Pasadena and about 1.1 feet of subsidence for bench mark S-54 per 100 feet of water-level decline in well LJ-65-14-908 in the area farther northwest of Pasadena. This difference in the relationship between subsidence and pressure decline is due to the amount of clay present in the interval affected by the decline in pressure; the greater the percentage of clay, the greater the amount of subsidence.

Figure 9 shows the general relationship in the Houston-Galveston region between the percentage of clay and the amount of subsidence due to pressure-head decline. The percentage of clay was determined from interpretation of electrical logs; the pressure-head decline was determined from measured water levels in wells; and subsidence values were taken from changes in nearby bench-mark elevations.

Records from compaction recorders in the Houston-Galveston region are in-sufficient to relate compaction to depth. Most of the compaction probably is occuring near surface because near-surface clays have been subjected to less overburden than deeper clay. In a test in California, Poland and Ireland (1965) found that only about 0.01 feet of a total of 1.2 feet of compaction occured below a depth of 1,930 feet.

EFFECTS OF SUBSIDENCE

The detrimental effects of land-surface subsidence are: (1) Structural damage, probably due to faulting, that has cracked buildings and disrupted pavements; (2) damage to well casings; and (3) submergence of coastal lowlands.

Winslow and Wood (1959) suggested two beneficial effects of subsidence: (1) About one-fifth of the water pumped from wells in the region has come from compaction of clay, and (2) subsidence has deepened the Houston Ship Channel and thus reduced the amount of dredging required to keep the channel at the required depth.

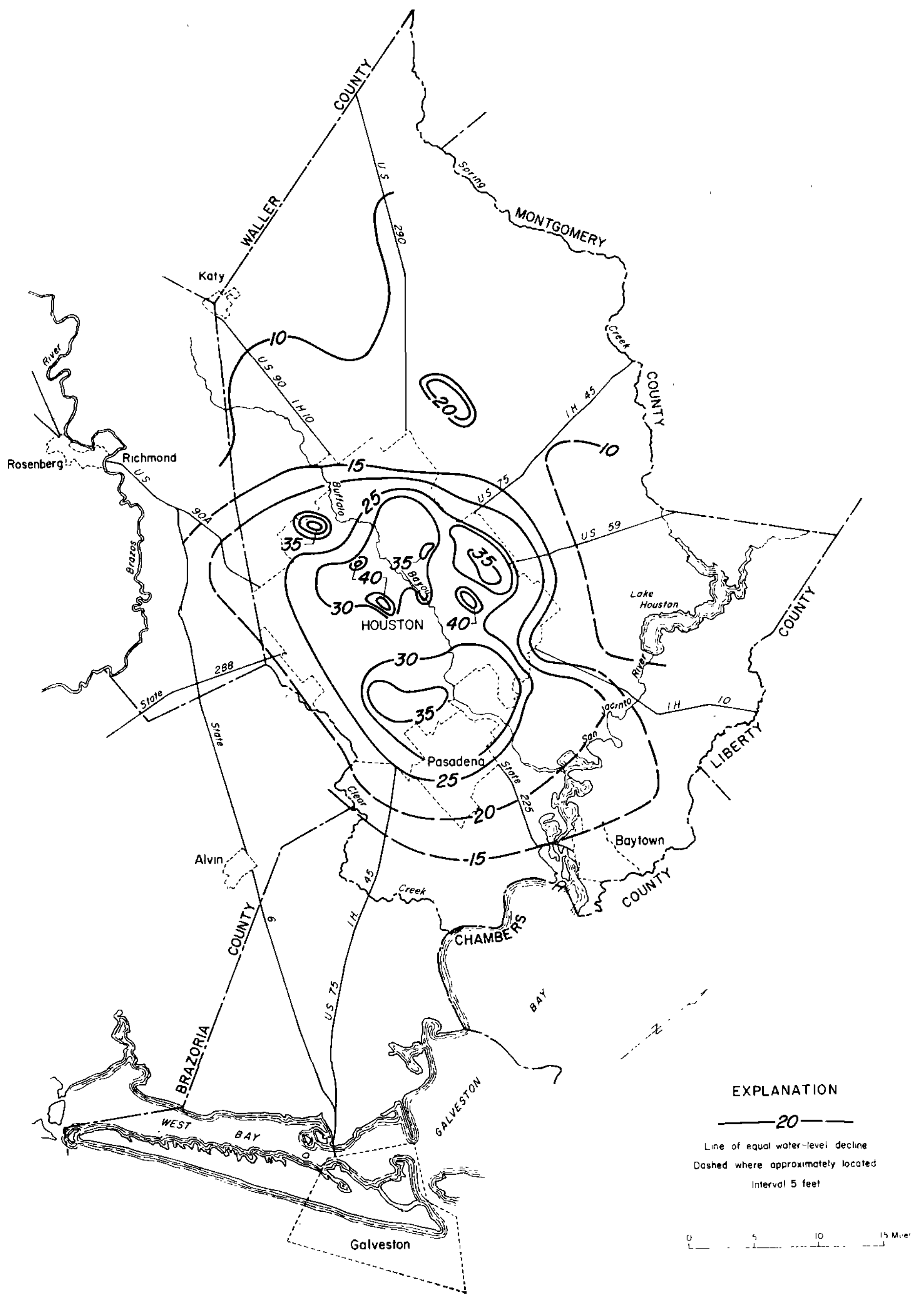

FIGURE 3. *Decline of water levels, 1959-64*

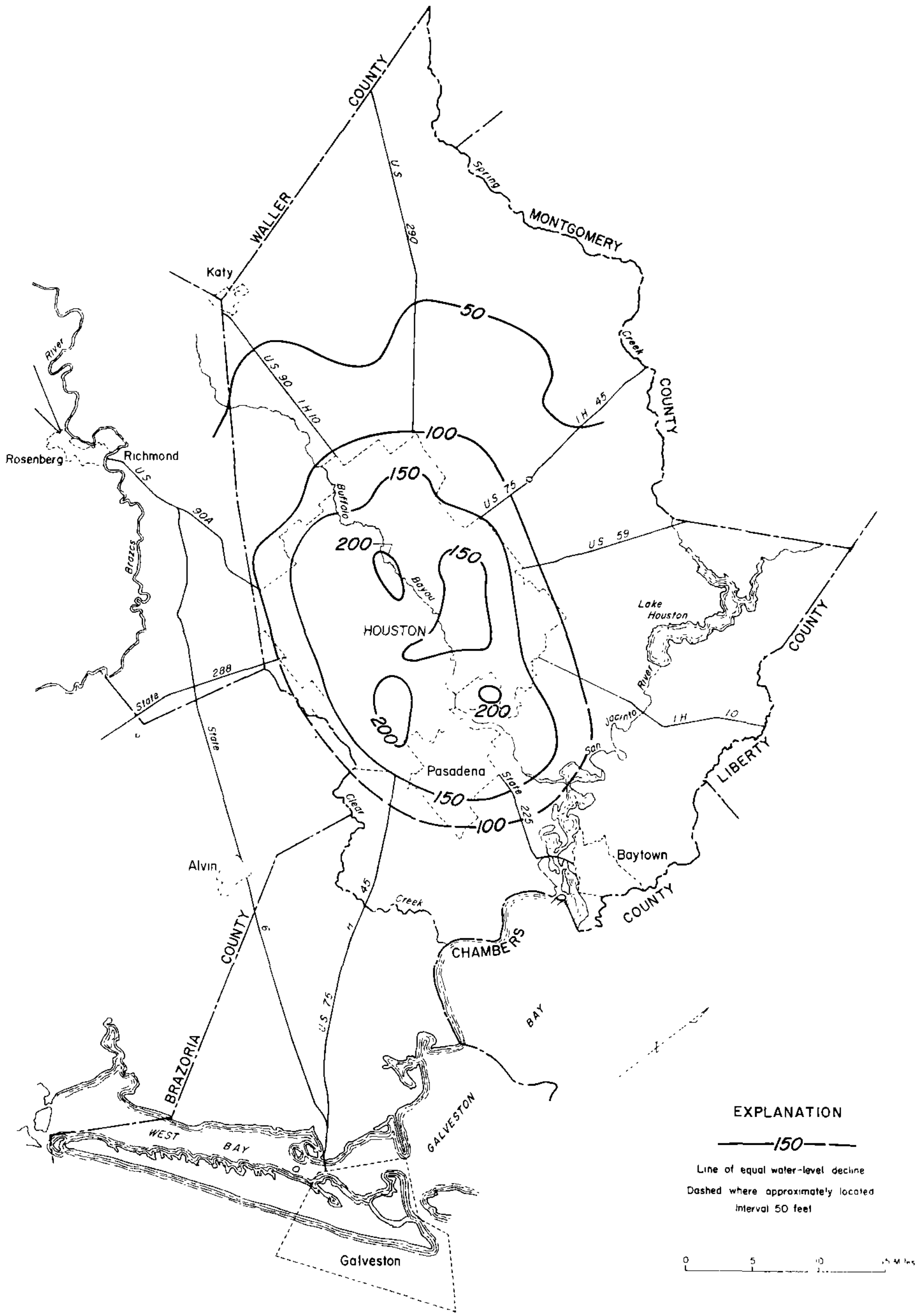

FIGURE 4. *Decline of water levels, 1943-64*

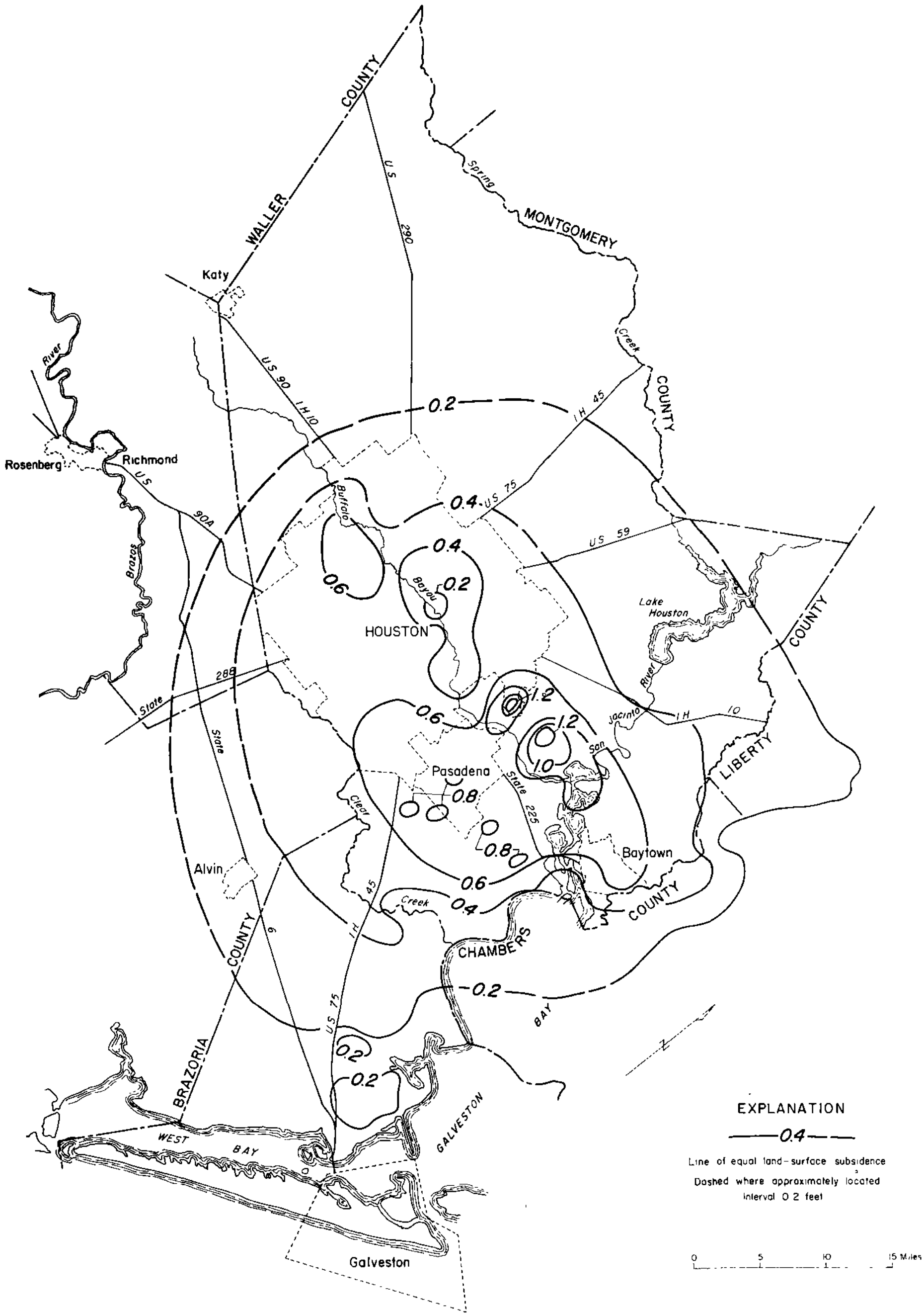

FIGURE 5. *Subsidence of the land surface, 1954-59*

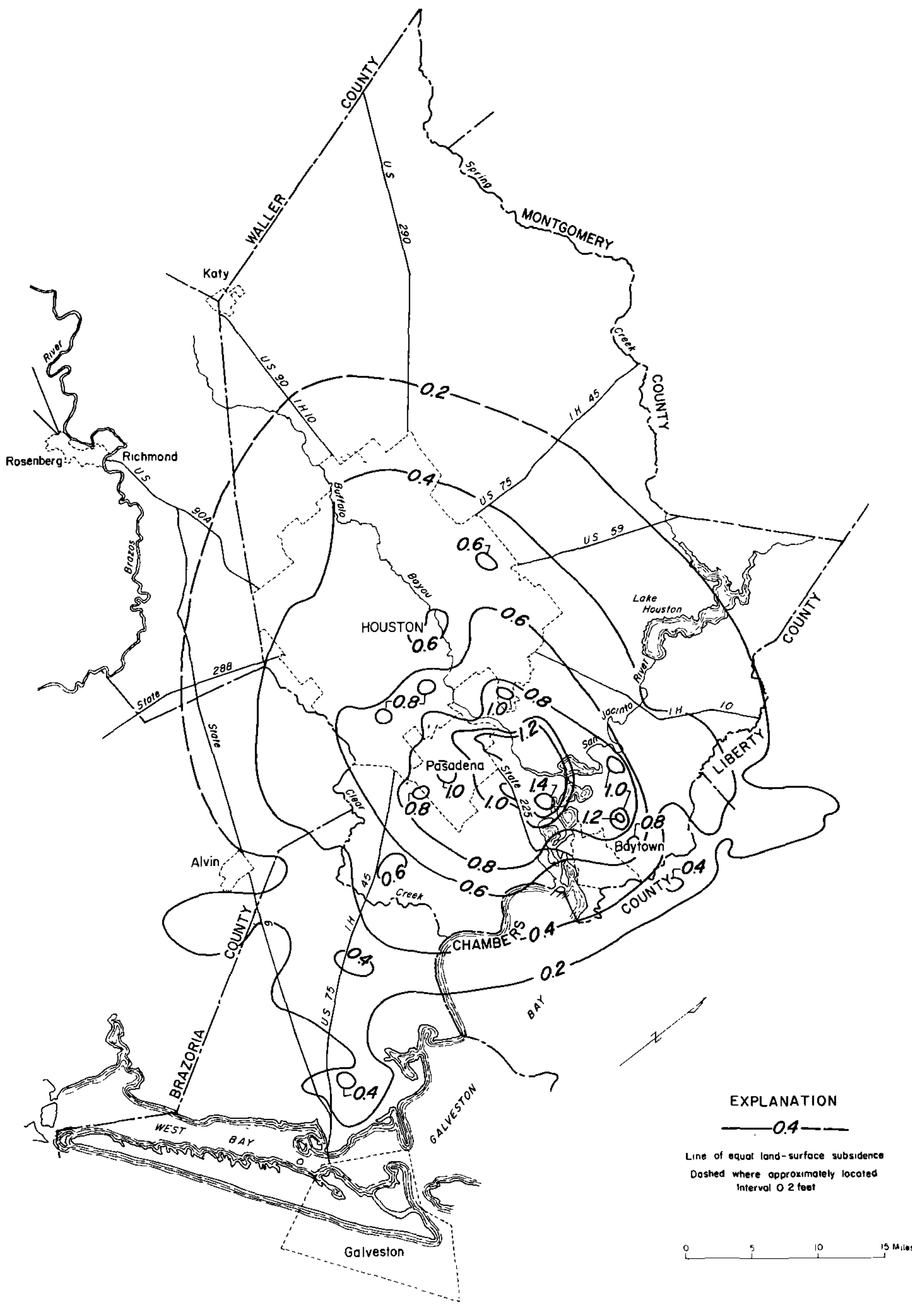

FIGURE 6. *Subsidence of the land surface, 1959-64*

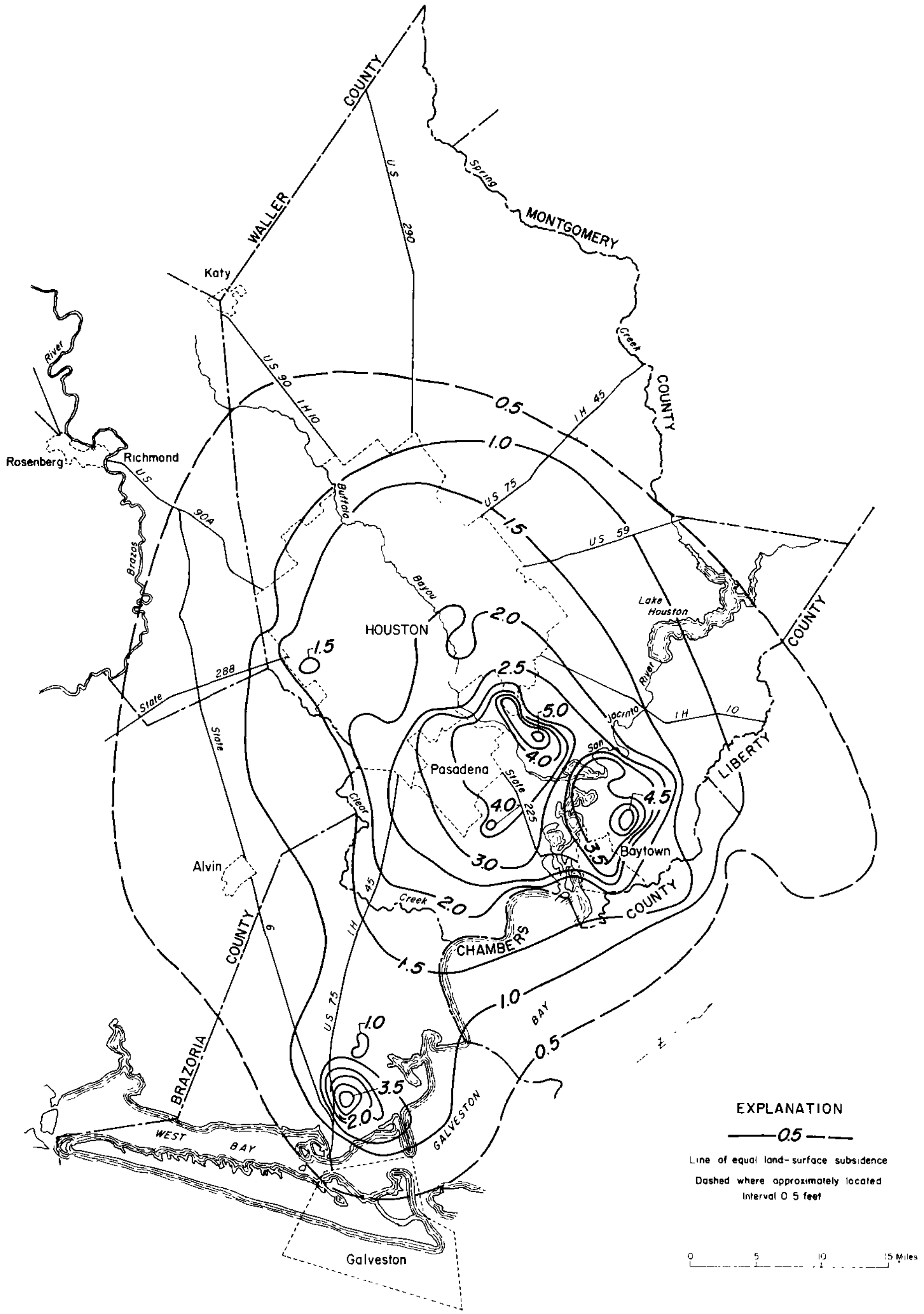

FIGURE 7. *Subsidence of the land surface in the Houston District, Texas, 1943-64*

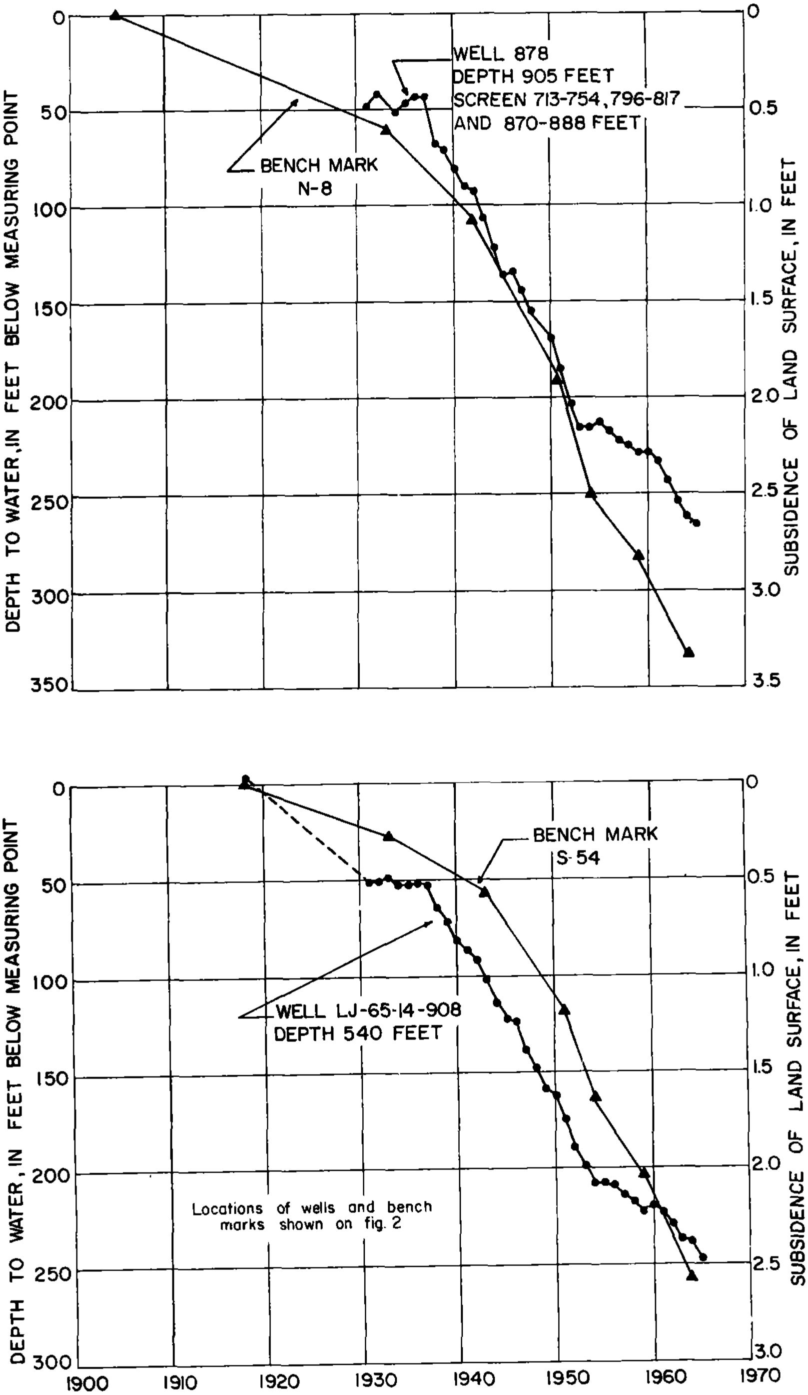

FIGURE 8. *Subsidence of bench marks compared to declines in water levels in nearby wells*

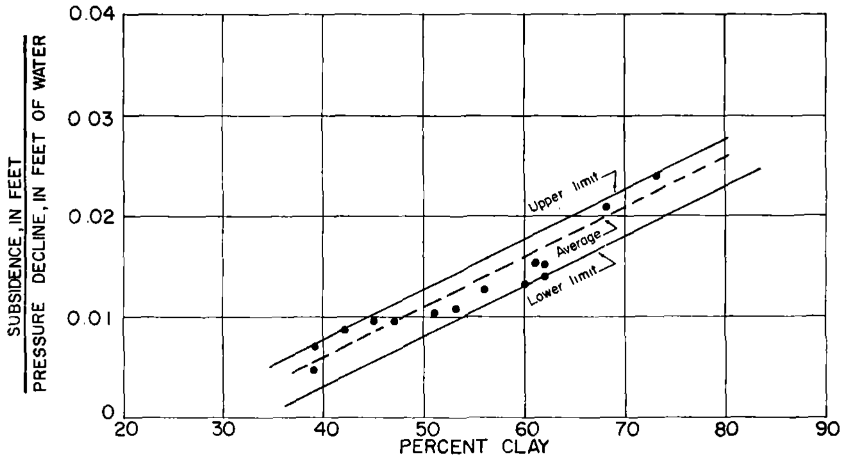

FIGURE 9. *Relation between percent clay and subsidence due to pressure decline*

CONCLUSIONS

Pumping of ground-water in the Houston-Galveston region will increase and subsidence will continue until additional surface-water sources are available.

After additional surface-water supplies become available, it is likely that there will be a decrease in the rate of ground-water withdrawal and possibly a stabilization of the pressure levels in the heavily pumped sands. The total amount of subsidence will depend upon the ultimate pressure-level decline which, in turn, is dependent upon the ground-water withdrawals.

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SURFACE DEFORMATION ASSOCIATED WITH OIL AND GAS FIELD OPERATIONS IN THE UNITED STATES

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ABSTRACT

Surface deformation associated with oil and gas field operations in the United States consists of (1) differential subsidence centering on the fields, (2) centripetally directed horizontal displacements, and (3) faulting. The number of documented examples and maximum measured movement for each effect are:

Effect	Number of fields		Maximum recorded movement		
	California	Texas	Amount (meters)	Period	Field
Differential subsidence	22	4	> 8.8	1928-1966	Wilmington, Calif.
Horizontal displacement	3	—	3.66	1937-1966	Wilmington, Calif.
Faulting	5	7	0.74	1932-1967	Buena Vista, Calif.

Faulting is commonly high-angle, normal, and peripheral to the subsidence bowl (Goose Creek and Mykawa, Texas; Inglewood and Kern Front, California), but may be low-angle, reverse, and central (Buena Vista, California).

Strain patterns developed over these fields are similar to those measured by D. V. Deere over a Texas salt dome where all three surface effects were observed during Frasch extraction of sulfur. In this case, a central compressional zone extended out to beyond the 0.35 S isobase (S = maximum subsidence) and a surrounding tensional zone extended to the periphery of the bowl, where high-angle normal faulting occurred.

RÉSUMÉ

La déformation superficielle subséquente à l'exploitation de gisements pétrolifères aux Etats-Unis se compose (1) d'un affaissement dont le maximum est centré au-dessus des gisements, (2) de déplacements horizontaux centripètes, et (3) de faillage. Le nombre de cas suivis et le déplacement maximum associé aux divers types de déformation sont repris ci-dessous :

Type de déformation	Nombre de champs		Déplacement maximum observé		
	Californie	Texas	Total (mètres)	Période	Champ
Subsidence	22	4	> 8,8	1928-1966	Wilmington (Cal.)
Déplacement horizontal	3	—	3,66	1937-1966	Wilmington (Cal.)
Faillage	5	7	0,74	1932-1967	Buena Vista (Cal.)

Les failles sont en général de pendage élevé, normales et en bordure du bassin de subsidence (Goose Creek et Mykawa (Texas) ; Inglewood et Kern Front (Calif.)), mais il en est de faible pendage, inverse et occupant le centre du bassin (Buena Vista, California).

Les types de déformation observés sont semblables à ceux qui ont été mesurés par D. V. Deere au sommet d'un dôme de sel (Texas). Ici, les trois effets de surface ont été observés lors de l'extraction du soufre par le procédé Frasch. Dans ce cas, une zone de compression centrale s'étendait jusqu'à l'isobase 0,35 S (S = subsidence maximum) et était ceinturée par une zone de tension périphérique où s'est produit un faillage normal avec pendages élevés.

INTRODUCTION

More than 40 known examples of differential subsidence, horizontal displacement, or surface faulting have been associated in time and space with operation of 27 California and Texas oil and gas fields (table 1). Altough the magnitudes of these movements rarely attain those developed over the Wilmington oil field, each effect has led to costly damage or destruction of surface structures. The necessity of avoiding such effects in urban areas emphasizes the importance of detecting and monitoring them; this necessity may require volume-for-volume replacement of withdrawn liquids.

DIFFERENTIAL SUBSIDENCE

Differential subsidence is the most common and widespread of the effects, but it is easily detected only in shoreline areas. Where level surveys have determined the size and shape of the subsidence bowl, it centers over and extends well beyond the producing area. Production in the listed fields comes from unconsolidated to poorly lithified and poorly sorted sands, generally Miocene or younger in age. Median depths of production range from about 360 to 3 900 m and exceed 1 800 m in only four cases.

Differential subsidence associated with oil field operations was first recognized in the Goose Creek oil field, on the Texas Gulf Coast. The producing strata consist of essentially unconsolidated sands and intercalated clay stringers; median depth of production is about 600 m. Production began in 1917; by 1925 subsidence centered over the producing area had exceeded 1 m and involved an area of about 11 km^2, and normal faults as long as 0.7 km, having displacements downware on the oilfield side, as great as 0.4 m, had formed along of the long margins of the eliptical subsidence bowl (fig. 1). The subsidence was accompanied by submergence of considerable near-sea level areas within the producing limits. Litigation was then initiated by the state over title to the newly-submerged "tidelands" and the value of the extracted minerals. Title was awarded to the oil field operators who contended that the subsidence was caused by removal of large volumes of materials and was therefore an act of man.

The most spectacular and costly case of differential subsidence is that of the Wilmington oil field, near Long Beach, California. By 1966, after 30 years of production and 8 of repressuring by injection, an elliptical area of more than 75 km^2 had subsided more than 8.8 m (see Poland and Davis, 1969, for a comprehensive, up-to-date review). Production is from poorly sorted, unconsolidated to semi-consolidated sand containing thin interbeds of silty shale and the median depth of production is about 1 000 m (fig. 2a). Successive collar surveys of well casings in the subsiding area at Wilmington for the first time have quantitatively related surface subsidence to subsurface compaction. Although the producing section extends to depths greater than 2 300 m, subsurface compaction was concentrated in the 650 to 1 200-meter interval (fig. 2b). Cumulative compaction of nearly 3 m had occurred in the Upper Terminal zone (960-1 125 m) by 1960; a similar amount occurred in the Tar and Ranger zones above, which aggregate about the same thickness (180 m) as the Upper Terminal. The cumulative compaction in the several zones agrees roughly with the surface subsidence at the well during the same time interval (Allen, 1968, p. 25). The surficial deposits above 650 m were in tension and well casings were stretched at least 0.6 m before full scale injection began in 1957; during the succeeding 2½ years of repressuring the casings were shortened as much as 0.75 m as the underlying producing zones expanded.

Large subsidence bowls have also been delineated over the Huntington Beach oil field, at the shoreline just southeast of Wilmington; over the Long Beach field immediately northeast of Wilmington; and over the Inglewood field, an inland field about 30 km northwest of Wilmington. Although differential subsidence over these fields has been minor

TABLE 1. Documented surface deformation over United States oil and gas fields
(Compiled from published and unpublished sources and field work by the authors during 1958-1969)

Field and Discovery Year		Maximum producing area (km²)	Median depth of production (m)	Differential subsidence[1]			Horizontal displacement		Surface faulting			
				Maximum measured (m)	Area of subsidence (km²)	Period of measurement	Maximum measured (m)	Period of measurement	Type[2]	Displacement (m)	Length (m)	Year first observed
California												
Buena Vista	1910	48	1130	0.27		1957-1964	0.39	1932-1959	Lr	0.74	2600	1932
				2.3		1942(?)-1964[3]						
Dominguez	1923	7	1430	>0.07		1945-1960						
Edison[4]	1928	6	1100	>0.09		1926-1965						
Fruitvale[4]	1928	14	1370	>0.04		1953-1965						
Greeley[4]	1936	9	3235	>0.01		1953-1965						
Huntington Beach	1920	16	930	1.22	37	1933-1965						
Inglewood	1924	5	900	1.73	11	1911-1963	0.76	1934-1963	Hn	>0.15	700	1957
Kern Front	1912	19	745	>0.34		1903-1968			Hn	>0.34	5000	1943?
Long Beach	1921	7	1690	>0.61	31	1925-1967						
McKittrick	1898	6	360	(?)					Hr	0.030	>8	1932
Midway-Sunset[4]												
Central Area	1901	65	555	>0.49		1935-1965						
Globe anticline	1912	15	1020	>0.43		1935-1965						
Sunset area	1900?	20	590	>0.18		1935-1965						
Paloma[4]	1939	23	3800	>0.07		1957-1965						
Playa del Rey	1929	2	1520	>0.29		1925-1937						
Rio Vista (gas)	1936	98	1300	>0.30		1939-1964						
River Island (gas)	1950	18	1250	>0.23		1935-1964						
San Emidio Nose[4]	1958	4	3900	>0.06		1935-1965						

TABLE 1 (*continued*)

Field and Discovery Year		Maximum producing area (km^2)	Medium depth of production (m)	Differential subsidence			Horizontal displacement		Surface faulting			
				Maximum measured (m)	Area of subsidence (km^2)	Period of measurement	Maximum measured (m)	Period of measurement	Type [2]	Displacement (m)	Length (m)	Year first observed
Santa Fe Springs	1919	6	1300	0.66	16	1927-1963						
Tejon, North[4]	1957	10	2800	>0.09		1935-1965						
Torrance	1922	27	1230	>0.10		1953-1960						
Unnamed (Orange Co.)	1909	5	1480	>0.05		1959-1964			Hr	0.05	305	1968
Wilmington	1936	29	1000	>8.84	>75	1928-1966	3.66	1937-1966				
Texas												
Clinton	1937	12	820						Hn	>0.64	>900	1962
Eureka Heights	1935	6	2540						Hn	0.24	>1000	1962
Goose Creek	1916	6	600	>1	11	1917-1925			Hn	0.41	>700	1925
Mykawa	1929	6	900	>0.1					Hn	Undermined	1000	1962
Saxet	1930	25	1800	>0.93		1942-1959			H	>0.61	2.5km	1950(?)
South Houston	1935	7	1200	0.09					Hn	0.46	>1000	1962
Webster	1937	11	1670						Hn	Undermined	>1000	1962

1. Determined with respect to local control near margins of fields; any regional subsidence gradient removed. In many cases, determ ined only for a point within the field and maximum subsidence not known.
2. H, high angle; L, low angle; n, normal; r, reverse.
3. Data courtesy R.D. Nason, 1969.
4. Data on differential subsidence courtesy B.E. Lofgren, 1969.

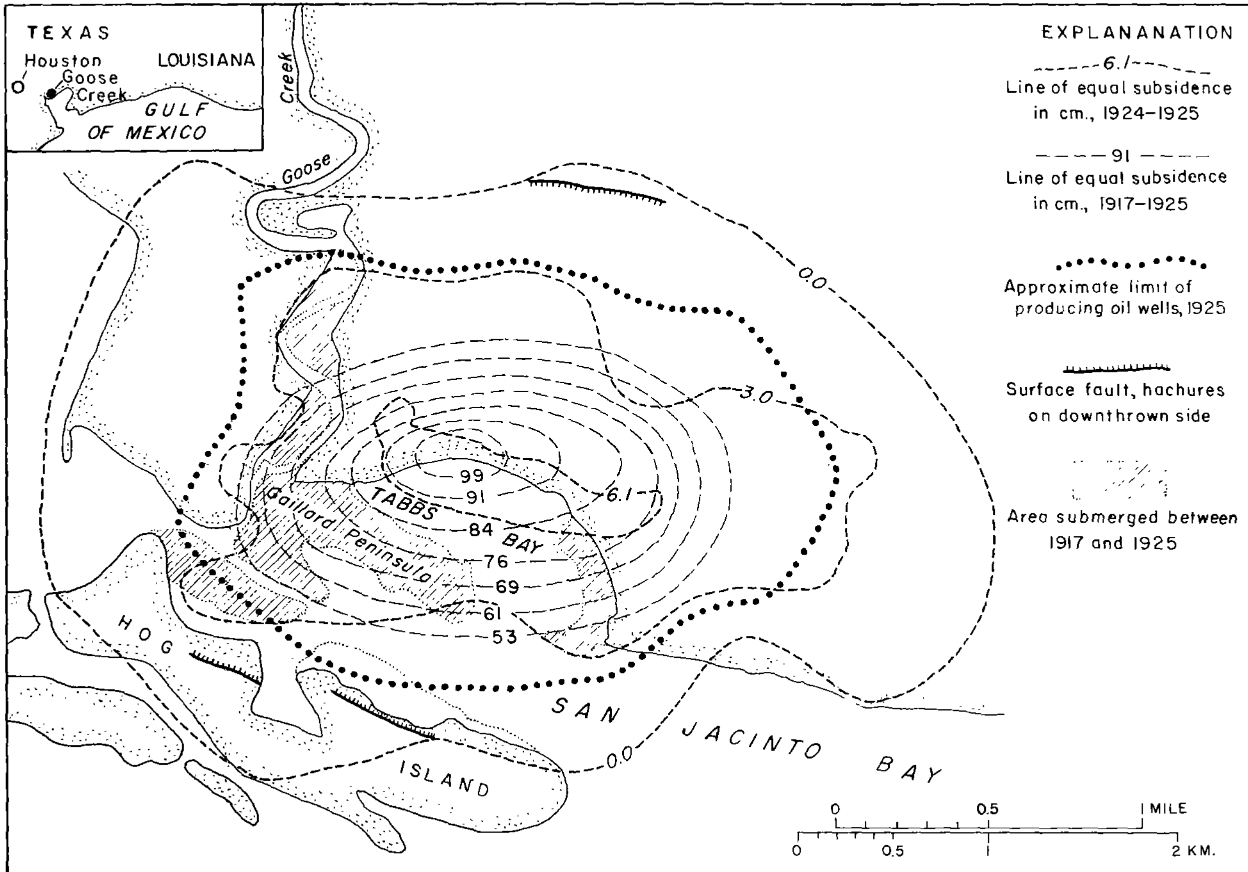

FIGURE 1. *Map of Goose Creek oil field, Texas, showing contours of equal subsidence for the periods 1917-1925 and 1924-1925, approximate limit of producing wells in 1925, and areas submerged during the period 1917-1925. Based on Poland and Davis (1969, fig. 4); Pratt and Johnson (1926, figures 2, 3, and 7)*

(less than 1.75 m) in comparison with that at Wilmington, that at Inglewood was accompanied by horizontal displacement and surface faulting that led to costly destruction of surface structures near and beyond the periphery of the bowl (see California Department of Water Resources, 1964).

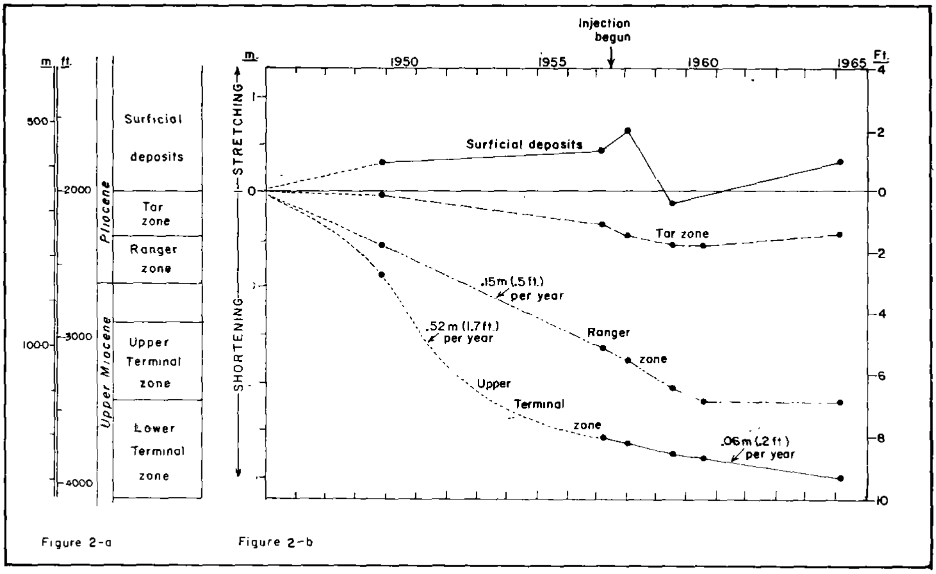

FIGURE 2. *Upper producing zones and compaction at the Wilmington oil field, southern California 2a, approximate thickness and age of upper producing zones near the center of subsidence; 2b, cumulative compaction in the upper three zones during 1945-1965 as based on casing joint measurements in a well (W-2) about 600 m southeast of the center of subsidence. Adapted from Allen (1968, figures 2 and 5)*

HORIZONTAL DISPLACEMENT

Centripetally directed horizontal displacements commonly accompany differential subsidence, but these can be documented only by triangulation and hence have been determined in only three United States oil fields. In the Inglewood and Wilmington fields the maximum horizontal displacement exceeded 35 percent of the maximum differential subsidence; it was located about halfway up the flanks of the subsidence bowl, from which location it decreased progressively to zero at both the center and periphery of the bowl. The maximum known horizontal displacement occurred within the Wilmington oil field (fig.3), where 3.66 m was measured between 1937 and 1966. This displacement resulted in horizontal strains greater than 1.2 percent. Between 1957 when repressurization began and 1967, several survey stations along the east part of the subsidence bowl (an aera where vertical rebound of about 17 percent of the total differential subsidence has occurred) recovered as much as 80 percent of their measured horizontal displacement.

SURFACE FAULTING

In addition to being the most prominent and easily documented effect, surface faulting may develop suddenly and it is therefore potentially more damaging to surface structures.

Subsidence-associated surface faulting is most commonly high-angle and normal, peripheral to the subsidence bowl and downthrown on the oil field side; it commonly trends subparallel to the isobase contours. Exemples of this type are over the Goose Creek and Mykawa fields in Texas, and over the Inglewood and Kern Front fields in California.

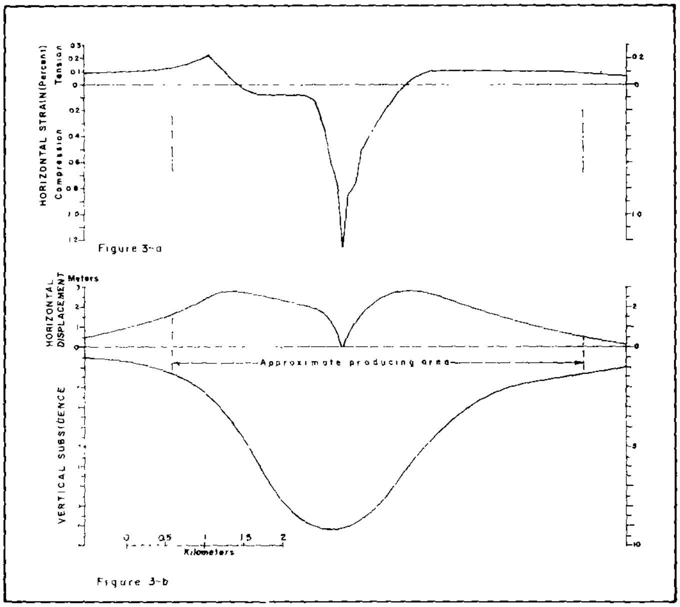

FIGURE 3. *Surface deformation over the Wilmington oil field. 3a, horizontal strain; 3b, horizontal displacement during 1937-1962 and vertical subsidence during 1928-1962. Profile is drawn along the minor axis of the subsidence bowl. Figure 3b from an unupblished study by Jan Law and D.R. Allen, Department of Oil Properties, Long Beach, California*

The surface fault at Kern Front is, at 5 km, the longest known; it follows a pre-existing subsurface fault of even greater extent that forms the east boundary of the Kern Front field. Displacement on the fault has not been great, but movement has been continuous for more than 20 years. The displacement has been described in the context of tectonic faulting unaccompanied by seismic activity (Hill, 1954, p. 11). However, the fact that it (1) is associated with measured differential subsidence, (2) continues with continuing extraction, (3) occurs along a pre-existing fault that dips toward the differential subsidence over the Kern Front field, and (4) has not been accompanied by seismicity, suggests that it is attributable to effects of subsurface compaction.

A second type of faulting, low-angle, reverse, and central to the subsidence bowl, has been recognized in only one field. A gently north-dipping reverse or thrust fault about 2.6 km long in the center of the Buena Vista oil field, California (fig. 4), has been known for more than 35 years. Dip slip of at least 0.74 m accumulated during the interval 1932-1967 (Nason and others, 1968, fig. 2), giving a average, fairly constant rate of about 2 cm/year – a rate that continues. Faulting between 1942 and 1964 was accompanied by differential subsidence exceeding 2.3 m, and by horizontal displacement of 0.39 m between 1932 and 1959. Even though the fault cannot be traced to depth in the numerous wells, it has long been cited as an example of active tectonism (Koch, 1933; Wilt, 1958); it has more recently been related to withdrawal of oil (Whitten, 1961, p. 319).

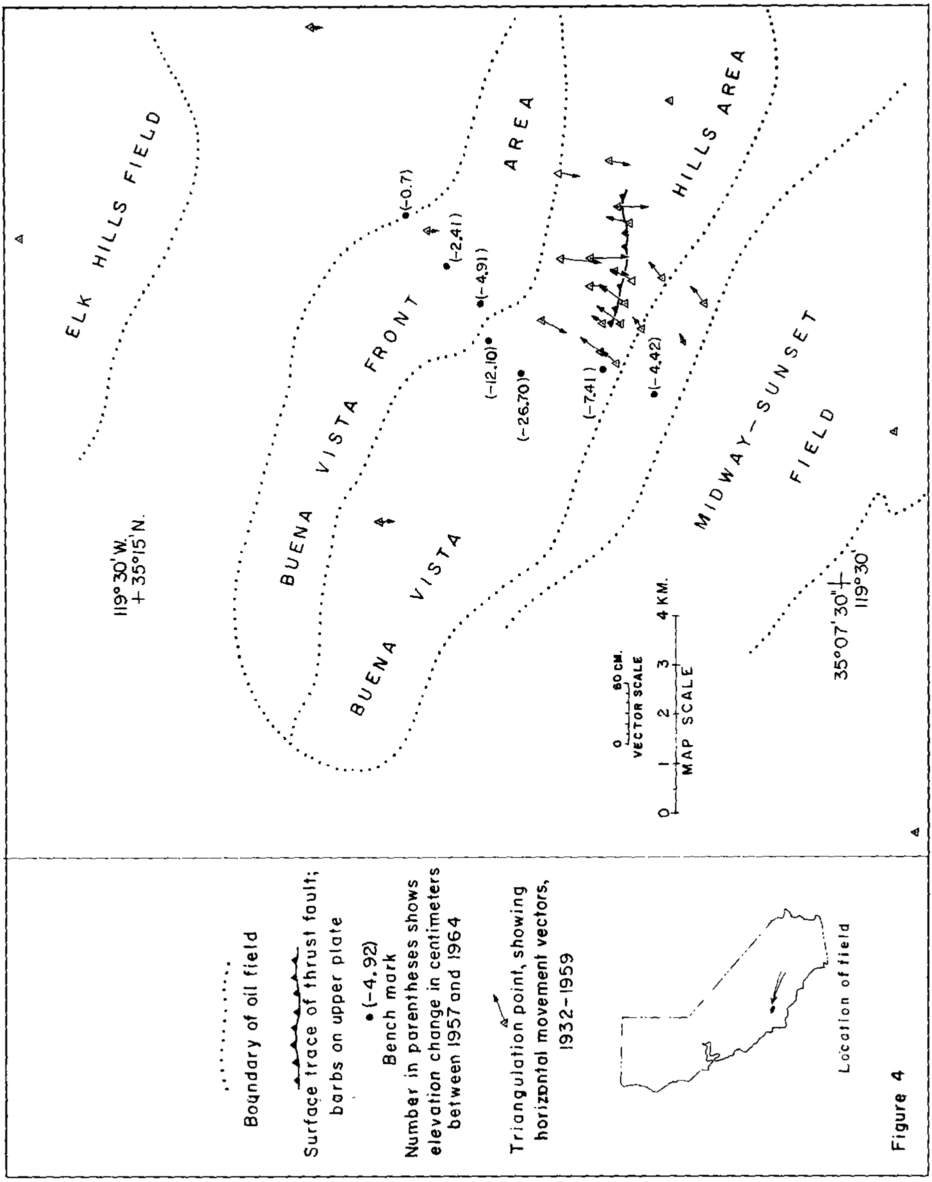

FIGURE 4. *Map of Buena Vista oil field, Kern County, California, showing (1) approximate boundaries of the Buena Vista Front and Buena Vista Hills areas of the Buena Vista field and two adjacent oil fields; (2) trace of active thrust fault along the south flank of the Buena Vista Hills area (Wilt 1958, pp. 170, 172); (3) adjusted elevation changes for the period 1957-1964 at selected bench marks (US Coast and Geodetic Survey, 1966, pp. 5-6, 18) and (4) horizontal displacement for the period 1932-1959 relative to an undefined network (Whitten, 1961, pp. 318-319)*

MODELS

The published literature contains few models that relate subsidence and other surface effects to extraction of subsurface materials. Probably the most carefully documented "controlled experiment" is the case history of surface effects measured during Frasch-process extraction of sulfur described by Deere (1961). Deere's results verify the analytical "tension center" model described by Stanford Research Institute (1949), and also resemble the results of analysis of cumultative surface deformation over the Wilmington oil field.

FRASCH-PROCESS CASE HISTORY

Deere (1961) has described differential subsidence, simultaneous horizontal displacements, and surface faulting associated with Frasch-process extraction of sulfur from a depth of 397-488 m within the cap rock of a Texas Gulf Coast salt dome; the cap is overlain by unconsolidated sands, gravels, clays, and clay-shales. During the first 31 months of operation, differential subsidence as great as 1.75 m developed over an elliptical area exceeding 5 km^2 that centered directly over the narrow linear producing zone. A normal fault about 650 m long, downdropped as much as 0,1 m on the mining side and peripheral to the subsidence bowl, developed suddenly during the fifth month of production. By the 31st month, it had

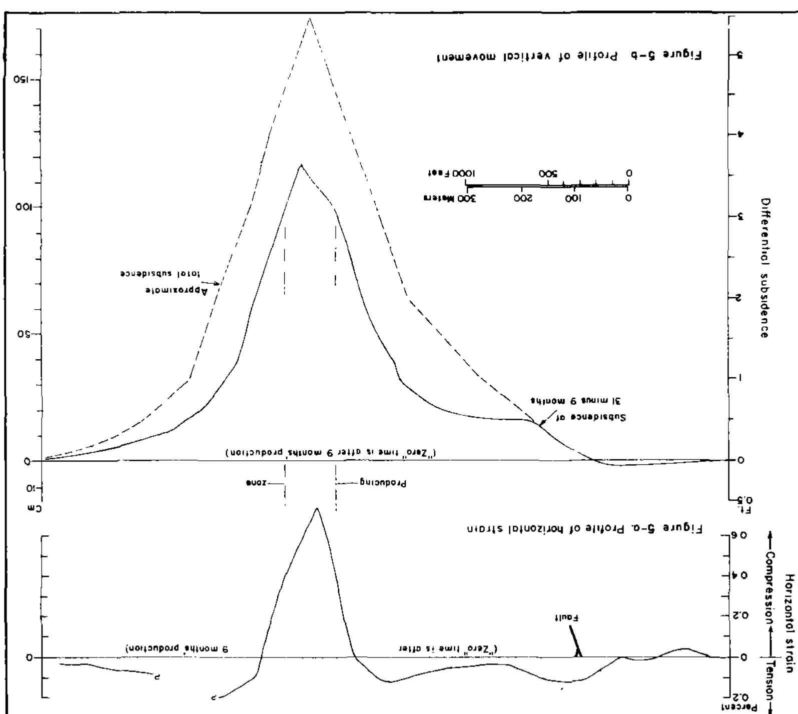

FIGURE 5. *Surface deformation during Frasch-process extraction of sulfur from cap rock of a Texas Gulf Coast salt dome. 5a, horizontal strain in percent developed between 9 and 31 months' production (after faulting had begun); 5b, vertical differential subsidence during the same period, and approxi mate total differential subsidence after 31 months production. Based on Deere (1961, figs. 1, 4, and 5)*

increased to about 800 m in length and 0.3 m in displacement. The fault dipped about 40° inward directly toward the top of the producing zone, and it formed at a point of maximum surface tension as determined by surveys (fig. 5).

Qualitative models

The "tension center" model was developed during a private study of the Wilmington oil field subsidence (Stanford Research Institute, 1949). The model assumes (1) a homogeneous, isotropic earth; (2) that all stresses remain within the elastic limit; (3) a spherical compacting region or "tension center";and (4) a negligible weight for the removed material. This model predicts a central zone of compression surrounded by an annular zone of tension that extends out to the periphery of the bowl[1], a strain pattern similar to that developed in the Frasch-mining case.

The quantitative results of Deere's investigation also support the qualitative models of Grant (1954), Rellensmann (1957), and Lee and Shen (1969). Grant (1954, p. 21) compared the subsiding prism of deposits at Wilmington to a bending beam, in which the greatest horizontal displacement of points on the surface occurs over the point of inflection (or point of greatest slope of the subsidence profile). Rellensmann (1957, fig. 2) presented dimensionless profiles of subsidence, horizontal displacement ("shift"), and horizontal strain as developed at the surface over relatively shallow mining excavations. Very similar

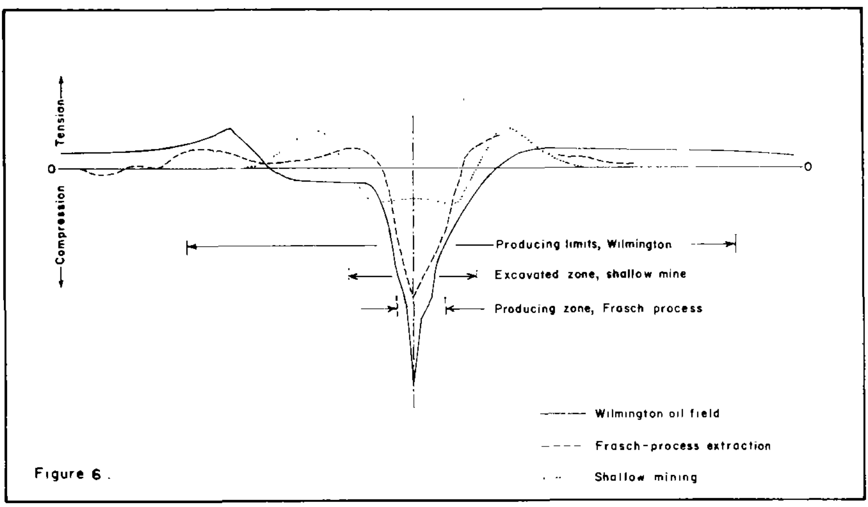

Figure 6. *Qualitative comparison of horizontal strains developed during subsidence over (1) the Wilmington oil field (relatively deep extraction of fluids over a broad area), (2) Frasch-process extraction of sulfur (from a narrow zone at intermediate depths), and (3) shallow mining of solid ores (Rellensman, 1957, fig. 2). Relative limits of producing areas are also indicated*

1. The Stanford Research Institute report (1949, pp. 76-68) presents an expression relating horizontal displacement (u) at a radius (r) from the center of subsidence to the differential subsidence (w) at that radius and the depth (h) beneath the original surface to the center of compaction: $u = rw/h$. By this expression, horizontal displacement varies from zero at the center of subsidence (where $r = 0$), through a maximum, and back to zero at the periphery (where $w = 0$).

dimensionless curves are presented by Lee and Shen (1969, fig. 1), who investigated horizontal displacement during subsidence due chiefly to surface loading; they supported the curves with both model studies and theoretical finite element analyses. A profile of the horizontal strain developed during subsidence over the Wilmington oil field (extraction of fluids over a broad area from a median depth of about 1 000 m) is compared in figure 6 with that developed during Frasch mining of sulfur (extraction from a narrow zone at a median depth of about 440 m), and with that developed over relatively shallow mining or tunnelling.

The following conclusions may be drawn from this brief review: (1) Differential subsidence commonly accompanies extraction of fluid or solid materials from poorly consolidated sequences; (2) subsidence is commonly accompanied by centripetally directed horizontal displacement, which varies from zero at the center of subsidence, through a maximum over the point of steepest slope of the subsidence profile, and back to zero at the periphery (as described by the tension center model and by Grant and Rellensmann); (3) the accompanying horizontal strains are most intense at or near the center of subsidence, where greatest compression is attained and thrust or reverse faulting may occur. In all cases analyzed, the greatest tension, and tensional faulting, develop in the outer parts of the subsidence bowl, peripheral to the annulus at which the steepest slope of the subsidence profile develops.

ACKNOWLEDGEMENTS

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DISCUSSION

Intervention of Dr. Sakuro Muraya (Japan)

Questiob:

This morning, Dr. Miyabe mentioned acute subsidence due to the earthquake in Tokyo. I think there were some earthquakes in Long Beach. Do you have any experience about influence of earthquake on the movement of surface?

Answer of Mr. Yerkes:

There has been earthquakes during oil production in Long Beach but the relations are complex and I do not clearly understand.

Intervention of Dr. Jose G. Mendez (Venezuela)

Question:

I think there have to be room for certain legal aspects. Who is respensible for the damage?

Answer of Mr. Yerkes:

I am not in the position to answer that.

SUBSIDENCE IN THE WILMINGTON OIL FIELD, LONG BEACH, CALIFORNIA, U.S.A.

M. N. MAYUGA[1] and D. R. ALLEN[2]

Abstract

The subsidence area is in the shape of an elliptical bowl superimposed on top of California's largest oil giant, the Wilmington Oil Field. The center of the bowl has subsided over 9 meters (29 feet) since 1926. Horizontal and vertical movements have caused extensive damage to wharves, pipelines, buildings, streets, bridges and oil wells necessitating costly repairs and remedial work, including the raising of land surface areas to prevent inundation by the sea. Remedial costs have already exceeded US$100 million. Most investigators

(1) Assistant Director and
(2) Subsidence Control Engineer, Department of Oil Properties, City of Long Beach, California, U.S.A.

agreed that the withdrawal of fluids and gas and the consequent reduction of subsurface pressures in the reservoirs caused compaction in the oil zones. A massive repressurization program, by injection of salt water into the oil reservoirs, has reduced the subsidence area from approximately 50 sq. kilometers to 8 sq. kilometers. The rate of subsidence at the historic center of the bowl has been reduced from a maximum of 75 cm (28 inches) per year in 1952 to 0.0 cm (0.0 inch) in 1968. A small surface rebound has occurred in areas of heaviest water injection.

RÉSUMÉ

La région subsidente a la forme d'une cuvette elliptique qui se superpose au top du plus grand champ pétrolier de Californie, le gisement de Wilmington. Au centre de la cuvette la subsidence a dépassé 9 mètres (29 pieds) depuis 1926. Les mouvements horizontaux et verticaux ont causé des dégats importants aux jetées, pipelines, immeubles, rues, ponts et puits de pétrole, nécessitant des réparations coûteuses et des travaux de protection, tels que l'élévation du niveau du sol pour empêcher les inondations par la mer. Les travaux de protection ont déjà coûté plus de $100 millions. La plupart des spécialistes sont d'accord que le soutirage de production d'huile et de gaz, et la réduction conséquente de pression dans les réservoirs a causé la compaction des zones productrices. Un programme de recompression massive, par injection d'eau salée dans les réservoirs, a réduit la région subsidente de 50 à 8 km^2, environ. Le taux de subsidence au centre de la cuvette a été réduit d'un maximum de 75 cm (28 inches) pa ran en 1952, à 0.0 cm (zéro inch) en 1968. Un léger gonflement de surface s'est produit dans les régions où l'injection d'eau a été la plus forte.

1. INTRODUCTION

Ranking high among the many causes of land subsidence are those related to man's exploitation of the earth's natural resources. One of the most widely known cases of induced subsidence occurred in Long Beach, California, USA. The subsidence in the Long Beach area has been related directly by most investigators to the production of oil and gas from the huge Wilmington Oil Field. The subsidence here has attracted world-wide attention because of its location and magnitude. Situated within the bowl of subsidence is one of California's most highly industrialized areas, including the Port of Long Beach and one of the United States Navy's most important shipyards. Figure 1 shows an airphoto of the area with contours of total subsidence as of October, 1968. Total vertical movement is about 9 meters (29 ft.) at the center of the bowl of subsidence. Horizontal movements of nearly 3 meters (10 ft) also have been measured within the area. There appears to be a definite relationship between the shape and location of the axis of the bowl of subsidence and that of the underlying Wilmington oil structure.

2. GEOLOGIC FEATURES

The Wilmington Oil Field, located near the southwestern margin of the Los Angeles Basin in Southern California was discovered in 1936. The geologic structure is a broad, assymetrical anticline broken by a series of transverse normal faults (fig. 1). The structure was "buried" or covered by approximately 550 to 600 meters of late Pliocene, Pleistocene and Recent sediments deposited almost horizontally over a Lower Pliocene-Upper Pliocene unconformity. The sediments above the unconformity contain no commercial oil and gas. Below the unconformity are seven major producing zones which range in age from Lower Pliocene to Upper Miocene (fig. 2). These productive zones span a vertical section of about 1 500 meters. Oil and gas are produced primarily from sands of varying thickness and consolidation which are interbedded with layers of shale or siltstone. The degree of consolidation of sediments is generally related to depth of burial. The sands at shallow depths are loosely consolidated and the shales become progressively softer and grade to claystones and mudstones toward the surface. Oil bearing sands are generally poorly sorted with a high percentage of fine materials. Porosities vary from 25 percent in the deep zones to approximately 35 to 40 percent in shallower zones. The "shales" at

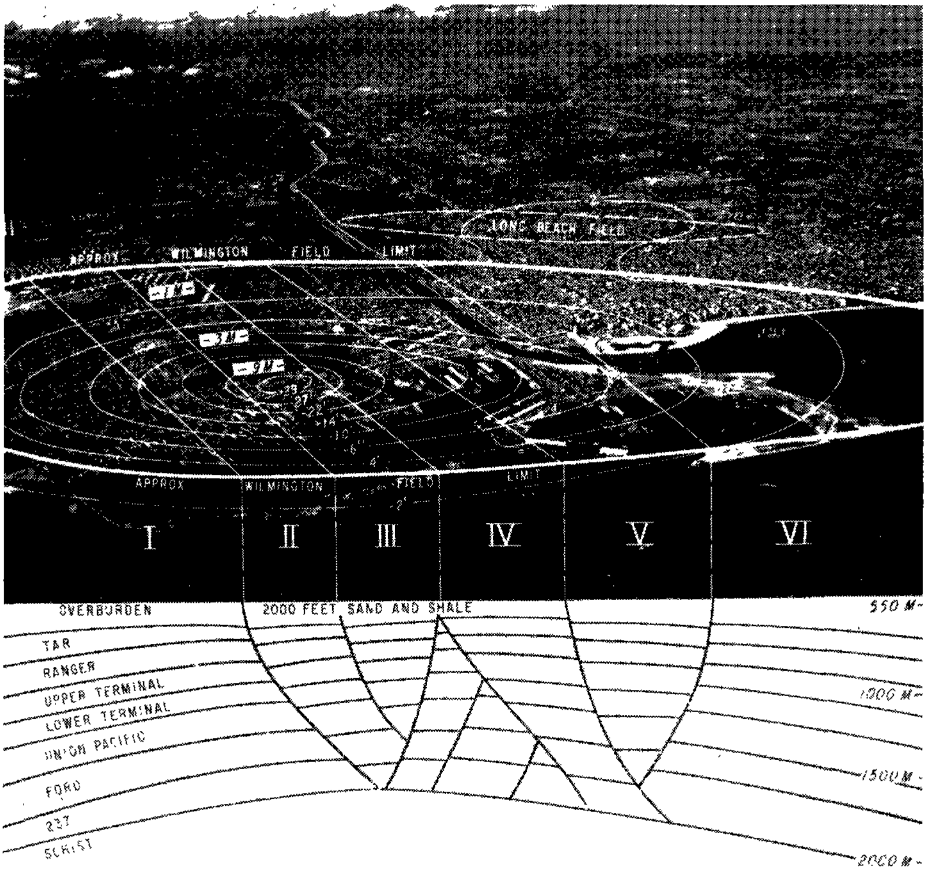

FIGURE 1. *Subsidence and Geologic Cross Section Wilmington Oil Field*

shallow depths are more accurately described as siltstones due to the high percentage of silt materials which they contain. The beds are either flat or dip gently near the crest of the structure. The primary production mechanism essentially has been a solution-gas drive. Due to a very limited water encroachment the pressure decline in the oil and gas reservoirs was relatively rapid. The substantial reduction of reservoir pressures and the compactability of rocks within the oil producing zones are considered by most investigators to be the primary causes of subsidence in the area.

The oil reservoirs were developed by zones and fault blocks. From November 1936 to July 1, 1969 the oil field has produced approximately 203, 360, 000 cubic meters (1.279 billion barrels) of oil.

3. SUBSIDENCE HISTORY

Small amounts of regional subsidence had been detected in the Long Beach-Wilmington-San Pedro area at various times prior to 1940, but little attention was given because the amount was very small. A noticeable amount of subsidence did not occur until after the major oil field development began in 1939-1940. By coincidence, the first major elevation changes were recorded in 1940 and 1941, when 40 centimeters (cm) (1.3 ft.) of land subsidence was observed at the easterly end of Terminal Island, apparently due to shallow dewatering operations in a nearby area for a large US Navy graving dock. It was assumed that the land subsidence would cease when the dewatering operations stopped. In July 1945, long after the dewatering operations had ceased, a survey by the US Coast and Geodetic Survey showed a surface subsidence of more than 122 cm (4 ft.) at the easterly end of Terminal Island. The rate of subsidence and the size of the affected area continued to increase during the following years. Continuing damage to surface and subsurface structures and the threat of inundation of the surface area caused serious concern. The average ground elevation of the harbor area prior to subsidence was only a few meters above the extreme high tides of the bay. As the ground subsided, the tidewater backed up through the storm drain systems at high tide and flooded the streets (fig. 5). By 1963, over 1 300 hectares of natural and artifically created industrial land which had been above high tide level before subsidence, had settled well below that level. Extensive diking, filling and land raising operations were undertaken throughout the harbor area. Remedial operations

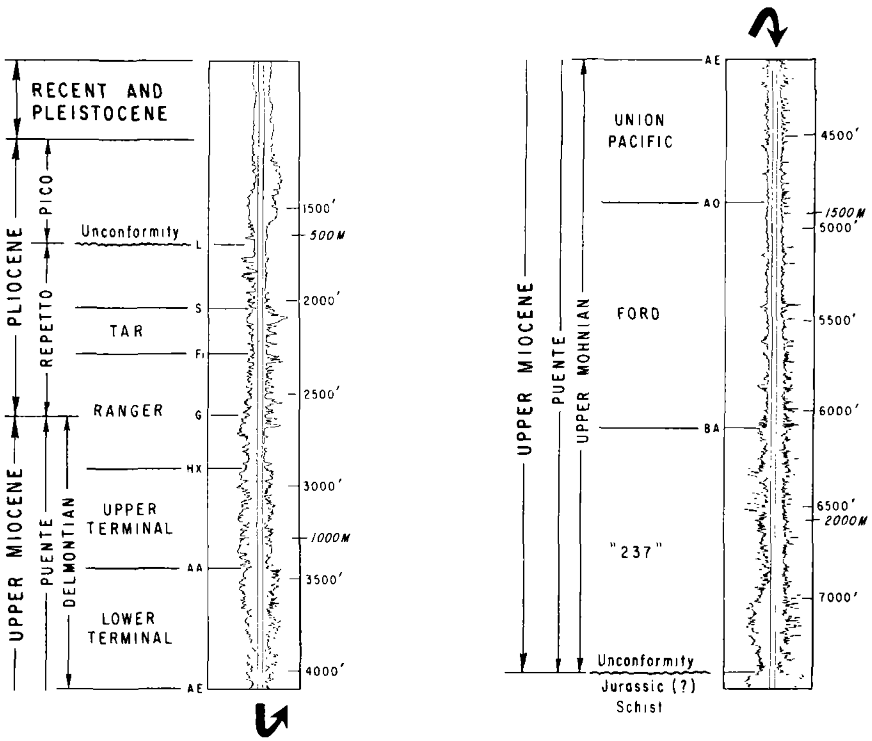

FIGURE 2. *Wilmington Oil Field—Composite log and Stratigraphic Units*

included raising and replacement of wharves, transit sheds, warehouses, oil wells, pipelines, and buildings of all types. The deepest part of the subsidence bowl, which is located over the crest of the oil structure, sank about 9 meters (29 ft.) between 1926 and 1968 (fig. 1). The maximum subsidence rate of 71 cm (28 in.) per year at the center of the bowl

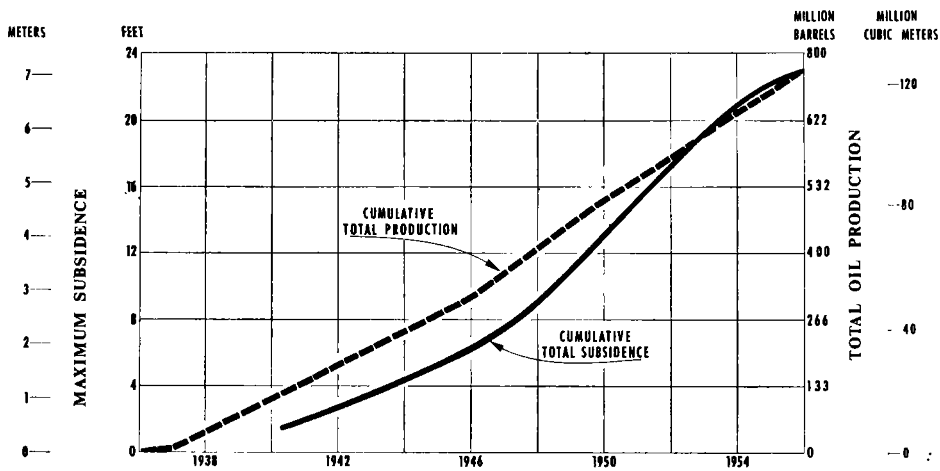

FIGURE 3. *Wilmington Oil Field—Relation of oil production to subsidence*

was reached in 1952 (fig. 4). The horizontal movements which accompanied the vertical land subsidence have caused extensive damage to many surface and subsurface structures necessitating costly repairs and replacements. Many oil wells have been damaged or destroyed by subsurface shearing associated with subsidence.

In order assist the harbor engineers in planning new construction and remedial work, various experts were engaged to predict the amount of ultimate subsidence in the area.

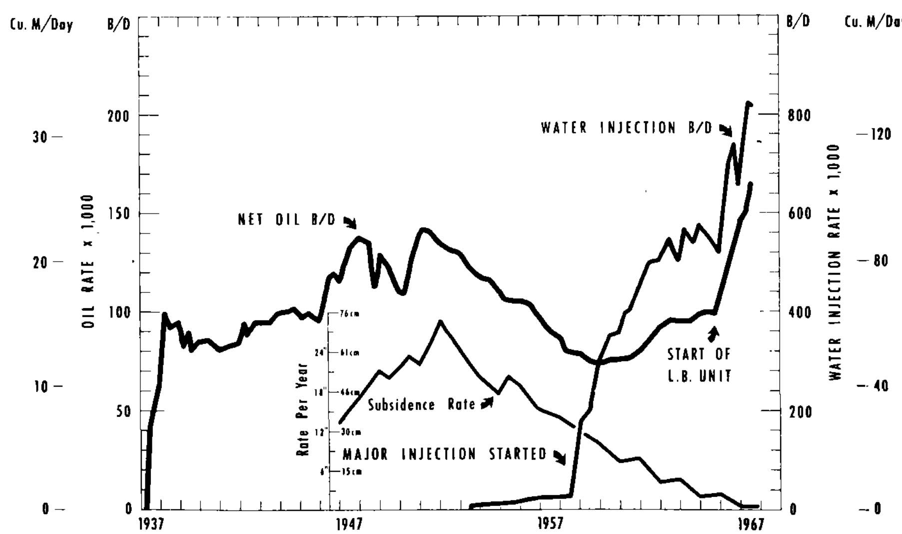

FIGURE 4. *Wilmington field Oil production rate VS. Subsidence & Water injection rates*

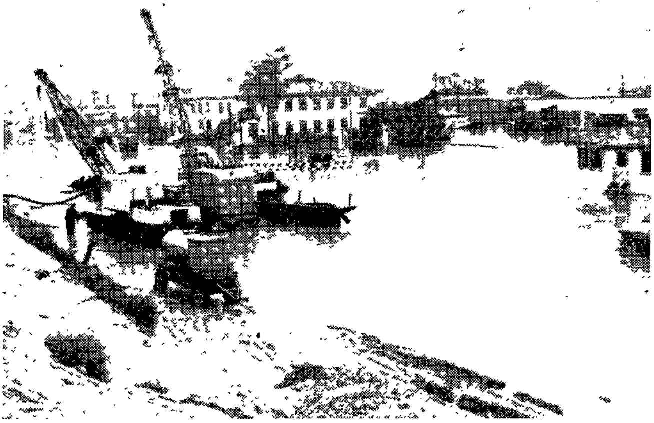

FIGURE 5. *Flooded Area Due to Subsidence*

FIGURE 6. *Buckled Pipelines*

Some early predictions ranged from 2.1 meters (7 ft.) to 3.6 meters (12 ft.) at the center of the bowl, but these were soon exceeded. Later estimates ranged as high as 22 meters with most investigators predicting between 9 to 13,5 meters (30 to 45 ft.).These predictions were made before it was known that repressurization of the oil reservoirs by water injection would stop subsidence.

A massive repressurization program, which started in 1958, has succesfully reduced the surface area and vertical rate of subsidence. The rate of vertical movement at the historic center of the bowl was reduced from a maximum of 71 cm (28 in.) in 1951 to 0.0 cm per year by 1968 (fig. 4).

4. EXTENT OF DAMAGE

The horizontal movements associated with the land subsidence built up great stresses in the surface and near-surface structures. The elastic limit of ordinary construction materials was easily exceeded. Evidence of horizontal movements was manifested on the surface by buckling of asphalt paving and railroad tracks. Buried pipelines often buckled when the overburden was removed, (fig. 6) showing the great stress imposed by the horizontal movements. Large buildings were among the most seriously affected structures due to the shortening of the ground, which pulled the foundation system with it, while the more rigid roof system successfully resisted the movement. The result was shear failures in the gunite walls and cracking of columns (fig. 7). A transit shed built with concrete walls and steel frames showed buckling of side trusses which caused compression failure of the concrete lintel in the exterior wall (fig. 8).

The Commodore Heim Bridge, a lift bridge which connects Terminal Island with the mainland to the north, suffered considerable damage (fig. 9). This bridge and its elevated approach roadways, about 1 220 meters (4 000 ft.) long, underwent approximately 2.3 meters (7.5 ft.) of shortening due to horizontal movements. The heavily reinforced concrete columns within the pier structure of the bridge were sheared off by the horizontal movements (fig. 10). The supporting towers moved horizontally and were tilted out of position making it impossible to operate the bridge.

Severe shear forces were imposed on the oil well casings by the earth movements and caused widespread casing damage. These subsurface stresses were relieved several times by sudden earthquake generating horizontal movements along claystone and soft shale beds between 450 and 600 meters below the surface. As a result of these movements, steel casings of several hundred oil wells were sheared or severely damaged along the planes of movement. Five such earthquakes were recorded between November 1949 and April 1961. A movement of 23 cm (9 in.) was observed along one subsurface horizon at about 470 meters (1 550 ft.) after one of the earthquakes. A slow continuing or "creeping" horizontal movement was also evident between the periods of earthquakes as many oil wells were continually being damaged along suspected of movements. Evidence of well damage was manifested by protrusions of tubing and casing at well heads, constriction of casing diameters, corkscrewing of pulled pipe and failure of liner hangers (fig. 15).

5. REMEDIAL WORK

(*a*) Surface Structures

As early as 1940, some remedial work was initiated at the waterfront area. As previously stated, it was imperative that the land area and the surface structures be protected from inundation by the sea. This protection took the form of nearly every type of engineering

FIGURE 7. *Shear Cracks on Wall*

construction work, including the raising of land areas with earth fills, raising of wharves and buildings, a complete replacement of badly damaged structures and facilities, construction of earth dikes, raising of bridges and approaches, increasing the height of bulkheads and rebuilding of railroad tracks and streets to provide access to the facilities. An interesting example of surface remedial work is shown in figure 13, a transit shed damaged by horizontal movements. To remedy the conditions, contraction-expansion joints were cut entirely though the width of the building approximately 60 meters (200 ft.) apart. As

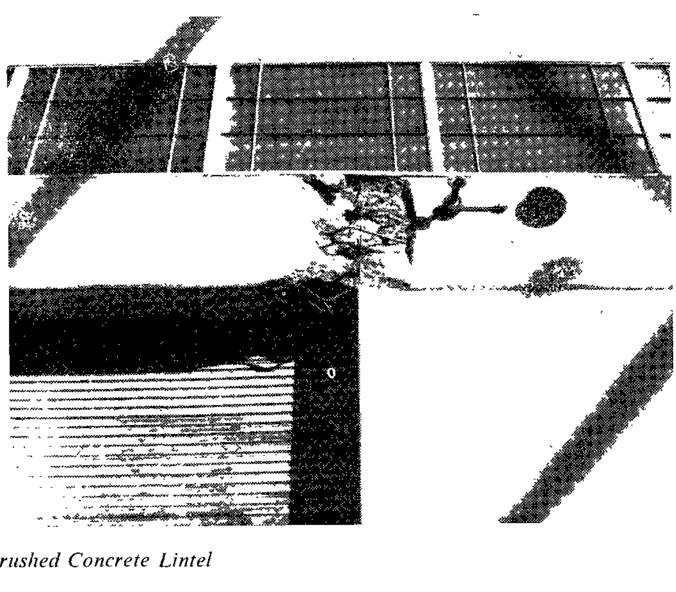

FIGURE 8. *Crushed Concrete Lintel*

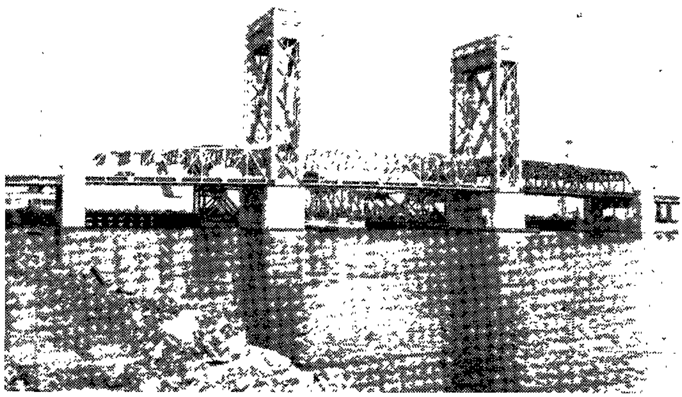

FIGURE 9. *Commodore Heim Bridge*

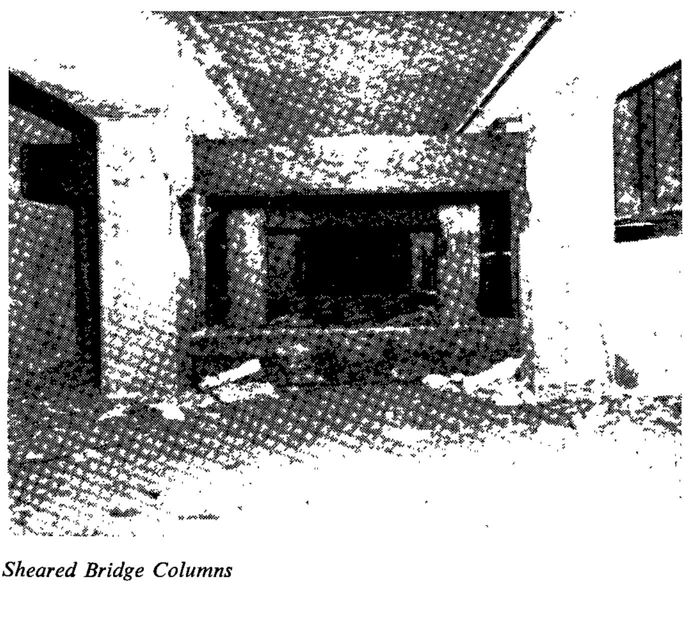

FIGURE 10. *Sheared Bridge Columns*

the threat of inundation of the area increased, however, it was physically lifted, underfilled around and lowered on new foundations at higher elevation. It has been estimated that over one hundred million dollars have been spent for surface remedial work due to subsidence.

(*b*) Oil Wells

To prevent inundation of oil wells in seriously affected areas, a large number of wellheads were raised during land fill operations (figs. 11 and 12). Oil wells which were damaged

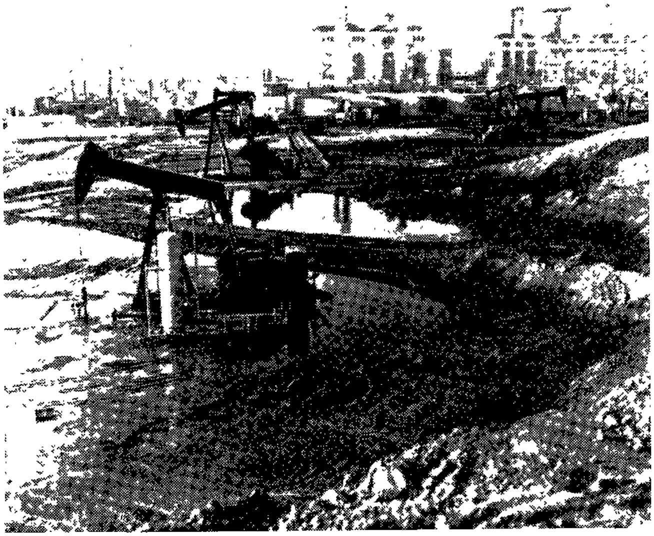

FIGURE 11. *Oil Wells Before Raising*

beyond repair by subsurface horizontal movement were abandoned, but many were replaced by new ones. Partially damaged wells were repaired by installing smaller diameter casing opposite the damaged section. In order to protect new oil wells being drilled within the area where subsurface horizontal movement was anticipated, a unique oil well completion technique was designed which allowed a small amount of subsurface horizontal movement to occur without shearing the well casing (fig. 16). The design provided for

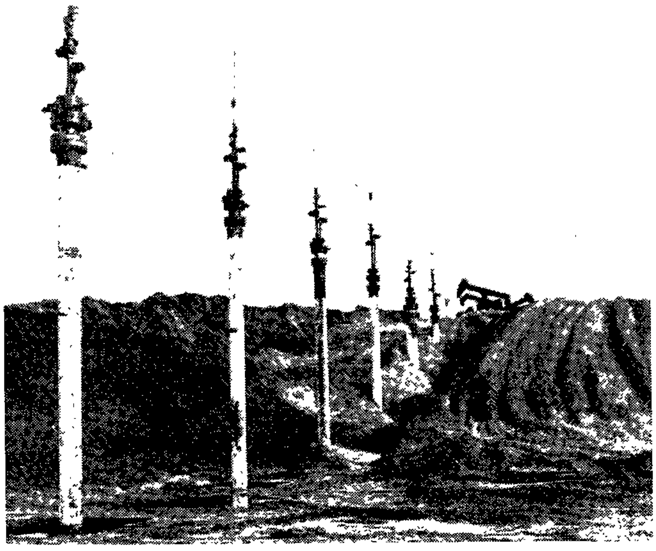

FIGURE 12. *Raising Oil Wells During Land Fill*

enlarging the usual 30 cm (12-1/4 in.) hole to 76 cm (30 in.) hole straddling the suspected interval between 425 meters (1 400 ft.) and 600 meters (2 000 ft.). Normal size casing was run inside the hole and the 76 cm (30 in.) cavity known as the "bell hole", was filled with high gel oil-base compound resembling asphaltic mastic. The technique was so successful that it became the standard completion method for many years in areas where subsurface movements were anticipated. It was discontinued during recent years due to the success in abating subsidence. It is estimated that the cost of damage to oil facilities due to subsidence has exceeded twenty million dollars.

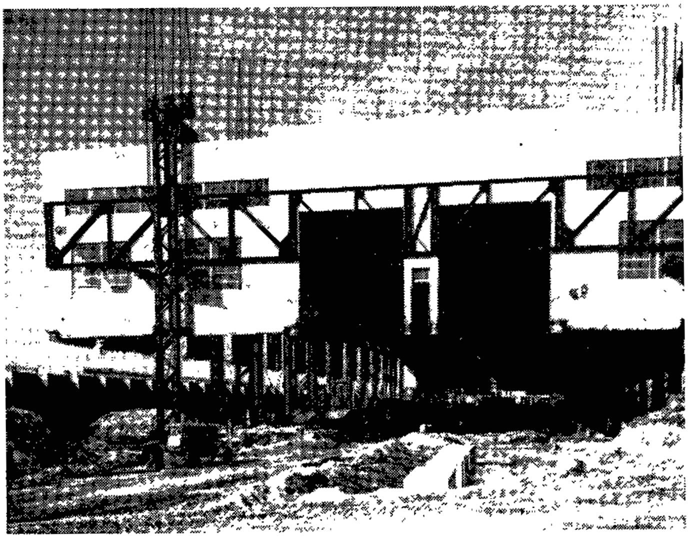

FIGURE 13. *Raising of Transit Shed*

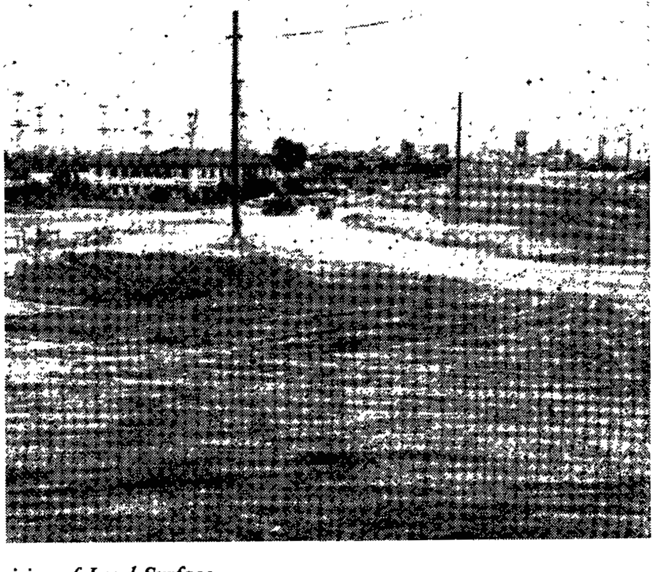

FIGURE 14. *Raising of Land Surface*

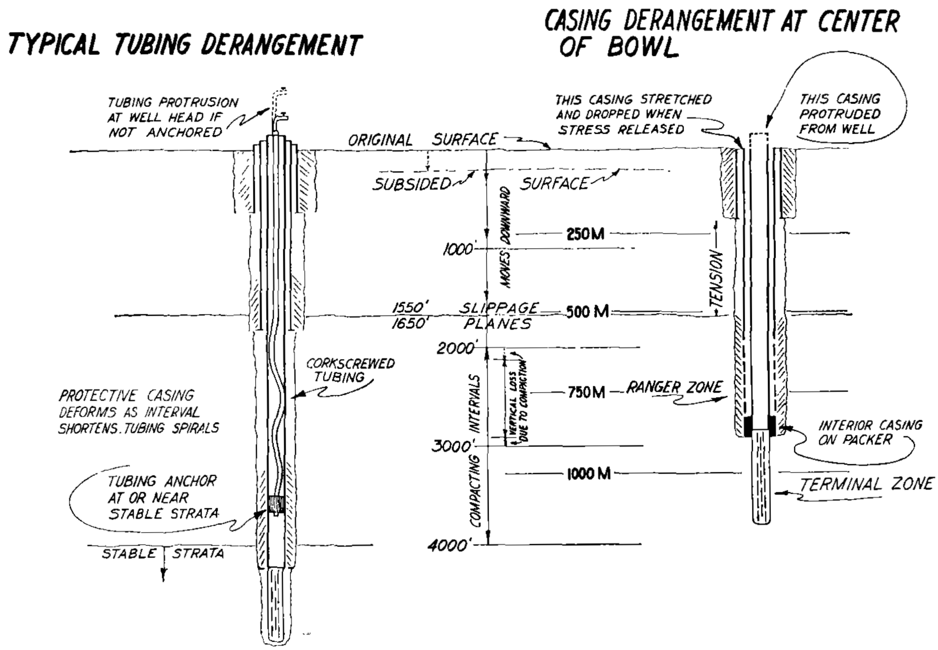

FIGURE 15. *Effect of Vertical Movement on Well Casing and Tubing*

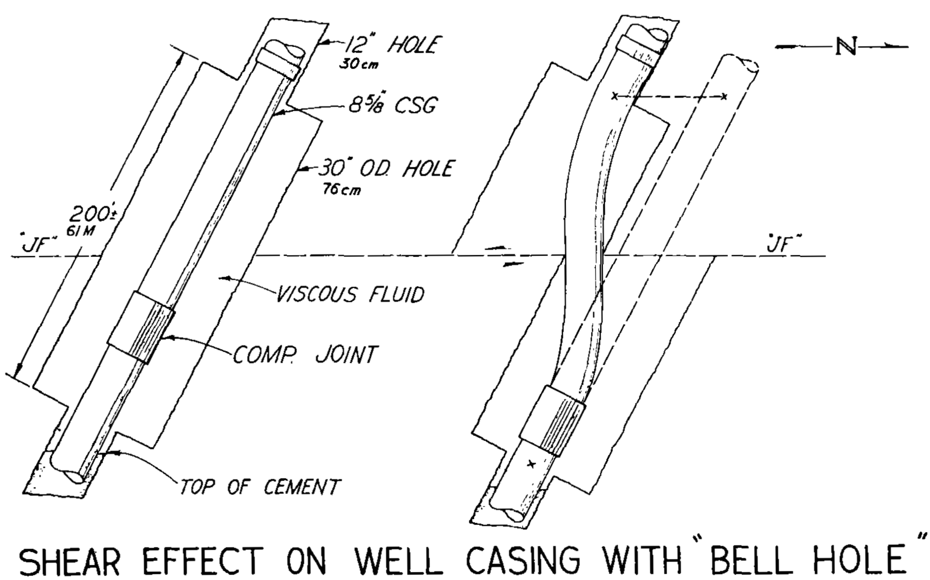

FIGURE 16. *"Bell Hole" Protection of Well Against Horizontal Movement*

6. CAUSE OF SUBSIDENCE

The cause of subsidence and the mechanics of compaction of subsurface rocks in the Wilmington Field are discussed in greater detail in the companion paper prepared for this Symposium (Allen and Mayuga, 1969). Several possible causes of subsidence were investigated by many authorities including geologists, engineers, soil mechanics experts, and mathematicians. Among the possible causes investigated were:

1. Lowering of hydraulic head due to ground water withdrawal;
2. Oil reservoir compaction due to gas and fluid withdrawal;
3. Compaction of shales and siltstones interbedded with the oil sands;
4. Surface loading by land fill and building facilities;
5. Vibrations due to land usage;
6. Regional tectonic movements and local movement along known faults in the field;
7. Lack of structural rigidity of the anticlinal structure and overlying sediments;
8. Lack of preconsolidation in sediments.

Regional tectonic movement, ground water withdrawals, surface loading and vibrations due to land usage may have all contributed to the land subsidence but the magnitude of vertical movement that has taken place is far greater than could be attributed to these causes. Most investigators agreed that the withdrawal of fluids and gas from the oil zones and the consequent lowering of subsurface pressure caused compaction in the oil sands and interfingered silts and shales. The relative amount contributed to subsidence by the shales and the sands has been a controversial issue. An interesting correlation of rate of subsidence with rate of oil production is shown in figure 4. The maximum subsidence rate of 75 cm (28 in.) per year was reached in 1952, only eight to nine months after the primary peak production of oil was reached in the area. Figure 1 also shows an interesting relationship between the deepest part of the subsidence bowl and the crest of the subsurface oil structure where the largest gross oil production per unit surface area had been obtained. Figure 3 shows a relationship between cumulative primary oil production and cumulative subsidence.

To determine the location and magnitude of compaction of the subsurface formations, a casing joint measuring method, using a magnetic collar detecting device, was developed. Results of these collar locating surveys are described in another paper dealing with mechanics of compaction (Allen and Mayuga, 1969). In general, most of the compaction apparently took place in the oil zones between 600 and 1 200 meters.

7. REPRESSURIZATION PROGRAM

Although surface remedial work which was previously described kept the area in operation, it was obvious to most observers the ultimate answer had to be the abatement of subsidence. The apparent solution to the problem, based on several studies, was to repressure the oil reservoirs by water injection. By 1961, after resolving the complex legal, engineering and economic problems involved, a full scale water injection operation was in progress in the Long Beach harbor area. Approximately 174,900 cubic meters (1.1 million barrels) of water per day are currently being injected into the field. It is estimated that a total of 366 million cubic meters (2,3 billion barrels) have been injected since the expansion of the waterflood operations in 1958. Subsidence has now been stopped over a large portion of the field and the area has been reduced from 50 square kilometers (20 sq. miles) to 8 square kilometers (3 sq. miles). A small rebound has occurred in areas of heaviest water injection. (Allen and Mayuga, 1969).

In addition to ameliorating subsidence, the water injection program has also been a great economic success, as shown by the increase in daily oil production since 1959 (fig. 4). Approximately 75 percent of the present daily production rate in the Long Beach harbor area is credited to water injection stimulation. Water currently being used for injection is sea water produced from shallow beds directly connected with the ocean. Produced oil field water is also being injected into the formations. Before the end of 1969, the largest operator in the field will commence the injection of "renovated" sewage water which will help reduce the use of high sulfate-bearing sea water and produced brine.

8. SUBSIDENCE SURVEILLANCE PROGRAM

As a subsidence surveillance program, the City of Long Beach establishes the elevation of approximately 900 bench marks within the affected area on a quarterly basis. Reservoirs are also being closely monitored by periodic subsurface pressure surveys in selected wells. Tidal gauges have been installed on the drilling islands off Long Beach as a means of detecting subsidence. Several strategically located wells are also surveyed periodically by the "collar counting" technique to detect any changes in casing joint lengths which would be an indication of subsurface compaction.

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DISCUSSION

Intervebtion of Dr. J.F. ENSLIN (Republic of South Africa)

Question:

What machinery, if any, exists in the U.S.A. whereby property owners may legally be entitled to claim compensation in the event of damage to their properties as a result of surface subsidence due to extraction of oil?

Answer of Dr. MAYUGA:

As you may have heard, we did have a lawsuit in Long Beach. The U.S. Navy, with their naval shipyard in the area and who has no interest at all in the oil, sued the City of Long Beach, the State of California, who is our partner in this oil production, and all the oil operators in the area for damage to their installations allegedly due to subsidence. That case was never actually adjudicated, because a compromise of financial settlement was made. So the responsibility was not actually established by the courts.

Most people who have been damaged in the Long Beach area are themselves involved in oil operation and received benefits from the oil production. I do not know yet what the responsibility would be or who would be liable, if someone files a claim. I think, this is a good case for lawyers. This will be argued for some time. We thought the U.S. Navy's case against the oil operators, including the City of Long Beach, would establish liability but it did not. The case was not adjudicated.

LAND SUBSIDENCE IN THE TOKYO DELTAIC PLAIN

Takamasa NAKANO, Hiroshi KADOMURA and Tware MATSUDA
(Tokyo Metropolitan University)

ABSTRACT

The following facts should be mentioned:

a) Core area of land subsidence expanded northward in the earliest period of land subsidence and in the periods before and after the end of the Second World War, and in the last several years several small core areas of land subsidence were separated.

b) Five levels of buried terraces, scarps and dissecting valleys were restored based on boring data. The core areas of land subsidence coincid with high density areas of buried valleys.

c) Eastward expansion of core area of land subsidence after the regulation of ground-water use in the western section of the Tokyo Lowland can also be pointed out.

RÉSUMÉ

Les faits suivants sont à prendre en considération :

a) Le noyau de l'affaissement s'est étendu vers le Nord au début de l'affaissement et au cours de la période avant et après la seconde guerre mondiale mais au cours des dernières années plusieurs autres centres d'affaissements se sont séparés.

b) Cinq niveaux de terrasses enfouies, d'escarpements et de vallées ont pu être établis en se basant sur les résultats des sondages. Les centres de l'affaissement coïncident avec les zones présentant une grande densité de vallées enfouics.

c) On peut aussi rappeler l'extension vers l'est de la zone d'affaissements massifs après les mesures pour régulariser l'utilisation de l'eau dans la partie ouest des régions basses de Tokyo.

1. INTRODUCTION

As a result of long discussions on land subsidence in the Tokyo Lowland among the specialists concerned, the following have been clarified.

1. Land subsidence has been caused mainly by the compaction of soft clayey layers within aquifers subject to over-pumping of ground water.
2. Compaction is progressing not only in Alluvium, but also in Diluvium.
3. Generally speaking, the amount of land subsidence is shown by the amount of compaction of Alluvium and that of the layers below Alluvium.
4. Relationship between the amount of land subsidence and the thickness of Alluvium is rather clear, particularly in case of land subsidence due to pumping of ground water from Alluvium and Upper Diluvium water bearing layers.

Unfortunately, however, areal characteristics of land subsidence have not been fully understood in previous studies. In this respect, the authors intended to analyse the characteristics of areal differentiation of land subsidence in the Tokyo Lowland and to consider the causes of such areal differentiation based on land subsidence data since 1930 and geological and geomorphological studies.

2. MIGRATION OF THE CORE-AREA OF LAND SUBSIDENCE

The rate and mode of land subsidence in the Tokyo Lowland have changed annually and regionally. Such changes are throught to have been caused both by the areal difference of ground conditions, above all by the areal variation in the thickness of soft compressible clayey layers, and by the areal difference of rates of lowering of groundwater levels due to pumping.

In order to make clear the regionality of land subsidence, areal variation of land subsidence is analyzed by using data obtained from repeated levelings during the following periods; 1930-1938, 1951-1955, 1961 and 1966, each of which represents *1*) peak period of subsidence before the Second World War, 2) the recurring period after the waı, 3) the peck period after the war, and 4) the declining period, respectively. In this analysis the area which subsided at a rate of more than 100 mm/year is defined as the Core-area of land subsidence because annual subsidence of 100 mm is thought to be an indicator showing the intensity of subsidence, and corresponds approximately to the maximum annual compaction withing the shallower deposits – shallower than 35-70 m, almost all of which are the Recent Deposits, as deduced from data obtained from the land subsidence gauge during the peak period after the war.

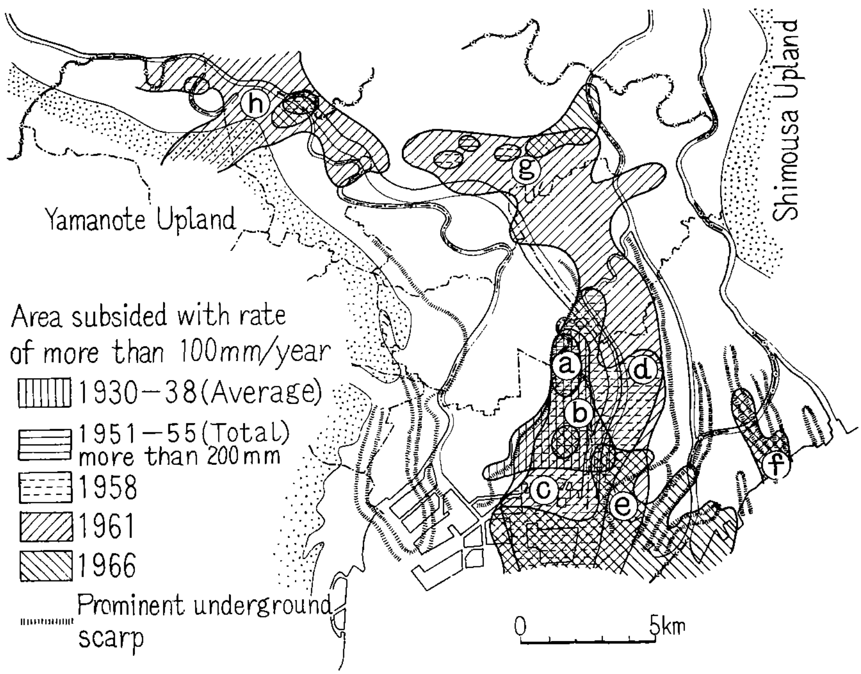


FIGURE 1. *Migration of core-area of land subsidence.* (a) *Azuma-Higashi;* (b) *Central Joto;* (c) *South Joto;* (d) *Central-North Edogawa*; (e) *Central-South Edogawa;* (f) *East-South Edogawa;* (g) *Adachi;* (h) *Johoku*

Areas *a)–h)* are the core-areas of land subsidence in the Tokyo lowland, and are located in the limited part of the lowland. As show on figure 1, annual and areal differentiations of the Core-areas are clearly pointed out as follows:

1. The core-areas until 1955, at the end of the recurring period, are found in the Koto District including the areas *a*) *b*) and *c*), and in the northern part *g*). Among these areas *a*) is noted as the area of maximum subsidence.
2. After 1957, the beginning of the peak period of subsidence after the war, the main body

of the core-area expanded rapidly not only over both the west and east bank areas of the Arakawa Discharge Channel, but also to the far northern part of the lowland

3. During 1966, the maximum period of subsidence, the maximum annual subsidence exceeded 180 mm in areas *a*), *b*), and *g*), and the core-area occupied the widest extent, about 69 km^2.
4. In the Johoku area (*h*) in the northwestern part of the lowland, the area has subsided at a rate of more than 100 mm/year since 1955, and extended even over the Yamanote Upland in the peak period.
5. Annual subsidence in the main body of the core-area has been decreasing since 1963, accompanying the decrease in total size of the core-area. The cause of decrease of subsidence is due mainly to the regulation of pumping of ground water in this area.
6. But, in the southeastern coastal area intensive subsidence with a rate of 100-180 mm/year has occurred since 1961. The southeastern coastal area located on the east bank of the Arakawa Discharge Channel is the major area of subsidence in recent years.

3. RESTORATION OF BURIED LANDFORMS AND THE SIGNIFICANCE TO LAND SUBSIDENCE

a) Buried landforms of the Tokyo lowland

Generally speaking, terraces and valleys are buried under the coastal lowlands, which were formed by rejuvenation due to eustatic change of sea level. These terraces and valleys consist of harder layers than the layers covering them and an unconformity is present between them. The soften sediments comprise the layers near the ground surface and are indication of their physical properties. From this standpoint, the upper loose sediments are defined as Alluvium in this study, and are essentially the same as the Recent Alluvium employed in geological studies. Alluvium consists mainly of marine clayey sediments (AC Alluvial clay in fig. 2) overlain by deltaic sandy sediments (US: upper sand). The clayey layer is divided into three parts according to N-value (fig. 2).

In order to investigate the distribution of the thickness of Alluvium, the authors tried to reconstitute the landforms buried by the Alluvium, that is, landforms expressed by the base of the Alluvium. In the first place, the cross sections along Subway No. 5 (A-A″) and Metropolitan Expressway No. 7 (B-B″) ware drawn as the datum profiles (fig. 2). Then analysing other data obtained from borings, the distribution of buried landforms was clarified as shown on the geomorphological map (fig. 5).

According to these data, landforms have been identfied:

1. Upper shallow terrace (Ia)

The terraces at about 10 meters depth below sea level are shown on the western end of the cross section A-A″ and along the eastern part of the cross section B-B″. They are buried coastal terraces with a width of 2-4 kilometers and are distributed along the fringe of the Diluvium uplands, Yamanote and Shomosa Uplands. The eastern terrace named Koiwa Daichi inclines southward gently and reaches 13 meters below sea level at its southern end.

2. Lower shallow terrace (Ib)

There is a terrace at a depth of 20-30 meters below sea level as shown in the eastern part of cross section A-A″. This buried coastal terrace named Urayasu Daichi is present under the southeastern part of the lowland. I*a* and I*b* consist of well consolidated sandy layers.

3. Upper middle terrace (IIa)

The terraces at a depth of approximately 30 meters below sea level are indicated between borings No. 10 and No. 21 of cross section A-A″ and along the western part of the cross section B-B″, but they are parts of the same terrace with broad distribution under the western area of the lowland. This terrace can be regarded as a buried river or fluvial terrace because it consists of gravelly layers overlain by tephra, Kanto Loam, of variable thickness.

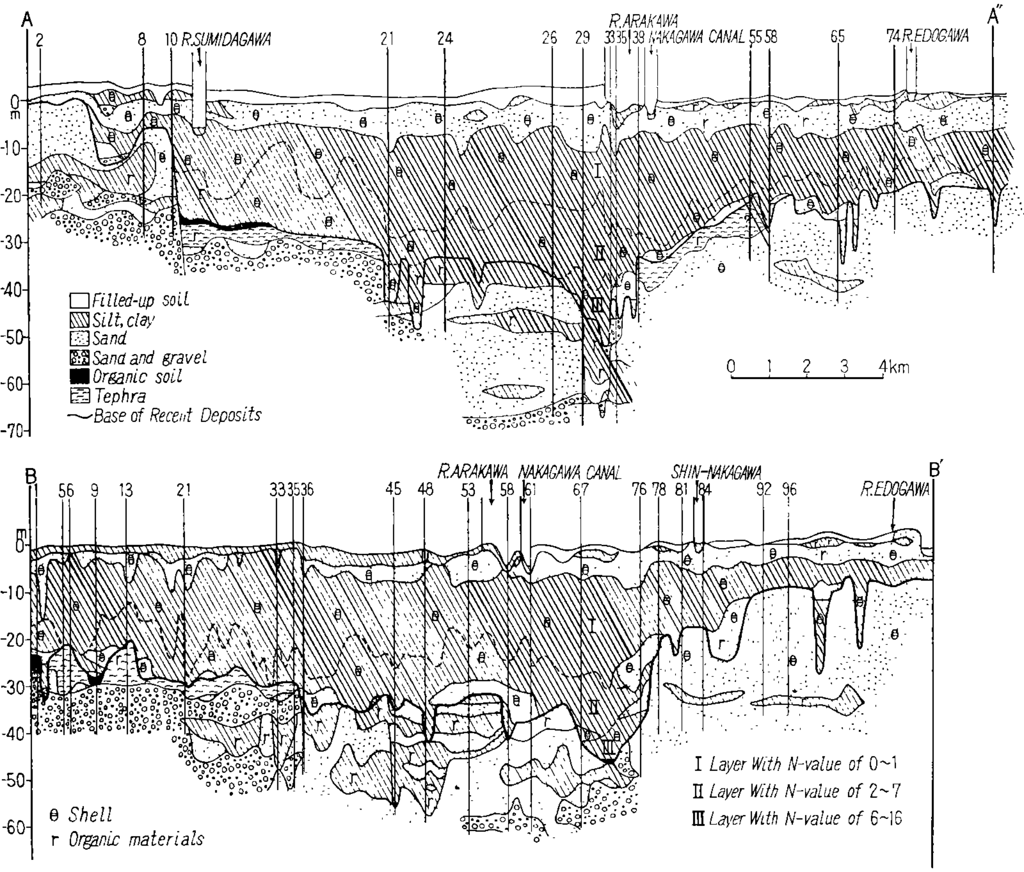


FIGURE 2. *Cross sections along Subway No. 5 (A-A″) and Metropolitan Expressway No. 7 (B-B″). Location of sections is shown in figure 5.*

4. Lower middle terrace (IIb)

This terrace is indicated between borings Nos. 38 and 55 of cross section A-A″. It slopes westward at about 20-30 meters below sea level, but its distribution is restricted to a small area in the southern part of the lowland and its origin remains unexplained.

5. Deeper terrace (III)

The lowest terrace under the central part of the lowland is shown in the middle of both cross sections A-A″ and B-B″, 30-40 meters below sea level. The materials comprising the terrace are clayey and sandy sediments, and vary both vertically and horizontally from place to place. They also vary in density, so it is often impossible to define the base of the Alluvium.

6. Underground valley

Several valleys dissect the terraces mentioned above and manifest themselves in both cross sections, though they are in different sizes. The largest is under the central part of the lowland at a depth of 60 meters below sea level and in the southern part of the lowland it is considered as a main valley.

As mentioned above, the upper shallow, buried terraces which are composed of well consolidated materials, are present beneath both sides of the lowlands. The upper shallow terraces are connected in the western part with the upper middle terrace, which is composed of Alluvial gravelly layers with tephra, and in the eastern part with the lower middle terrace. Also the lower terrace is present between the middle terraces where the several valleys modify these landforms.

The idealized schematic profile showing subsurface geology and landforms is presented in figure 3. The depths of buried terraces and valleys and the thickness of Alluvium covering them are given in table 1.

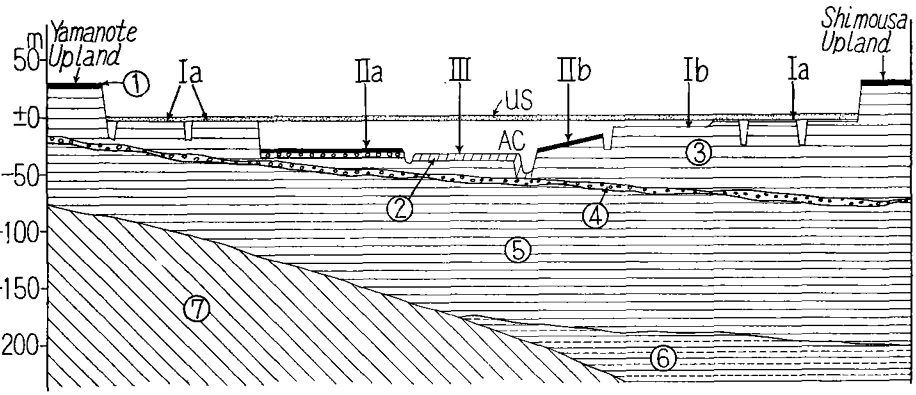


FIGURE 3. *Simplified geological section of the Tokyo Lowland. Buried terrace — I*a*: Higher upper; I*b*: Lower upper; II*a*: Higher middle; II*b*: Lower middle; III: Lower. Stratigraphy — Alluvium: US: Upper Sand; AC: Silt and Clay; Diluvium & older: 1: Tephra; 2: Uppermost Clay; 3–5: Tokyo; 6: Narita G.; 7: Miura G.*

TABLE 1. Depht of the buried landforms and thickness of Alluvium (in meters below sea leve

Buried landforms	Depth	Upper sand (US) (3-20)	Alluvial clay (AC) I (0-1)	II (2-7)	III (6-16)
Upper shallow terrace	0–10		0–6	0	0
Lower shallow terrace	20–30		5–10	2–5	0
Upper middle terrace	30	4–8	6–16	4–8	0
Lower middle terrace	30–40		10–20	2–6	0
Deeper terrace	30–40		11–22	6–12	0
Valley bottom	30–60		11–22	6–12	0–12

Figures in the parenthese denote N-value.

b) RELATIONSHIP BETWEEN LAND SUBSIDENCE AND BURIED LANDFORMS

The areas subsided at a rate of more than 100 mm/year, in the core-area of land subsidence which is limited to the central part of the Tokyo Lowland, as disccussed previously. In order to explain such areal differentiation of land subsidence the realtionship between and subsidence and ground conditions is analyzed annually and regionally by using restored buried landforms.

The relationship between cross sections of buried landforms along Metropolitan Expressway No. 7 and Subway No. 5, and the amount of subsidence measured by bench marks near the sections during 1930-1938, 1961 and 1966 is shown in figure 4. The following apparent relationships can be recognized from this illustration:

1. The difference in the rates of subsidence is shown clearly between areas on both sides of the lowland which are underlain shallow buried terraces, and areas in the central part of lowland where Alluvium is thickest.
2. The eastern edge of the core-area of subsidence (subsidence rate of more than 100 mm/year) has never extended beyond the underground scarp limiting the western margin of the Koiwa, the Nishi-Ichinoe and the Urayusu Daichi. The western edge of the core-area coincides approximately with the eastern scarp of the Honjo Daichi, and has never extended far over the terrace area even in the peak period of subsidence. In the southern part of the Tokyo lowland, the main part of the core-area of subsidence almost always has been limited to the central part of the lowland, where the lowest buried terrace and valleys are deeper than 30 m below sea level—the area where the total thickness of the Alluvium exceeds 30 m.
3. In the southeastern part of the lowland subsidence at a rate of more than 100 mm/year has occurred recently in and around deep buried valleys in the Urayasu Terrace.
4. Comparing the rate of subsidence over the Honjo Daichi with that over the eastern shallow terrace group, the following contrasts can be recognized: Although mean annual subsidence of 50-100 mm was observed over the Honjo Terrace during 1930-

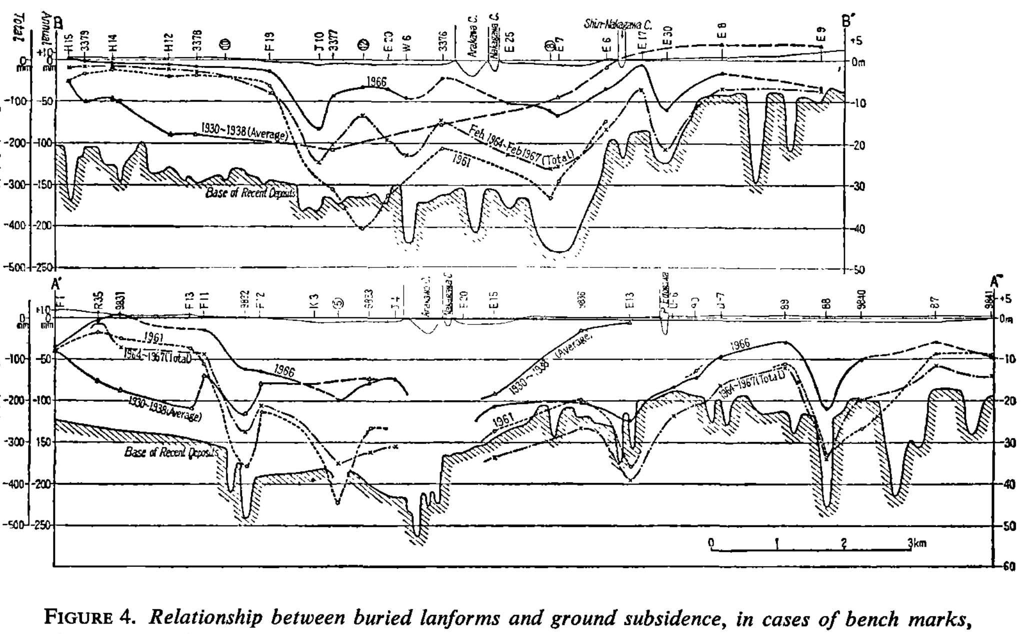


FIGURE 4. *Relationship between buried lanforms and ground subsidence, in cases of bench marks, along Metropolitan Expressway No. 7 (B-B') and Subway No. 5 (A'-A''). Location of sections is shown in figure 5*

1938, slight uplift of land surface was recorded over the Koiwa and the Urayasu Daichi during the same period. However, during 1961 and 1966, annual subsidence over the Koiwa and the Urayasu Daichi was 30 mm greater than that over the Honjo Daichi. This may reflect the annual variation of ground-water level in these areas.

From the above, it is natural to conclude that buried landforms which determine the areal variation in the thickness of soft compressible clayey layers has played an important rol in the magnitude regionality of land subsidence. But, in order to prove the exact role of buried landforms, it is necessary to determine the ratio of the relative amount of soil compaction from the surface to the base of Alluvium to the total amount of subsidence.

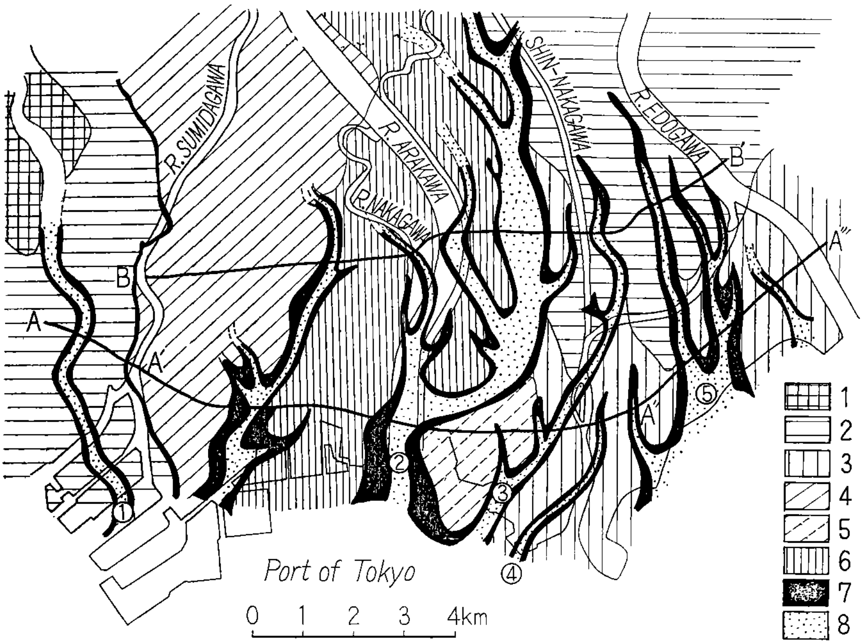


FIGURE 5. *Geomorphological classification of buried landforms in the south Tokyo Lowland. 1: Yamanote Upland; Buried terrace — 2: Higher upper (Ia); 3: Lower upper (Ib); 4: Higher middle (IIa); 5: Lower Middle (IIb); 6: Lower (III); 7: Buried scarp; 8: Buried valley — (1): Showadori-dani; (2): Main valley; (3): Myokenjima-dani; (4): Urayasu-dani; (5): Maeno-dani; A-A''' & B-B': Location of cross sections in figure 2 and figure 4*

According to observed data on partial soil compaction using standard iron tubes, 60-100% of the total subsidence has been caused by the compaction of soil layers within the shallow deposits, shallower than 35-70 m, which corresponds approximately to the thickness of Alluvium. Among the shallower deposits, the marine clay layer (Ac), the distribution of which is governed by buried landforms, is the main compaction layer because of its engineering properties and thickness. (figs. 2-3, tab. I).

Therefore, buried landforms are significant indicators of the areal characteristics of land subsidence.

4. CONCLUSION

In conclusion, the following facts are presented

a) Core-areas of land subsidence expanded northward in the earliest period ofland subsidence and in the end periods before and after the end of the Second World War. In the last several years several additional small core areas of land subsidence have been identified.

b) Five levels of buried terraces scarps and dissecting valleys were recognized based on boring data. The core-areas of land subsidence coincided with high density areas of buried valleys.

c) Eastward expansion of the core-area of land subsidence after the regulation of ground water use in the western section of the Tokyo Lowland can also be recognized.

REVIEWS OF LAND SUBSIDENCE RESEARCHES IN TOKYO

Yoshi INABA, Iwao ABE, Susumu IWASAKI, Shigeru AOKI, Takeshi ENDO and Ryozo KAIDO

ABSTRACT

The present paper deals with an historical review of development of land subsidence research in Tokyo during about thirty years. Three periods are distinguished; the purposes, methods and results of the research in each period are briefly summarized in relation to advancement of the land subsidence phenomena in Tokyo.

RÉSUMÉ

Une revue historique des recherches sur les affaissements à Tokyo au cours des 30 dernières années est présentée. Trois périodes peuvent être distinguées ; les buts, méthodes et résultats des recherches dans chaque période sont brièvement résumés en relation avec l'avancement de la subsidence à Tokyo.

1. INTRODUCTION

The land subsidence in the lowland of Tokyo was discovered as a result of repeated precise levelling which had been carried out in 1924, in order to study the post-seismic crustal disturbance associated with the Kwanto Earthquake of 1923. The recognition of such an abnormal subsidence of land is said to be the first one in Japan or elsewhere.

The present paper is a historical review of the progress of the land subsidence phenomena in Tokyo and of the related investigations.

The writers' acknowledgements are given to Dr. Naomi Miyabe for his kind supervision and for reading this manuscript, and to the members of our Institute for their valuable assistance in the preparation of this manuscript.

2. HISTORICAL REVIEW

A historical review of the land subsidence observed in Tokyo and the related explorative activities is shown in table 1. Three periods can be distinguished in the history of development of the land subsidence phenomena and the related exploration in Tokyo. These are summarized in the following paragraphs.

TABLE 1. Historical Review of Land Subsidence and Related Explorations in Tokyo

Year	Phenomena, Disasters, Countermeasures	Explorations, surveys
1923	(Kwanto Earthquake occurred).	Precise levelling started.
1924	Abnormal subsidence of bench marks in Koto Delta discovered.	
1929		Geological maps of Tokyo and Yokohama published.
1932	The degree of land subsidence became remarkable.	
1933		Observation well with compaction recorder constructed, and soil tests undertaken.
1938	Land subsidence advanced.	Repeated precise leveliling in Koto Delta.
1940		Fukugawa observation well of 35 m depth constructed
1941	(The 2nd World War broke out)	Tokyo Metropolitan Conferences for Land Subsidence Problems made plan for land subsidence exploration. Six observation wells of 32-35 m depth constructed.
1942		Azuma observation well of 19 m depth constructed.
1944	Land subsidence had not advanced.	
1945	(Koto region attacked and devastated by bombers. The 2nd World War ended).	Observations well destroyed. Precise levelling for all bench marks stopped. Local levelling continued.
1957	Subsided area invaded by high water influx caused by typhoon.	Relevelling started.
1949	Subsided area invaded by high water influx caused by typhoon. Land subsidence revided.	

Year	Phenomena, Disasters, Countermeasures	Explorations, surveys
1951		A series of land subsidence exploration for five years started.
1952	Iron Tube of observation wells sinking.	Kameido observation well of 60.5 m constructed. Amount of industrial ground water withdrawal surveyed. Chemical components of ground water studied.
1953	The degree of land subsidence in the Johoku district became remarkable.	Two observation wells of ground water table constructed.
1954		Regional hydrogeological survey in and around Tokyo by G.S.J. started. Amount of industrial ground water withdrawal resurveyed.
1955	Land subsidence in the Koto Delta remarkably advanced.	Azuma-B observation well of 115 m depth constructed. Future amount of land subsidence for five years estimated.
1957	Local land subsidence occurred along river valley in upland area of Tokyo.	Adachi observation well of 115 m depth constructed. Survey on amount of ground water withdrawal in whole lowland area started.
1959	(Ise Bay Typhoon struck Nagoya City).	Geological map of Tokyo Published.
1960		Repeated precise levelling for embankment of subsidence started. Future amount of land subsidence reestimated.
1961	Control of industrial ground water withdrawal in the Koto Delta decided.	Group of observation wells of different depths constructed at several localities, i.e., Nanagochi No. 1 (50.5 m)—No. 1 (27 m depth)—No. 4 (290 m depth) etc.
1962	Control of ground water withdrawal for building use decided.	Hydrogeological map published.
1963	Control of industrial ground water withdrawal in Johoku district decided.	Three observation wells constructed.
1966	Rate of land subsidence decreasing. Ground water table rising. Shrinkage of deeper soil layers advanced. Construction of outer embankment of Koto Delta completed.	Observation well of 450 m depth constructed.
1968		Four observation wells constructed in upland aarea. Geological map of Tokyo published.

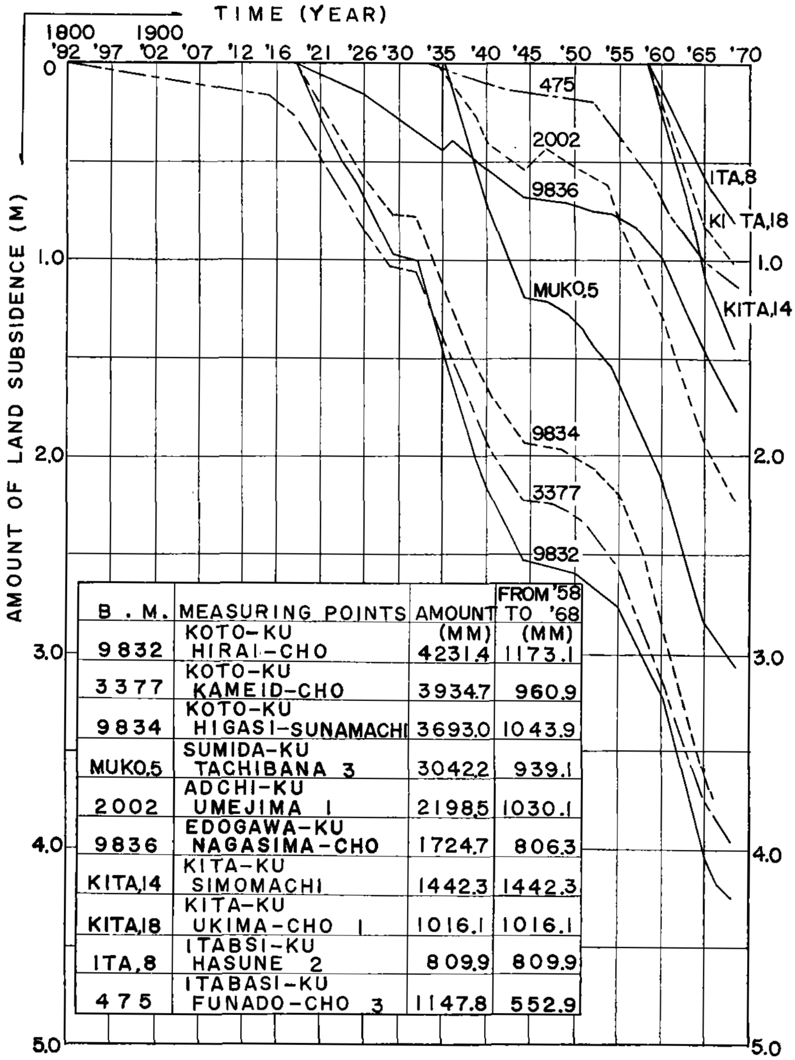
2.1 The first period (1923 to 1945)

Since the discovery of land subsidence in the lowland of Tokyo following the severe earthquake, it became quite remarkable (see fig. 1). As a result of repeated precise levelling, the several bench marks in the Koto Delta had undergone subsidence of 10 cm or more/year during the years 1938 to 1940 (see fig. 2).

The exploration and research during this first period were concentrated on the cause of such an abnormal subsidence of land. In the early phase of this period, the cause of the land subsidence was thought to be one of the following.

(i) Crustal or tectonic movement.

(ii) Shrinkage of subsurface layer.



B. M.	MEASURING POINTS	AMOUNT (MM)	FROM '58 TO '68 (MM)
9832	KOTO-KU HIRAI-CHO	4231.4	1173.1
3377	KOTO-KU KAMEID-CHO	3934.7	960.9
9834	KOTO-KU HIGASI-SUNAMACHI	3693.0	1043.9
MUKO.5	SUMIDA-KU TACHIBANA 3	3042.2	939.1
2002	ADCHI-KU UMEJIMA 1	2198.5	1030.1
9836	EDOGAWA-KU NAGASIMA-CHO	1724.7	806.3
KITA.14	KITA-KU SIMOMACHI	1442.3	1442.3
KITA.18	KITA-KU UKIMA-CHO 1	1016.1	1016.1
ITA.8	ITABSI-KU HASUNE 2	809.9	809.9
475	ITABASI-KU FUNADO-CHO 3	1147.8	552.9

Figure 1. *Total subsidence of several bench marks in the lowland of Tokyo*

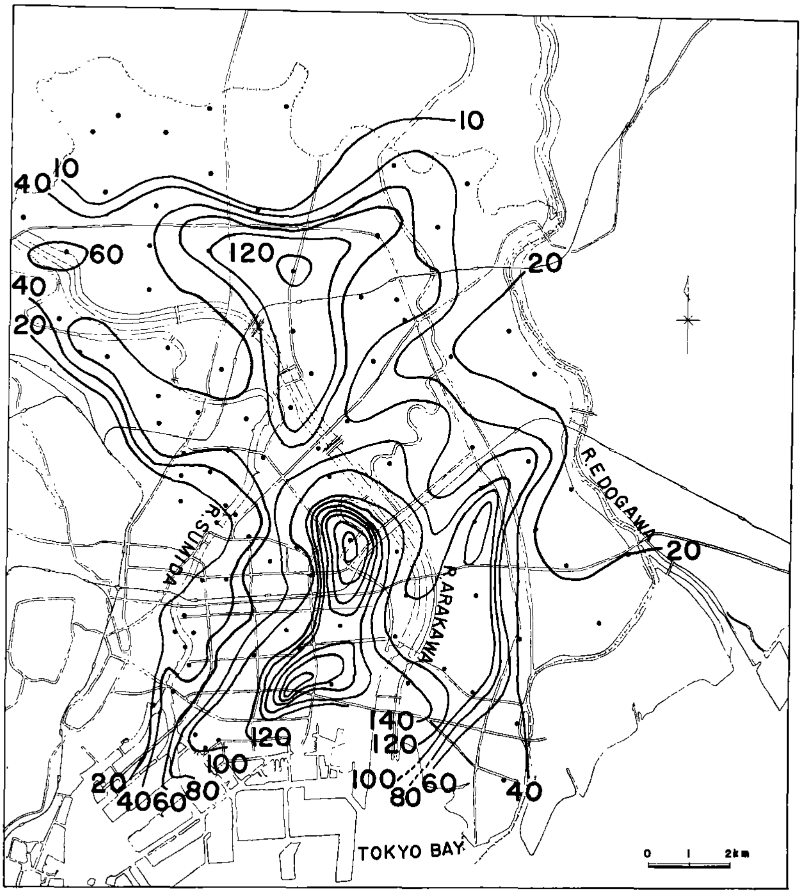


FIGURE 2. *Land subsidence during 1938-1940 (in mm)*

In association with the advancement of the land subsidence, the apparent lift up of masonry buildings and deep well pumps had become remarkable in the subsided area. The foundation of masonry buildings and the bottom of deep wells in the affected area usually rest on the deep-seated pleistocene sand and gravel beds which are covered by the holocene soft soil layer. Therefore, such phenomena suggested that the land subsidence had been caused by shrinkage of the layer of holocene deposits. In order to carry out research on the said phenomena, the first observation well of 35 m depth was contructed by Miyabe at the Kazuya Primary School, in the Koto district, in 1933. This well was equipped with a type of compaction recorder. Subsequently, several wells of the same type were constructed at various locations by the Tokyo Metropolitan Government during 1940 and 1941.

As a result of observation at the said wells, the shrinkage of the subsurface soft soil layer was recorded and, therefore, the land subsidence research was then concentrated on and analysis of the cause of shrinkage of this soft soil layer. In connection with the construction of the observation wells, soil tests on the samples derived from the bore holes had been undertaken for the purpose of physical and mechanical analysis of the soil layer shrinkage.

In the later phase of this period, various theories concerning the cause of shrinkage were presented as follows:

The shrinkage of the soft soil layer was caused (i) by direct compaction due to the excessive weight of structures, or to traffic vibration in the urban areas; (ii) by natural consolidation of the soft clay layer; (iii) by the pumping up of ground water; (iv) by the variation of atmospheric pressure, tidal movement, and so on.

In 1941, when the 2nd World War Broke out, the Tokyo Metropolitan Conference for Land Subsidence Problems had made plans for land subsidence explorations, such as the construction of deeper ground water observation wells, ground water surveys by the injection of isotopes, geological explorations by means of test drilling, and so on, but these plans could not be put into practise due to advancement of the war. In 1945, the lowland area, especially the Koto district of Tokyo, was attacked and destroyed by large bombers, and thus the observations at several wells and precise levelling for the whole bench mark network within the said area was stopped, except for local levelling of the top-marks of the iron tube at the observation wells.

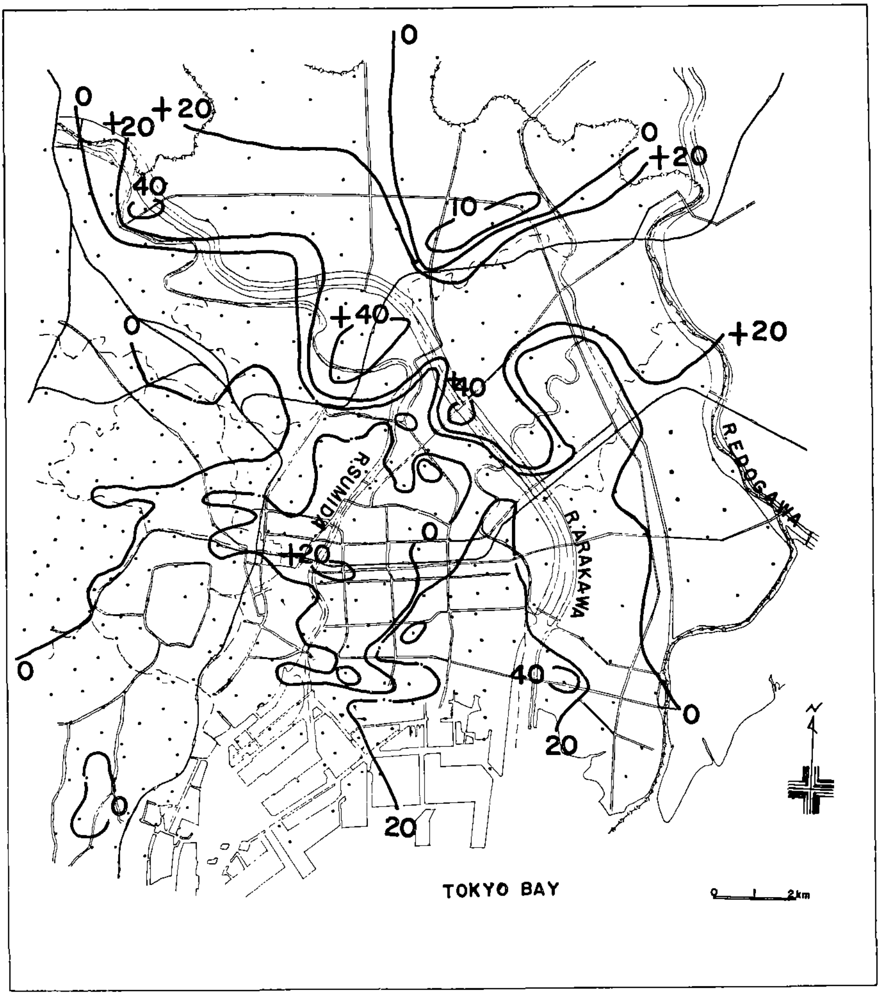


FIGURE 3. *Land subsidence during 1944-1947 (in mm)*

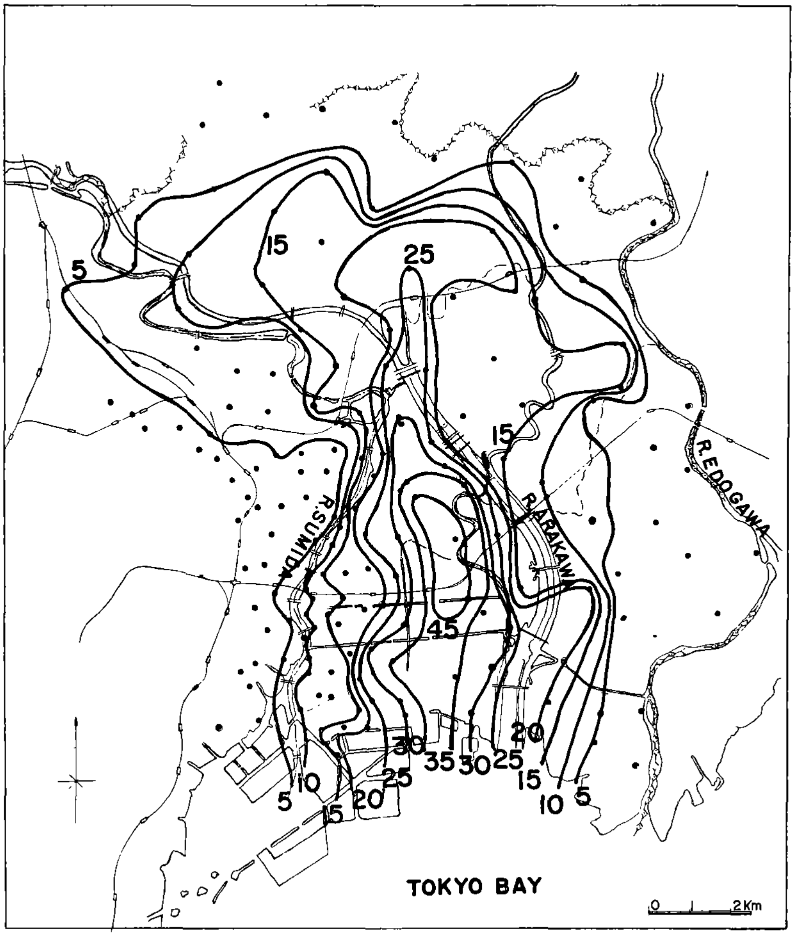


FIGURE 4. *Land subsidence during 1951-1952 (in mm)*

2.2 THE 2ND PERIOD (1946 to 1960)

During the years 1944 to 1947, no advancement of the land subsidence in the area under consideration was noticeable from the local levelling on the top-marks of the iron tubes (see figs. 1 and 3). Subsequently, the result of relevelling which was executed after 1947 showed the revival of the land subsidence (see figs 1 and 4). These facts suggested that the land subsidence of the lowland area of Tokyo had a close relationship with industrial activity in the said area, and particularly with pumping up of ground water for industrial use.

The first survey on the amount of industrial ground water withdrawal was carried out by the Tokyo Institute of Civil Engineering in 1952, and it has been continued by the Bureau of Capital City Development, Tokyo Metropolitan Government. Continuous records of the variation of the ground water table were started in 1952 at the Kameido observation well (of 60.5 m depth), and now 25 observation wells have been constructed, as shown in figure 5. As a result of the above-mentioned survey and observations, (see

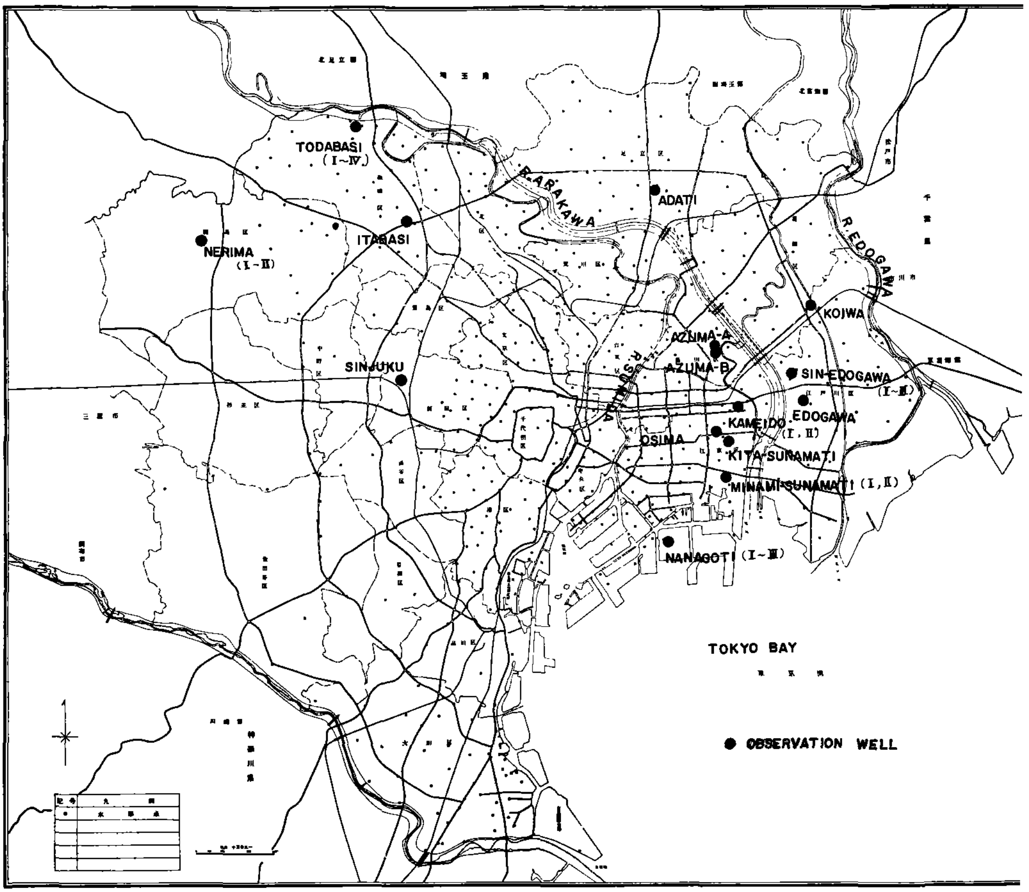


FIGURE 5. *Distribution of observation wells in Tokyo*

fig. 6), it was recognized that a close relationship exists between the lowering of the confined ground water table, the amount of the ground water withdrawal for industrial activities, and advancement of land subsidence. The amount of ground water withdrawal at the factories within the Koto delta was approximately 190,000 m^3/day in 1961; it is considered to be the maximum within the said area during this second period.

A regional hydrogeological survey in and around Tokyo was been undertaken by the Geological Survey of Japan between 1954 and 1962, the result of which was published as basic data for the control of ground water withdrawal. These surveys, however, were for the purpose of developing of ground water resources in and around Tokyo for industrial use, not for the purpose of controlling the land subsidence due to ground water withdrawal.

On the other hand, the high influx caused by typhoons had frequently invaded the lowland or the so-called "zero meter" area of Tokyo in the early phase of this period (see fig. 7). In order to construct the anti-high water embankment around the Koto Delta, the Tokyo Metropolitan Government made a series of land subsidence explorations and surveys for the five years, starting in 1951. The research, supported by the Tokyo Metropolitan Conferences for the Land Subsidence Problems, was concentrated on the estimation of the future land subsidence in Tokyo, based on Terzaghi's consolidation theory

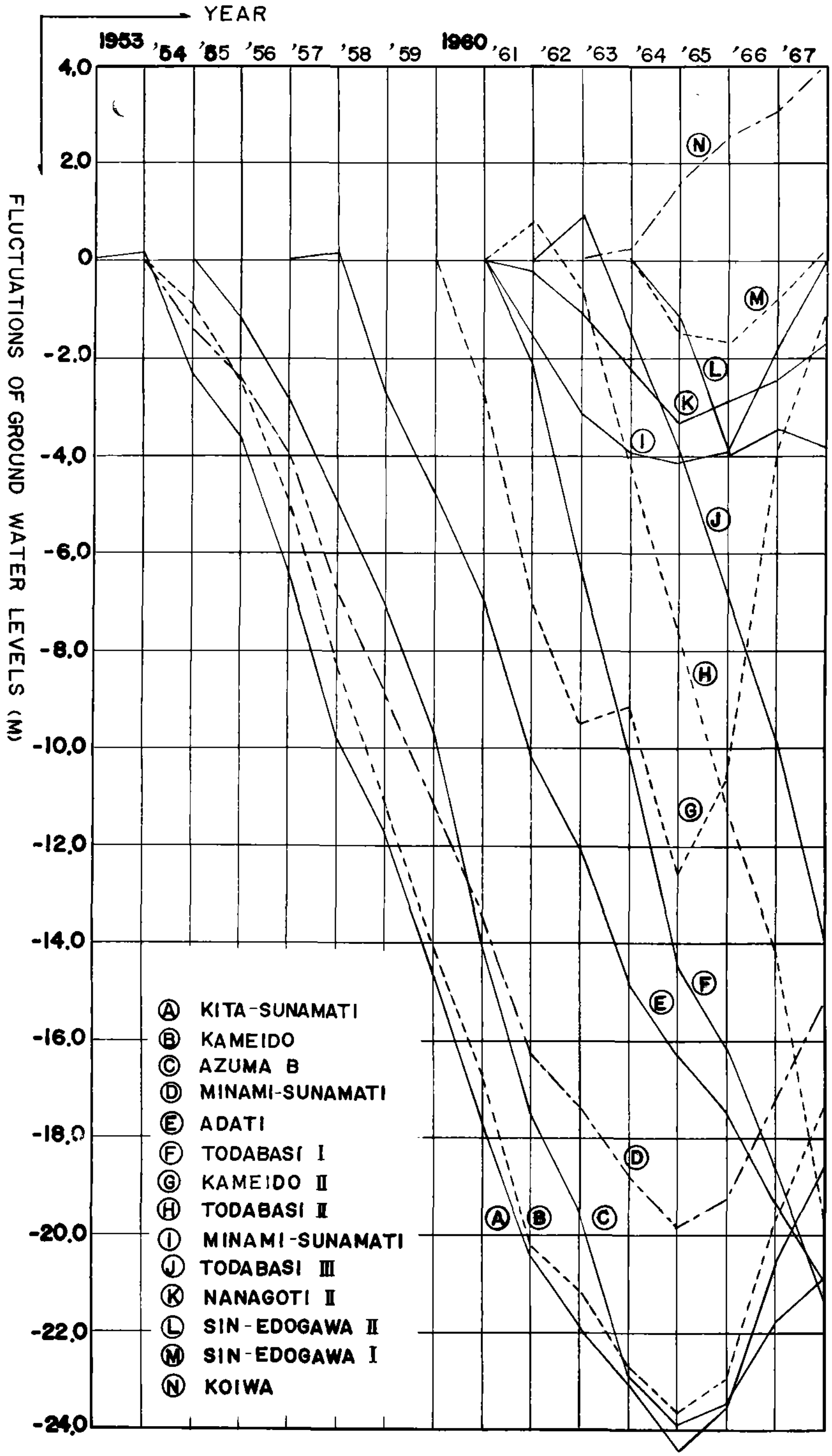


FIGURE 6. *Secular trend of ground water tables at observation wells*

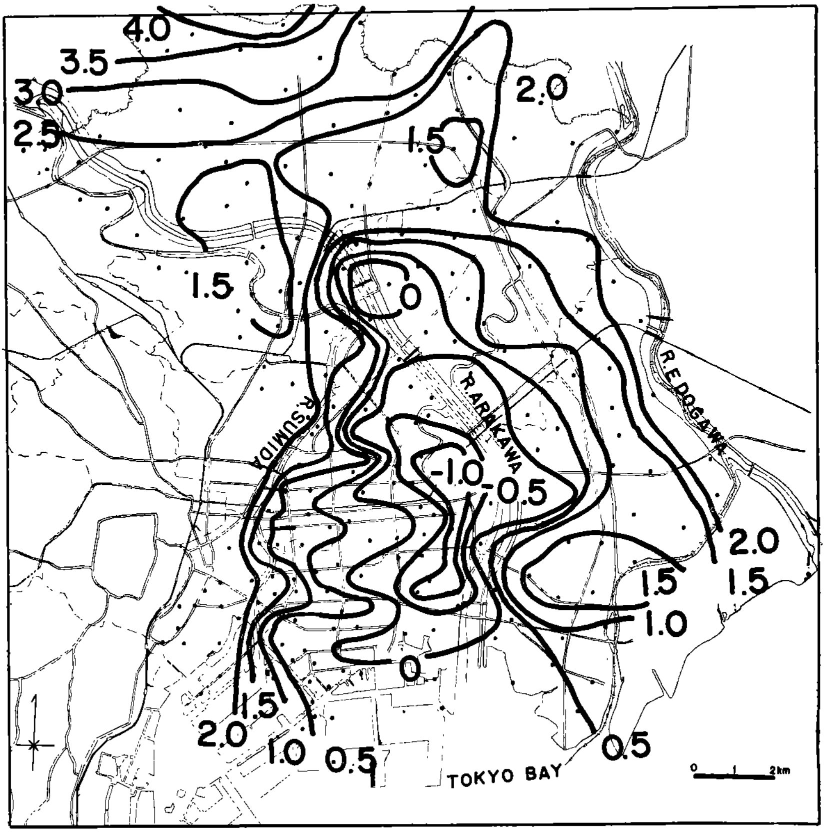


FIGURE 7. *Height distribution in the lowland Area in 1951 (in meters)*

and other methods. As a result of this research the estimated amount of the land subsidence for the five years following 1955 was approximately 1 meter. In comparison with the actual rate of land subsidence for the same five years, this estimation was re-examined by statistical analysis in 1960.

2.3 THE 3RD PERIOD (1961 TO THE PRESENT TIME)

Based on the various explorations and research, the Government decided to control the ground water withdrawal for industrial use within the Koto Delta after 1961. Since the progress of control of the ground water in Tokyo is discussed in another paper of this Bulletin (by Athara, Ugata, Tanaka and Myiazawa), the present paper will not deal with it.

Since control was applied on ground water withdrawal, the continuous observation of the ground water table at the observation wells and the repetition of precise levelling have been carried out. As a result, the ground water table of several observation wells in the Koto Delta has changed from lowering to rising, and the rate of the land subsidence has been decreasing since 1965 (see figs. 1 and 6).

Since 1952, it had been noted that the iron tube of observation wells, which were set up on the subsurface pleistocene sand and gravel or about 30 to 70 m depth, had been sinking every year. Such a phenomenon suggests that the shrinkage of the subsurface

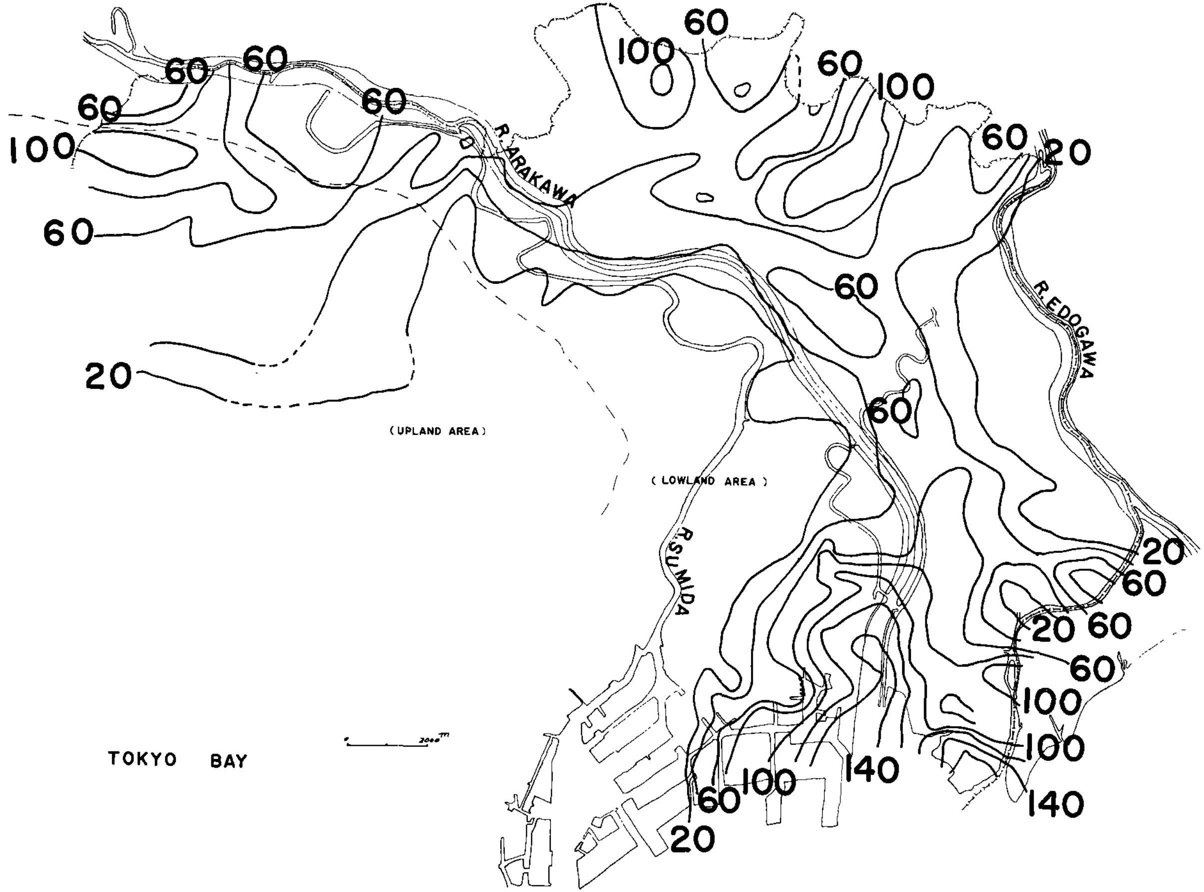


FIGURE 8. *Land subsidence during 1967-1968 (in mm)*

layers might occur not only in the holocene soft clay layer, but also in the lower or deeper pleistocene bed. A group of observation wells have been constructed in several localities for the purpose of an analysis of such a partial compaction of the subsurface layers. These are composed of two or four wells with different depth in the same locality. As a result of observation since 1961, the ratio of shrinkage between the shallower subsurface layer and the deeper one has been estimated as approximately 50%. In association with the analysis of partial compaction of the soil layer, mineralogical and paleontological analysis of soil samples of the geological and stratigraphical structures have also been undertaken.

Recently, in spite of control of the ground water withdrawal, the land subsidence and the overall lowering of the ground water table have continued, and the area of subsidence has been extending towards the surrounding lowland and upland areas as shown in figure 8. Judging from the recent geological research, the lower pleistocene deposits which contain abundant confined water are widely distributed within the large sedimentary basin of the southern Kwanto region. The removal and variation of the ground water contained in the said deposits, which might have a close relationship with the land subsidence, should be studied from a view point of hydrologic water balance within the ground water basin.

The recent and the near-future projects of the land subsidence investigation should be concentrated on an analysis of the mechanism of partial compaction of different layers and on an analysis of water balance of the ground water basin. Also, repeated precise levelling and the various observations should be continued in order to provide information on the development of the land subsidence.

RESULTS OF REPEATED PRECISE LEVELLINGS IN LAND SUBSIDENCE AREA IN TOKYO

Susumu IWASAKI, Ryozo KAIDO and Naomi MIYABE

ABSTRACT

The amounts of land subsidence in the Koto region of Tokyo, the well known subsidence area, have been measured once a year by repeating precise levelling there.

By using as data the yearly amounts of land subsidence thus measured, the gradual expansion of the subsidence area was studied after the manner in which the phenomenon of diffusion are analyzed. The local values of constants *D*, analogues to the diffusion constant, are calculated and their distributions are examined. As a result, the distribution of *D* thus obtained may be regarded as some indication of underground hydrogeological structures which are closely related to the occurrence of land subsidence.

RÉSUMÉ

Le nivellement précis et répété, effectué une fois par an, donne toutes les mesures de l'affaissement de la région de Koto, zone bien connue de la région de Tokyo pour son affaissement.

En utilisant les données annuelles de l'affaissement ainsi mesurées, on a étudié l'extension de la zone d'affaissement de la même manière que le phénomène de la diffusion a été traité. Les valeurs locales des constantes *D*, analogues à la constante de diffusion, sont calculées, et leur distribution est examinée. Par conséquent, la distribution des constantes *D* ainsi obtenues peut être considérée comme une certaine indication des structures hydrogéologiques souterraines, qui ont un rapport étroit avec la production de l'affaissement.

1. For the purpose of determining the distribution of land surface subsidence and its development, the precise levelling surveys have been carried out repeatedly in the land subsidence area in Tokyo. Changes in elevation were measured by these repeated levellings at more than 300 bench-marks in the area under consideration.

In Tokyo, such precise levelling surveys had been repeated at two years intervals during the period from 1938 to 1944, in the main part of the subsidence area, that is, the eastern part of the city of Tokyo. The precise levelling survey has been executed every year since 1951, when the development of land subsidence has brought about serious disturbance in the active industrial area.

On executing the levelling survey in the subsidence area, it is required that it be carried out as quickly as possible because the land surface undergoes continual subsidence and the measured heights at the bench marks may not present an accurate picture of condition if the survey operation takes too long. For example, if there is a difference in vertical displacements of 36 mm between two adjoining bench marks during a year, which is a modest possible amount, the height-difference between these two bench-marks may increase or decrease by about 0.1 mm a day on the average. The height-difference will consequently deviate by about 6 mm from the correct value, for the interval of two months. This amount of error, if included, is too large in comparison with the value which would be required from the specified accuracy.

It is also required that the results of the survey be as accurate as those of a first order survey operation.

The levelling survey is executed during the first three months of every year which may favour the requirement of quick survey. The accuracy of the results of these levelling surveys was tested statistically by taking frequency distribution of disclosures between forward and backward sighting from one bench-mark to another. The frequency distribution of the disclosures is obtained as shown in figure 1, from which the constant h^2 of the Gaussian distribution curve

$$f(\varepsilon) = \frac{h}{2\sqrt{\pi}} e^{-h^2\varepsilon^2} \tag{1}$$

is deduced to be 0.288, and this value of h leads to the probable error of about ± 1.1 mm for single observation. This value of the probable error may be satisfactory for the present discussion.

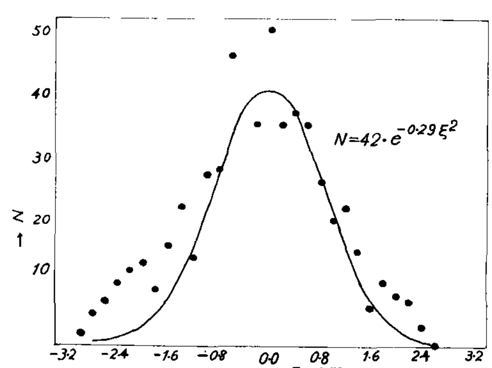


FIGURE 1. *Frequency distribution of disclosures of levelling survey operations*

2. As a result of repeated precise levellings, the areal distributions of annual subsidence are obtained for the period from 1961 to 1968, in an area of more than 100 km^2 in the eastern and northern parts of Tokyo. Of these, the distribution of subsidence during the period 1961-1962 and that during 1967-1968 are shown, as examples of distribution patterns of subsidence, in figure 2 and figure 3. Of course, the distributions of annual subsidence before 1961 also are available. They are omitted, because this paper deals with recent gradual expansion of the subsidence area correlated with the phenomenon of diffusion, by using the distribution of annual subsidence obtained for every year since 1961.

As has already been pointed out by Wadati [1], the rate of subsidence of the land surface of clayey soil layer due to the lowering of the ground-water level is given by

$$u = \partial S/\partial t = k(p-p_0), \tag{2}$$

where:

p is the pressure of the ground water in the underlying aquifer, and k and p_0 are constants.

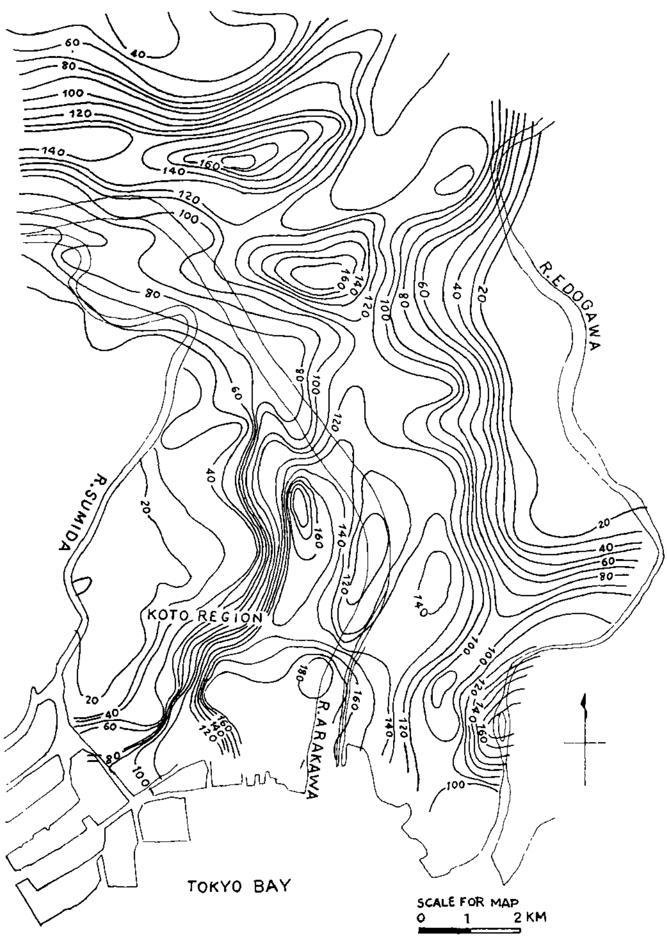


FIGURE 2. *Distribution of vertical displacements (subsidences, negative signs being disregarded) in Koto Region, for the period 1961-1962*

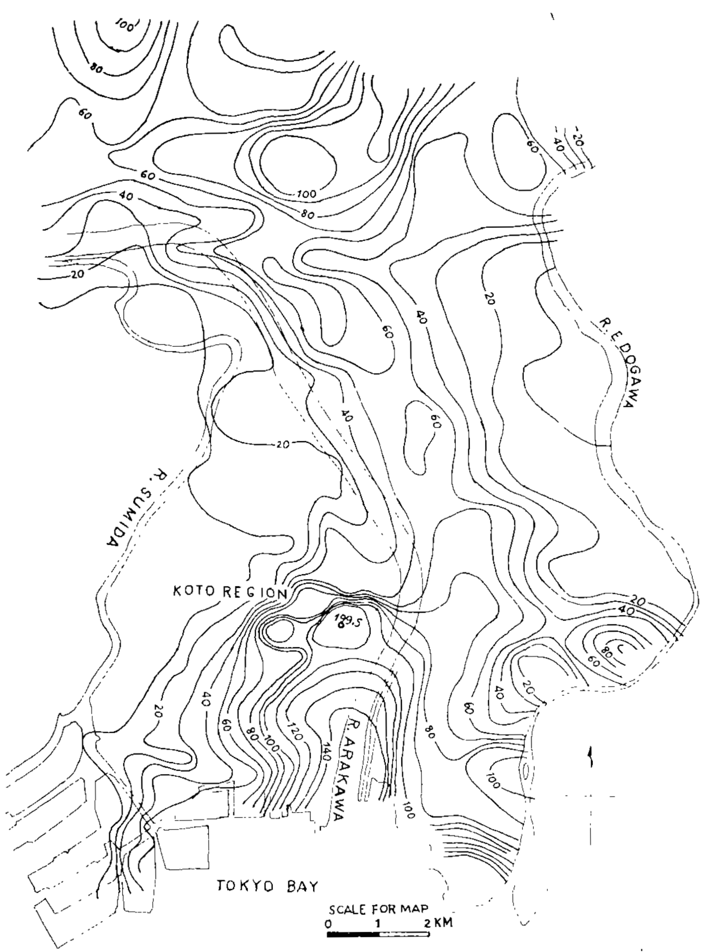


FIGURE 3. *Distribution of vertical displacements (subsidences, negative signs being diregarded) in Koto Region, for the period 1967-1968*

Since p may be regarded as proportional to the height of the ground-water level, the above mentioned relation indicates that the rate of land subsidence is proportional to the height of the ground-water level.

The equation which determines the height of the ground-water level, when the ground water is moving through the porous media, the aquifer, will be deduced from Navier Stokes's hydrodynamical equation, taking into consideration Darcy's law and assuming that the flow of water through the aquifer is sufficiently slow, so that the vertical movement of water particle is negligible and horizontal velocity of water is uniform with regard to depth. Thus we can introduce the equation which is satisfied by the height of the ground water level ζ at any place and at any time, as

$$\frac{\partial \zeta}{\partial t} = \frac{kH}{\mu}\left(\frac{\partial^2 \zeta}{\partial x^2} + \frac{\partial^2 \zeta}{\partial y^2}\right), \tag{3}$$

which is similar with the equation of diffusion.

We therefore assume that, considering a field of subsidence, the field intensity U, that may represent the yearly amount of subsidence, should satisfy an equation like (3), that is,

$$\frac{\partial U}{\partial t} = D\left(\frac{\partial^2 U}{\partial x} + \frac{\partial^2 U}{\partial y^2}\right), \tag{4}$$

where D denotes the constant analogous to the diffusion constant.

3. The constant "D", defined above, are calculated by using as the data the values of annual subsidence at bench-marks distibuted in the eastern part of Tokyo, following the procedure given below.

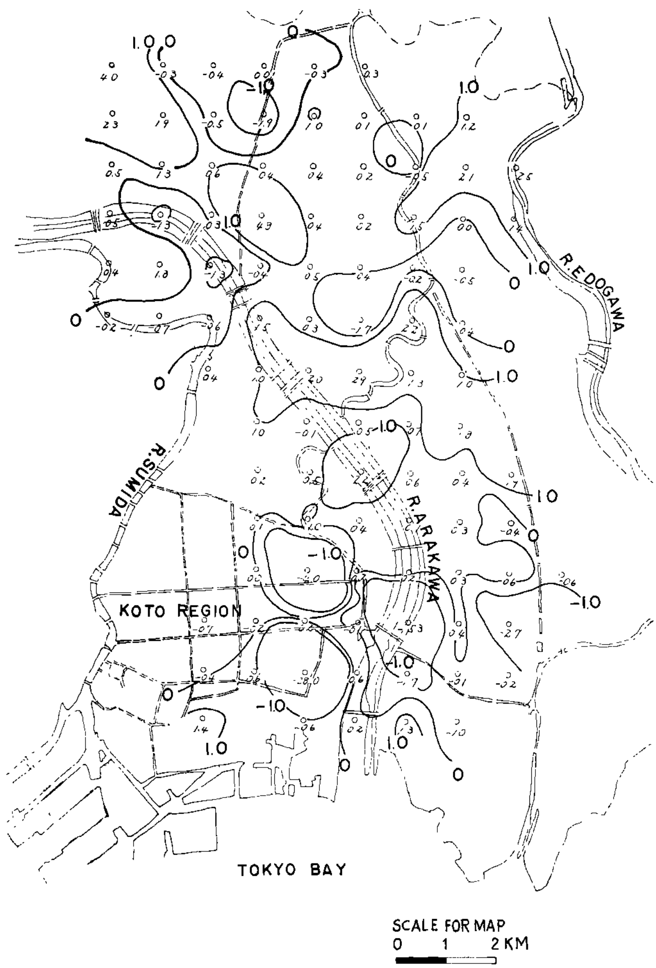


FIGURE 4. *Distribution of „D", for the period 1961-1962*

On executing the calculation, a grid of one kilometre mesh is set over the land subsidence area under consideration. From the distribution of annual vertical displacement (annual subsidence) of the bench-marks measured for the period from the epochs

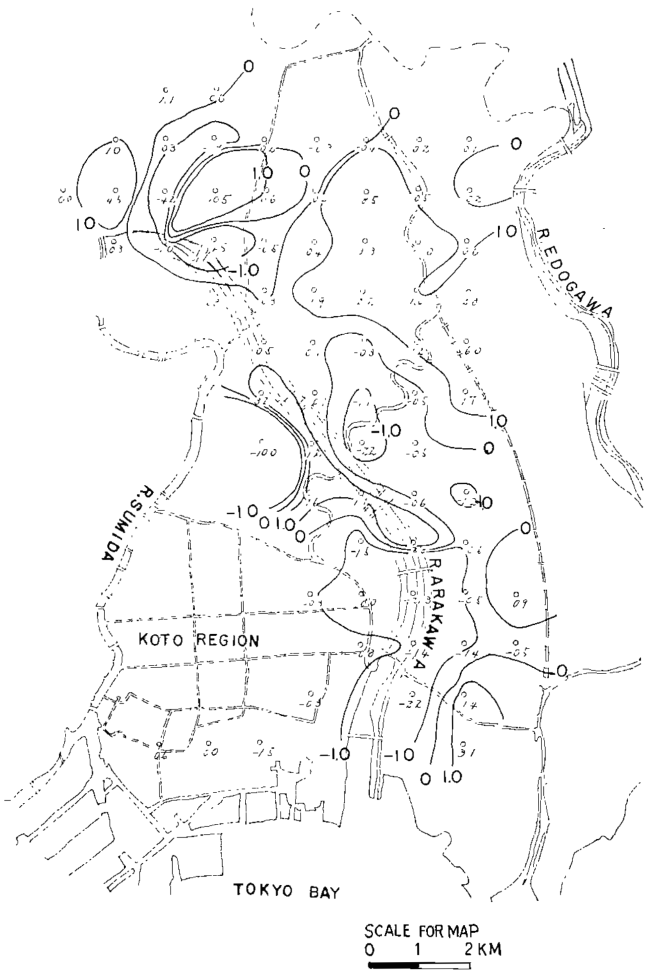


FIGURE 5. *Distribution of „D", for the period 1967-1968*

t_{k-1} to t_k, the vertical displacement in the same period at each mesh point, U_{ij}^k is estimated, of which x- and y- coordinates are designated by x_i and y_j. Then the values of $(\partial^2 U/\partial \times^2 + \partial^2 U/\partial y^2)$ are calculated by

$$(1/h^2)\,(U^K_{i+1,j} + U^K_{i-1,j} + U^K_{i,j+1} + U^K_{i,j-1} - 4\,U^K_{i,j}), \tag{5}$$

while $\partial U/\partial t$ is given by

$$(1/\tau)\,(U^{K+1}_{i,j} - U^K_{i,j}).$$

Therefore the values of D at (x_i, y_j), D_{ij} are given by

$$D_{ij} = (h^2/\tau)\,(U^{K+1}_{i,j} - U^K_{i,j}) / (U^K_{i+1,j} + U^K_{i-1,j} + U^K_{i,j+1} + U^K_{i,j-1} - 4\,U^K_{i,j}) \tag{6}$$

where τ is the time interval between the successive survey operations, normally taken as 1 year, and h the separation of adjoining mesh points, that is, one kilometre in our case. Therefore the value of h^2/τ is to be taken as unity, and the values of D_{ij} are given in the unit of km^2 per year.

Two examples of distribution pattern of D_{ij} thus calculated for 1961-1962 and 1967-1968 are shown in figures 4 and 5.

5. From the distribution of D_{ij}, it may be noted that there are several areas where the values of D_{ij} are negative. Although we are not sure what this negative D_{ij} means, it may at least be suggested that no expansion of the subsidence area will be expected in these areas of negative D_{ij}.

It may also be considered that, because the rate of land subsidence is assumed to be proportional to the effective height (negative) of the ground-water level which is largely affected by the water flow through the aquifer under consideration, the distribution of the values of D_{ij} obtained above may be some indication of the under-ground distribution of

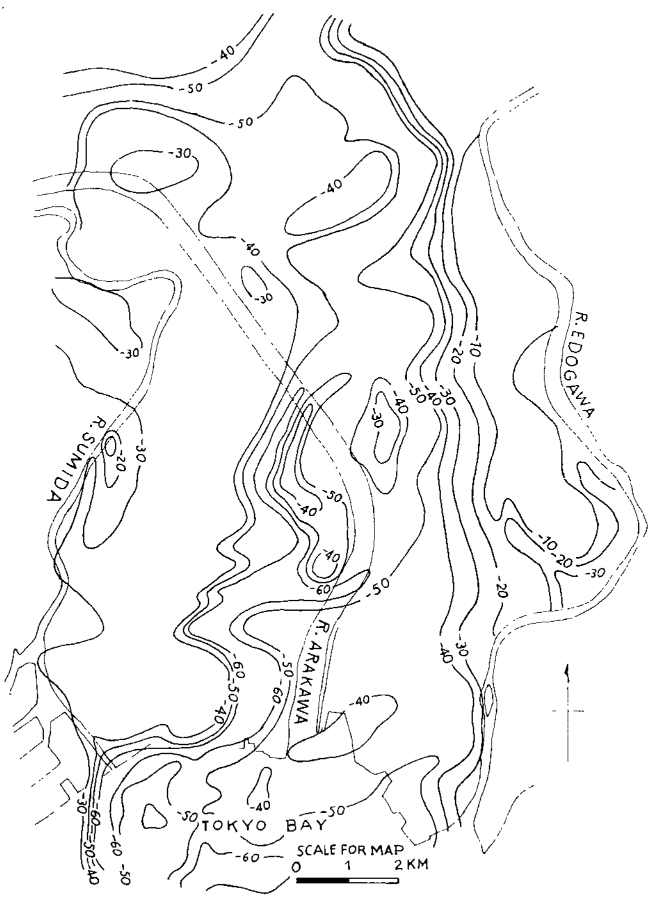


FIGURE 6. *Diluvium surface topography in Koto Region, reproduced from geological map of Tokyo, published by Tokyo Institute of Civil Engineering*

a certain parameter which controls the ground-water flow. One of the parameters considered as the controlling factor is the water permeability of the aquifer, multiplied by the gradient of ground-water pressure, which describes the lateral flow of ground water according of Darcy's law. The distribution of D_{ij} obtained above may therefore be some indication of the distribution of the ground-water flow or that of the water permeability within the aquifer concerned. If so, the distribution of D_{ij} may have some relation to the underground geological features, since the water permeability within the aquifer is dependent on the size of particles which constitute the soil layer and its compactness.

Referring to the geological map of Tokyo [2], the geological structure in the region under consideration is such that the water bearing sandy layer of diluvial deposits is covered with the clayey soil layer of alluvial deposits.

Therefore, the configuration of contour lines which represent the surface topography of diluvium, shown reproduced in figure 6, may have some relation to the distribution of

The fact that there are several regions where the values of D_{ij} are in turn positive and negative, cannot yet be explained. When this question is answered, the solution of the problem of unusual distribution of land subsidence will be approached. The complete explanation of areal extension of land subsidence will be approached by further advancement of research on the related phenomena, such as the behaviour of the ground water on the basis of water balance and so on.

In conclusion, the authors wish to express their sincere thanks to the colleagues and the staff members of Tokyo Institute of Civil Engineering for their kind assistances in preparing this report.

D_{ij} given in figures 4 and 5. On comparing the distribution of D_{ij} with the diluvium surface topography, it may be noted that the areas of negative D_{ij} tend to be concentrated in the zone of the diluvial valley, and in the coincident industrial centre, where the withdrawal of the ground water is most actively carried on. It is also observed that, in several parts of the region, the values of D_{ij} are always positive, while, in some other parts, the values of D_{ij} are sometimes positive and sometimes negative, though the diluvium topographic features which favour the distribution of D_{ij} is not observable.

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LAND SUBSIDENCE IN OSAKA

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ABSTRACT

One of the typical examples of land subsidence in Japan can be seen in Osaka city, where the maximum subsidence amounts to about 2.5 m during the last 25 years. Such remarkable ground subsidence is due mainly to the consolidation of clay strata caused by the decrease in the artesian pressure. This can be confirmed by the results of the following investigation.

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(1) There was a close correspondence between the level of ground water and the actual rate of subsidence. When the water level recovered, the subsidence became slight and sometimes it stopped.
(2) The distribution of the pre-consolidation pressure in the normally consolidated alluvial clay in Osaka shows the decrease in the artesian head in the aquifer.

In the Symposium the following topics on the land subsidence in Osaka will be reported:

(*a*) History and outline of the land subsidence;
(*b*) Geological constitution and some physical properties of earth materials;
(*c*) Review of research on the land subsidence in Osaka;
(*d*) Field instrumentation and the results measured;
(*e*) Regulation against the use of ground water and its result on the land subsidence.

RÉSUMÉ

L'un des exemples typiques d'affaissements au Japon se trouve à Osaka, où la subsidence maximale s'est élevée à environ 2,5 m au cours des 25 dernières années. Cet affaissement est dû en ordre principal à la consolidation des couches d'argile causée par la diminution de la pression artésienne. Ceci peut être confirmé par les résultats des recherches suivantes :

1. Il y a une correspondance étroite entre le niveau de l'eau souterraine et le taux actuel d'affaissement. Quand le niveau de l'eau remonte, l'affaissement devient faible et parfois s'arrête.
2. La distribution de la pression de pré-consolidation dans l'argile alluviale normalement consolidée montre la diminution de la pression artésienne dans l'aquifère.

Les points suivants sur les affaissements à Osaka seront présentés :

a) Histoire et description des affaissements ;
b) Constitution géologique et quelques propriétés physiques des échantillons de sol ;
c) Revue des recherches sur les affaissements à Osaka ;
d) Instrumentation et résultats de mesures ;
e) Mesures régulatrices de l'usage de l'eau souterraine et leurs résultats sur les affaissements.

1. HISTORY AND OUTLINE OF THE LAND SUBSIDENCE

Osaka, the second largest city in Japan, has developed on an alluvial plain of ancient sediment from the Rivers Yodo and Yamato. The alluvial layer consists in the upper part of a sand stratum several meters thick, originally deposited as the top bed of a recent delta, and in the lower part as an alluvial clay. The average thickness of the clay layer is about 15 m, but it becomes thicker as it goes nearer to the coastel zone. Below the Alluvium there is a very thick Diluvium deposit of Pleistocene age, which consists of alternating layers of sand and clay.

In early years, the ground water level in the City was very high and it is reported that even artesian wells could be found in some parts of the City. However, the use of ground water was gradually increased and land subsidence due to the withdrawal of the ground water began to appear.

In the period from 1885 to 1928, when pumping of underground water was not so heavy as in later years, the rate of subsidence in the City was very slight, holding at the almost uniform rate of 6 to 13 mm per year. This slight subsidence is considered to be the result of the natural movement of the earth's crust and the natural consolidation of the newly deposited alluvial clay. After 1928, however, the rate of subsidence increased markedly with increasing use of underground water. Investigating the old data of leveling, the presage of the subsidence due to withdrawal of underground water can be seen in about 1928. This date almost coincides with the time when the use of underground water for industrial purpose became active. Subsequently, a remarkable rate of subsidence began to appear in about 1934, From this period, the precise leveling for the wider part of the City was frequently performed by the Municipal Authorities, following the suggestion of

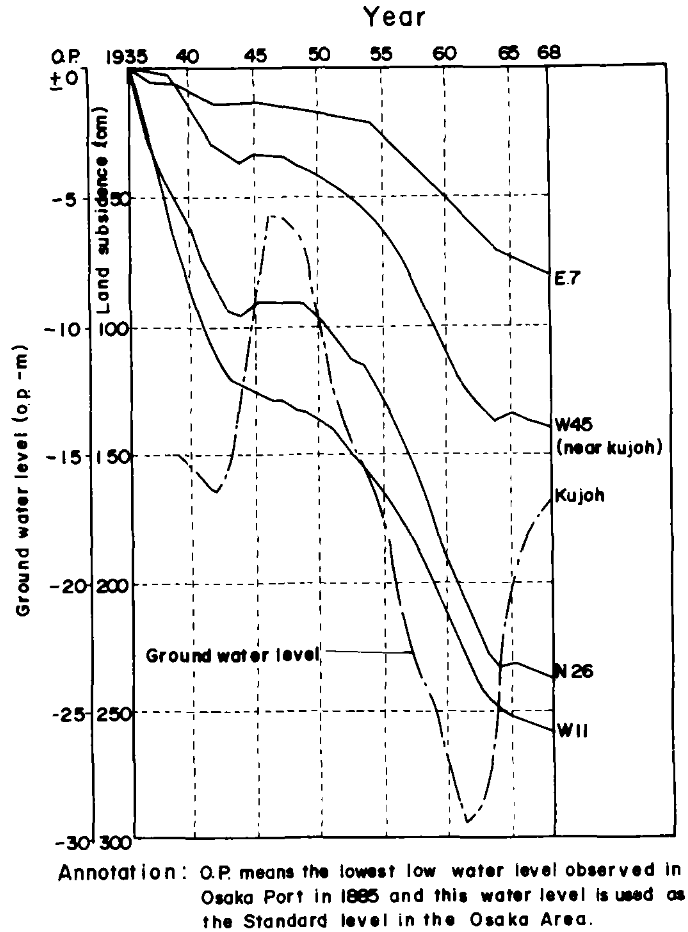

FIGURE 1-1. *Annual variation of ground water level and amount of land subsidence in Western Osaka*

Prof. M. Imamura. Besides such leveling, the consolidated amount of the soil layer and the artesian head in the aquifer at O.P.[1]—176 m were observed by the apparatus which had been installed by Dr. K. Wadachi at Kujho-Park in the City in 1938. Figure 1-1 shows the amount of land subsidence of various bench marks in Western Osaka and the annual variation of the artesian head at the Kujoh-well stated above. From this figure, the annual amount of land subsidence in the early period amounts to more than 15 cm. Figure 1-2 shows the monthly variation of the quantity of the water pumped in Osaka City, the variation in artesian head, and the rate of land subsidence measured at the Kujho-well. In figure 1-2.1, the quantity of industrial water supplied by the newly established Industrial

1. O.P. (Osaka Peil) means the lowest low-water level observed in Osaka Port in 1885 and this level is used as the standard datum in Osaka area.

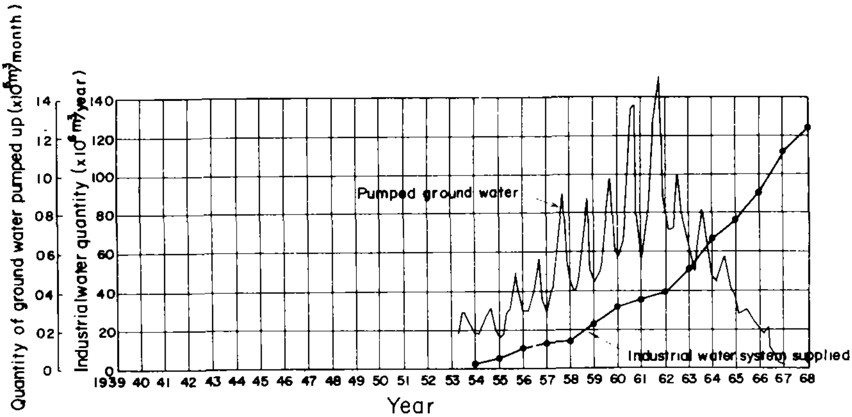

FIGURE 1-2.1. *Monthly variation of water quantity pumped from wells and variation of water buantity supplied by industrial waterworks in Osaka City*

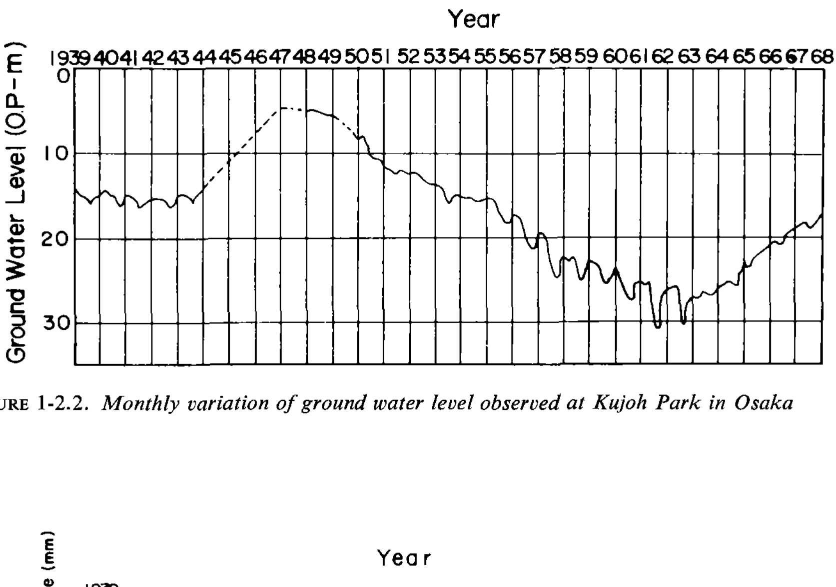

FIGURE 1-2.2. *Monthly variation of ground water level observed at Kujoh Park in Osaka*

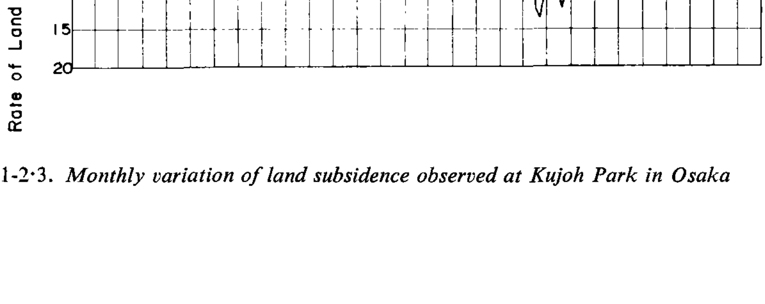

FIGURE 1-2·3. *Monthly variation of land subsidence observed at Kujoh Park in Osaka*

Water Works of Osaka is also illustrated. The peak points in figure 1-2.1 and figure 1-2.2 lie in 1961, while that in figure 1-2.3 lies in 1958. In spite of some unconformity in the details, however, the general variation in the artesian water level and that in the rate of subsidence show a close correspondence and suggests the existence of strong relationship between both phenomena. In figure 1-2.3, the period when land subsidence stopped corresponds to the period of recovered level of the underground water (fig. 1-2.2). This was caused by pumping being stopped due to the destruction of the City by the heavy bombing in the World War II. The total amount of subsidence during thirty five years from 1935 to 1968 in Osaka is shown by the isopleth in figure 1-3. The figure shows that the subsidence is larger nearer the coastul zone, and that a zone of almost no subsidence is left in the hilly part in the middle of Osaka, where the very thin alluvial layer is covered.

The pumping wells have various depths, but most wells are shallower than −200 m. The use of underground water fluctuates every day, every week, and every season. This

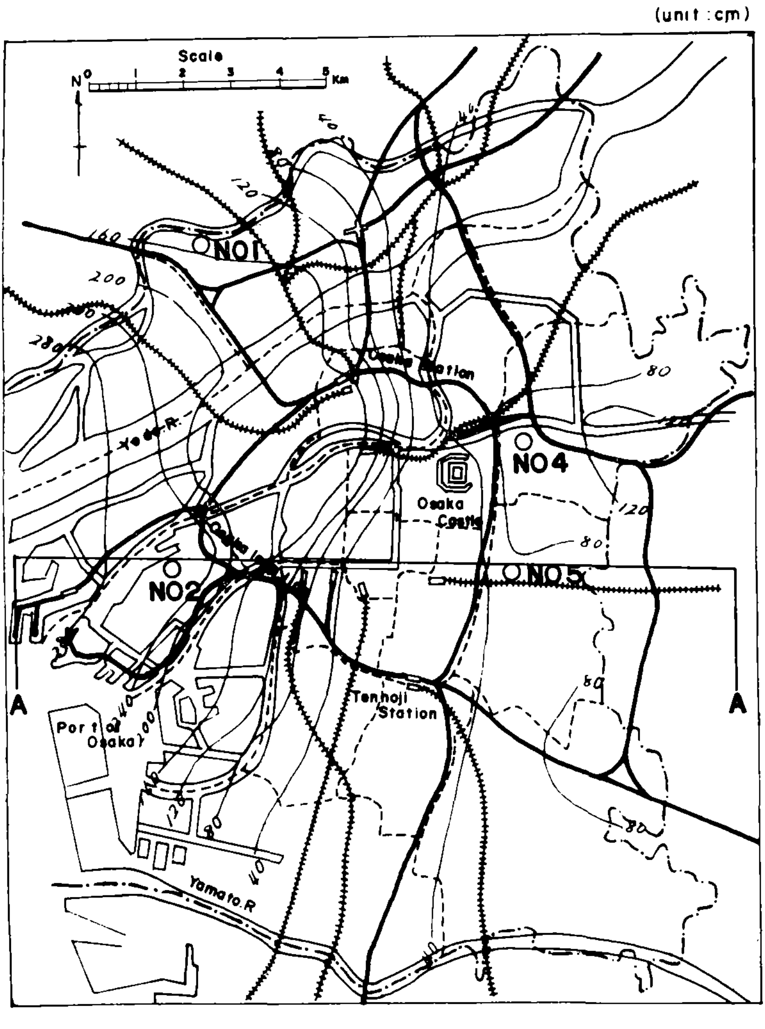

FIGURE 1-3. *Isopleth of amount of land subsidence in Osaka City from 1935 to 1968*

is reflected in the artesian water level—levels during the night, Sunday and winter were mostly higher than usual.

Due to the remarkable land subsidence stated above, the ground height of a part of Western Osaka has lowered below sea level and the City is exposed to the danger of flooding caused by a high tide in Osaka Bay. Figure 1-4 shows the ground height of Western Osaka in 1961.

The flood feared was unfortunately realized in 1934 by the Muroto Typhoon, one of the biggest typhoons which have ever attacked Osaka. As a result an area of about 49 sq km was flooded by the high tide of O.P. +4.20 m.

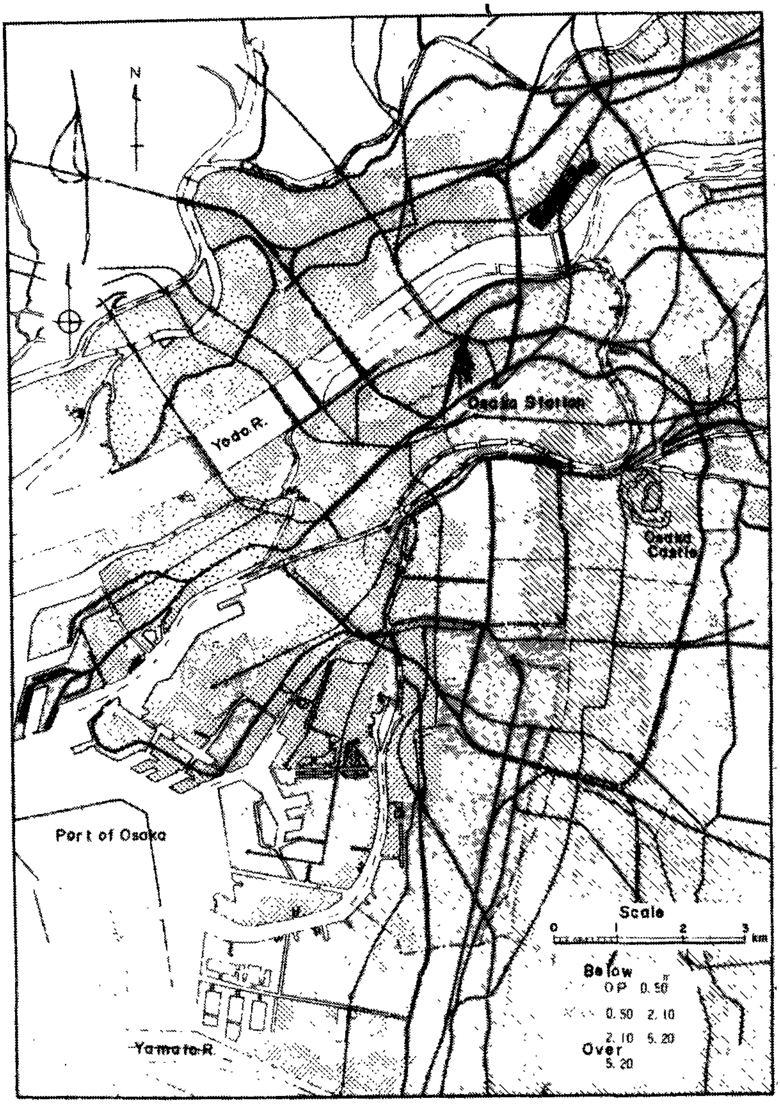

FIGURE 1-4. *Ground height of Osaka City in 1961*

In order to prevent such a disaster in the future, more satisfactory dikes have been repaired and built. Besides these prevention works, the use of underground water has gradually been regulated in phase with the completion of the industrial water-supply works planned as the substitute of the underground water. (see fig. 1-2.1).

Owing to the regulation against the use of ground water in Osaka City land subsidence in the City gradually decreased and is almost stopped at present. This result can be recognized by comparing figure 1-3 and figure 1-5; the latter shows the isopleth of the amount of land subsidence from 1963 to 1968.

In spite of the success in preventing the land subsidence in Osaka City, however, the land subsidence in Eastern and Northern Osaka has increased remarkably during the last few years because these regions have been developed lately and many factories using much

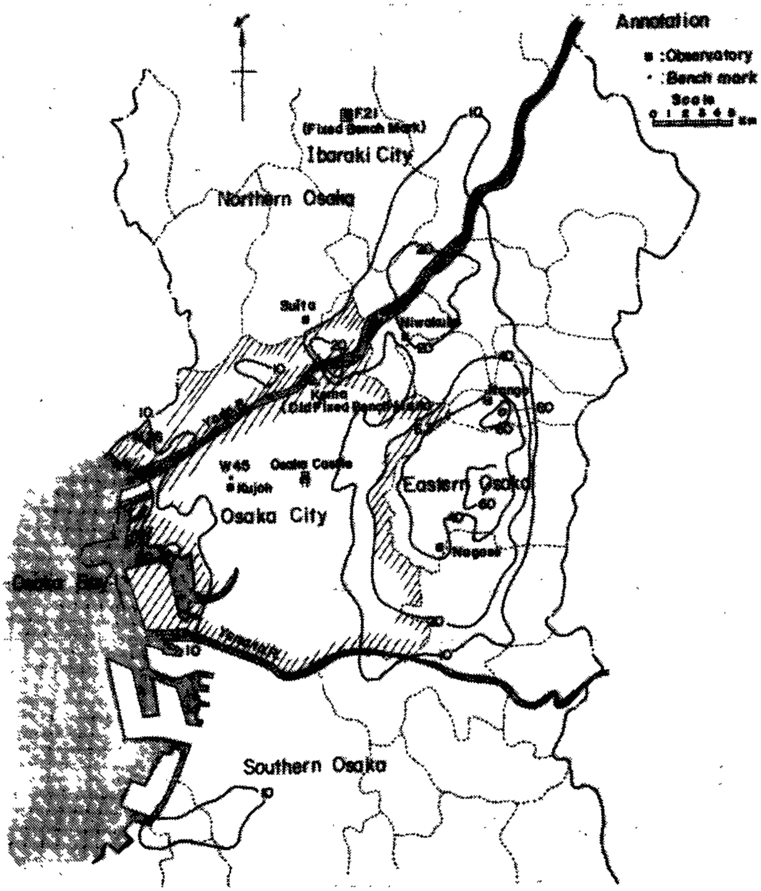

FIGURE 1-5. *Isopleth of amount of land subsidence in Osaka area from 1963 to 1968 (unit: cm)*

underground water have been built. Figure 1-6 shows the amount of land subsidence in Eastern and Northern Osaka measured since 1965 at several bench marks. This figure shows that the subsidence at Nagase observation station come sup a remarkable 220 mm during only three years. Figure 1-7 gives the monthly variation of ground water level and the rate of land subsidence of the several observation stations in Eastern and Northern Osaka. From figures 1-7.1 and 1-7.2, it seems that the shallower the ground water level is, the less the rate of land subsidence becomes.

In order to prevent land subsidence in Eastern and Northern Osaka, the use of underground water for industrial purposes has been regulated by law since 1965 in the Northern

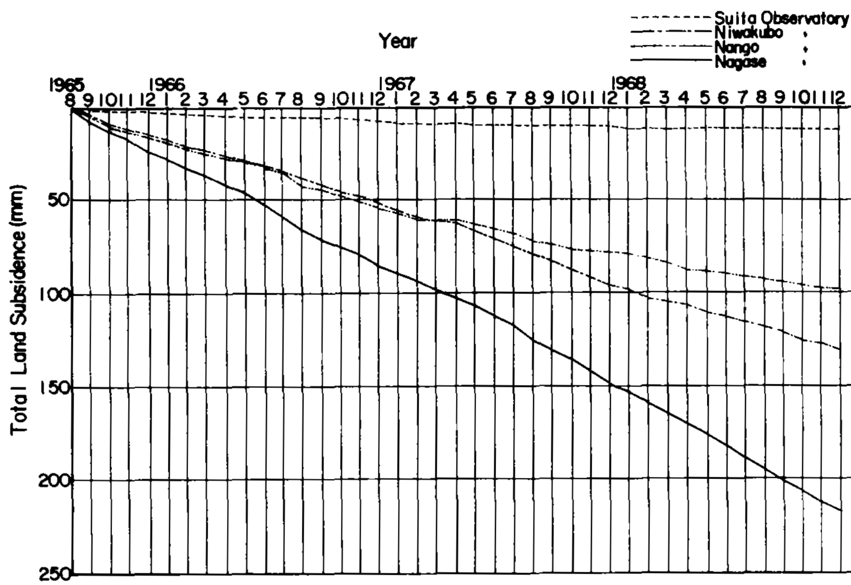

FIGURE 1-6. *Amount of land subsidence in Northern and Eastern Osaka*

region and since 1966 in the Eastern region. The industrial water supply service to substitute for the underground water has been proceeding and the preventive effect should be realized in the near future.

The annual land subsidence from 1967 to 1968 is illustrated in figure 1-8. This figure shows that the land subsidence in Osaka City has almost stopped except for the

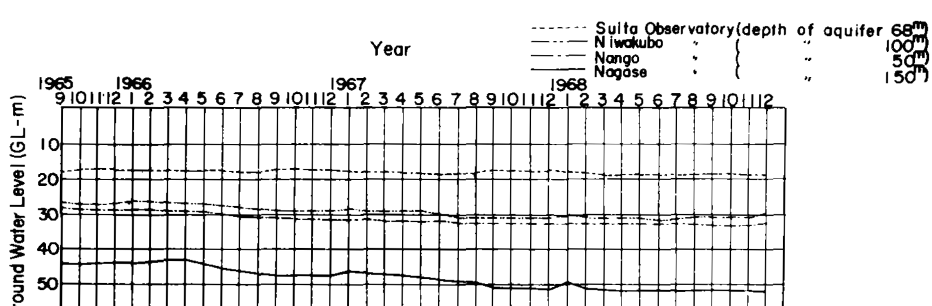

FIGURE 1-7.1. *Monthly variation of ground water level in Northern and Eastern Osaka*

coastel zone arround the Port of Osaka. However, the land subsidence in Eastern Osaka is still active and there is a large area of land subsidence whose maximum amount is 16 cm. Besides these land subsidences, it can be found that a new area around Kishiwada City in the Southern Osaka is developing. Though the size of the area is still very small, it may be important to pay close attention to it.

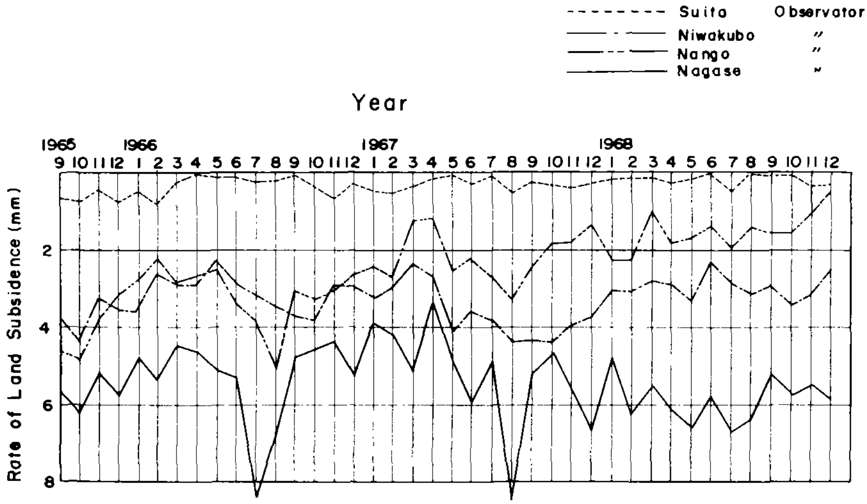

FIGURE 1-7·2. *Monthly variation of land subsidence in Northern and Eastern Osaka*

In order to protect the City from high tides, as described previously, high-tide preventive dikes with a height 5 m above O.P. were constructed along the bay coast and both sides of rivers running through the City by the Osaka Prefectural Government and the Osaka Municipal Office. The total length of those dikes amounts to about 124 kilometers.

Furthermore, in order to withstand a severe high tide caused by an extra large typhoon, the Osaka Prefectural Government now has a project to raise the height of the existing high-tide preventive dikes up to 6.6 m above O.P. and is constructing high-tide preventive locks across the rivers at the most effective down-stream sites.

2. GEOLOGICAL CONSTITUTION AND SOME PHYSICAL PROPERTIES OF EARTH MATERIALS

(1) SUBSURFACE GEOLOGY OF OSAKA BASIN

By correlating the deep boring logs and geological investigations, the generalized stratigraphy of the Osaka Basin can be shown in table 2-1. As for the shallow part of the subsurface structure, its geological profile along the line A-A in figure 1-3 is shown in figure 2-1. As shown in this figure, the alluvial layer of about 20$\sim$30 m thick is covering the main part of the ground surface. Beneath the alluvial layer is a diluvial layer which belongs to the Pleistocene age and is of thicknesses up to several hundred meters, as shown in the boring logs of figure 2-2. In the plio and middle- early Pleistocene sediments, there generally exist 10 marine clay layers which are numbered in series as Ma-1, Ma-2, ... and Ma-10 from bottom to top. Layer Ma-3, contains a special tuff of russet colour which is designated as Azuki-tuff and is used as a key bed. The Pleistocene deposit above Azuki-tuff is conventionally designated as the upper division of the Osaka Group.

Classifying the clay layers from their preconsolidation stresses, the clay in the alluvial layer belongs to the normally consolidated clay and that in the diluvial layers to the overconsolidated clay, Therefore, the alluvial layer has the most effect on the land subsidence

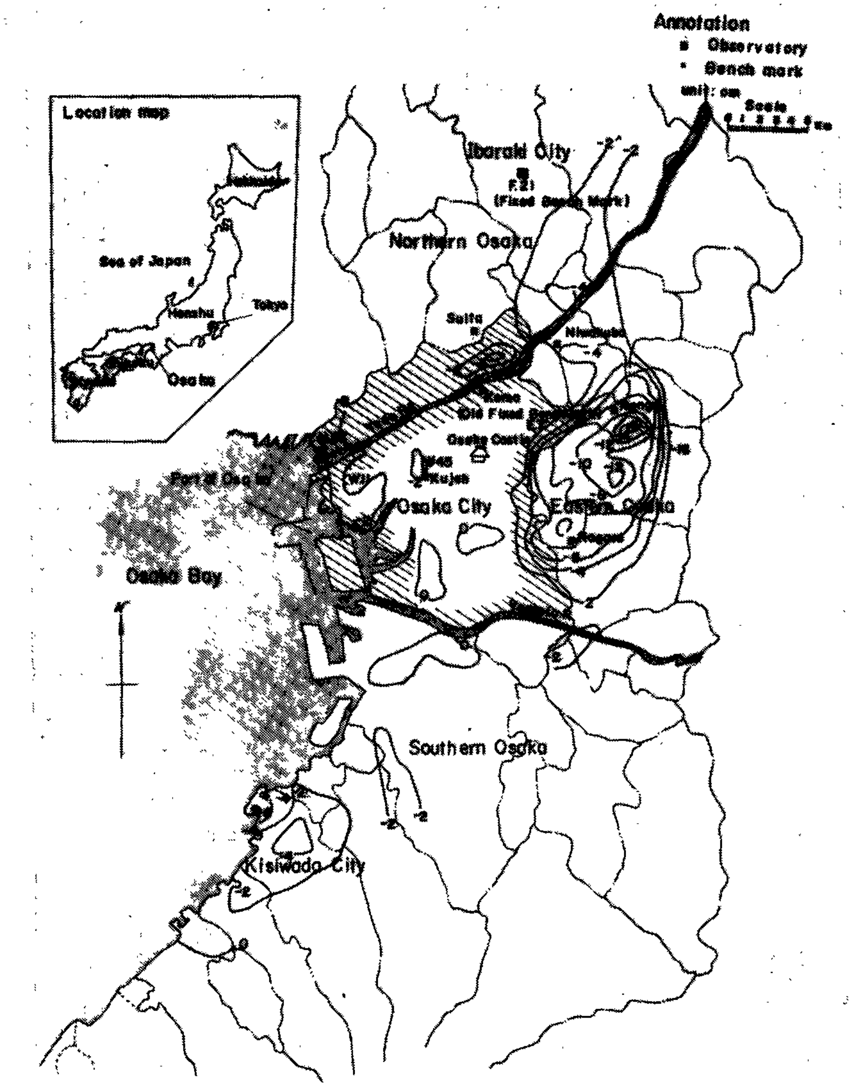

FIGURE 1-8. *Isopleth of land subsidence in Osaka area (1967-1968)*

and the layer about 100 m deep, which is supposed to be a deposit of the hidden terrace, also has a serious effect on it. Furthermore, it seems that the upper division of the Osaka Group also has a similar effect on the subsidence.

(2) General properties of clay

The compressibility of Osaka clay was investigated on undisturbed samples obtained from deep borings; Boring No. 1 of 700 m depth in Northern Osaka, No. 4 of 500 m depth, and No. 5 of 700 m depth in Eastern Osaka.

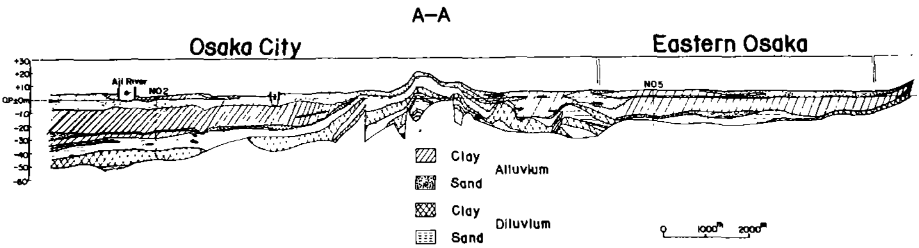

Figure 2-1. *Geological profile along A-A line in figure 1-3*

As shown in figure 2-3, most of the clays in Osaka can be classified into inorganic clays of high plastisity when their liquid limits and plastic indices are plotted on the plasticity chart.

Their activity numbers (A), or the ratios of the plastic index to the percentage of clay fraction, are shown in figure 2-4. As shown in this figure, most of clays belong to the active clay, since the A is greater than 1.25. In general, the larger the activity number of clay is, the greater becomes its volume change due to the decrease in its water content from the liquid limit to the shrinkage limit. Numbers affixed to the plots in figures 2-3 and 2-4 show the series numbers of marine clays shown in figure 2-1.

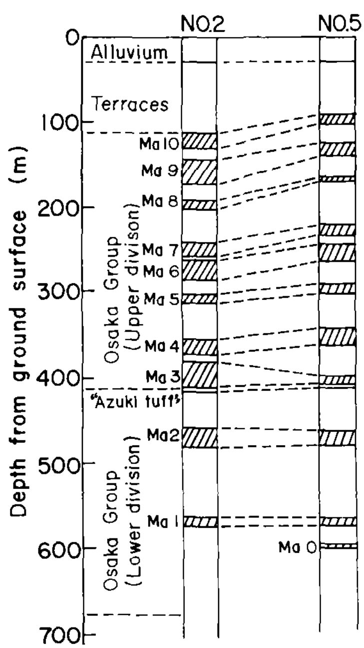

Figure 2-2. *Boring logs of Borings No. 2 and No. 5*

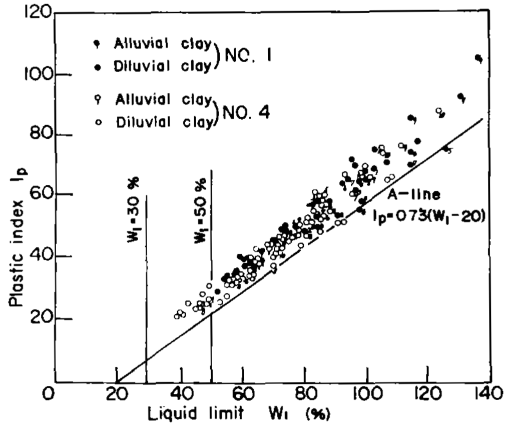

FIGURE 2-3. *Liquid limit and plastic index of Osaka clays plotted in plastisity chart*

The consolidated volume ($\Delta V/V$) of the normally consolidated clay due to the stress ($p_c + \Delta p$) can be represented by the following equation

$$\frac{\Delta V}{V} = \frac{C_c}{1+e} \cdot \log \frac{p_c + \Delta p}{p_c},$$

where:

p_c the preconsolidation stress of the clay;
e the void ratio of the clay before consolidation, and
C_c the compression index.

The values of C_c obtained by standard oedometer tests are plotted against liquid limit (w_e) as shown in figure 2-5. From this figure the following rough relation may be obtained on an average,

$$C_c = 0.017\,(w_c - 37).$$

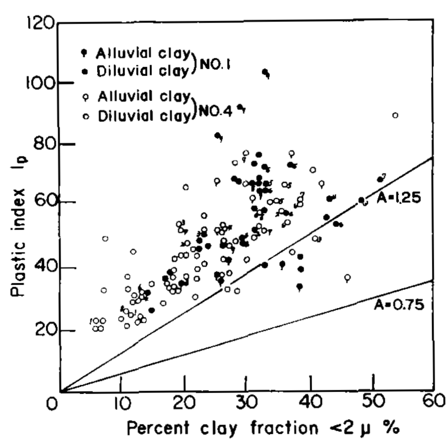

FIGURE 2-4. *Activity numbers of Osaka clays*

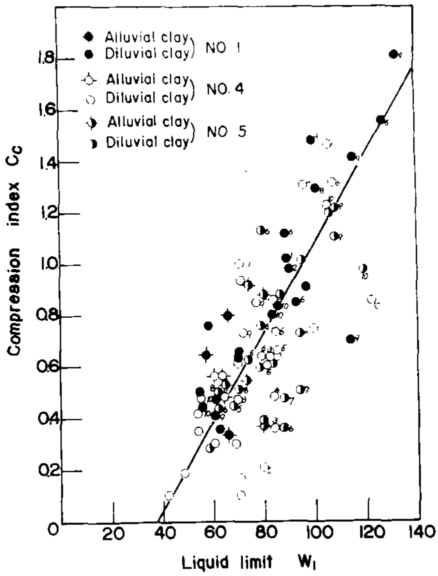

FIGURE 2-5. *Relationship between compression index C_c and liquid limit w_e*

Figure 2-6 shows the distribution of preconsolidation stress, p_c, plotted against the depth from the ground surface. In this figure, p_c seems to increase in proportion to the depth and is generally larger than the present effective over-burden pressure. However, the plots of p_c for the alluvial clays existing shallower than about 20 m lie on the over-

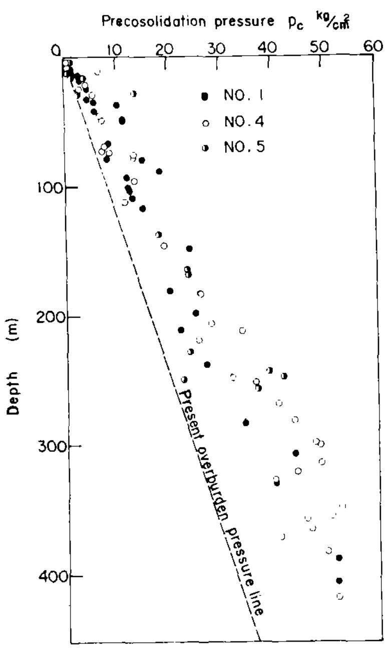

FIGURE 2-6. *Distribution of preconsolidation stress measured by oedometer tests*

TABLE 2.1. Generalized stratigraphy of the Osaka district

Age	Formation
Recent	Alluvium
Late Pleistocene	Terraces Hidden Terraces ?
Middle-Early Pleistocene	Osaka Group (Upper division) "Azuki tuff"
Plio-Pleistocene	Osaka Group (Lower division)
Pliocene Late Miocene	Nijo Group Kobe Group
	Basement complex (chiefly granitic)

burden pressure line. Therefore, it can be said that those clays shallower than about 20 m belong to the normally consolidated clay and those deeper than about 20 m to the over-consolidated clay.

(3) THE EFFECT OF LAND SUBSIDENCE ON THE PRECONSOLIDATION STRESS

Generally, the total stress applied to a clay due to the over-burden load is supported by both pore water pressure and effective stress, where the effective stress means the inter-particle stress of the clay. Therefore, if the pore water pressure is decreased due to the lowering of the artesian head, the effective stress of the clay is increased by the same amount as the decrease in the pore water pressure. Consequently, the clay is consolidated in proportion to the increase in the effective stress.

The maximum effective stress which was ever applied to a clay is termed the pre-consolidation stress of the clay. This pre-consolidation stress of a clay located at a certain depth in the ground can be measured by the oedometer test on the undisturbed clay sample. From this stress, the pore water pressure, u, at that depth can be calculated as the difference between the total stress and this stress. Figure 2-7 shows the p_c-line, or the

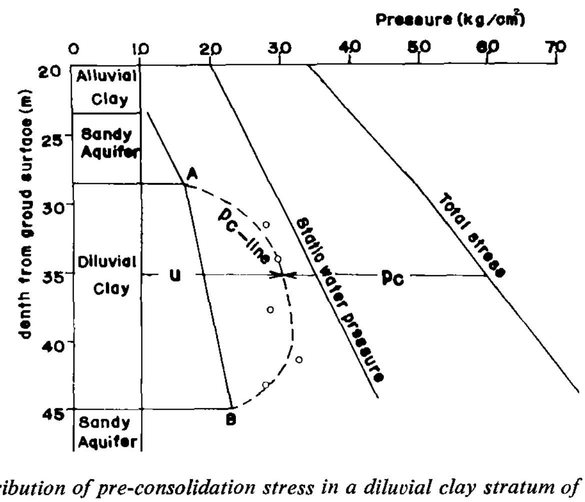

FIGURE 2-7. *Distribution of pre-consolidation stress in a diluvial clay stratum of land-subsided area*

distribution curve of the pre-consolidation stress, of a diluvial clay at a location in Osaka City.

In figure 2-7, the "total stress line" is calculated from the wet densities of undisturbed soil samples, the "static water pressure line" is drawn from the origin whose level is taken at the original underground water surface before land subsidence, the "p_c-line" is obtained by plotting the pre-consolidation stress p_c leftwards from the "total stress line", and the "AB-line" is drawn by connecting the underground water levels in the upper aquifer and the lower one at the time when samples were obtained. From this figure, it is suggested that the consolidation in the clay stratum, or the land subsidence, was in progress at that time as far as the artesian heads remained as they were. From the "p_c-line", moreover, the degree of consolidation at that time and the subsequent amount of settlement due to the future consolidation of the clay stratum can be calculated by applying the theory o consolidation under an assumed relative position of "AB-line".

3. REVIEW OF RESEARCHES ON THE LAND SUBSIDENCE IN OSAKA

There has been a variety of research and investigation on land subsidence in Osaka. Some scientific results of these studies will be introduced in this section and publications are listed under "references" at the end of this paper.

The first investigation of the land subsidence in Osaka was performed by K. Wadachi as chief of the science section in the Natural Disaster Research Institute, Japan Science Promotion Society. He established the instrumentation (fig. 4-4) to observe the land subsidence and the artesian head in the aquifer at Kujoh Park in 1938, and verified that there was a linear relationship between the rate of land subsidence and the head difference between the artesian head and a certain level. In order to analyse such a relationship, he and T. Hirono extended the basic equation of consolidation proposed by Terzaghi by assuming the clay skeleton as a Voigt body (1942).

Y. Ishii, as a research engineer in the Ministry of Transportation, explored the Osaka soil, mainly throughout the alluvial layer. Besides, he also analysed the basic equation of consolidation by assuming another mechanical model (1949). This model consists of the series connection of a elastic spring and a Voigt element with a Newtonian viscosity. By comparing the coefficients of the model with the results of soil testing on undisturbed samples obtained by the exploration boring, he described the characteristics of land subsidence in Osaka.

As for the rheological property of clays, S. Murayama and T. Shibata proposed a new model consisting of an independent Hookean spring connected in series with a modified Voigt element, with the viscosity element in the Voigt model being deduced statistically by applying the rate process. The increase in the elasticities of the proposed model due to the consolidation was also analysed (1958, 1964).

Besides the above stated research on the role clay layers have in land subsidence, the influence of sandy aquifers on the land subsidence was investigated by S. Hayami, a professor in Kyoto University. He observed the variation in the modulus of compressibility of the sandy aquifer through repetitious pumping tests on the well in the field (1952-1955). Because the modulus of compressibility of the sandy aquifer during the draw-down period of the underground water level was found to be larger than that of the recovery period, he supposed that the sandy aquifer was compacted by the fluctuation in the artesian head. In order to verify these conclusion he and K. Akai performed model tests using a tank of large scale (1956, 1957).

From the investigations and researches stated above, it can be confirmed that the land subsidence in Osaka is caused by the excess withdrawal of underground water. However, the mechanism of the land subsidence is not so simple because two main causes of land subsidence seem to exist in Osaka; the consolidation of clayey layer due to lowering of artesian head and the repetitious compaction of the loose sandy aquifer due to the fluctuation of the artesian head.

Other research on land subsidence in Osaka is the geological investigation of the subsurface structure of Osaka Basin. This is being performed by S. Matsushita, a professor in Kyoto University, J. Iwatsu, N. Ikebe and J. Takenaka, professors in Osaka City University.

4. FIELD OBSERVATIONS AND RESULTS OBTAINED

In order to investigate the real state of the land subsidence, the following observations and surveys have been performed.

(1) Leveling of land subsidence area

First order precise leveling of Osaka City was begun in 1885 by the Japanese Government, as a part of a survey net covering whole area of Japan. For this leveling, 12 bench marks were set in Osaka City at first. After land subsidence was noticed, more bench marks were established and additional leveling has been performed by the Municipal Authorities since 1934. After land subsidence extended to the surrounding regions around the City, the leveling of these regions has been performed by the Osaka Prefectural Government. At present, 576 bench marks in all are set in the whole area as listed in table-4.1. Such leveling is generally performed every winter, because the rate of land subsidence shows the least amount in winter and such tendency may help to promote the accuracy of the leveling. As the result of this leveling, various kinds of contour maps, annual variation curves of land subsidence, etc. have been produced.

Table 4-1. Bench marks in Osaka

Name of region	Nos. of B.M.
Osaka city	239
Northern Osaka	84
Eastern Osaka	136
Southern Osaka	117
Total	576

(2) Survey about pumping wells

A survey of pumping wells (their location, their purpose, numbers, quantity of water pumped, their depth, etc.) was performed. One of the results in Osaka City is shown in figure 4-1. The decrease in use of underground water since 1962 is shown in figure 4-1 and is mainly due to the effects of regulation and the law. The results of a survey in 1966 for the whole area of Osaka, except Osaka City, is shown in figure 4-2. Of the total quantity of water used (200 million cubic meters), the water used for buildings (mainly for air conditioning) represents 47 percent. Figure 4-3 shows the monthly variation of water quantity pumped from wells in Eastern Osaka in 1968, showing a peak in August.

(3) Geological and soil-mechanical investigation in Osaka

Explanation about this subject is described in section 2.

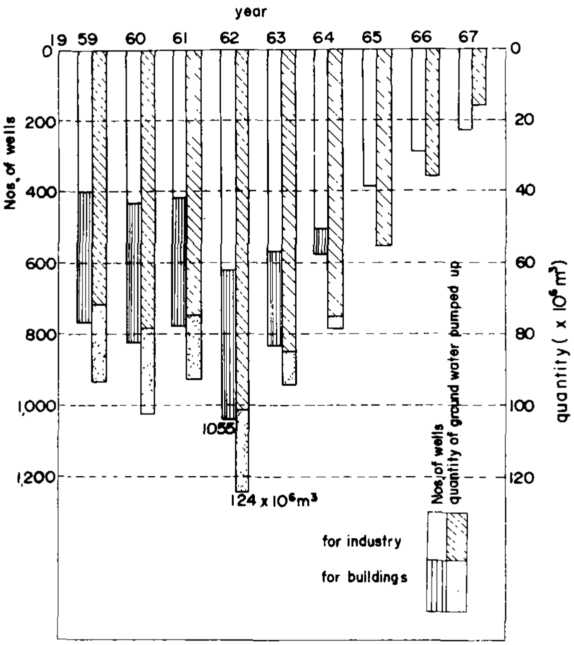

FIGURE 4-1. *Numbers of wells and amount of ground water pumped*

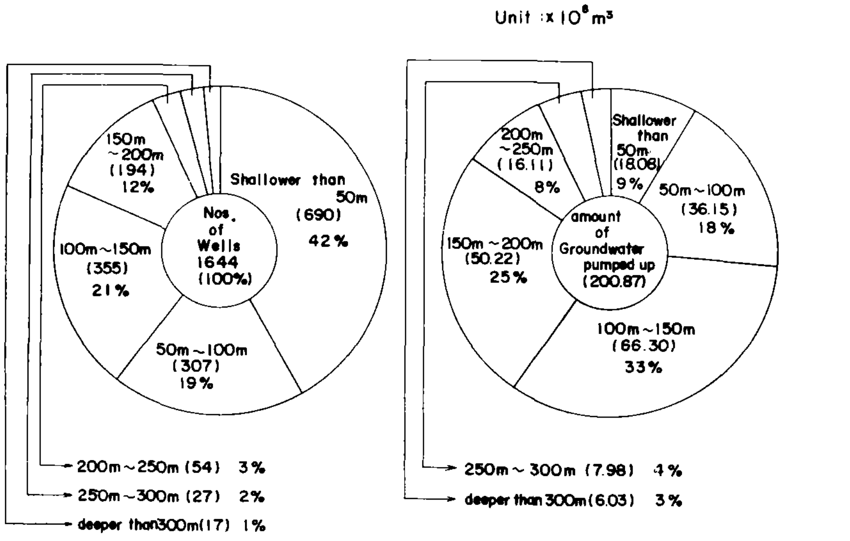

FIGURE 4-2. *Distribution of depth of wells and quantity of under ground water pumped (Whole area of Osaka except Osaka City)*

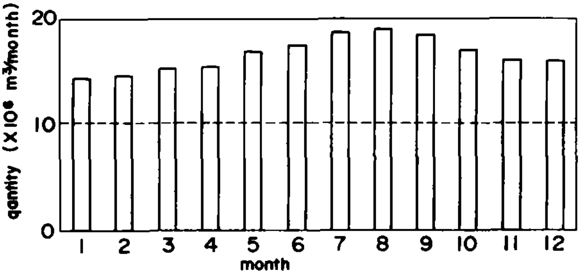

FIGURE 4-3. *Monthly variation of water quantity pumped from wells in 1968 (Whole area of Osaka except Osaka City)*

(4) OBSERVATION OF SUBSIDENCE OF SOIL STRATA AND GROUND WATER LEVEL

The subsidence of soil strata and the ground water level of an aquifer are observed by the instrumentation whose schematic sketch is shown in figure 4-4. This instrumentation consists of two concentric steel pipes, of about 30 cm and 10 cm in diameters, inserted into a vertical bore hole. These pipes are fixed in a sandy aquifer. The bottom part of the outer pipe is perforated to make a strainer. If the land subsidence occurs in the soil strata between the ground surface and the aquifer, these steel pipes stick out of the ground.

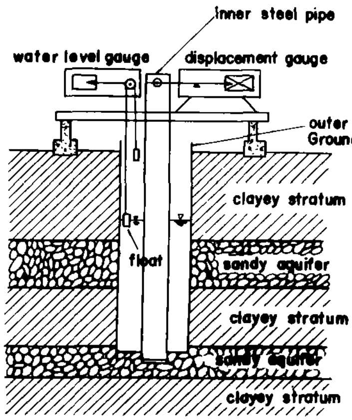

FIGURE 4-4. *Outline of observation apparatus*

Therefore, the relative displacement between the inner steel pipe and the ground surface, measured by a gage set near the head of inner pipe, represents the amount of land subsidence from the surface to the bottom of the steel pipe. An automatic recording gage of a certain magnification is usually used for the displacement gage. The underground water level can be measured by the automatic water gage placed between the inner pipe and outer one Figure 4-5 shows an example of continuous records measured automatically at the Kujoh observatory in August 1957. The locations of such observatories have been shown in figure 1-5, and their instrumentations in table 4-2.

If the instrument pipes are set on the aquifers of various depths, the amounts of land subsidence between the pipes of various depths can be observed. Examples of such observed results are shown in figure 4-6.

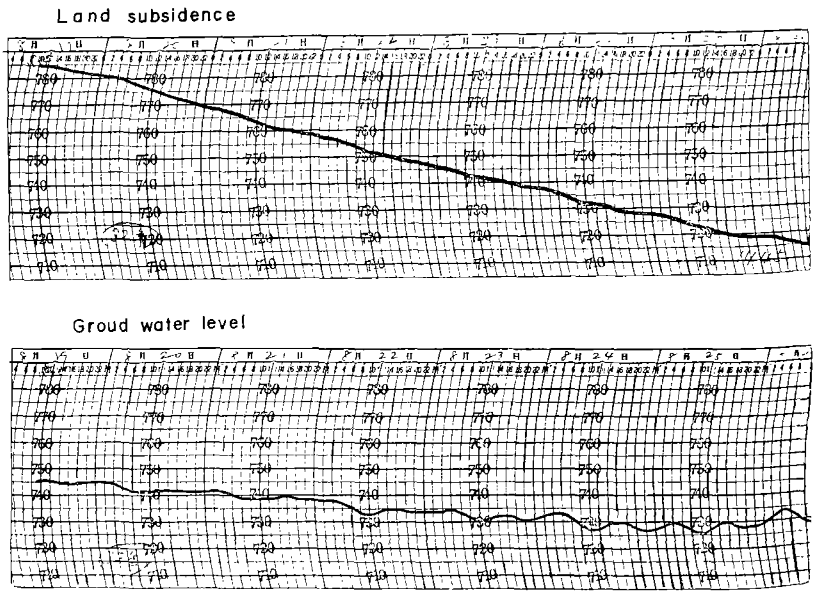

FIGURE 4-5. *Records of land subsidence and ground water level measured at Kujoh Park 1957 (25th/Aug. was Sunday)*

TABLE 4-2. Number of observatories in Osaka

Kind of Observation	Nos.
Observatories for Land Subsidence	29
Observatories for Ground Water Level	43

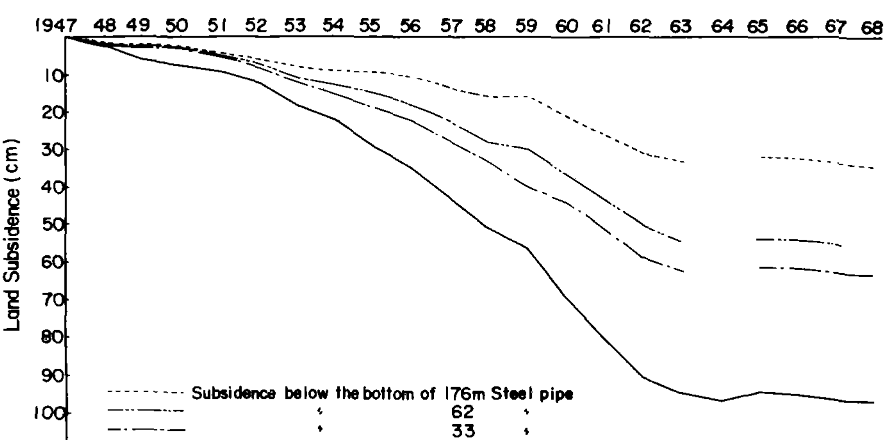

FIGURE 4-6.1. *Vertical distribution of land subsidence in Western Osaka (at Kujoh Park, 1947-1968)*

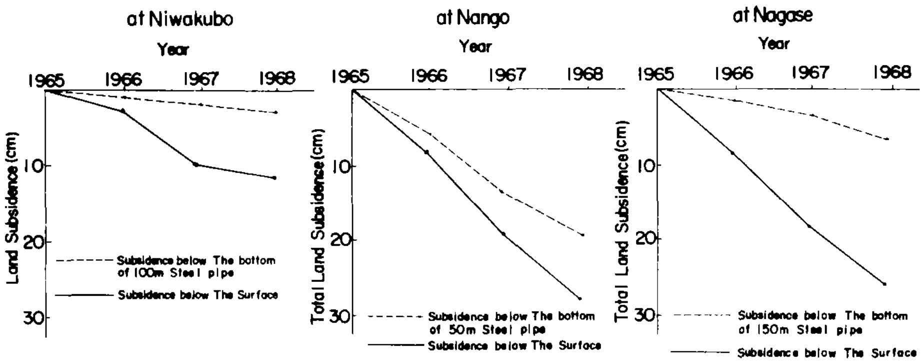

FIGURE 4-6.2. *Vertical distribution of land subsidence in Eastern Osaka (1965-1968)*

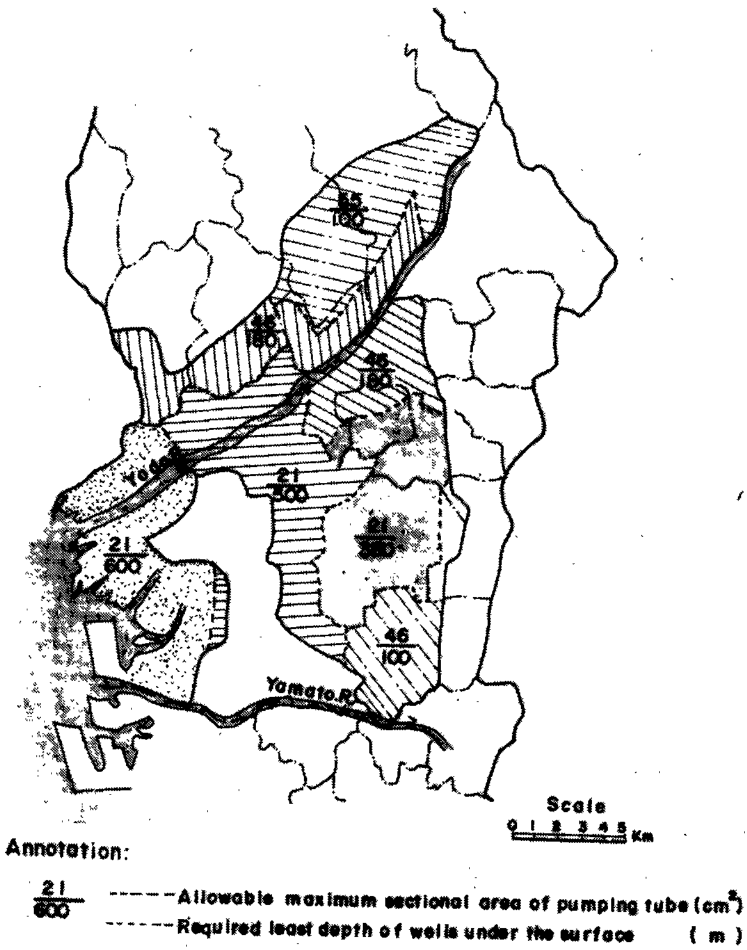

FIGURE 5-1. *Regulated district against the use of ground water for industry*

(5) Investigation of aquifers and ground water

By using test wells in the field, the permeability and compressibility of aquifers were investigated under the guidance of Prof. S. Hayami. Investigation of a recharge well was also tested in the field.

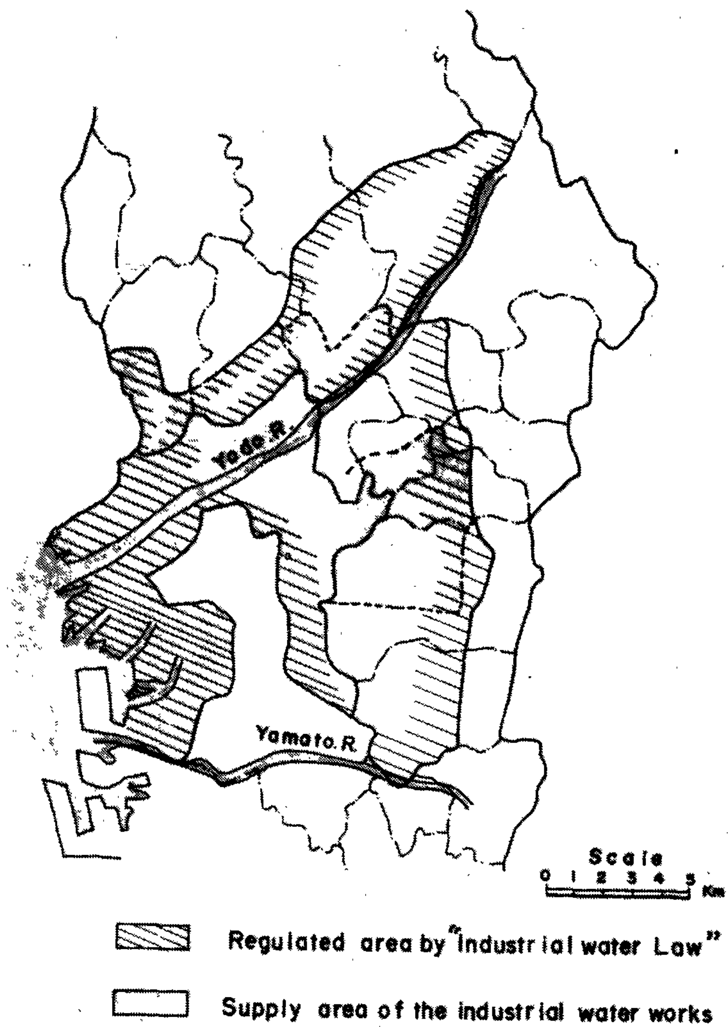

Figure 5-2. *Supply area of industrial water works*

The ability of a well to supply ground water was sometimes determined by a pumping test of underground water. In this test, the ability of the well to supply water was determined by the quantity of water pumped, which corresponded to the inflective point of the curve representing the relation between quantity of water and water level during the test.

Since the underground water of different sources contains different amounts of soluble chemicals, the underground water system in Osaka was investigated by the examination of chemical analysis of the ground water.

5. REGULATION AGAINST THE USE OF GROUND WATER AND ITS EFFECTS ON THE LAND SUBSIDENCE

In order to prevent land subsidence the use of underground water had to be regulated. Besides the regulation, the river water had to be substituted for the underground water by establishing the industrial water works.

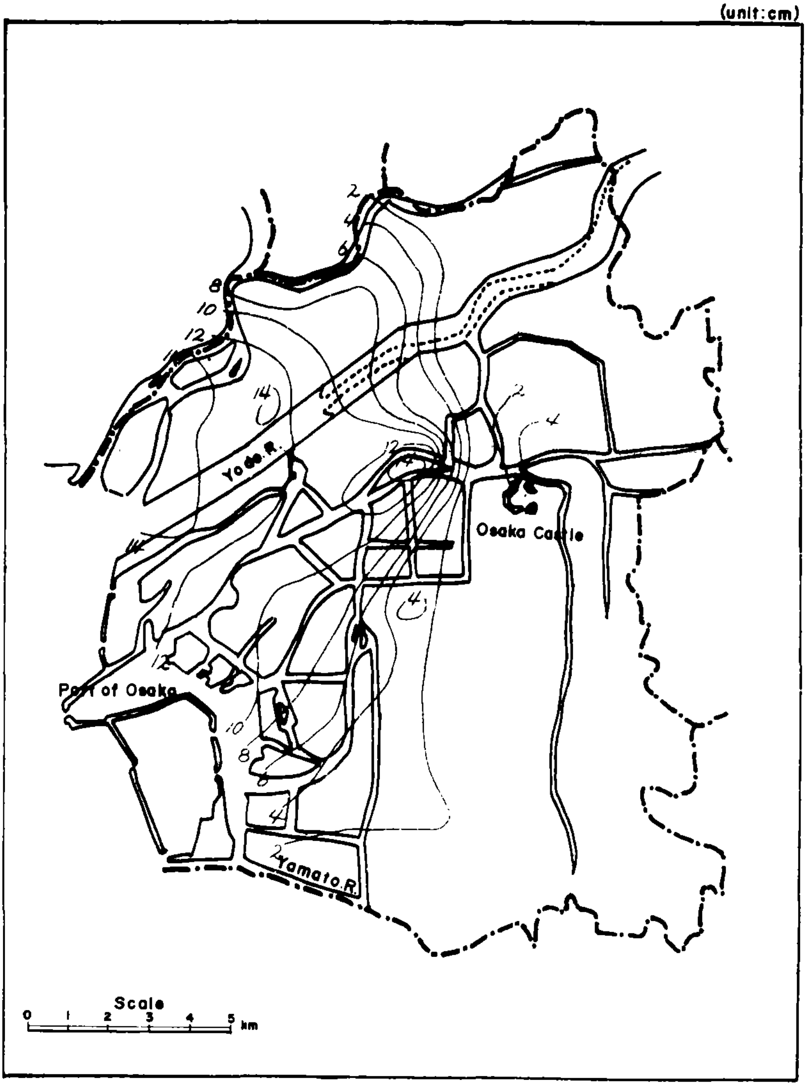

FIGURE 5-3.1. *Isopleth of amount of land subsidence in Osaka (1960)*

The regulation in Osaka City was decided first by the Industrial Water Law enacted in 1956 in order to prohibit constructing new wells. Afterwards, this law and its regulated district were reformed to be severer. According to the existing law, reformed in 1962, the districts regulated against the use of ground water for industrial purposes are shown in figure 5-1. In each district, the maximum sectional area of a newly establishing pumping-tube and the minimum depth of a well were limited as described in figure 5-1. Figure 5-2 shows the present water supply area of industrial water in Osaka. The quantity supplied has previously been shown in figure 1-2.1.

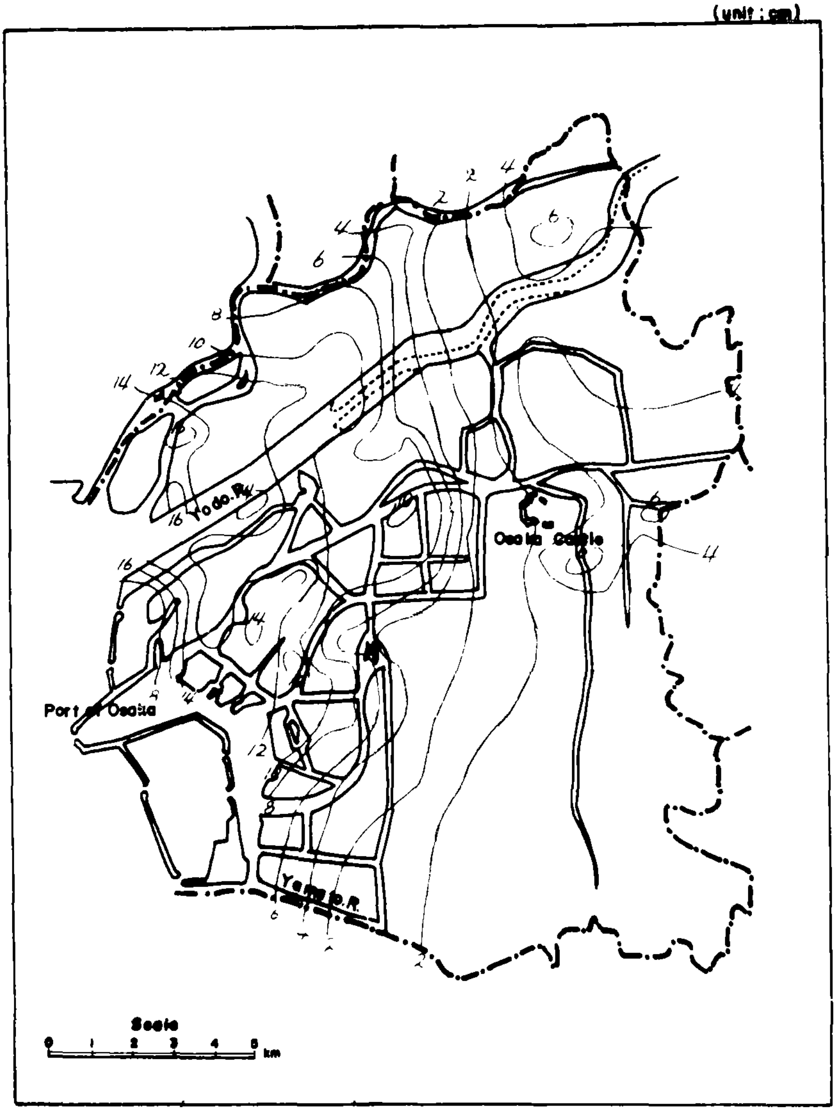

FIGURE 5-3.2. *Isopleth of amount of land subsidence in Osaka (1962)*

As the underground water began to be used not only for industrial purpose but also for the buildings, the regulation of ground water for buildings was enacted in 1959 for a certain part of Osaka City. But the regulated district was extended to the entire City in 1962. The cooling tower method was substituted for use of underground water for buildings.

The effect of these regulations on the land subsidence in the City can be observed in figure 5.3.1 to 5.3.4. The regulations in the Eastern and Northern regions were described previously in section 1.

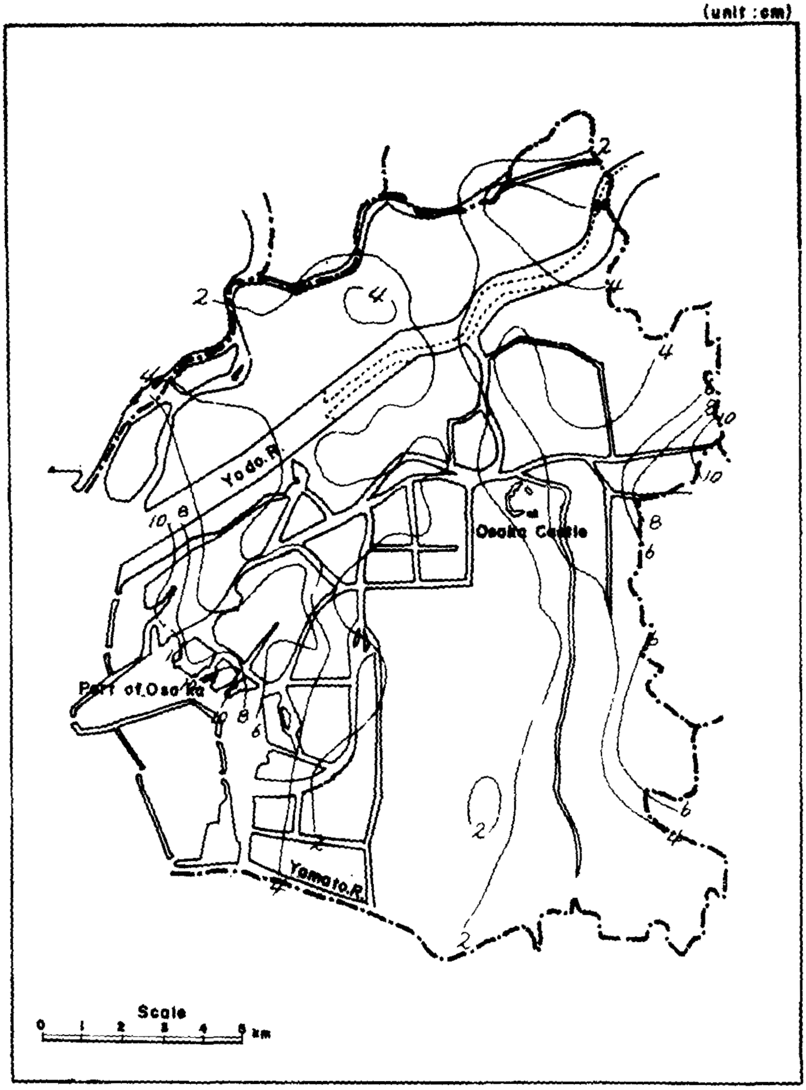

FIGURE 5-3.3. *Isopleth of amount of land subsidence in Osaka (1964)*

6. ACKNOWLEDGEMENT AND INFORMATION

This paper introduces data on the land subsidence in Osaka obtained through various fields and many investigators. Data in Sections 1, 4 and 5 were obtained from the engineers in the Osaka Prefectural Government and the Osaka Municipal Office. The contents of Section 2. (1) and 2. (2) have been summarized by Dr. J. Takenaka, Prof. in Osaka City University and Mr. N. Yagi, Assist. Prof. in Kyoto University respectively. The writer wishes to express his deep appreciation to them for their sincere co-operation.

As this paper is too short to report sufficient data on the land subsidence in Osaka, another more detailed report is now being prepared for distribution at the Symposium by the editorial committee, which consists of the engineers in the Osaka Prefectural Government and the Osaka Municipal Office and the researchers participating in the investigations.

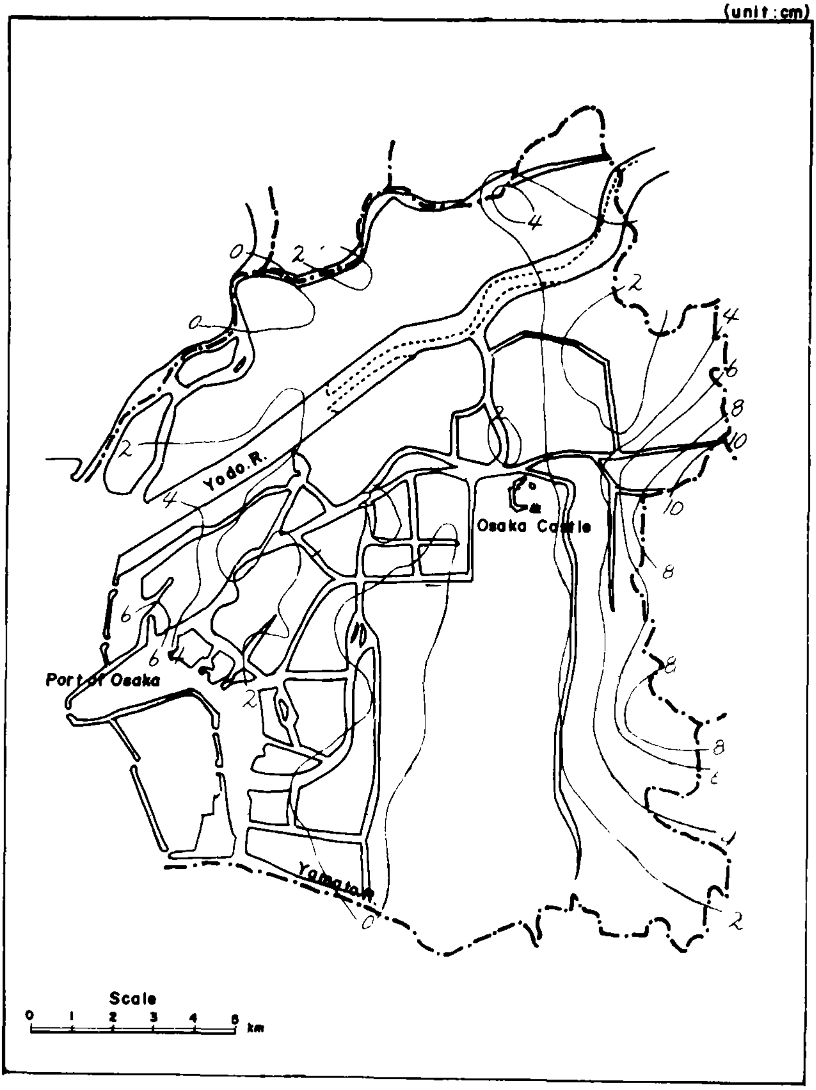

FIGURE 5-3.4. *Isopleth of amount of land subsidence in Osaka (1966)*

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ANALYSIS OF LAND SUBSIDENCE IN NIIGATA

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ABSTRACT

The land subsidence in the Niigata area was analyzed by means of consolidation theory. Pumping the underground water from the deep sandy layers for extracting methane gas caused a time-dependent loading to the clay layers. An analytical solution of consolidation under the load increasing linearly with time was obtained in terms of the vertical strain. A numerical method of analysis for the load decreasing linearly and then becoming constant was developed considering the difference in values of soil constants for consolidation and for rebound respectively. These two methods were combined for analyzing the subsidence in the Niigata area, and the results compared favourably with the observed settlement.

RÉSUMÉ

L'affaissement du sol à Niigata a été analysé par le moyen de la théorie de consolidation. Le pompage de l'eau souterraine à partir de la couche profonde sableuse pour extraire le gas méthane a causé la mise en charge, variable avec le temps, de la couche de l'argile. Une solution analytique de la consolidation sous la charge augmentant avec le temps est obtenue en ce qui concerne la déformation verticale. Une méthode numérique d'analyse de la charge augmentant d'abord linéairement et devenant ensuite constante est développée en tenant compte de la différence dans les valeurs des constantes du sol à la consolidation et au regonflement. Ces deux méthodes sont combinées pour analyser le tassement à Niigata, et les résultats ont été favorablement comparés avec les valeurs observées.

1. INTRODUCTION

Niigata City and the neighbouring area have suffered a severe land subsidence since the 1950's. The greatest rate of settlement of 53 cm per year was observed around the Port of Niigata in the period of 1958-1959 as shown in figure 1 (1st District Bureau for Port Construction *et al.*, 1963). Many of the port and coast facilities, river and road embankments, and farms and factories had gone out of use.

Every possible reason of land subsidence was investigated, and finally it was concluded that the main source of the subsidence was the pumping of underground water for extracting methane gas. Natural methane gas in the Niigata area is dissolved in the water of deep sandy aquifers. The ratio of the volume of the dissolved gas to the water was one to one, and the greatest rate of the pumpage was 17,000,000 cubic meters per month. It caused the water heads of the aquifers to be below sea level by a maximum amount of 44 m, and resulted in a considerable consolidation of the clay strata.

While large scale countermeasures were rushed in the subsiding area, various investigations were carried out; leveling, tide observations, borings and soil tests, measurements of compression and water table in each stratum by means of observation wells, and the statistical survey of a quantity of the pumped water and gas. Subsidence analyses were made by means of the consolidation theory and others, and future settlement was predicted with satisfactory accuracy at each time.

Pumping restrictions, have been enforced four times from 1959 through 1962. It resulted in a marked recovery of the water heads in the aquifers, hence a new loading condition for the clay strata, as illustrated in figure 2. The writer developed a new method for analyzing consolidation under such time-dependent loading (Okumura and Moto, 1967), and applied it to the analysis of the land subsidence in the Niigata area.

2. THEORETICAL BASIS

2.1. Consolidation under increasing load

According to Mikasa's theory (1963), it is more convenient to analyze consolidation in terms of strain than in terms of excess pore water pressure, since the problem of pore pressure set-up in this case can be replaced by the problem of boundary conditions.

If the change in thickness of a clay layer and the influence of its own weight are neglected, and the coefficient of consolidation, C_v, is assumed constant, the fundamental differential equation of one dimensional consolidation in terms of compression strain, ε, is given as,

$$\frac{\partial \varepsilon}{\partial t} = C_v \frac{\partial^2 \varepsilon}{\partial z^2} \tag{1}$$

Assuming that the total stress distribution is linear with depth and the load increases at a constant rate up to time t_1, as shown in figure 2, the boundary and initial conditions become,

$$\left.\begin{aligned} \varepsilon(0, t) &= m_v p_1\, t/t_1 \\ \varepsilon(2H, t) &= m_v p_2\, t/t_1 \\ \varepsilon(z, 0) &= 0 \end{aligned}\right\} \tag{2}$$

where the coefficient of volume compressibility, m_v, is assumed constant. The solution of equation (1) under the condition of equation (2) can be obtained by the theory of conduction of heat (Carslaw and Jaeger, 1959) as follows,

$$\varepsilon = \frac{m_v}{T_1}\left[T\left(p_1 + (p_2 - p_1)\frac{z}{2H}\right) - \frac{p_1+p_2}{2} F_1(T, z/2H) + \frac{p_2-p_1}{2} F_2(T, z/2H)\right] \tag{3}$$

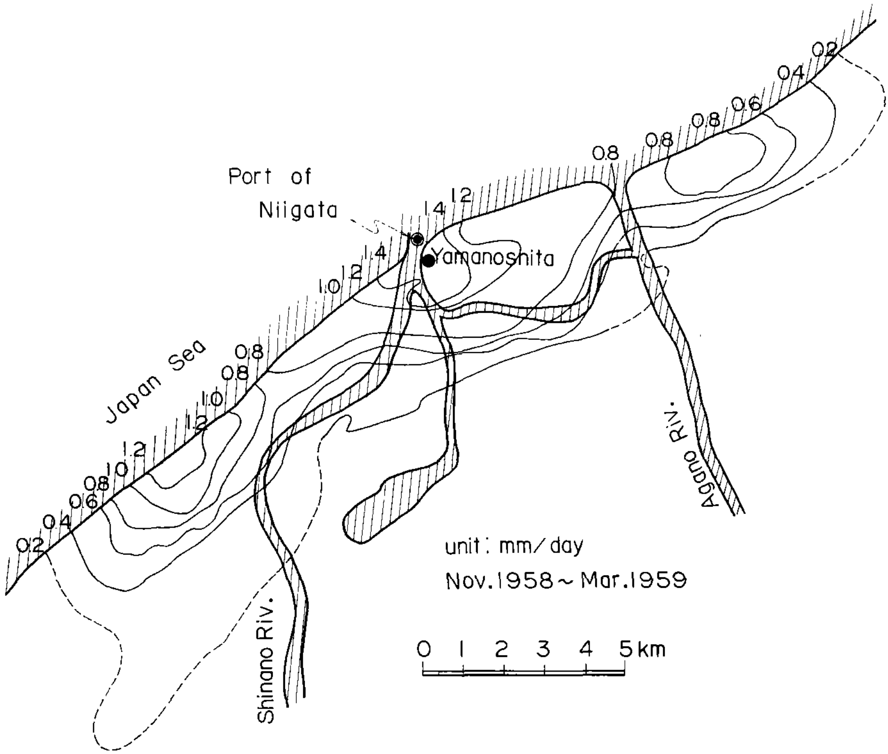

FIGURE 1. *Extraordinary rate of subsidence in and around Niigata City*

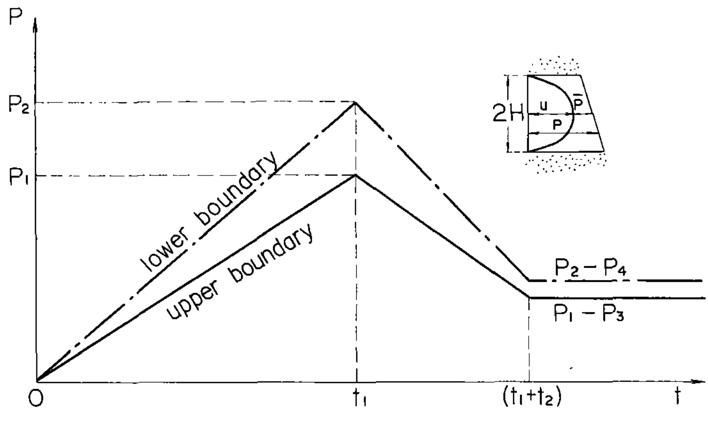

FIGURE 2. *Typical loading condition*

in which T is the time factor ($T = C_v t/H^2$), T_1 is the time factor at time t_1, and $F\ (T, z/2H)$ is referred to the coefficient of strain and expressed in the forms,

$$F_1(T,\ z/2H) = \frac{16}{\pi^3} \sum_{n=1,3,5,\dots}^{\infty} \frac{1}{n^3}(1-e^{-n^2\pi^2 T/4}) \sin\frac{n\pi z}{2H} \tag{4a}$$

$$F_2(T,\ z/2H) = \frac{16}{\pi^3} \sum_{n=2,4,6,\dots}^{\infty} \frac{1}{n^3}(1-e^{-n^2\pi^2 T/4}) \sin\frac{n\pi z}{2H} \tag{4b}$$

The relationship between the strain and the excess pore water pressure, u, is,

$$\begin{aligned} \varepsilon &= m_v(p-u) \\ &= m_v\left[\frac{t}{t_1}\left(p_1 + (p_2-p_1)\frac{z}{2H}\right) - u\right] \end{aligned} \tag{5}$$

where p is the total pressure at the depth and time under consideration. The solution in terms of excess pore pressure is then written as,

$$u = \frac{1}{T_1}\left[\frac{p_1+p_2}{2} F_1(T,\ z/2H) - \frac{p_2-p_1}{2} F_2(T_1\ z/2H)\right] \tag{6}$$

This corresponds to Terzaghi-Frölich's solution (1936). And the condition of $p_1 = p_2$ gives,

$$u = \frac{p_1}{T_1} F_1(T_1\ z/2H) \tag{7}$$

This corresponds to Schiffman's solution (1963).

Denothing the degree of consolidation, U, as the ratio of the mean effective pressure to the mean total pressure at that time, the expression for the degree of consolidation is,

$$\begin{aligned} U &= 1 - \frac{\int_0^{2H} u\,dz}{\int_0^{2H} p\,dz} \\ &= 1 - \frac{1}{T}\left(\frac{1}{3} - \frac{32}{\pi^4} \sum_{n=1,3,5,\dots}^{\infty} \frac{1}{n^4} e^{-n^2\pi^2 T/4}\right) \\ &\equiv U_0(T) \end{aligned} \tag{8}$$

in which $U_0\ (T)$ shall be called the coefficient of degree of consolidation and is independent of the magnitude of pressures p_1 and p_2 respectively. Using the coefficient of degree of consolidation, the expression for the settlement, S, becomes,

$$
\begin{aligned}
S &= \int_0^{2H} \varepsilon dz \\
&= 2Hm_v \frac{p_1+p_2}{2} \frac{t}{t_1} U_0(T)
\end{aligned} \tag{9}
$$

It may be said that the settlement in this case is expressed as the product of the thickness of the clay layer, the coefficient of volume compressibility, the mean total pressure at that time, and the degree of consolidation.

2.2. Consolidation under decreasing load

When a load decreases before a corresponding consolidation of a clay layer is completed the following situation may occur: the effective pressure in a part of the layer continues to increase, whereas it decreases in the other part, causing this part of the soil to swell or rebound. In such a case, the compound phenomenon of consolidation and rebound should be considered.

The coefficient of volume expansibility, m_{vr}, of the clay will be smaller than the coefficient of volume compressibility by the factor of about 10 (fig. 6), and it will be dependent on the magnitude of the preconsolidation load and the overconsolidation ratio. As far as the writer is aware, however, no established relationship between the above factors has been reported. Similarly much ambiguity exists in the coefficient of rebound, C_{vr}, which is a counterpart of the coefficient of consolidation, C_v. If the coefficient of permeability does not change, however, the coefficient of rebound will become m_v/m_{vr} times the coefficient of consolidation. Therefore, in this section it may be assumed,

$$
\left.\begin{aligned}
m_{vr} &= m_v/r \\
C_{vr} &= rC_v
\end{aligned}\right\} \tag{10}
$$

where r is a constant for a particular soil.

In the consolidation phenomenon, including rebound, the strain depends upon the stress history. Thus the choice of the strain as the dependent variable may not be relevant. The fundamental differential equation of consolidation including rebound, under an assumption of linear stress distribution, is represented as a function of excess pore water pressure,

$$
\frac{\partial u}{\partial t} = \begin{Bmatrix} C_v \\ rC_v \end{Bmatrix} \frac{\partial^2 u}{\partial z^2} + \frac{\partial p}{\partial t} \tag{11}
$$

The choice of the coefficients in braces depends on whether the effective stress, $\bar{p}$, increases or decreases with time, and may be written as follows,

$$
\left.\begin{aligned}
&C_v \cdots \text{for} \quad \frac{\partial \bar{p}}{\partial t} \geqq 0 \quad \text{i.e.} \quad \frac{\partial^2 u}{\partial z^2} \leqq 0 \\
&rC_v \cdots \text{for} \quad \frac{\partial \bar{p}}{\partial t} < 0 \quad \text{i.e.} \quad \frac{\partial\ u}{\partial z^2} > 0
\end{aligned}\right\} \tag{12}
$$

Under the loading condition shown in figure 2 ($t_1 \leqq t \leqq t_1 + t_2$), the fundamental equation, the boundary conditions, and the initial condition are represented respectively as follows,

$$\frac{\partial u}{\partial t} = \begin{Bmatrix} C_v \\ rC_v \end{Bmatrix} \frac{\partial^2 u}{\partial z^2} - \frac{1}{t_2}\left(p_3 + (p_4 - p_3)\frac{z}{2H}\right) \tag{13}$$

$$\left.\begin{aligned} &u(0, t) = 0 \\ &u(2H, t) = 0 \\ &u(z, 0) = \frac{1}{T_1}\left(\frac{p_1+p_2}{2}F_1(T_1, z/2H) - \frac{p_2-p_1}{2}F_2(T_1, z/2H)\right) \end{aligned}\right\} \tag{14}$$

Since the above equation can not be solved analytically, a numerical method has to be used. Dividing the whole layer of clay into α slices as shown in figure 3, equation (13) can be written in the form of a finite difference equation,

$$\frac{\Delta u_i}{\Delta t} = \begin{Bmatrix} C_v \\ rC_v \end{Bmatrix} \frac{u_{i-1}^j + u_{i+1}^j - 2u_i^j}{\Delta z^2} - \frac{1}{t_2}\left(p_3 + (p_4 - p_3)\frac{i}{\alpha}\right) \tag{15}$$

The excess pore pressure at time $(t+\Delta t)$ can then be computed by the following equation, using the excess pore pressures at time t,

$$u_i^{j+1} = u_i^j + \begin{Bmatrix} \beta/r \\ \beta \end{Bmatrix}(u_{i-1}^j + u_{i+1}^j - 2u_i^j) - \frac{1}{t_2}\left(p_3 + (p_4 - p_3)\frac{i}{\alpha}\right)\Delta t \tag{16}$$

in which

$$\beta = rC_v \Delta t/\Delta z^2 \tag{17}$$

$$\left.\begin{aligned} &u_0^j = u_\alpha^j = 0 \\ &u_i^0 = \frac{1}{T_1}\left(\frac{p_1+p_2}{2}F_1(T_1, i/\alpha) - \frac{p_2-p_1}{2}F_2(T_1, i/\alpha)\right) \end{aligned}\right\} \tag{18}$$

i: grid number in z-axis, from 1 to $(\alpha - 1)$
j: grid number in t-axis, from 0 to $t_2/\Delta t$.

In some cases it will be probable that the effective stress in some slices decreases at first and then increases. Such situations may be encountered when the rate of loading changes after a period of time, as shown in figure 2 ($t > t_1 + t_2$). In this case equation (12) alone is not sufficient. Assuming that the same consolidation parameters are applicable to both rebound and recompression, as far as the effective pressure does not exceed the preconsolidation pressure, the choice of the coefficients in equation (16) may be extended approximately as,

$$\beta/r \ldots \quad \text{for} \quad F \leqq 0 \quad \text{and} \quad 3\bar{p}_i^j - \bar{p}_i^{j-1} \geqq 2\bar{p}_{i\max} \tag{19a}$$

$$\beta \quad \ldots \quad \text{for} \quad F \leqq 0 \quad \text{and} \quad 3\,\bar{p}_i^j - \bar{p}_i^{j-1} < 2\,\bar{p}_{i\max} \tag{19b}$$

$$\beta \quad \ldots \quad \text{for} \quad F > 0 \tag{19c}$$

in which

$$F = u_{i-1}^j + u_{i+1}^j - 2u_i^j \tag{20}$$

$$\begin{aligned} \bar{p}_i^j &= p_i^j - u_i^j \\ &= p_1 - \frac{p_3 j \Delta t}{t_2} + \left\{ p_2 - p_1 - \frac{j \Delta t}{t_2}(p_4 - p_3) \right\} \frac{i}{\alpha} - u_i^j \end{aligned} \tag{21}$$

and $\bar{p}_{i\max}$ is the maximum value of $\bar{p}_i^j$ in $t \leqq t_1 + \Delta t\,(j-1)$.

Applying the trapezoid formula, the settlement, S^j, of the layer is represented as follows,

$$\begin{aligned} S^j &= \frac{1}{2}(\varepsilon_0^j + \varepsilon_\alpha^j)\Delta z + \sum_{i=1}^{\alpha-1} \varepsilon_i^j \Delta z \\ &= m_v \Delta z \left[\frac{p_1 + p_2}{2} - \frac{j \Delta t}{2r\,t_2}(p_3 + p_4) + \sum_{i=1}^{\alpha-1} \left\{ \begin{matrix} p_i^j \\ \left(1 - \frac{1}{r}\right)\bar{p}_{i\max} + \frac{\bar{p}_i^j}{r} \end{matrix} \right\} \right] \end{aligned} \tag{22}$$

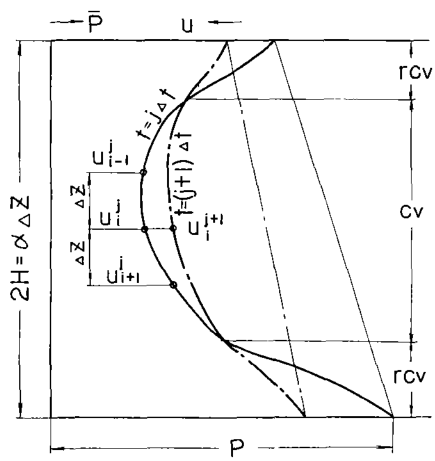

FIGURE 3. *Key sketch to numerical analysis*

The third term of the right hand side of equation (22) should be,

$$\left. \begin{aligned} &\sum_{i=1}^{\alpha-1} \bar{p}_i^j && \cdots \text{for condition of equation (19}a\text{)} \\ &\sum_{i=1}^{\alpha-1} \left[\left(1 - \frac{1}{r}\right)\bar{p}_{i\max} + \frac{\bar{p}_i^j}{r} \right] && \cdots \text{ for condition of equations (19}b\text{) and (19}c\text{)} \end{aligned} \right\} \tag{23}$$

The degree of consolidation, defined in the same way as that for increasing load, is represented by the trapezoid formula as follows,

$$U^j = 1 - \frac{2}{\alpha\{p_1+p_2-(p_3+p_4)j\Delta t/t_2\}} \sum_{i=1}^{\alpha-1} u_i^j \tag{24}$$

When the negative excess pore pressure is prevalent within the layer, the degree of consolidation may become more than 100 percent.

After the load has become constant as shown in figure 2 ($t > t_1 + t_2$), the differential equation of consolidation including rebound is represented,

$$\frac{\partial u}{\partial t} = \begin{Bmatrix} C_v \\ r C_v \end{Bmatrix} \frac{\partial^2 u}{\partial z^2} \tag{25}$$

And the fundamental equation for numerical analysis becomes,

$$u_i^{j+1} = u_i^j + \begin{Bmatrix} \beta/r \\ \beta \end{Bmatrix} (u_{i-1}^j + u_{i+1}^j - 2u_i^j) \tag{26}$$

in which

$u_o^j = u_\alpha^j = 0$

$u_i^{t_2/\Delta t}$: from equation (16)

i: from 1 to $(\alpha-1)$;
j: from $(t_2/\Delta t)$ to infinity.

The effective pressure, the settlement and the degree of consolidation of the layer become,

$$\bar{p}_i^j = p_1 - p_3 + (p_2 - p_1 - p_4 + p_3)\frac{i}{\alpha} - u_i^j \tag{27}$$

$$S^j = m_v \Delta z \left[\frac{p_1+p_2}{2} - \frac{p_3+p_4}{2r} + \sum_{i=1}^{\alpha-1} \begin{Bmatrix} \bar{p}_i^j \\ \left(1-\frac{1}{r}\right)\bar{p}_{i\max} + \frac{\bar{p}_i^j}{r} \end{Bmatrix} \right] \tag{28}$$

$$U^j = 1 - \frac{2}{\alpha(p_1+p_2-p_3-p_4)} \sum_{i=1}^{\alpha-1} u_i^j \tag{29}$$

The choice of the terms in $\{\quad\}$ in equations (25), (26) and (28) should be made after equations (19) and (23).

3. SOIL CONSTANTS AND LOADING CONDITIONS

In order to measure the water head of the aquifers and the compression of the clay layers, several observation wells were installed in the Yamanoshita area. The wells consist of an outer steel pipe with a filter tip through which water in the aquifer may enter freely, and of an inner pipe supported by the frictionless centralizer through the outer pipe and based on the aquifer. The water head of the aquifer was recorded automatically by measuring the water table in between both pipes, and the compression settlement down to the aquifer was automatically recorded by the movement of the inner pipe relative to the ground surface.

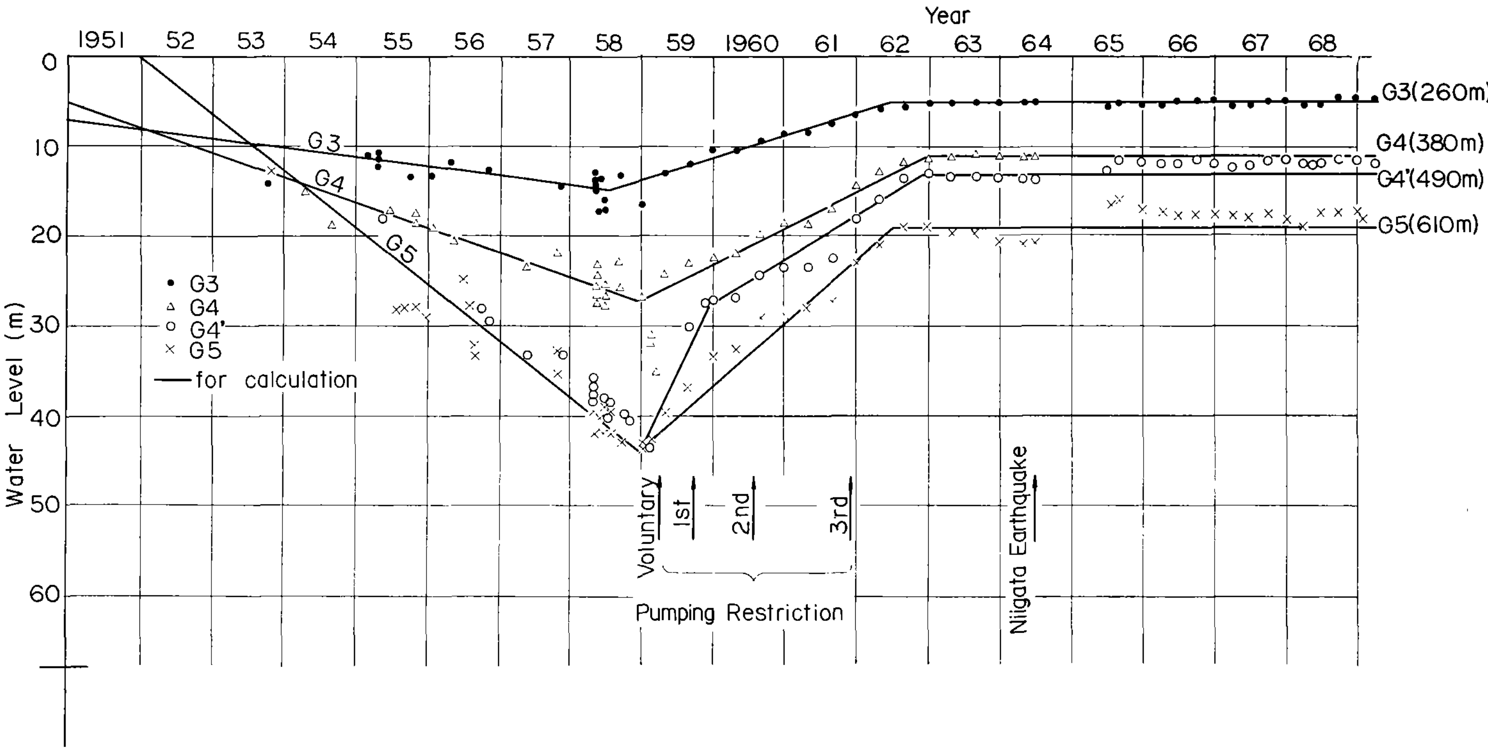

FIGURE 4. *Observed change in underground water level*

Figure 4 shows the underground water head of each aquifer in meters below the sea level, which was supplemented with the record taken from the industrial wells for gas. These decreases in water head may lead to additional loads both for the clay layers and the sandy aquifers. An example of the settlement record by the well is shown in figure 8.

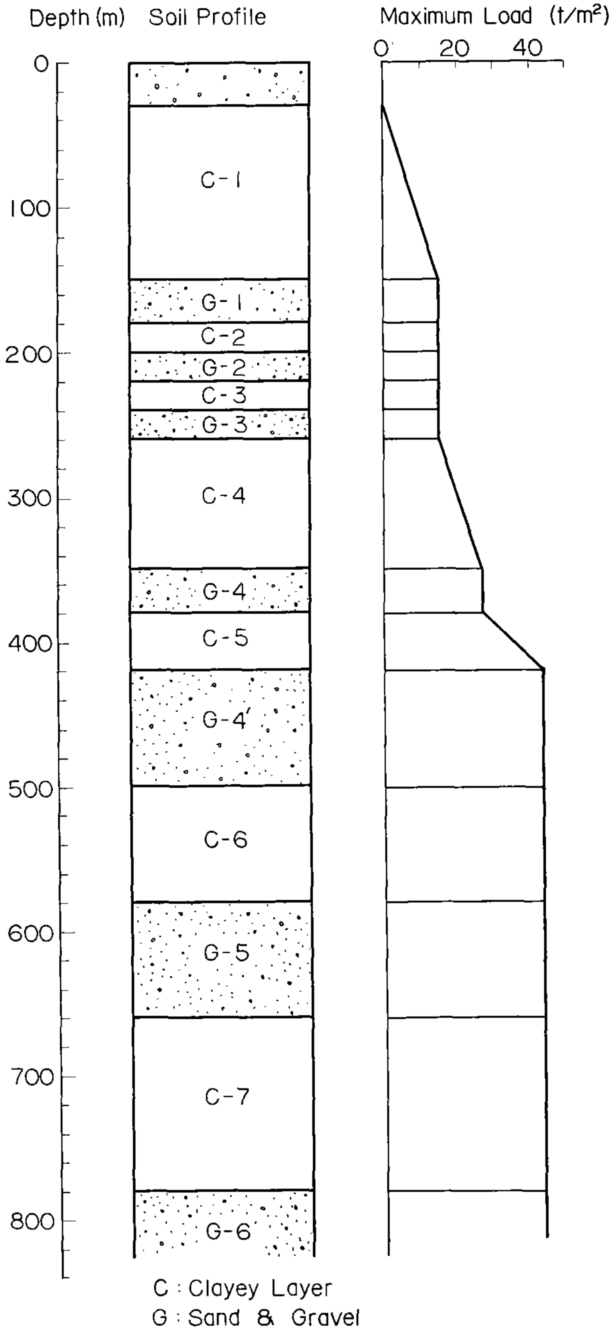

FIGURE 5. *Simplified soil profile and loading condition for calculation in Yamanoshita*

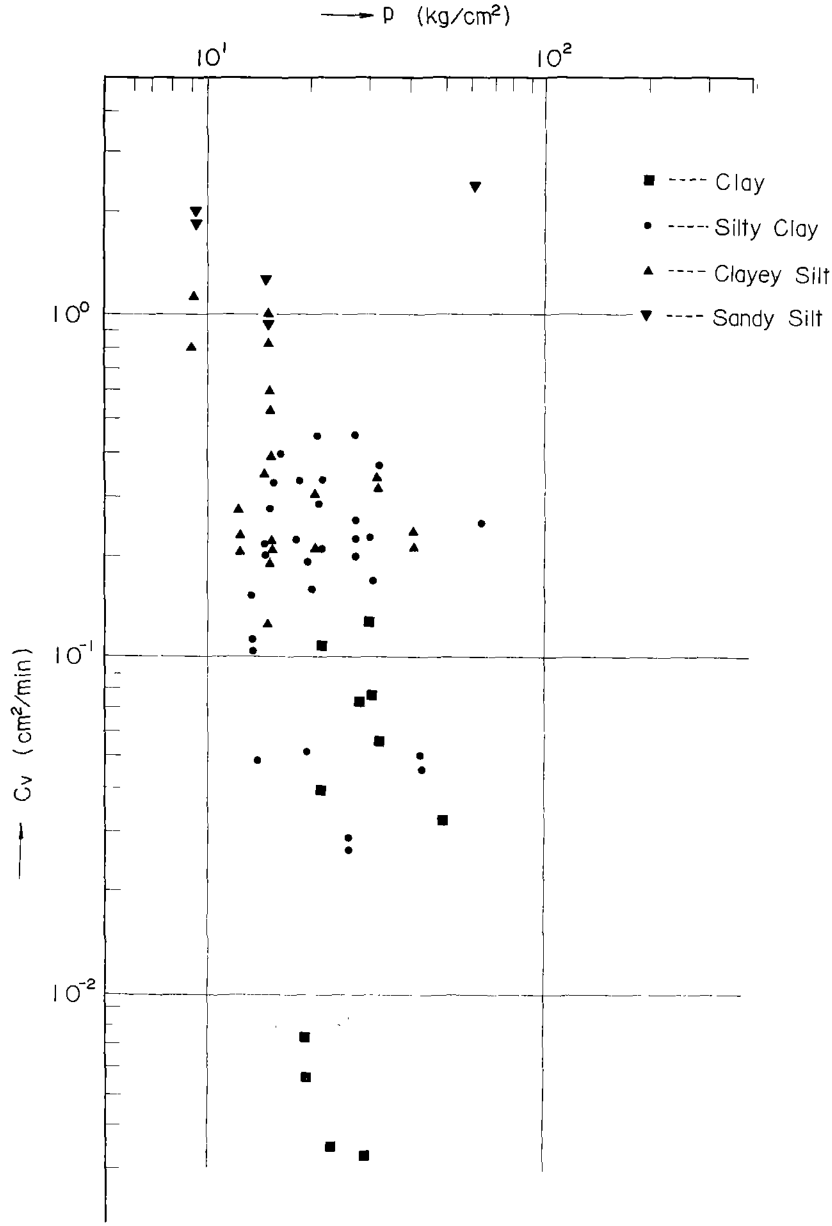

FIGURE 6. *Coefficient of consolidation (Virgin compression)*

In connection with installing the observation wells several borings were carried out down to the depth of 1200m, and undisturbed clay samples were taken with the tin-wall fixed-piston sampler from the clay layer above 500 m depth, as well as some disturbed samples from the layer above 1200 m depth. Consolidation tests, unconfined compression tests, and classification tests were performed on these samples.

The soil profile is shown in simplified form in figure 5. Except for the thin surface layer, alternate silty-clay and gravelly-sand strata down to the depth of about 600 m are considered to be diluvial deposits.

A large capacity oedometer, with maximum load intensity of 200 kg/cm², was used to investigate the consolidation properties of the clay samples. Test results in figure 6 and 7 show that, in spite of extremely high overburden pressure, the coefficient of volume compressibility and the coefficient of consolidation are not so different from those of typical alluvial soils in Japan. The preconsolidation pressure determined by Casagrande's method was slightly larger than the estimated overburden pressure, but it was not so clear.

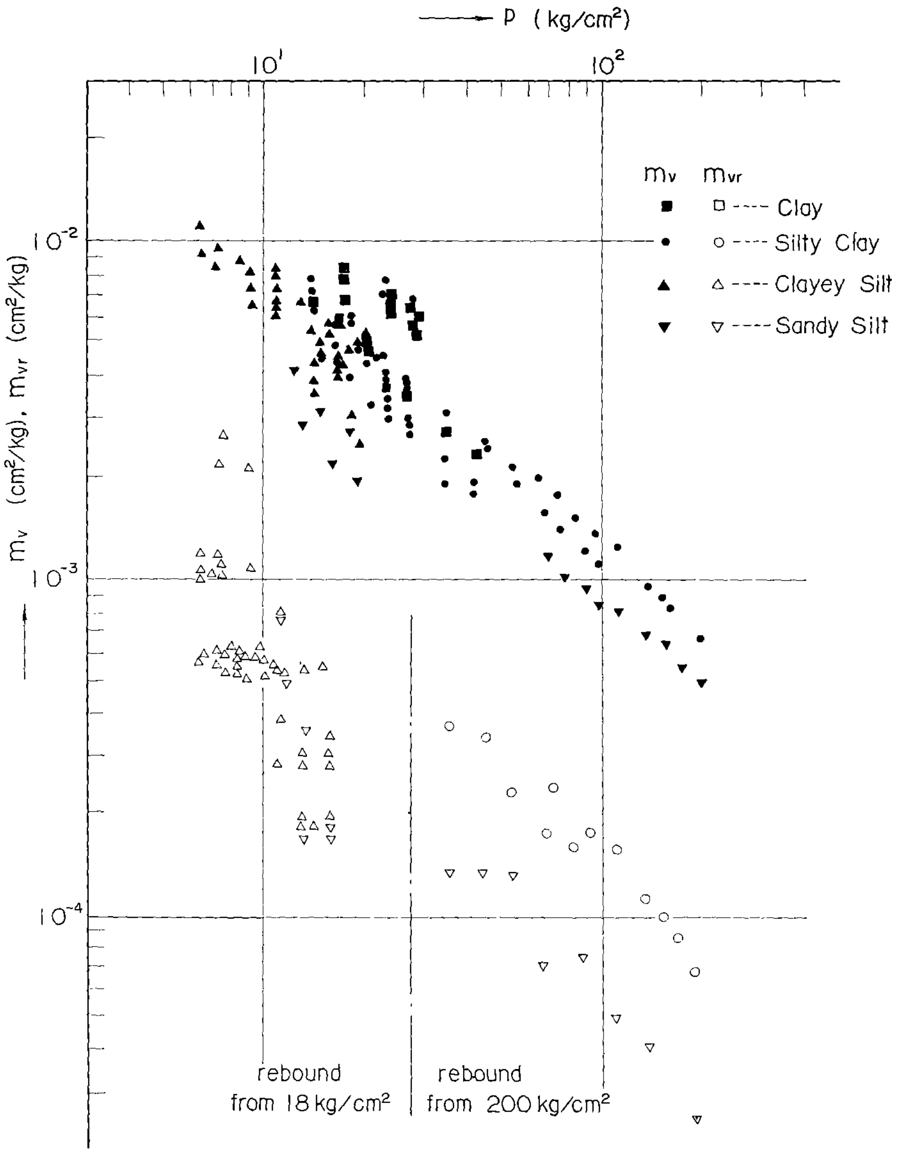

FIGURE 7. *Coefficient of volume compressibility and volume expansibility (Virgin compression)*

Rebound tests in which the pressure was released from the highest consolidation pressure gave the ratio of the coefficients of volume compressibility to expansibility to be 10 to 50 as shown in figure 6.

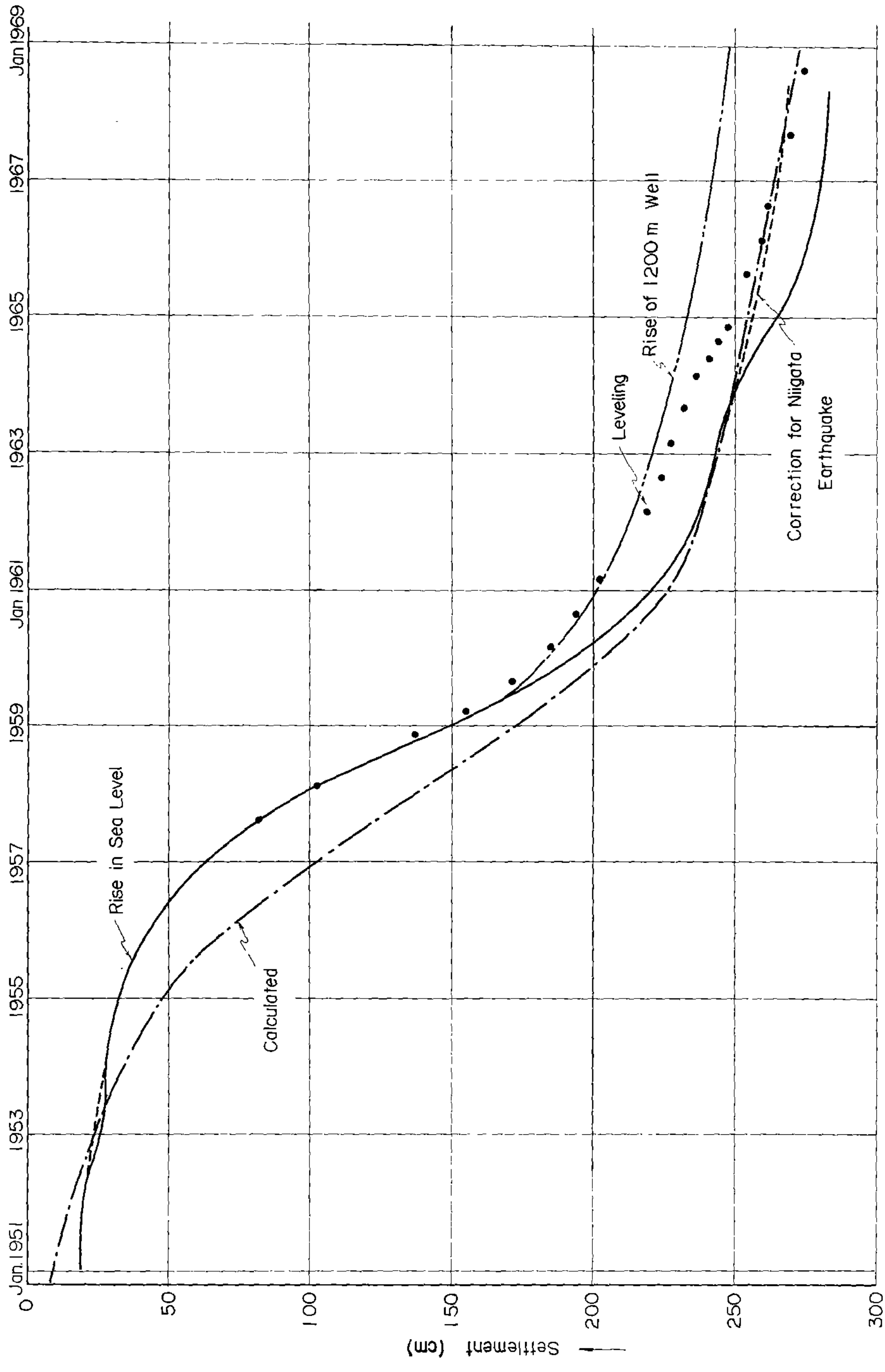

FIGURE 8. *Land subsidence in Yamanoshita*

4. COMPARISON OF CALCULATED RESULTS WITH THE OBSERVED SETTLEMENT

For simplicity of analysis it is assumed that consolidation loads change linearly with time as shown in figure 4, and that they are applied to each layer as shown in figure 5. After a number of trial calculations, the soil constants considered to be relevant to the analysis are as follows,

m_v (clay)	8×10^{-3} cm^2/kg	m_v (sand)	4×10^{-4} cm^2/kg
m_{vr} (clay)	1.6×10^{-4} cm^2/kg	m_{vr} (sand)	0
C_v (clay)	0.1 cm^2/min for increasing load 0.2 cm^2/min for decreasing load		
C_v (sand)	∞		
C_{vr} (clay)	10 cm^2/min		
Overconsolidation pressure	1 kg/cm^2		

Three kinds of settlement records are available, as shown in figure 8. The record of leveling survey will be most reliable, but there is no detailed record until 1957. Harmonic analysis of the tide level gives a smooth curve of relative settlement, but it may be somewhat overestimated in comparison with that obtained by the leveling. The rise in the inner pipe founded at 1200 m shows the least settlement, which may be to some extent due to the friction between the pipe and the soil. Therefore, none of these three records is satisfactorily reliable. Moreover, a considerable earth crust movement by the Niigata Earthquake complicated the settlement record.

The calculated settlement is shown in figure 8. As compared with the observed settlement record, the rate of calculated settlement is found to be greater in the period until 1956, and smaller in the subsequent years. This difference in the rate of settlement may be partly due to an ambiguity in assessing the loading condition in the early period. (See fig. 4) However, the calculated result compares favourably with the observed settlement, as a whole, and may be used for estimating future settlement with satisfactory accuracy.

5. CONCLUSION

A newly developed method for analysis of consolidation under time-dependent loading was applied to the analysis of land subsidence in the Niigata area. Although the soil constants, the loading conditions, and even the observed settlement were not fully reliable, the calculated result compared favourably with the observed settlement as a whole. The present method may be applicable to the analysis of consolidation in which the applied load is partly removed before the consolidation is completed, e.g., in the pre-loading method of a road embankment.

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NIIGATA GROUND SUBSIDENCE AND GROUND WATER CHANGE

Takuzo HIRONO

ABSTRACT

Periodogram and Fourier analysis were applied to the data obtained from observation wells installed for measuring ground subsidence at Yamanoshita ward, Niigata City, in 1958. The items of data analysed are the sea level at the harbour and the atmospheric pressure at the meteorological observatory in addition to the ground subsidence and the ground water level at the observation wells with depths of 20, 130, 260, 380 and 610 m respectively. The analyses were made for the period from July to October, 1958 (about 120 days).

The ground surfaces changes periodically with the ground water. For shorter periods (< 30 days) the ground surface undulates with amplitude proportional to that of the ground water without any significant phase-lag. But the longer the period (> 100 days), the later becomes the motion of the ground with respect to the ground water up to 90° out of phase. The permeability of the soil is inversely proportional to the period.

RÉSUMÉ

L'utilisation de périodogrammes et de l'analyse de Fourier a permis l'étude des données fournies par les puits d'observation installés pour mesurer l'affaissement du sol à Yamanoshita, Cité de Niigata, en 1958. Les données analysées sont le niveau de la mer dans le port et la pression atmosphérique à l'observatoire météorologique, ainsi que l'affaissement du sol et le niveau de l'eau souterraine à des puits ayant des profondeurs respectives de 20, 130, 260, 380 et 610 m. Les analyses ont été faites pour une période allant de juillet à octobre 1958 (sur 240 jours).

Le niveau du sol change périodiquement avec le niveau de l'eau souterraine. Pour les plus courtes périodes (< 30 jours), le niveau du sol fluctue avec des amplitudes proportionnelles à celles de l'eau souterraine presque sans déphasage, mais pour les périodes plus longues (> 100 jours), le déphasage du mouvement du sol par rapport à celui de l'eau souterraine atteint 90°. La perméabilité du sol est inversément proportionnelle à la période.

1. INTRODUCTION

The Niigata district has suffered severe ground subsidence since 1956. In this district, the land had been subsiding before that time, though the speed was less than 10 mm a year. In 1958, yearly sinking at Sekiya, Niigata city, amounted 16 cm and, in the worst years of 1959 and 1960, subsidence attained 32 cm per year at Ono. Since 1958, the precise levellings have been repeated over the whole area of the Niigata plain, and a number of observation wells, a device to record the amount of subsidence (contraction of layers to be exact) and the level of ground water, have been installed at places where sinking has been very conspicuous.

The examination of the results thus obtained revealed that the layers at the depth from 380 m to 610 m were conspicuously contracting. This fact suggested that the real cause of the subsidence may be attributed to the drop in hydralic pressure of confined aquifer, accompanying natural gas mining. Then, regulations were frequently enforced to restrict the pumping up of a certain quantity of ground water. Consequently, the slow down in the speed of the subsidence in the area is achieved.

This paper deals with analysis of records obtained from a group of the observation wells at Yamanoshita, Niigata city, installed by the First Regional Harbor Construction Bureau, Ministry of Transportation. Author's discussions are mainly concentrated on the effect of the pumping regulations on preventing subsidence.

2. GENERAL VIEW OF NIIGATA GROUND SUBSIDENCE

Most part of the Niigata plain, roughly 8,300 km^2, is a victim of ground subsidence. The severest parts extend along the Shinano river and may be divided into two parts. One is in the down part of the river forming a strip area of 30 km × 7 km along the coast of the Sea of Japan with Niigata city in its center (the Niigata area). The other one occupys a rhombic area with its side being 15 km long, half way up the river, with Shirone city in its center (the Shirone area). Figure 1 shows the distribution of the total amount of subsi-

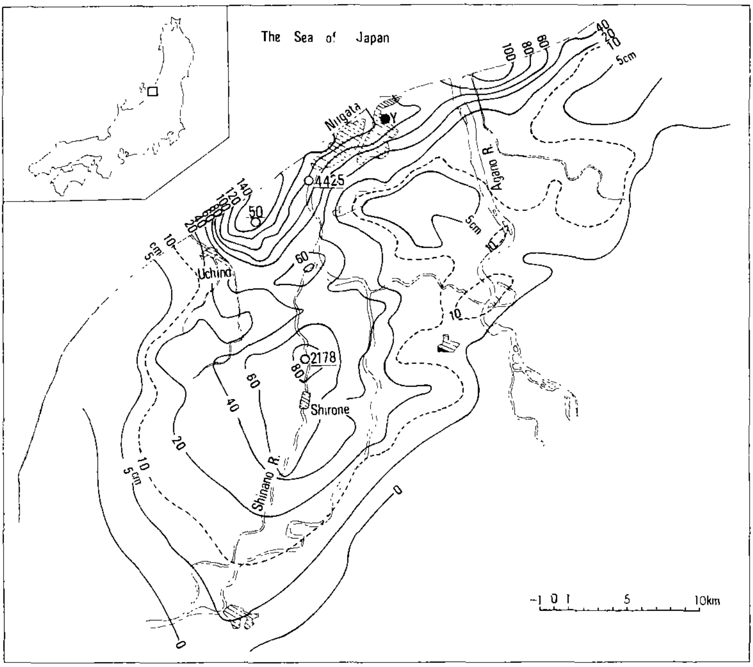

FIGURE 1. *Ground subsidence in the Niigata district (1959-1968). Y: Yamanoshita.* 50, 2178 *and* 4425*: No. of bench marks referred to figure 2*

dence which occurred during nine years from Sept. 1, 1959 to Sept. 1, 1968. The places of the most remarkable subsidence are Aoyama in the Niigata area and Ajikata in the Shirone area, where the total amounts attained 194.4 cm and 96.3 cm respectively. The progress of the subsidence is shown in figure 2 in which the integrated vertical displacements of several representative bench-marks (cf. fig. 1) obtained as the results of the precise levelling are given. Due to the subsidence, the Niigata area has been threatened with the invasion of sea water, and the Shirone area with the interruption of irrigation.

In October, 1958, the Cosmos Project was undertaken for the period of one month to test whether or not the height of the ground water level has an ability of controlling the subsidence. The wells for natural gas mining around Yamanoshita were divided into four groups by two quadrantal lines crossing at Yamanoshita. Natural gas industries in the city stopped pumping in turn from one group to another for a few days each, and the subsidence was examined simultaneously by the observation wells. Figure 4a and 4b are the records thus obtained which display the slightly slackened speed of subsidence at the 610 m well but rather sudden sink at the 380 m well making the conclusion ambiguous.

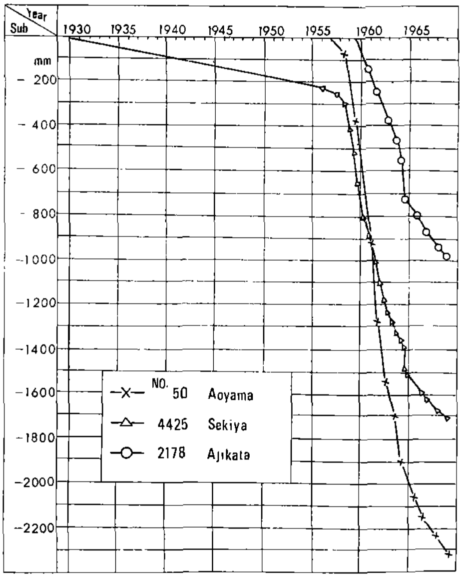

FIGURE 2. *Variation of heights at some bench marks. The locations of B. M. are referred to figure 1*

The last phenomenon, however, was proved later on as have derived from the loose structure of the bottom of the observation well.

In order to get any decisive result, a long term regulation of their own accord for water puping was put to effect. This is No. 1 regulation listed in table 1 and the results are shown in figures 4c, 4d and 4e. The subsidence speed surely diminished as shown in the figure. But sea level, which was also expected to diminish in rising, apparently increased. This unexpected gradual rise of the sea level was caused by the lowering in atmospheric pressure in the ratio of 1.9 cm/mb.

Encouraged by this fact the regulation was stepped up to an higher degree. The details of the regulations enforced up to the present are as follows;

TABLE 1.

No.	Month	Area	Dgree of regulation	Regulated amout
1	Feb., 1959	Harbor area in the city	partial	67,000 kl/day
2	Sep.-Nov., 1959	The whole city	partial	113,000 kl/day
3	July, 1960	Inner and outer area of the city	shallower than G5(610 m) partial	142,000 kl/day
4	Nov., 1961	Within 5 km out of the city border	shallower than G6(800 m) complete	170,000 kl/day
5	July, 1968	Within 10-5 km out of the city border	shallower than G3(260 m) complete	14,500 kl/day

Now, the speed of subsidence reduced to a few centimeter per year within the city but at both Uchino and Shirone, located in the partially regulated area, a speed of 8 cm/year were still observed during the period from 1967 to 1968.

3. OBSERVATION WELLS

In the subsidence, there have been 40 observation wells installed successively since 1953, of which 17 are located in Niigata city and 19 in the Shirone area. Most of the observation wells are of similar structure. For an example, the 610 m, observation well at Yamanoshita, Niigata city, is introduced here. It is constructed with double iron tubes having the diameters of 76 mm and 140 mm respectively. The inner tube stands on a cylindrical shoe fixed in a gravel layer, and the bottom end of the outer tube is capable of sliding inside the same shoe, whenever it is forced to move downwards by the earth's frictional force. The shoe has many slits on its wall through which water in the aquifer is led into the space between the two tubes. If the layers which is penetrated by the iron tubes contract, the

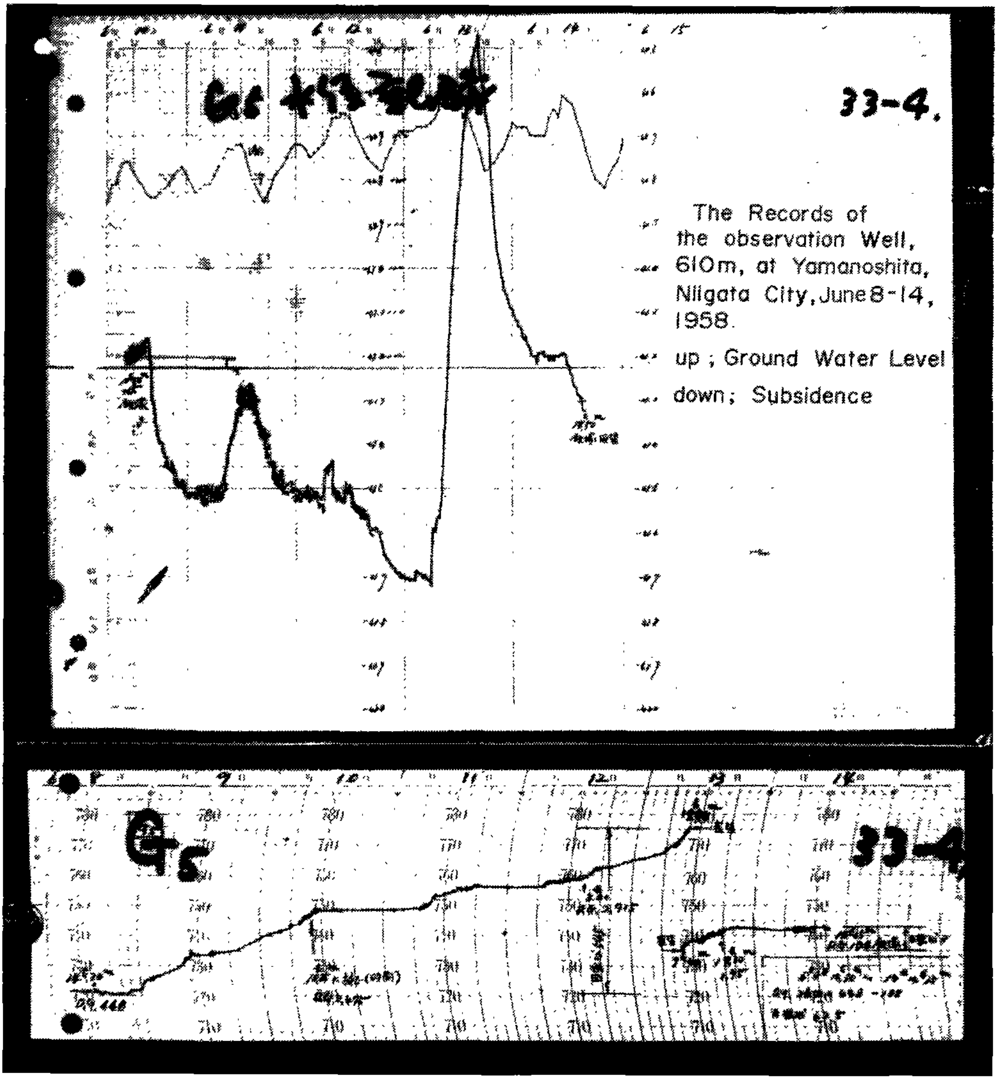

FIGURE 3. *Records of the observation well. 610 m, at Yamanoshita, Niigata city, June 8-14, 1958 Top: ground water level. Bottom: subsidence*

inner tube apparently rises above the ground surface. The amount of the rise is recorded on a recorder whose magnification is 20, and the ground water level is also recorded at the reduction of 10:1. For this purpose, the diameter of the outer tube is widen at the end portion near the surface and a small buoy is set afloat on the water surface between the double tubes.

At Yamanoshita, where the subsidence has once been very conspicuous (fig. 1), there are six observation wells involving that mentioned above; the others being those with depths of 20 m, 260 m (G3), 380 m (G4), 490 m (G4′) and 1200 m (G7). The bracketed symbols designate the aquifers at different depths, named with respect to natural gas mining. All of these observation facilities were destroyed by the Niigata earthquake of 1964, and only the 260 m, 490 m and 610 m wells were restored one year after the earthquake. The data used here are all from them except for the 20 m well which was located at Yamada, one kilometer distant from the site, until it was broken by the earthquake. Data of barometric pressure and sea level used here are due to the Niigata District Meteorological Observatory and the Niigata harbor each 4 km and 0.5 km distant from Yamanoshita.

4. RECORDS AND DATA

An example of the actual record of the observation well of 610 m depth is shown in figure 3. Curves obtained by connecting these records successively for a month (October,

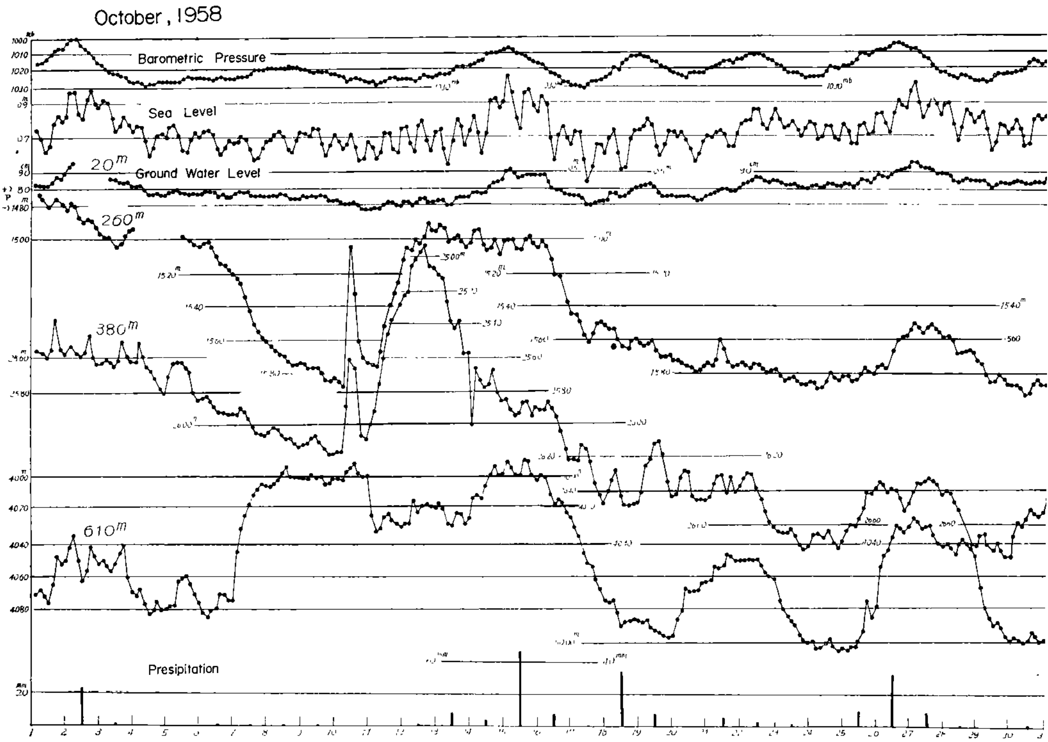

FIGURE 4a. *Ground water level recorded by the observation wells at Yamanoshita, and other elements. The data are plotted at every 3 hours for a month (October, 1958). The abnormal uprisings of water levels are due to tentative, near-by, pumping stops for the days, 1-4, 7-10, 14-16, 20-22, 26-31 for the Cosmos Project*

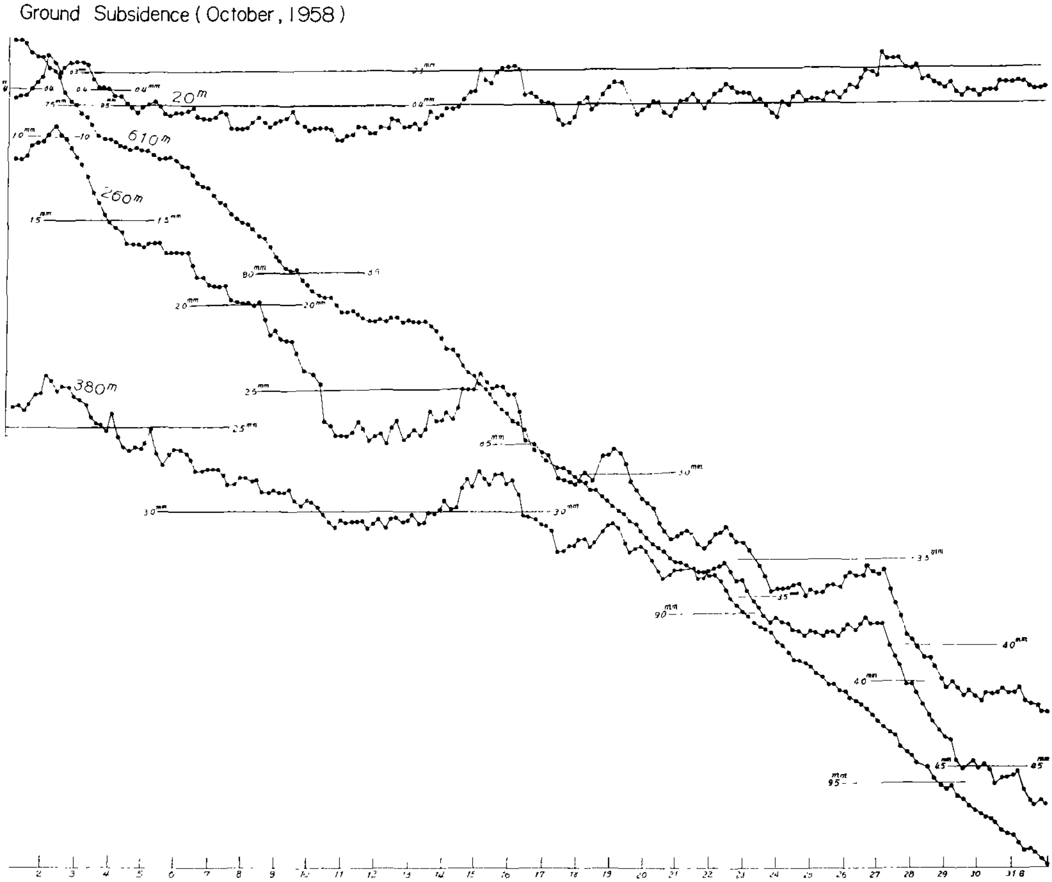

FIGURE 4b. *Ground subsidences recorded by the observation wells for the same period as in figure 4a*

1958) are shown in figure 4a, b in which the data of the barograph and the mareograph are given in parallel. Incidentally, these are the records of the Cosmos Project stated above and some levels of ground water extraordinarily change, but it can be said that they are characterized by undulation which looks parallel to that of sea level, and this tendency is occasionally disturbed by the irregular change, clearly of artificial origin, especially in the case of the deeper wells than 260 m. On the other hand, the subsidences impressed us with the strong responces to the atmospheric pressure different in degree with the wells rather than to the ground water change.

Period analyses are done with the data of about eight months, beginning from August 1st, 1958, involving the terms of the Cosmos Project and No. 1 and 2 pumping regulations in the earliest stage of the conspicuous sinking. The data applied to it are the values observed at 2 a.m. of every two days. The whole data are illustrated in figures 4c and 4d. In the latter, the data of the subsidences obtained from the observation wells other than those at Yamanoshita are also shown. At Yamanoshita, the amounts of the subsidences of different wells for the term in the proportion of 100:89:29:13 in the order of the 1200 m, 610 m, 380 m and 260 m wells in which 100 stands for 348 mm/year.

This form of the data for subsidence is not pertinent for the analysis and what used here is the successive difference between the 2 a.m. values of every two days (fig. 4e). It may

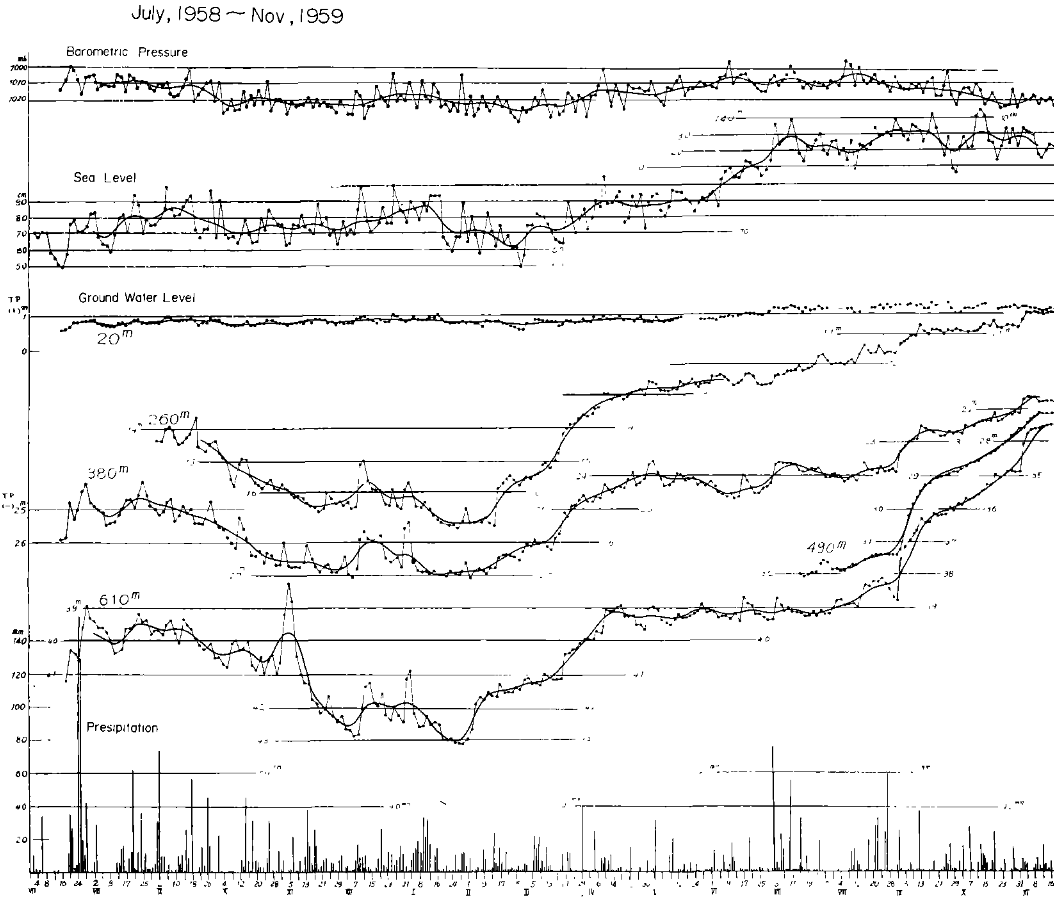

FIGURE 4c. *Ground water levels of the observation wells at Yamanoshita, and other elements. The data are plotted every 2 days for 8 months. The abnormal rising of the ground water levels are mainly due to pumping regulations started in February, September and October*

express a speed of subsidence per 4 days, while the former a displacement. The data for the sea level and the barometric pressure are of the speed type in reference to the 24 hours' mean values. In performing the analysis, the data arranged in the order of time were cut at every nth value, superposed, summed up over every column and divided by the number of superposition (cf. fig. 7). Then, the fundamental amplitude and its phase angle with the resulting curve was calculated. This process was repeated for the value of n from 3 to 15.

In executing Fourier analysis, the data of the displacement type are used for the sea level and the barometric pressure. The analysis was carried out by picking up every ten days value from the same data as in the periodic analysis, after smoothed by Spencer's method (figs. 4c and 4e).

5. GROUND WATER LEVEL AND THE SPEED OF SUBSIDENCE

(a) Ground water

Figure 5 shows the amplitude spectra for the ground water thus obtained and their phase

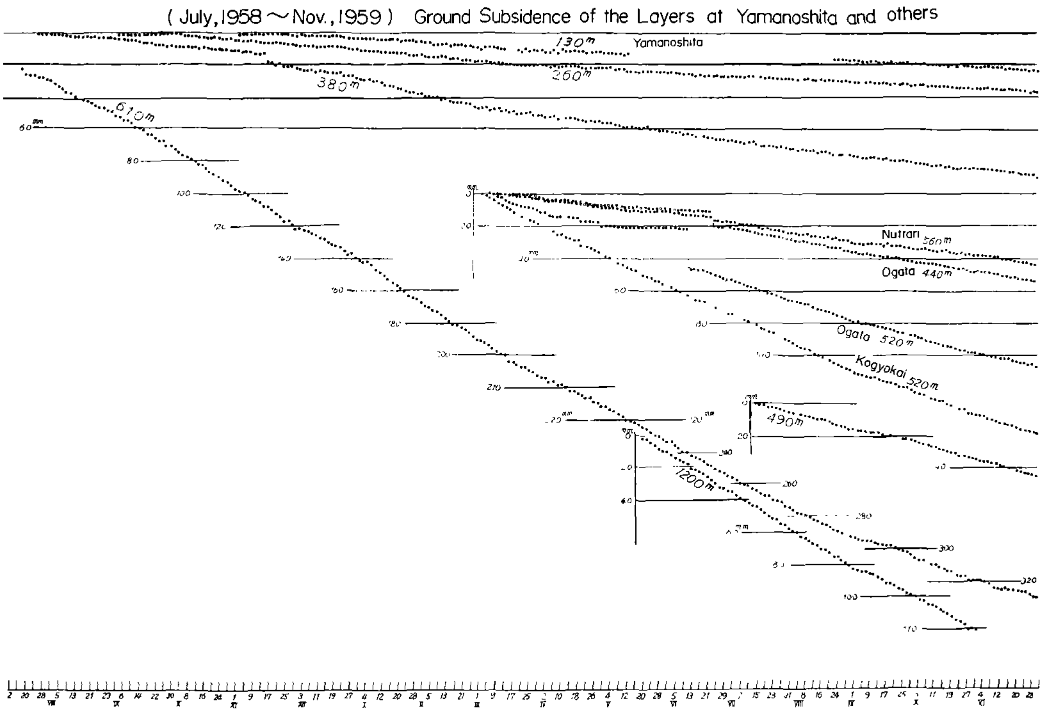

FIGURE 4d. *Ground subsidences of the observation wells at Yamanoshita and other places for the same period as in figure 4c*

differences in reference to those of the sea level. The actual period corresponding to the value of n is $2n$ days as seen from the nature of the data stated above.

Only two peaks (predominant values) are seen in the spectrum of the barometric pressure, but in that of the sea level, two more peaks appear in addition. They are occurring at $n = 11$ and 14, which correspond approximately to 21 days and 28 days, multiples of 7 days. The predominance of fluctuations with the amplitudes of $n = 4$ and 7 may also be notable in the same spectrum. This fact suggests that they might have been caused by some artificial origin with a seven days period through periodic inclination of the sea bottom. The amplitude ratio of sea level versus barometric pressure is definitely 1.9 cm/mb for the predominant periods, $n = 6$ and 9, though the values for other n are rather scattered around 1.9, from 0.6 to 6 in cm/mb.

In the spectrum of the ground water level, peaks corresponding to those of sea level may be recognized in the case of the 20 m well, but it becomes more difficult to find out any correspondence between them for the wells deeper than 260 m. The phase of the fluctuations in the ground water level is retardative of that of the sea level by about 30° for the 20 m well (See fig. 5b). In the case of the 130 m well, the ground water level was not observed. But the phase relation between the subsidence and the sea level enable us to estimate the retardation also about 30°.

The disagreement between phases of the fluctuations in ground water level of the deeper wells and those of the sea level may be due to the artificial seven day period disturbances already stated. As to the existence of the fluctuation with period of 7 days, the result from the superposition in the case of $n = 7$ shown in figure 6 may be presented as an evidence.

The amplitude ratioes of the fluctuations in the ground water to those of the sea level

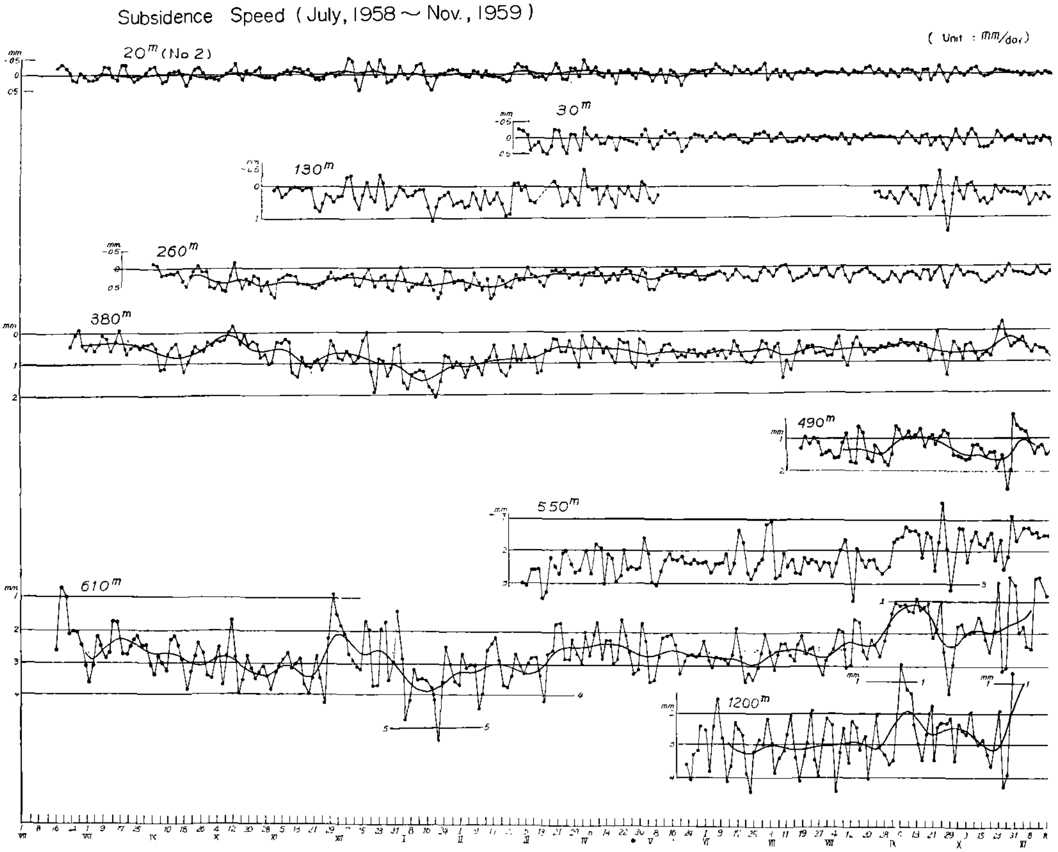

FIGURE 4e. *Subsidence speeds of the observation wells at Yamanoshita for the same period as in figure 4c. Figure 4d and 4e are the whole data with which period analysis and Fourier one were made*

for the predominant periods are not uniform. It is the case even for the 20 m well; they are 0.18 for n = 6 and 9, and 0.29 for n = 7, 11 and 14. The difference is considered as due to the effect of the atmospheric pressure.

(b) SUBSIDENCE SPEED

Figure 7 shows similar spectra concerning the subsidence speed and their phase lag in reference to those of the ground water. The peaks in the spectra for the subsidence speed agree with those of the ground water level in the cases of the shallower wells, but do not in the cases of the deeper wells. The reasons for this may be:

(1) Interference of components having periods close to each other.
(2) The subsidence observed with a deep well is the integrated compactions of various layers, yielded in relation to ground water level in respective confined aquifers.
(3) The barometric pressure obviously affect the subsidence so that the ground surface is depressed when pressure becomes higher.

The subsidence speed observed at the 20 m well has phase lag from 70° to 90° behind the ground water level for all values of n except for 5 and 10. It implies that subsidence occurs almost in phase with ground water. Assuming that the peaks of the ground water

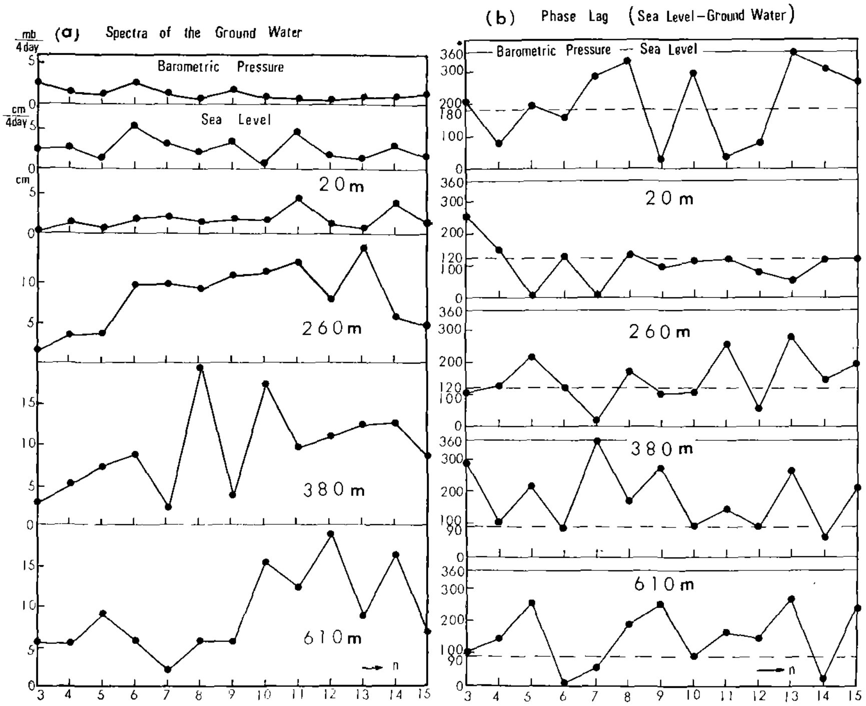

FIGURE 5. *Spectra of the ground water and the other elements and their phase lags from the sea level (Period analysis). The positive sense of the axes of both elements are taken upwards*

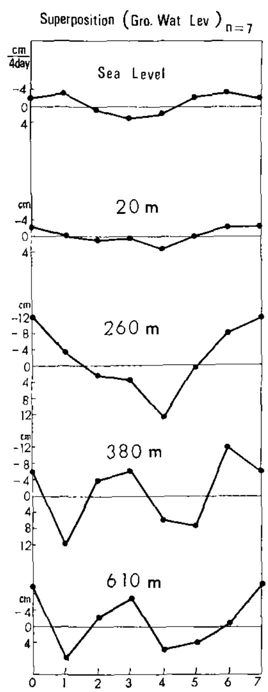

FIGURE 6. *The result of the superposition of data for ground water for* n = *7 showing the existence of a 7 day period*

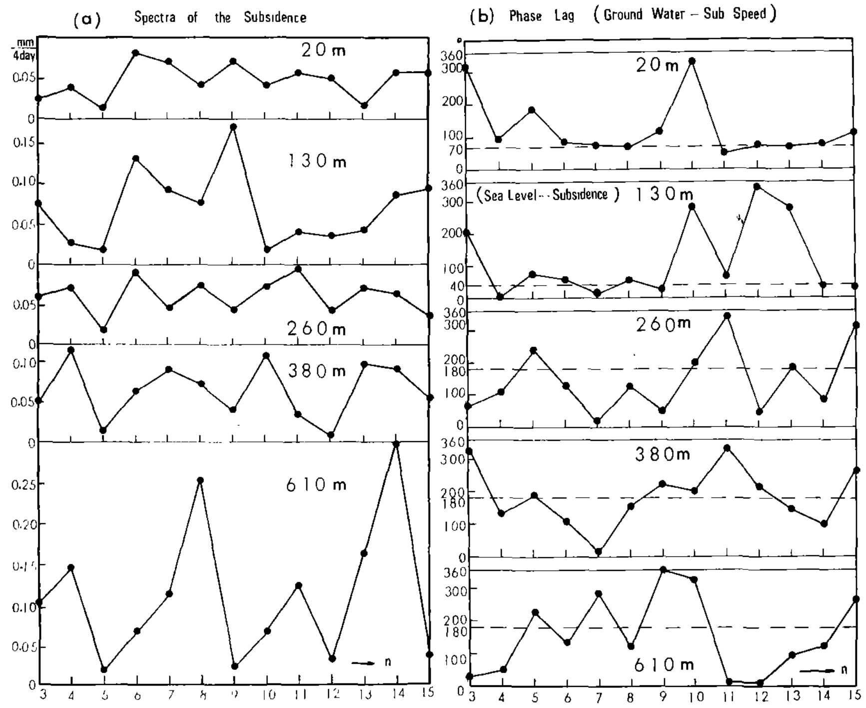

FIGURE 7. *Spectra of the ground subsidence and their phase lags from the ground water (Period analysis). The positive axis of the subsidence is taken upwards except for the 20 m well for which it is downwards*

at $n = 6$ and 9 for the 20 m well cause the subsidences in proportion to their amplitudes and that if they are not actually in proportion, the deviations from it should be the effect of the atmospheric pressure acting in proportion to the deviation, we get the ratio of amplitude of subsidence versus that of ground water at 0.024 and that of subsidence versus barometric pressure at 0.0036 mm/mb. Assuming further that the variation with predominant periods of the sea level are in proportion to that of the ground water for the 130 m well, we get 0.023 mm/mb as the ratio of subsidence versus atmospheric pressure for the well.

For the wells deeper than 260 m, conditions are entirely different, though they are also very sensitive for the variation in the atmospheric pressure as is understood by comparing figure 4a with 4b. In many cases, as seen in figure 7, phase lags of the variations in subsidence observed at the deeper wells are not close to 90° but nearly zero (360°) or 180°. Minute examination of the result of the superposition (for instance, cf. fig. 6) reveals that the fluctuation with the period of about half n is not smoothed away because of the shortage in number of superposition. In addition, the barometric pressure seems to emphasize the fluctuations with shorter period. Consequently, the form of the result of the superposition for subsidence speed sometimes looks entirely different from that of ground water. But considering such effect of the barometric pressure as mentioned above, it may be comprehensible that the phase lag of the subsidence behind the water is nearly zero. Figure 8 shows an example in which fundamental forms of the results of the superposition with the data from the 260 m and 380 m wells are rather distinct.

Besides the period analysis mentioned above, the 24-ordinate Fourier analysis was made with the data of 240 days. This was done for the purpose of elimination of small

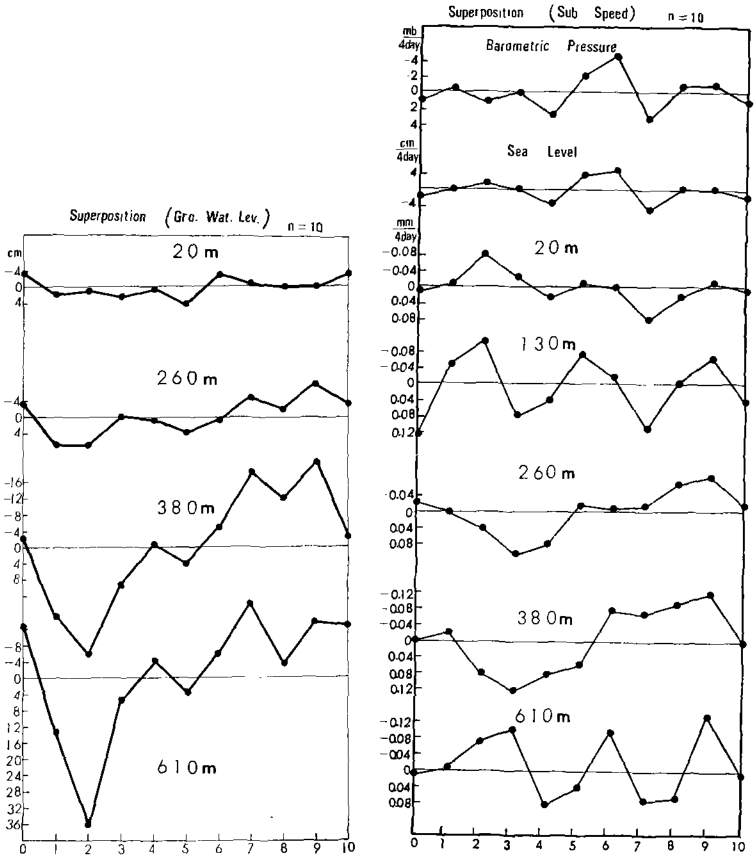

FIGURE 8. *The result of the superposition of data for* n = *10, showing the ground water nearly proportional to the speed of subsidence for the 260 m and 380 m wells and the large effect of the atmospheric pressure on the subsidence speeds*

disturbances with shorter periods that mask the general trend of the phenomena. The results are shown in figure 9. As seen in the figure, there are a few peaks of the coefficients. But the peaks with the ground water do not coincide with those with the subsidence. Since the violent change of the ground water level in the 380 m and 610 m wells are supposed to have been expressed by the first few terms of the Fourier series, any predominant ones among them except for the fundamental one may not have any special meanings. Only common peaks corresponding to $n = 5$ may be significant in figure 9a, and it has the period of 48 days, a period very close to a multiple of 7 days, suggesting that it has been brought about by the artificial origin. On the other hand, in figure 9b which concerns with the subsidence, common peaks corresponding to $n = 9$ (27 days) are notable, instead of $n = 5$. They also may be due to the atificial origin.

The amplitude of the fluctuation in the ground water level of the 20 m wells are very small as compared with those of the deeper ones, but the amplitude of the fluctuation in the subsidence of the 20 m well is not so small as compared with the others. At the 20 m well, subsidence and ground water changes occur simultaneously even for a long period, but the subsidence speed is almost in phase with the ground water for $n = 1$, the fundamental term, in the case of the 610 m and 380 m wells (cf. fig. 9c). This is the same conclusion as obtained in the period analysis.

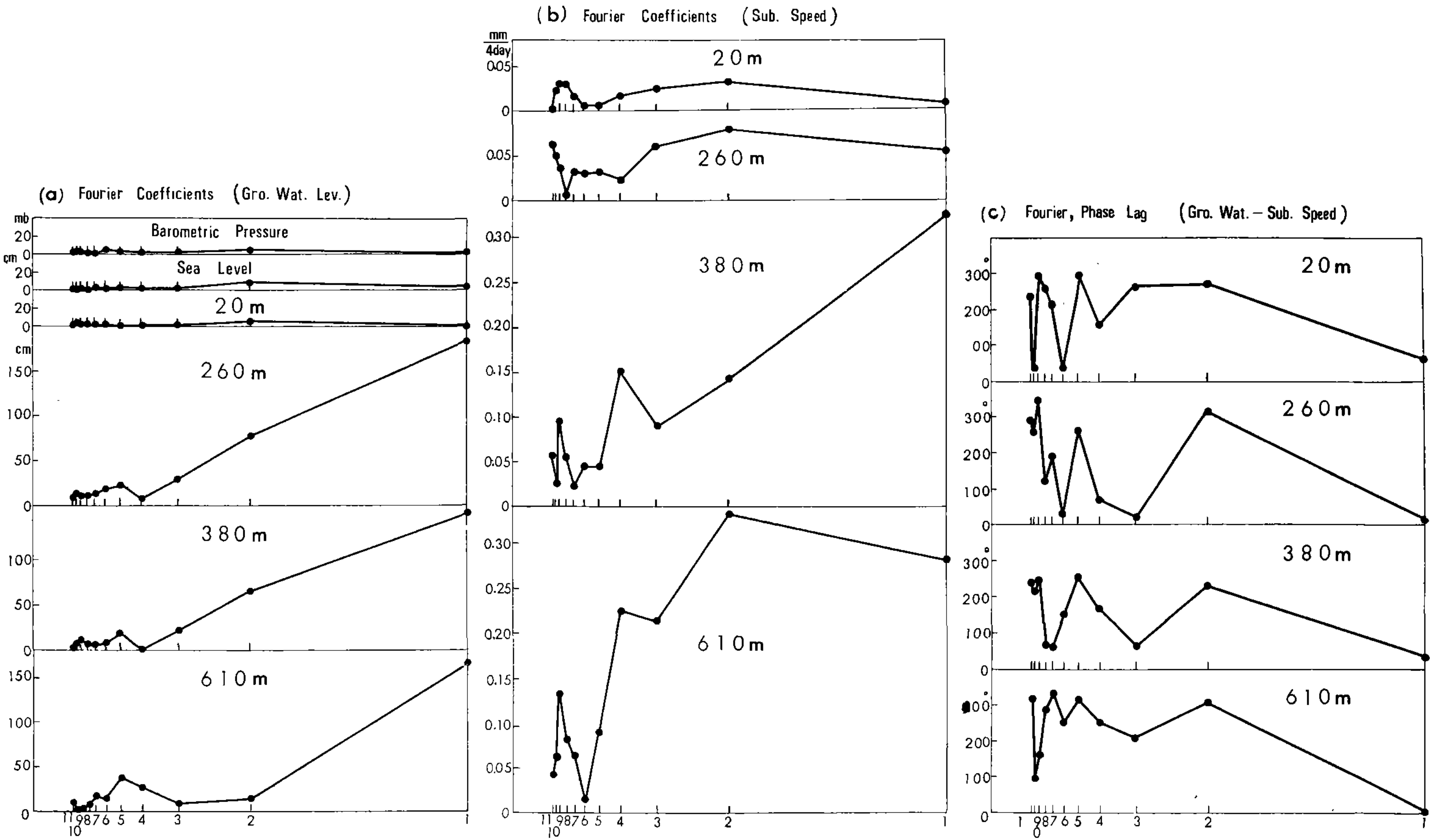

FIGURE 9. *The result of Fourier analysis*

From the discussion mentioned above, it is conceivable that when the periodic change in the ground water level is small enough in amplitude, the subsequent motion of the ground surface responds without time lag as if it is caused by a volume change of sandy transition layer, situating between the gravel layer and the silt or clay layer, while the sufficiently large amplitude affect the water pressure in the silt-like layer over the transition layer.

6. STANDARD LEVELS OF THE GROUND WATER

The above statement concerns with the analysis of the data shorter than one year. But the data for about 10 years from the beginning of the severe subsidence are the object of discussion hereafter.

In figure 10 are shown monthly mean values of the subsidence speed per day and the height of ground water level of the 610 m well together with those of barometric pressure and mean sea level, the former two changing gradually in parallel with each other, that is, the subsidence speed decays with rising ground water level. It is also observed that one year period of subsidence speed can be clearly traced in parallel with the barometric pressure without any phase lag larger than one month. The ratio of the undulation in the subsidence speed to that in the barometric change is roughly estimated as 0.08 mm/day.mb. Although the ratio of ground water level to the mean sea level with respect to the change with yearly period is roughly 0.2-0.3, it is not appear distinctly as in the case just stated above, because the range of the change of the ground water level is extremely large. A similar character may be put to the behavior of the variation in subsidence measured as referred to the 380 m well.

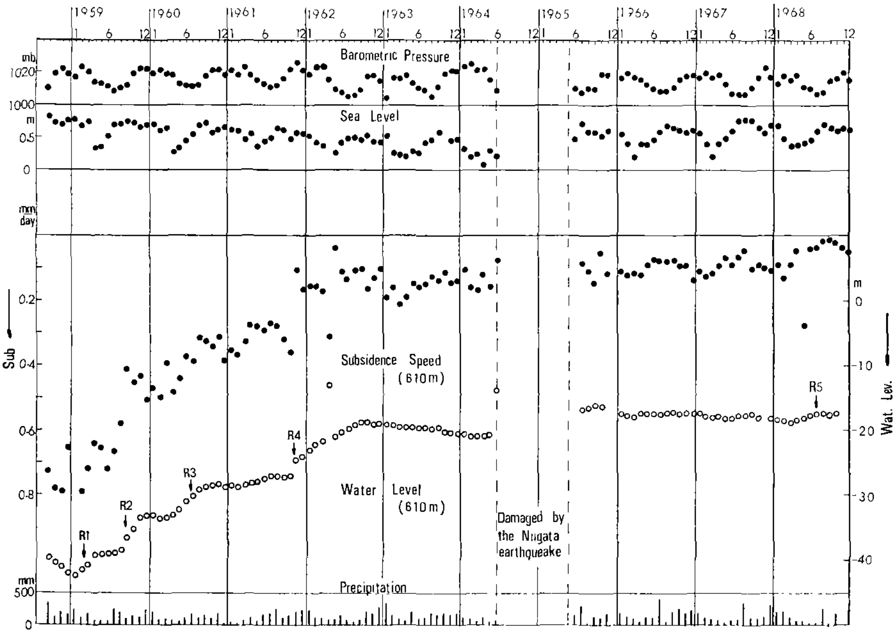

FIGURE 10. *Subsidence speed and ground water level of the 610 m well at Yamanoshita with the other elements. Monthly mean values are plotted for every elements. Rs show the times of enforcement of the pumping regulations*

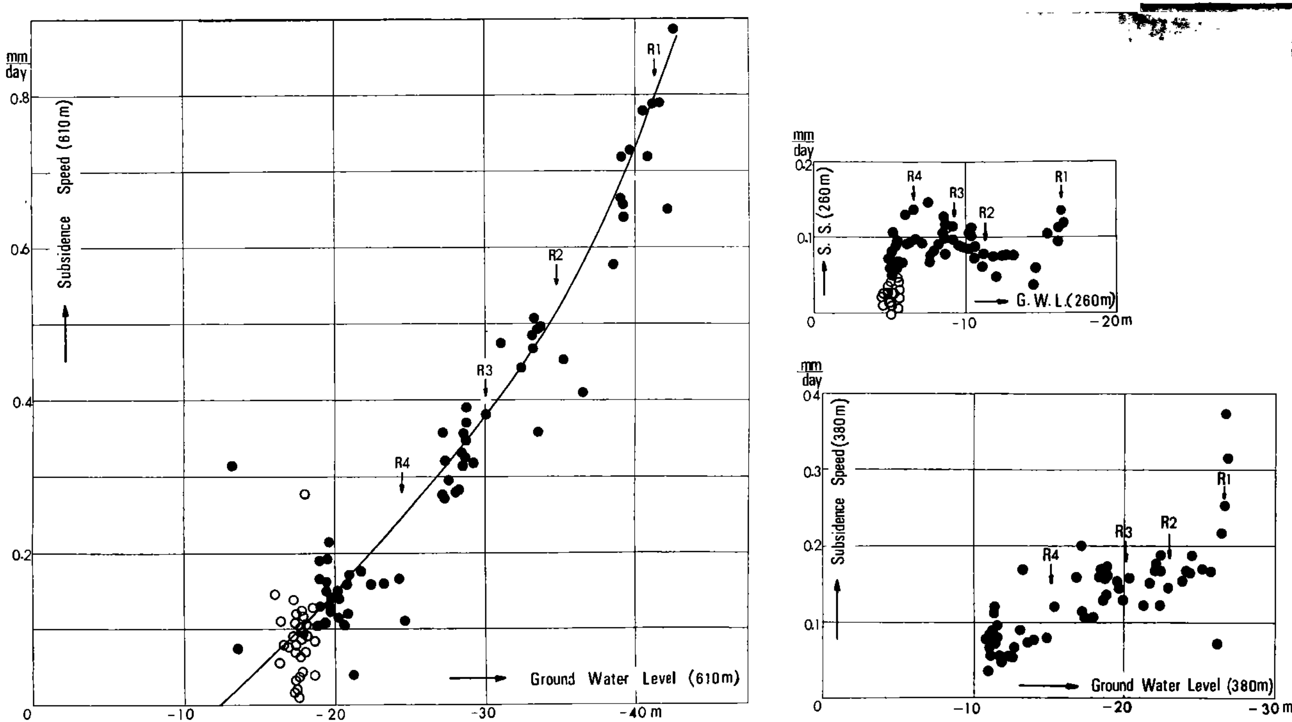

FIGURE 11. *Relations between ground water levels and subsidences observed by the observation wells at Yamanoshita. White circles are the data obtained after the Niigata earthquake*

Now, along the curve of the ground water level in figure 10, are seen the signs of *R* at 5 places. They indicate the times when the regulation on the pumping of ground water were enforced and we see a marked decrement of the subsidence speed just after every moment when the regulation is put to effect. It took about a few months for the 610 m well to attain the stationary state after the regulation is effected for the first three cases and about six months for the fourth case.

Figure 11 shows the relation of the subsidence speed versus the ground water level, in which (11a), (11b) and (11c) are for the cases of the 610 m, 380 m, 260 m wells respectively. Series of points in every diagram should be thought to shift with the time lapse generally from upper right to the lower left in the figure. Except for the case of the 260 m well, close relation is clearly seen between rise of the ground water level and decay of the subsidence speed. The ratio *k* of the subsidence speed to the ground water level for the 610 m well is about 0.02 mm/day.m., with the ground water level of -20 m, while it is 0.25 and 0.05 respectively when the ground water level is about -30 m and -40 m.

The cause of this variety of the ratio may be the influence of the contraction of the upper layers controlled by the water of the other aquifers. One of the main aquifer shallower than 610 m is that of 380 m, and the value of *k* with regard to this aquifer may be taken as constant, say, about 0.008 mm/day.m. (-cf. 11b). On the contrary, in the case of the 260 m well, the subsidence speed did not change appreciably for a while in spite of the monotonous rising of the ground water level due to the pumping regulations. But after the fourth regulation of July, 1958, which involved entire suspension of pumping within Niigata city, the situation is reversed. The level of the ground water stopped changing while the speed of subsidence continued diminishing. This may prove that the shallow layers above 260 m have stopped shrinking. It is a matter of course that similar phenomena have occurred in the cases of the 380 m well and the 610 m well after the fourth regulation (cf. 11a, 11b).

From figure 11e, -3 m may be taken as an upper limit of the level of the ground water for the 260 m well to stop subsidence. The curve expressing the tendency of the relation between the subsidence speed and the level of the ground water suggests -12 m as an upper limit for the 610 m well, but taking the changed condition just mentioned which occurred after the fourth regulation into consideration, a deeper level, say, -15 m, may be safely taken as a limit. Similarly, for the 380 m well, though the limiting point seems to be -2 m from general tendency in figure 11b, it may be considered to be enough to be taken as deep as -6 m for it.

Figure 12 shows the relation between the speed of subsidence at the 610 m well and that at the 380 m well. In this case, too, the ratio of the former to the latter varies depending on the stage of the regulation. Before the third regulation, the speed of subsidence at

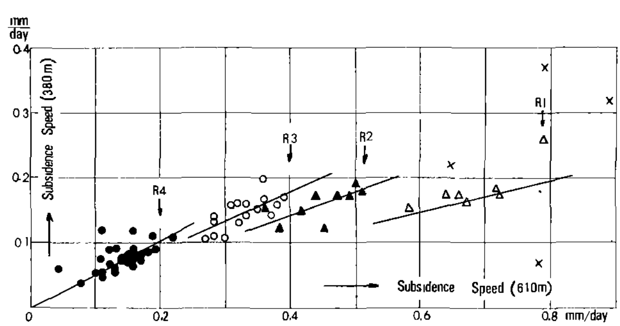

FIGURE 12. *Relation between two subsidence speeds of the 610 m and 380 m wells. Any group of data with similar marks occupies common interval of any two successive regulations*

the 610 m well diminishes more rapidly than the other but finally attains to a constant ratio of 2:1 to the other, showing that only the layer between 610 m to 380 m was recovering from the contraction before the third regulation. After that, the contraction of the layer above 380 m stats to slow down and tends to decay after the fourth regulation.

The points in figure 12 may be devided into four groups according to the intervals of the regulations. The points in each group are apparently on a line passing through the origin showing that the barometric pressure affects the subsidence speeds at both wells in proportion to the speed. It suggests that the atmospheric pressure affects homogeneously the whole layers.

Figure 13 shows the relation between the subsidence speed at the 610 m well and that at the 1200 m well. In this case, too, the ratio of subsidence speed at one well to the other is different according to the stage of the regulation. When the speed was large the ratio was about 5:7, and when small, it is 1:1. This figure indicates that the major part of Niigata ground subsidence is due to the contraction of the layer lying between 380 m to 610 m.

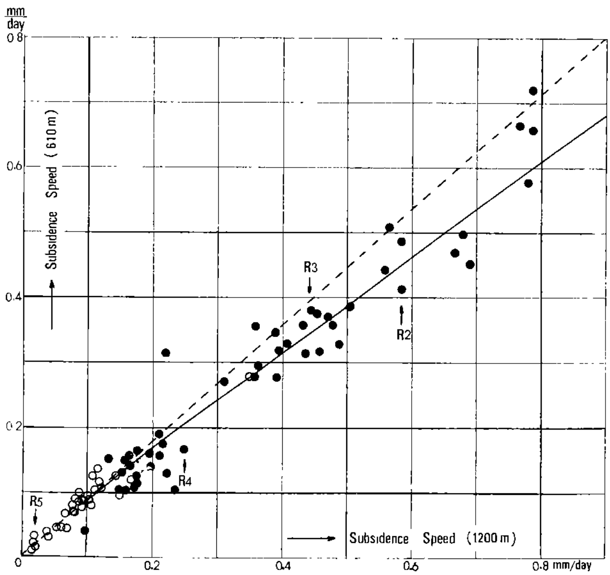

FIGURE 13. *Relation between two subsidence speeds of the 610 m and 1200 m well*

7. CONCLUSION

The ground subsidence occurring in the Niigata plain is extremely larger in scale and depth than those in Tokyo and Osaka. It is of some different nature compared with similar phenomenon in other regions, probably due to the difference in soil character. The layers are composed of sandy and silt-like soil, poor in clay, and contain ground water with soluble gas. This may be one of the reasons why the periodic components in the ground subsidence appear subsequent to the periodic change in the barometric pressure.

The complex structure of the layers in this district had made it difficult to obtain any positive results from the analysis of the data for short interval of time, say, less than a year. The sufficiently long term data now available, however, made it possible to distinguish a number of small disturbances, natural or artificial, from the general trend and to ascertain the validity of the results obtained with the short interval of data. Thus, the facts displayed in the present papers approve us to conclude that the subsidence in the

Niigata district is also be attributable to the same cause as in Tokyo and Osaka. In these cities, the proof of the theory that the subsidence is caused by the overpumping of the ground water, was given when the factories in the cities were destroyed by the War. But in the case of Niigata it was given by a series of regulations on pumping of ground water. The regulations have resulted in a large slow down of subsidence, although there are still some locations outside of the city where people suffer from subsidence as in the Uchino and Shirone areas.

There are some problems left in future to be solved. One is that concerning the efficiency of the recharge of ground water. Since 1961, the pumping regulation has been enforced with a net value of pumping equal to the total amount of the pumped ground water substracted by the recharged amount. But its efficiency has not been assured yet.

Another problem is to examine the possibility of the rise of the ground surface by supplying sufficient ground water pressure. In the case of Tokyo and Osaka it was recognized difficult to raise it, but in the case of Niigata, where the nature of soil is somewhat different from those of the other regions, it is worth while to extend examination in this direction. Study in these problems, which relate to each other, will be a clue to better methods to prevent further subsidence and to improve the conditions.

ACKNOWLEDGMENT

The author thanks Dr. N. Miyabe for inspecting this paper. The author also thanks the authorities of the Ministries of International Trade Industry, Agriculture and Forestry, Construction, and Transportations for the data presented through the Special Committee for Niigata Land Subsidence under the superintendence of the Science and Technology Agency and the Land Subsidence Subcommittee, Conservation and Disaster Prevention Committee, STA.

This research was supported in part by the Science and Technology Agency, Niigata Prefecture and Niigata City.

DISCUSSION

Intervention of Dr. Kiyoo WADATI (Japan)

Question:

The land subsidence of the Niigata region is mainly caused by the withdrawal of groundwater from deeper layers when compared with the cases of Tokyo and Osaka. Did the close relation between the subsidence rate and groundwater level exist even for a short period such as 5 or 10 days as it was reported in the investigation of Osaka and Tokyo?

Answer of Dr. HIRONO:

The linear relationship is observed in long term series of more than one month, however, we could not establish it for short periods of several days. This, I think, is attributable to the strong disturbances due to other influences such as atmospheric pressure, tides, etc. as well as the thickness of the sand layers of Niigata. In other words, I wanted to point out that for a long enough time period, the rate of subsidence corresponds to the change of water level. As to the detailed report on the short term change of several days, I would like to refer to my present paper.

A LINEAR RELATIONSHIP BETWEEN LIQUID PRODUCTION AND OIL-FIELD SUBSIDENCE

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U.S. Geological Survey, Menlo Park and Sacramento, California

ABSTRACT

Correlations between production and subsidence in the Inglewood oil field of California indicate that subsidence has varied approximately linearly with net liquid production. Similar relations in three Venezuelan fields and the Wilmington (during the primary production stage) and Huntington Beach fields in California, suggest that this linearity may be a general phenomenon. The ratio V_s (volume of subsidence)/V_p (volume of liquid production) ranges from 0.08 in the Inglewood field to nearly 1 in one Venezuelan field. Early production stages commonly have been characterized by non-linearity.

The observed linearity is imperfectly understood. If compaction is considered a linear function of effective stress over the relevant stress range, simple analogy with a tightly confined artesian system (where production must derive from liquid expansion and/or reservoir compaction) indicates that V_s/V_p should remain: (1) constant and close to but > 0 for those fields characterized by slightly compressible reservoir materials (and invariant liquid compressibilities); and (2) constant and close to but < 1 for those fields characterized by highly compressible reservoir materials. Changes in V_s/V_p through time may be related to changes in the produced gas/liquid ratio.

RÉSUMÉ

Les corrélations entre la production et la subsidence dans le champ pétrolifère d'Inglewood (California) montre que l'affaissement a varié approximativement de façon linéaire avec la production nette de liquide. Une relation semblable existe dans trois champs au Vénézuela et dans les champs de Wilmington (phase de production principale) et de Huntington Beach (California), suggérant que cette corrélation linéaire peut être un phénomène général. Le rapport V_s (volume de subsidence)/V_p (volume de production) varie entre 0,08 dans le champ d'Inglewood et près de 1 dans un des champs vénézuéliens. Pendant les premières phases de production cette corrélation n'est généralement pas linéaire.

La relation linéaire observée est imparfaitement comprise. Si la compaction est considérée comme étant une fonction linéaire de la tension régnante, pour un domaine de valeurs de tension raisonnable, la simple analogie avec un système artésien fortement enfermé (où la production doit dériver de l'expansion du fluide et/ ou de la compaction du réservoir) montre que le rapport V_s/V_p doit rester (1) constant et faiblement > 0 pour les systèmes à matériaux de réservoir faiblement compressibles (avec compressibilités des fluides invariantes), ou (2) constant et faiblement < 1 pour les systèmes à matériaux de réservoir fortement compressibles. La variation de V_s/V_p avec le temps est peut-être due à celle du rapport gaz/liquide produit.

INTRODUCTION

Investigation of the differential subsidence centering on the Inglewood oil field of California has identified an approximately linear relationship between liquid production and various aspects of subsidence. This relationship was unexpected here, for its existence had been specifically denied in the well-studied Wilmington oil field (Gilluly and Grant, 1949, pp. 501-502), where the subsidence was generally considered directly proportional to measured reservoir-pressure decline or to various logarithmic expressions of liquid production (Gilluly and Grant, 1949, pp. 463, 502-518; Miller and Somerton, 1955; Hudson, 1957, pp. 43-59). None of these relationships were recognized in the Inglewood field. This paper examines the basis and character of the linear relationship identified in the Inglewood field, the extent of its occurrence in other oil fields, and a possible explanation for its existence.

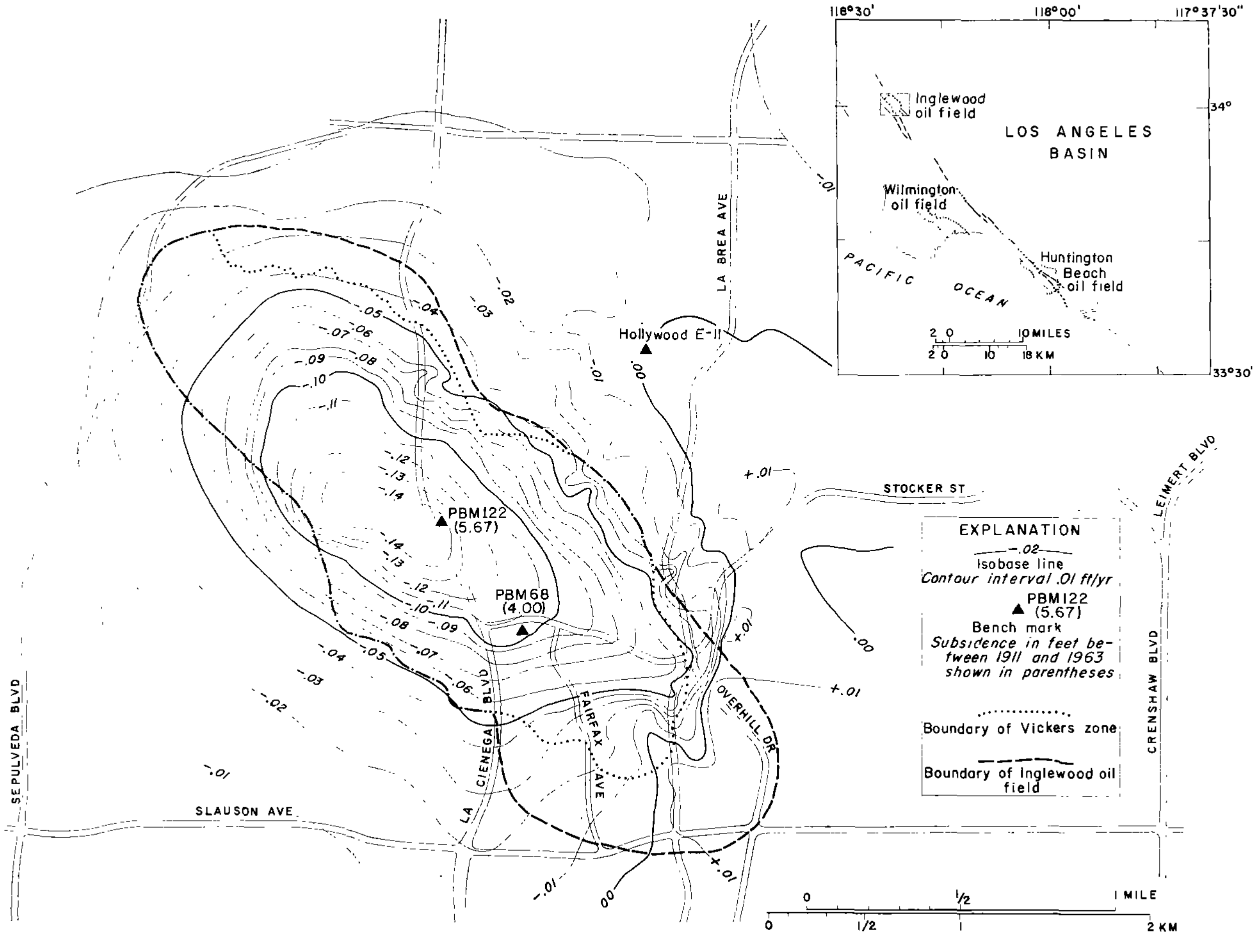
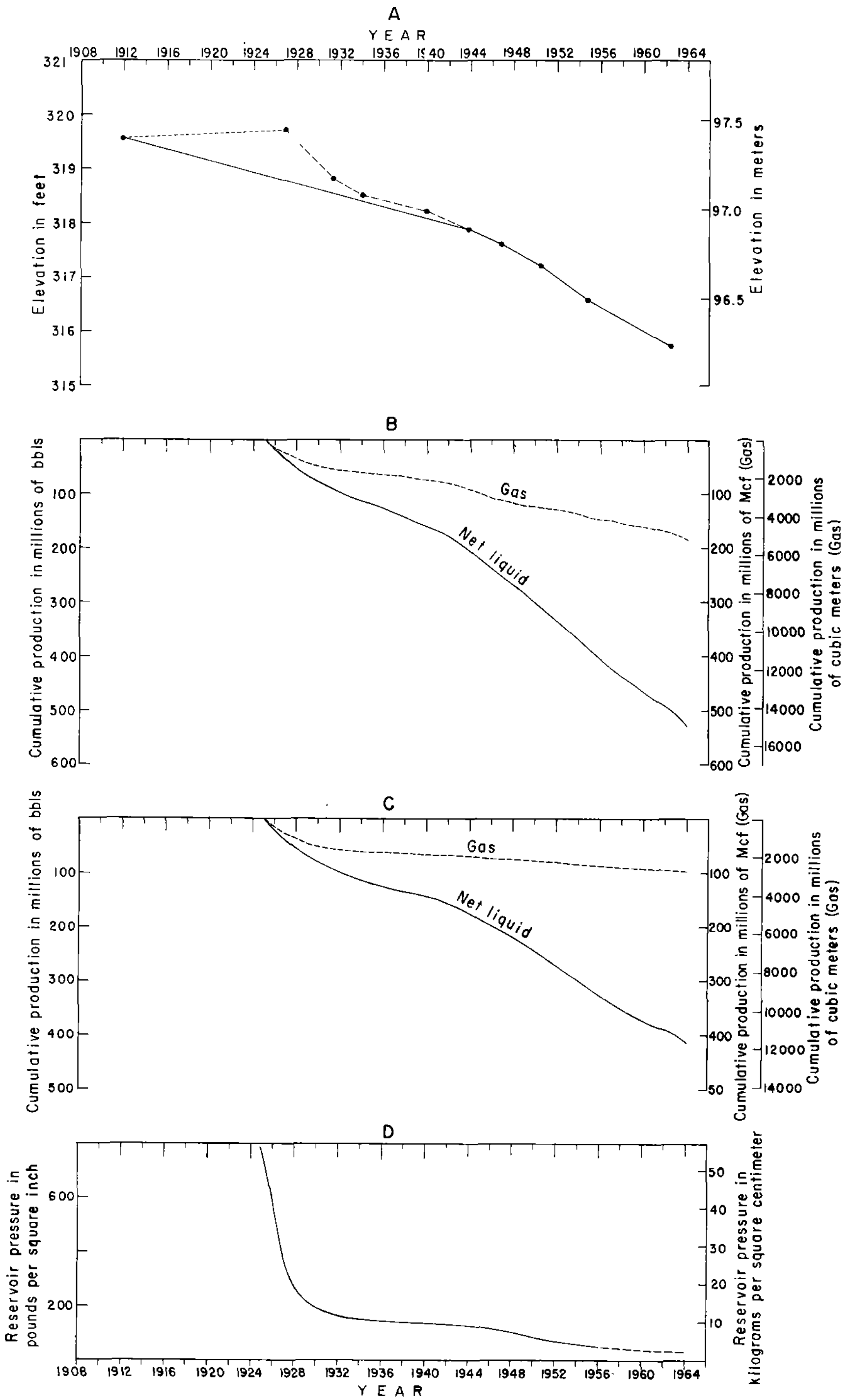


FIGURE 1. *Average annual elevation changes over the Inglewood oil field, with respect to bench mark Hollywood E-11; 1958 to 1962 (Los Angeles Department of Water and Power, 1963, Unpublished data)*



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SUBSIDENCE IN THE INGLEWOOD FIELD

The Inglewood oil field was discovered in 1924 in the northwest part of the Los Angeles basin (fig. 1). It is underlain by a sequence of gently to moderately arched and conspicuously faulted Cenozoic sedimentary and volcanic rocks (Driver, 1943, p. 307; Yerkes and others, 1965, pl. 4). Through 1963 about three-quarters of the oil and about four-fifths of the net liquid had been produced from the relatively shallow Vickers zone (fig. 2c), which ranges in depth from about 800 to 3100 feet and ranges in age from middle to upper-Pliocene (Standard Oil Company of California, 1966, unpublished data).

Sub sidence over the Inglewood field (fig. 1) was first recognized in 1943 and has since been closely monitored by the Los Angeles Department of Water and Power (1963, unpublished data). Vertical movement at PBM 68 (fig. 1), the only bench mark within the subsiding area that was leveled before production began and has been releveled from time to time since, is shown in figure 2A. Subsidence at PBM 122 (fig. 1) closely approximates the maximum subsidence within the field; calculation of the cumulative subsidence at this bench mark has been based in part on a comparison with measured subsidence at PBM 68.

The indirect comparisons between subsidence and production shown in figure 2 indicate both a coincidence in time between the beginning of production and the beginning of subsidence at PBM 68 and a close correspondence between rates of net liquid production and rates of subsidence . Direct comparisons between liquid production and subsidence derive in part from data presented in figure 2 and are shown here in figure 3.

The generally linear relationships between net-liquid production and the depth and volume of subsidence are clearly evident; correlations between other measures of liquid production and subsidence are less pronounced. Linearity between net-liquid production and subsidence is especially pronounced if consideration is restricted to the Vickers zone. Several observations indicate that nearly all of the subsidence derives from compaction of the Vickers zone: (1) through 1963 about 72.5 percent of the oil and about 78 percent of the net-liquid production had come from the Vickers zone; (2) roughly 30 percent of both the subsidence at PBM 68 and the liquid production between 1911 and 1963 had occurred by 1934, up to which time there had been almost no production from zones other than the Vickers; (3) variations in the subsidence rate have been apparently independent of production from zones other than the Vickers, whereas there is a close correspondence between the rates of subsidence and the rates of production from the Vickers zone.

The ratio of V_s (volume of subsidence) to V_p (volume of net-liquid production) for the full period of measurement shown in figure 3, ranges from 0.08 for the entire field to 0.10 where V_p is limited to production from the Vickers zone. The slope of the clearly linear portion of the V_s/V_p plot for the Vickers zone alone is nearly identical with that obtained through comparison of V_s with V_p over the entire period of observation; the slope of the linear segment of the V_s/V_p plot for the full field (about 0.074), on the other hand, is somewhat less than the ratio of V_s to V_p for the full period of production.

FIGURE 2. A. *Subsidence at PBM 68 with respect to bench mark Hollywood E-11. Long-dashed-line portions calculated from comparisons with vertical movements at nearby bench marks; short-dashed-line portion covers period devoid of measured elevations within the subsidence area centering on the Inglewood oil field.* B. *Cumulative net-liquid (stock tank oil plus water produced, minus water injected) and gas production from the Inglewood oil field through 1963. Compiled from production statistics of the California Division of Oil and Gas.* C. *Cumulative net-liquid and gas production from the Vickers zone of the Inglewood oil field. Compiled and computed from production statistics of the Conservation Committee of California Oil Producers.* D. *Fluid pressure decline midway through the central Vickers zone.Calculated from fluid pressure decline curve for the upper Vickers East zone (California Department of Water Resources, 1964,* p. *16, pl. 9)*

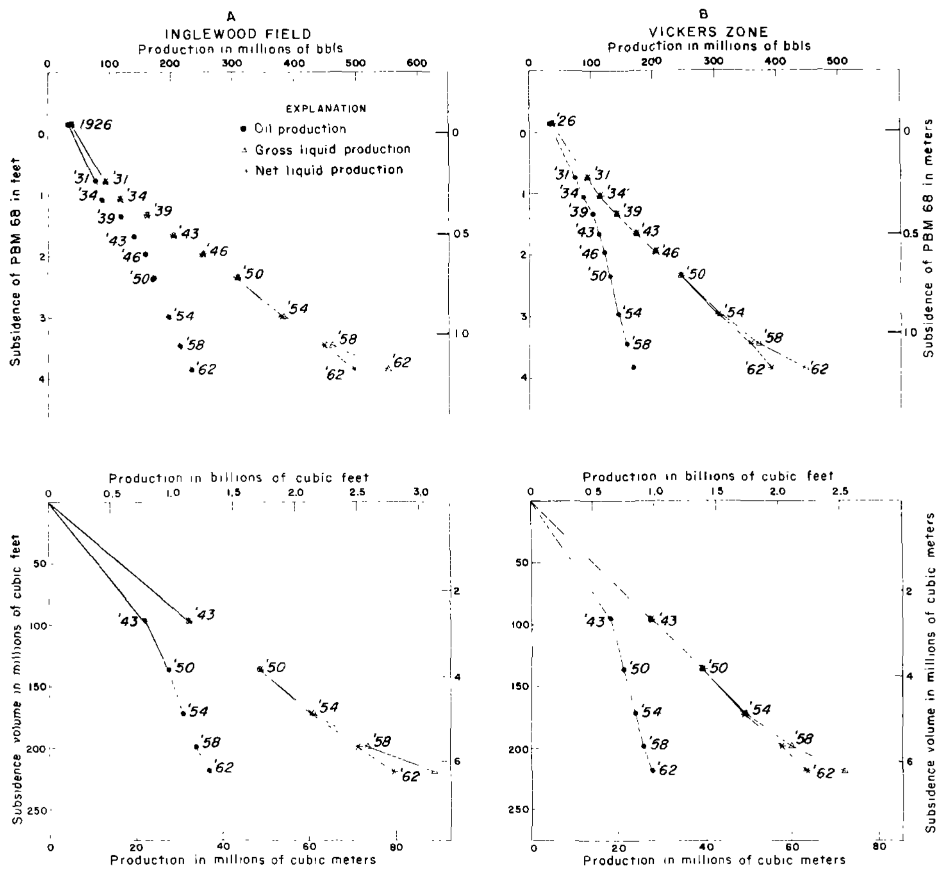

FIGURE 3. *Cumulative oil, gross-liquid, and net-liquid production from both (A) the entire field and (B) the Vickers zone of the Inglewood oil field versus cumulative subsidence at PBM 68 since 1911 and cumulative volume of subsidence over the field since 1911. Calculation of successive volumes of subsidence based on the assumption that their shapes have closely approximated inverted elliptical cones. See figure 2 for explanation of dashed-line portions*

LIQUID PRODUCTION-SUBSIDENCE RELATIONSHIPS IN OTHER OIL FIELDS

There are very few published examples of oil-field subsidence in which both the production and subsidence are well enough known to be compared over extended periods of time. The generally missing element is frequently repeated leveling over a suitably dense survey network.

BOLIVAR COAST OIL FIELDS

The best published comparisons between liquid production and subsidence come from three unidentified oil fields along the Bolivar Coast of Lake Maracaibo, Venezuela (van der Knaap and van der Vlis, 1967). Plots of cumulative gross-liquid production versus cumulative volume of subsidence for each of these fields are shown in figure 4. "Gross liquid" is, in this instance, almost certainly equal to "net liquid," for none of the fields in this area are known to have been repressurized through water flooding. Field A is thought to have lost liquids through migration to adjacent fields (van der Knaap and van der Vlis, 1967, p. 94); correction for this loss as production would reduce the average slope of curve A, possibly by simply shifting it to the right and thereby requiring that the pre-1935 segment of the curve be shallower than the post-1935 leg. Ratios of V_s to V_p for the entire period of measurement for each of these three fields are: (A) 0.85; (B) 0.69; (C) 0.62. Slopes of V_s/V_p over the linear portions of the three curves are: (A) 0.78; (B) 0.98; (C) 0.92.

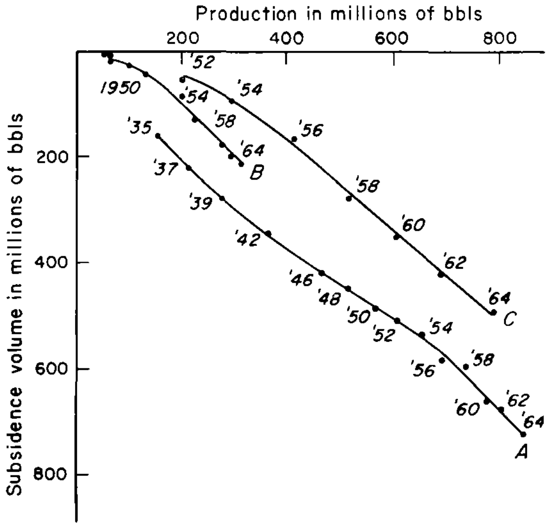

FIGURE 4. *Cumulative volume of subsidence versus cumulative gross-liquid production for three unidentified Bolivar Coast oil fields (after van der Knaap and van der Vlis, 1967, p. 94). Techniques utilized in measuring the volumes of subsidence have not been specified*

WILMINGTON OIL FIELD

The well-known Wilmington oil field, located along the south coastal section of the Los Angeles basin, is one of the few additional fields for which the data permit a comparison between liquid production and subsidence. Owing, however, to the absence of offshore control and the presence of interfering subsidence domains, it has not been possible to measure or calculate, other than very roughly, the cumulative subsidence volumes over successive production intervals. Thus, although no attempt is made here to compare liquid production with successive volumes of subsidence, we may compare various aspects of liquid production with subsidence at a point (fig. 5).

According to records of the Long Beach Department of Oil Properties (D.R. Allen, 1969, written communication), the ratio of V_s to V_p was about 0.62 for the period between 1936 (when production began) and 1956 (immediately preceding the initiation of full-scale water flooding). Our own crude estimate, as derived from the addition of three successive volumes of subsidence whose configurations are assumed to have approximated inverted elliptical cones, indicates that the maximum value of V_s to V_p for the period 1936 through mid-1957 was no more than about 0.7; subjective correction for the changing configuration of the subsidence bowl between 1946 and 1951 would reduce this ratio to about 0.55-0.60, a figure in reasonable agreement with that suggested by the Department of Oil Properties data.

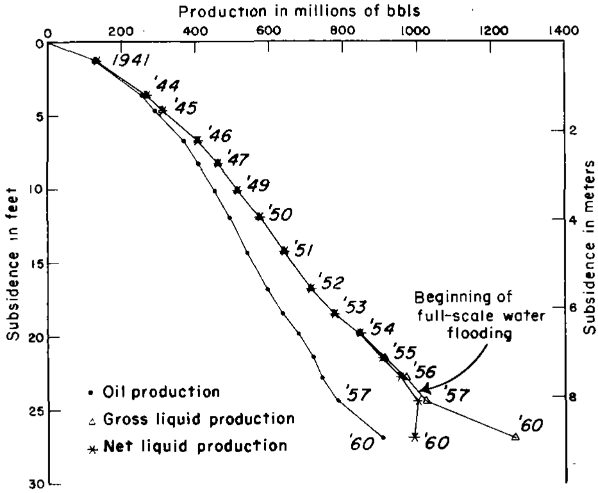

FIGURE 5. *Cumulative oil, gross-liquid, and net-liquid production from the Wilmington oil field plotted against cumulative subsidence near the center of the Wilmington subsidence bowl. Prepared from production statistics and subsidence data compiled by the California Division of Oil and Gas, and subsidence data presented by Gilluly and Grant (1949, pp. 471-473, 527), and Hudson (1957, table V).*

HUNTINGTON BEACH OIL FIELD

The Huntington Beach field, which is also located along the south coastal section of the Los Angeles basis, is the only other field for which available data permit a comparison between liquid production and subsidence. We have, however, been unable to develop any reliable estimates of successive changes in the volume of subsidence through all or even part of the production period of the Huntington Beach field. Although subsidence between 1920 (when production began in the Huntington Beach field) and 1933 has not been reliably determined at any bench mark within the zone of differential subsidence, we can compare the post-1933 subsidence at selected bench marks with various aspects of liquid production (fig. 6).

GENERAL OBSERVATIONS

Several conclusions emerge from direct comparisons between the various measures of liquid production and subsidence in the six fields examined above:

(1) All show at least a crudely developed linear relationship between cumulative net-liquid production and one or more measures of subsidence. Plots of subsidence against only oil or gross-liquid production are generally far less linear in character. Data from the Inglewood field suggest that subsidence at a point provides at least a rough index of the changing volume of subsidence.

(2) Departures from linearity seem to have characterized the early production stages in at least five of the six fields. Subsidence rates in the Bolivar Coast and Wilmington fields were, in proportion to their production rates, relatively low during the early years of development; whereas subsidence rates over the Inglewood field (and probably the Huntington Beach field) are believed to have been relatively high during the early production years.

(3) V_s/V_p, whether derived from the full period of observation or the slopes of the linear portions of the subsidence-production curves, ranged over nearly an order of magnitude – that is, from 0.08 or 0.10 in the Inglewood field to nearly 1 over the clearly linear parts of the curves for two of the Bolivar Coast fields.

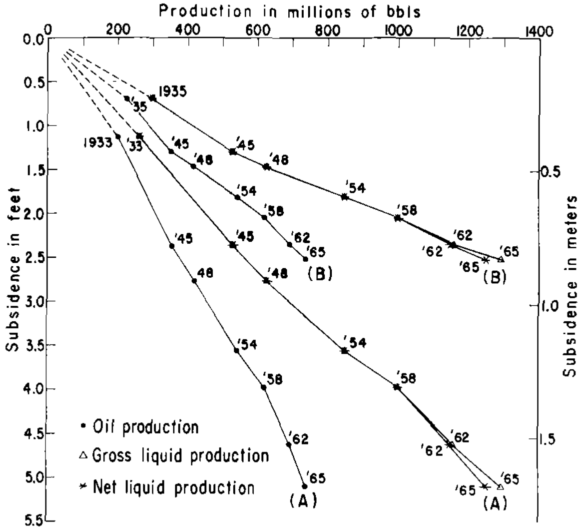

FIGURE 6. *Cumulative oil, gross-liquid, and net-liquid production from the Huntington Beach oil field plotted against subsidence at bench marks located (A) near the center of subsidence, and (B) midway up the southeast limb of the subsidence bowl. Prepared from production statistics of the California Division of Oil and Gas and elevation data of the US Coast and Geodetic Survey and the Orange County Office of County Surveyor and Road Commissioner. Easily related elevation measurements have been available only since 1933; estimates of subsidence since 1920 shown by dashed lines.*

EXPLANATION

The general theory advanced in explanation of reservoir compaction and resultant oil-field subsidence (Gilluly and Grant, 1949) is, in its broad outlines, beyond challenge. Thus Terzaghi's principle, which relates increased effective stress directly to fluid-pressure decline, probably is validly applied to the multifluid-phase system. Yet in seeming opposition to this generalization, measured reservoir pressure decline within the Vickers zone was disproportionately high with respect to surface subsidence during the early production years (fig. 2a and d); a similar situation is believed to have prevailed in the Wilmington field (City of Long Beach, 1967, unpublished data). Whatever the relationship, then, between measured reservoir pressure decline and compaction, the two are certainly not directly proportional.

The most likely explanation for the poor correlation between reservoir-pressure decline and subsidence (or compaction) is that pressure decline as measured at individual producing wells is generally non-representative of the average or systemic decline over the field as a whole. Thus in examining this problem in the Wilmington field, Miller and Somerton (1955, p. 70) observed that "reductions in average pressure in the reservoir are virtually impossible to determine with a satisfactory degree of accuracy." This deduction, coupled with the observed linearity between net-liquid production and subsidence, suggests that the liquid production may constitute a better index of average reservoir-pressure decline than that obtained through down-hole measurements.

The approximately linear relationship between net-liquid production and subsidence remains imperfectly understood; a general explanation may be offered, however, through simple analogy with a tightly confined artesian system of infinite areal extent. The artesian coefficient of storage may be defined as the volume of water released from storage within a column of aquifer underlying a unit surface area during a decline in head of unity. In an artesian system that is hydraulically isolated from any free-water surface, the volume of water represented by the storage coefficient will be derived entirely from the expansion of the confined water and compaction of the reservoir skeleton. Thus the total volume of reservoir compaction must be linearly related to cumulative production, provided only that the bulk modulus of elasticity of the water and the modulus of compression of the reservoir skeleton remain invariant over the relevant stress interval (see Jacob, 1940, p. 575-577). In the case of a well field in which the liquid-extraction flux is very high (that is, one characterized by closely spaced wells and high production rates) and hydraulic diffusivity[1] is very low, fluid-pressure decline will be expressed chiefly as mutually interfering cones of depression surrounding individual wells and will be largely confined to the main body of the well field. Thus production will be obtained chiefly from liquid expansion and reservoir compaction within the areal limits of the well field itself, rather than by extraction (and consequent but almost unmeasureable subsidence) from an extensive peripheral region. Under these circumstances the average pressure decline at any location within the field (and the consequent increase in effective stress and resultant compaction) will tend with time to become approximately linearly related to cumultative production. More significantly, subsidence will be restricted to a well-defined area within which it can be measured with some degree of accuracy.

The system described above becomes directly comparable with an oil field if two restrictions are imposed on the producing oil field: (1) the proportion of gas in the net fluid produced must remain constant so that the effect of adding the produced gas to the cumulative liquid production curve will uniformly change its slope but not its form (a restriction dictated by the presumption that the expansive effect of the gas is a function of its concentration in the fluid system); (2) the compressibilities of both brine and oil in the reservoir state must be sufficiently close as to be considered identical. The second required restriction is considered the most vulnerable feature of this model.

Liquid production from a petroleum system in which the reservoir solids are only slightly compressible and the reservoir fluids are relatively highly compressible, will consist of: (1) a relatively large volume attributable to fluid expansion; and (2) a relatively small volume attributable to compaction. Because the reservoir pressure decline, and thus the increased effective stress and the compaction, are directly proportional to liquid production, V_s/V_p for this system should be constant and charaterized by values close to but greater than 0. This model may be typified by the Inglewood field, which has been identified as an essentially solution-gas-drive field (California Department of Water Resources, 1964, p. 16). Liquid production from a system in which the reservoir skeleton is highly compressible and the reservoir fluids are only slightly compressible, constitutes the opposite extreme, and will consist of: (1) a relatively large volume attributable to reservoir compaction; and (2) a relatively small volume attributable to fluid expansion. Because reservoir compaction due to increased effective stress is again directly proportional to liquid production, V_s/V_p for this system will again be constant, but characterized by values less than but close to 1. This system may be typified by some of the Bolivar Coast fields, which are thought to be driven by formation compaction (van der Knaap and van der Vlis,

1. Hydraulic diffusivity, a term analogous to thermal diffusivity, is defined as the transmissivity of an aquifer (hydraulic conductivity times thickness) divided by its storage coefficient. This ratio determines the rate at which a head change propagates through the aquifer.

1967, pp. 94-95). The Wilmington field may be an example of one intermediate between the two cited extremes.

Departures from linearity may be related to changes in fluid or skeletal compressibilities. It is likely, however, that the observed departures in the early parts of the curves (figs. 3, 4, and 5) are due chiefly to changes in the produced gas-net liquid ratio. Thus relatively low gas production from the Wilmington (as deduced from production statistics of the California Division of Oil and Gas) and Bolivar Coast fields (as inferred from the changing gas-oil ratio – see Davila and others, 1951, p. 211) during their initial development, was associated with relatively low subsidence rates. The Inglewood field, on the other hand, was apparently characterized by both high gas production and relatively rapid subsidence during its early development (see fig. 2).

CONCLUSION

If linearity between net-liquid production and subsidence is indeed a general characteristic of subsiding oil fields, its recognition may supplement existing predictive techniques. Because the older techniques have been based largely or exclusively on the Wilmington experience, the relationship described here may have broader applicability in estimating future subsidence over recognized examples of subsiding fields.

Acknowledgment

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DISCUSSION

Intervention of Dr. Naomi Miyabe (Japan):

Question:

How did you compute the subsidence during its early stages?

Answer of Dr. Castle:

As I indicated earlier, there was only one bench mark within the Inglewood field that was leveled prior to the initiation of production in 1924 that has been releveled from time to time since; it was first releveled in 1943. However, several nearby bench marks were leveled between 1926 and 1943. By comparing the ratios of vertical movement at these benches with the movement at the older bench since 1943 and then extrapolating backwards, we can estimate the approximate subsidence at the critical bench during various pre-1943 periods.

Question of Dr. Miyabe:

Do you think that the subsidence was directly related to gas production?

Answer of Dr. Castle:

We feel that the relatively high gas production from the Inglewood field during its early production years contributed to the relatively high subsidence during this same period.

Intervention of Dr. Manuel N. Mayuga (USA):

Question:

Yesterday I was asked about the legal aspects of subsidence. I wonder if you would be free to comment on the legal aspects of the Inglewood oil-field subsidence with respect to structural damage generated in this area?

Answer of Dr. Castle:

As Dr. Mayuga has indicated, serious structural damage has been associated with the Inglewood oil-field subsidence. A reservoir located on the edge of the subsidence dish failed in 1943. The California Department of Water Resources concluded that the failure was attributable to faulting through the floor of the reservoir. We are not engineers and cannot make judgments on the failure of engineered structures. We have, however, attributed the indicated faulting to oil-field operations. Suits have been brought against the operators of the oil field and it is my understanding that they will be tried this fall.

Comment of Dr. Mayuga:

What I was trying to point out is that subsidence has led to a number of suits now pending in U.S. courts. Accordingly, there exists a problem of legal responsibility for any damage on the surface associated with subsidence.

Answer of Dr. Castle:

I agree. I might make one observation that could be considered an indirect response to your comment: this relates to litigation over the subsidence centering on the Goose Creek oil field in Texas.

The state of Texas retains title to tidelands oil. When the Goose Creek field sank beneath the waters of Galveston Bay the state claimed that title automatically transferred to them. The operators, according to Pratt and Johnson, contended that they were responsible for the subsidence and that it could not be considered "an act of God." A decision was rendered in favor of the defendant; that is, the operators retained title.

Intervention of Mr. Dennis R. Allen (USA):

Comment:

I have comment to make regarding the described relationship between production and subsidence. It has been our experience in the Wilmington field that the major cause of the subsidence is the reduction in petroleum reservoir pressures. When fluid pressure is reduced, loading is transferred to the grains. We feel that the linear relation between production and subsidence may thus be only coincidental and that deformation of the reservoir skeleton may be time dependent. Rapid declines in reservoir pressure could lead to limited compaction over the rapid decline intervals and thence be followed by significant compaction, largely in response to the earlier loss of fluid pressure. Under these circumstances, rapid subsidence could be fortuitously associated with high liquid production immediatly following a large loss in reservoir pressure.

Answer of Dr CASTLE:

I don't feel that our observation is inconsistent with your experience with respect to the relation between pressure decline and subsidence, What we have tried to explain here is the relation between fluid production on the one hand and average pressure decline and compaction on the other hand. Creep or lag effects may be important and we understand that they have actually been documented in the Wilmington field. However, the fact that we see this linear relation between liquid production and subsidence in every field for which we have data, argues that it must be more than simple coincidence.
Furthermore, the direct and nearly linear relations between rates of liquid production and rates of subsidence implies that production does indeed constitute a reasonable index of average reservoir pressure decline.

Intervention of Prof. Kenneth E. LEE (USA):

Question:

You mentioned that the model used to explain the linear relation between subsidence and discharge requires that the compressibilities of both fluid and solid must remain constant over the range of pressures involved. In an oil field the fluid pressures would be largely made up of gas pressure, and the compressibility would increase considerably as the pressure decreases. On the other hand, a decrease in fluid pressure leads to an increase in effective stress in the solid phase, and in a typical soil this usually leads to a decrease in compressibility.

If wonder if the assumption of constant compressibility could be an oversimplification of an overall gross effect made up of the increase in compressibility of the fluid phase and decrease in compressibility of the solid phase as the fluid is withdrawn and the fluid pressure decreases?

Answer of Dr. CASTLE:

Let me take your second point first. We doubt that the two suggested effects (namely, the increasing compressibility of the fluid mixture and the decreasing compressibility of the reservoir skeleton) could so neatly compensate as to produce a linear relation between liquid production and subsidence. Since the relation occurs in every field we have examined, its attribution to this compensating mechanism would demand a large element of coincidence. Furthermore, we would guess, depending in part on how both gas and water are produced, that the compressibility of the fluid mixture generally increases with increasing production. We admit, however, that we are working with a very limited number of examples.

We fully agree with your suggestion that our assumption of constant compressibility of the fluid mixture may be an oversimplification; we view it as a serious defect of the proposed model. That the linear relation is less perfect than it might be, could indeed derive from the changing compressibility of the liquid mixture. On the other hand, the available data indicate that the assumption of an invariant compressibility of the reservoir skeleton is a valid approximationat least over the elevated stress levels and limited stress ranges encountered in the depletion of a petroleum reservoir.

LAND SUBSIDENCE RELATED TO DECLINE OF ARTESIAN HEAD AT BATON ROUGE, LOWER MISSISSIPPI VALLEY, U.S.A.

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ABSTRACT

Comparison of precise levelling in the Baton Rouge area of the Lower Mississippi Valley indicates as much as 30 cm of subsidence of the land surface during the period 1900-1965. Maximum subsidence is centered in the Baton Rouge industrial district, the area of greatest withdrawals from wells and maximum decline of artesian head. Lines of equal subsidence for the period 1934 to 1965 show a bowl-shaped depression slightly elongated east-west; the 5-cm subsidence line encloses an area of about 250 sq. mi. (650 sq. km).

The areal distribution of subsidence corresponds closely with distribution of decline in head in the confined aquifer system. Increases in rate of head decline in heavily pumped zones are reflected in accelerating subsidence. Presumably head decline in the multiple aquifer system of the Baton Rouge area has caused compaction of fine-grained sediments interbedded with and separating the beds of water-bearing sand. Average head decline in the aquifer system in the area of maximum subsidence approximates 200 feet (61 m) since pumping of confined water began about 1890. The ratio of subsidence to head decline is about 0.5 foot (15 cm) of subsidence for each 100 feet (30 m) of head decline. Subsidence can be expected to continue with continuing head decline.

RÉSUMÉ

Une étude comparative utilisant un nivellement de précision dans la région de Baton Rouge, dans la basse vallée du Mississippi, a mis en évidence un affaissement du sol atteignant en surface jusqu'à 30 cm pour la période 1900-1965. L'affaissement maximal a son centre dans la zone industrielle de Baton Rouge, là où le débit prélevé par des forages profonds est le plus élevé et où la réduction de la hauteur de remontée de la nappe artésienne est la plus forte. Les lignes d'égal affaissement pour la période 1934 à 1965 présentent une dépression en forme de bol légèrement allongée, suivant la direction Est-Ouest ; la ligne correspondant à un affaissement de 5 cm délimite une superficie d'environ 250 miles carrés (650 km^2).

La répartition spatiale de cet affaissement correspond de façon très précise à la répartition de la diminution de la charge de la nappe artésienne.

L'augmentation de cette réduction de la charge dans les zones où le pompage est le plus intense se répercute dans une accélération de l'affaissement. On suppose que la diminution de la charge dans le système aquifère complexe de la région de Baton Rouge a provoqué la compaction des couches de sédiments à fine granulométrie intercalées entre les lits de sables aquifères et séparant ces lits. La perte de charge moyenne dans le système aquifère dans la zone où l'affaissement est maximal est voisine de 200 pieds (61 m), alors que le pompage de l'eau de la nappe artésienne a commencé vers 1890. Le rapport entre la hauteur d'affaissement et la perte de charge est d'environ 0,5 pied (15 cm) de hauteur d'affaissement pour 100 pieds (30 m) de perte de charge. On peut s'attendre à ce que l'affaissement du sol se poursuive avec la continuation de la diminution de la charge de la nappe artésienne.

INTRODUCTION

For many years subsidence of the land surface in response to decline of artesian head has been occurring in several places in the United States and in other parts of the world. Such subsidence was recognized in the Gulf Coast region as early as 1954 in the Houston, Texas area (Winslow and Doyel, 1954), some 250 mi. (400 km) west of Baton Rouge.

Heavy ground water withdrawal accompanied by sharp decline in artesian head suggested the possibility of similar land-surface change in the vicinity of Baton Rouge. Extensive releveling in the Baton Rouge area in 1964-65 by the US Coast and Geodetic Survey provided an opportunity for comparison with previous levelings of 1880, 1900, 1929, 1934-35, and 1938.

Subsidence exceeding 5 cm in the period 1934-65 has been found in an area of about 250 sq. mi (650 sq. km) centering on the Baton Rouge industrial district (fig. 1), where ground-water withdrawal and head declines have been greatest. The fault zone which

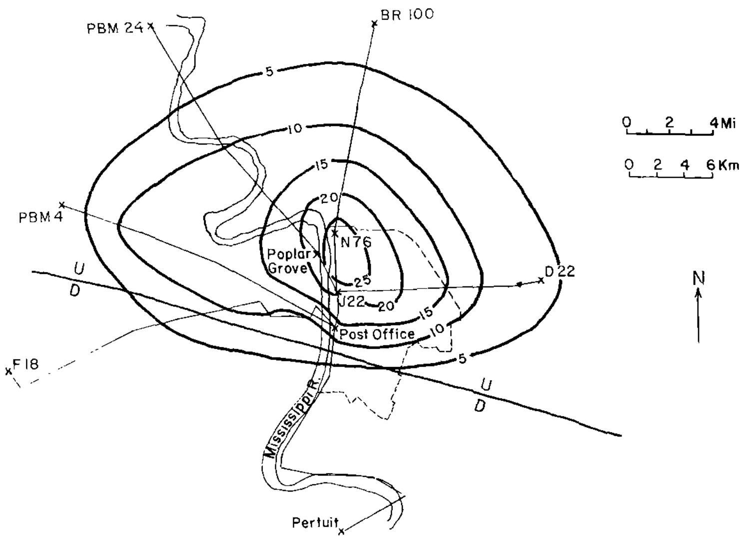

FIGURE 1. *Lines of equal subsidence for approximate period 1934-1965, interval 5 cm*

crosses the area from east to west and acts as a barrier to ground-water movement (Meyer and Rollo, 1965) also has a marked effect on the distribution of land subsidence. Substantial subsidence is restricted to the area north of the zone; to the south, where artesian head decline is small, subsidence is minor.

The greatest observed subsidence is at BM (Bench Mark) Poplar Grove, directly across the Mississippi River from the Baton Rouge industrial district, where a 30 cm change has been recorded over the period 1900-64 (fig. 2).

However, bench marks in the industrial district proper have subsied at a slightly greater rate over the period 1935-65.

As of 1968 no serious damage attributable to subsidence had been reported at Baton Rouge, but subsidence greatly complicates establishment of correct vertical control for engineering work. Moreover, if subsidence continues unabated, which seems likely in view of increasing ground-water withdrawal, it may result in costly damage to structures that depend upon maintaining a uniform grade, such as sewers, levees, and channel and drainage works.

GEOLOGIC SETTING

Baton Rouge is located in the lower Mississippi River Valley about 160 mi.(260 km) north west of the mouth of the Mississippi River, north of but very near the hinge line which separates areas of active downwarping from areas of uplift in the Gulf Coast Geosyncline. The city is on a Pleistocene terrace against which the Mississippi River now impinges.

West of the Mississippi River the recent valley extends for many tens of miles and is underlain by Quaternary sand and gravel aquifers only sparsely represented on the east bank of the river at Baton Rouge.

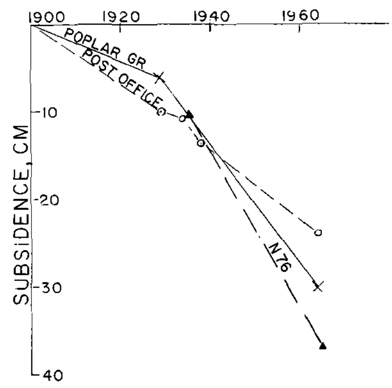

FIGURE 2. *Subsidence versus time at three bench marks near Baton Rouge. Locations shown on figure 1*

The fresh-water-bearing beds at Baton Rouge range in age from upper Miocene to Pleistocene and dip gently southward. One structural feature of major importance with respect to occurrence of ground water is an east-west trending fault zone which truncates the fresh-water-bearing sands of the area near the southern border of the city (fig. 1,3). This fault zone, having a total displacement in excess of 250 feet (75 m) within 2,000 feet (610 m) of the land surface, effectively isolates hydraulically the areas north and south of it Water levels in wells approximatelly 1,500 feet (460 m) apart on opposite sides of the fault and tapping the same aquifer differ by about 200 feet (fig. 4). Artesian flow still occurs south of the fault zone. The east-west elongation of the area of subsidence is no doubt related to the fact that only a small pressure decline has occurred south of this fault zone.

THE AQUIFER SYSTEM

As a matter of convenience the major fresh-water-bearing sands of the Baton Rouge area were named for the depth at which they occur beneath the land surface in the industrial district (Meyer and Turcan, 1955). These aquifers include the "400-foot," "600-foot" "800-foot," "1,000-foot," "1,100-foot," "1,500-foot," "1,700-foot," "2,000-foot," and "2,800-foot" sands (fig. 3). The "800-foot," "1,000-foot." and "1,700-foot" sands are little used, and in 1966 less than 5 percent of the total pumpage was from these three aquifers. For the purpose of discussion the "400-foot" and "600-foot" sands are considered as a unit because many of the wells in these aquifers are completed in both sands. Thus, an independent evaluation of pumpage and water levels for these two sands is not possible. The "2,000-foot" sand is now the most heavily pumped aquifer in the area. Water levels in this sand are now more than 180 feet (55 m) below mean sea level (fig. 4) and 345 feet (105 m) lower than pre-pumping levels. Water levels south of the east-west fault zone are still as much as 74 feet (23 m) above mean sea level, or about 260 feet (79 m) higher than in the center of the industrial district, about 6 mi. (10 km) to the north.

The principal difficulty of this multiple aquifer system, at least in relation to subsidence is to evaluate the subsidence of the land surface with respect to some composite decline in water level. This problem is discussed in another section.

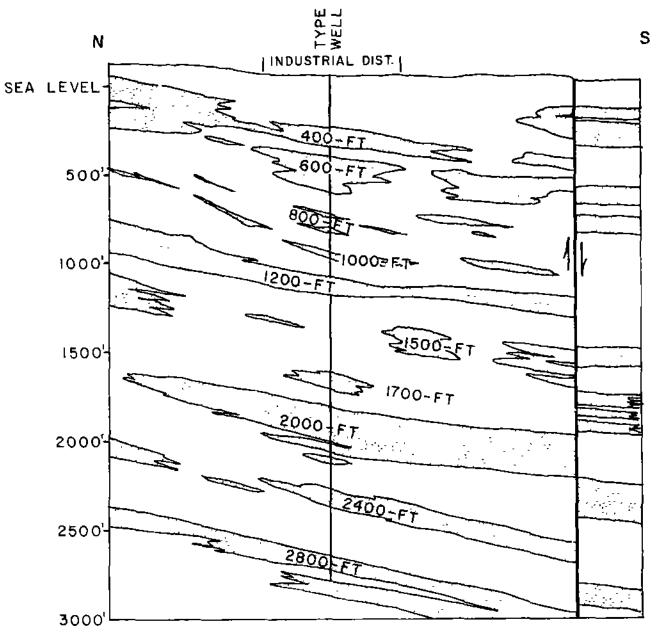

FIGURE 3. *Schematic geologic section extending north-south through Baton Rouge. Sand zones are named for their general depth in the industrial area*

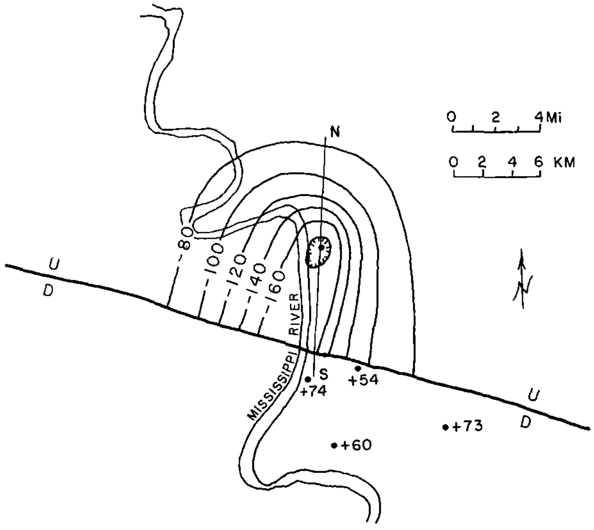

FIGURE 4. *Lines of equal elevation of head (in feet) in the "2,000-foot" sand, autumn 1966. Figures south of east-west fault are water-level elevations at individual wells; however, the number of control points does not permit contouring*

HISTORICAL BACKGROUND

The earliest information on water wells in the Baton Rouge area (Harris and Fuller, 1904) mentions an "old well" drilled in 1892 for the waterworks in the downtown district and a new well which had a yield of 500,000 gpd (1 900 m^3 per day) and flowed at the surface. It is safe to assume that little use had been made of the ground-water resources prior to 1890. The first significant increase in use of the ground-water system was during the period 1909-15 when the Standard Oil (Humble Oil and Refining Co). plant began operations in the industrial district. During that period this company alone drilled at least 16 wells into the "400- and 600-foot" sands and two wells into the "1,200-foot" sand. Figure 5 shows

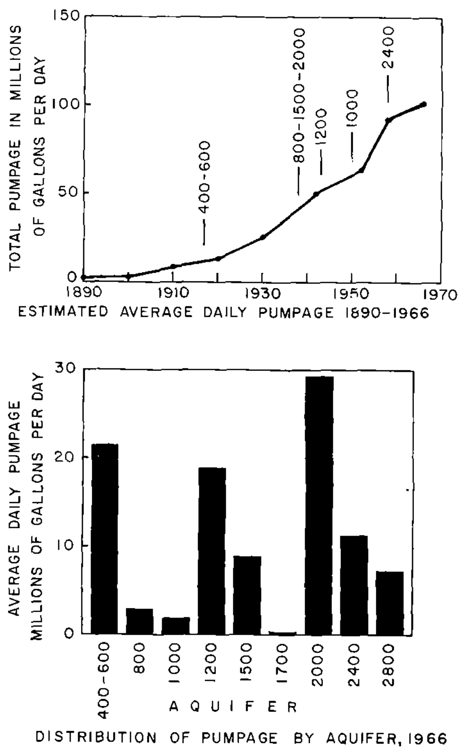

FIGURE 5. *Estimated daily pumpage in the Baton Rouge area.* A. *Estimated total daily ground-water pumpage, 1890-1966. Vertical arrows indicate approximate beginning of rapid head decline in aquifers indicated.* B. *Average daily pumpage by aquifers as of 1966*

the estimated pumpage from each of the major aquifers in the Baton Rouge area for 1966 and a graph of the total pumpage vs. time since 1890. Estimated decline in head from pre-pumping conditions until 1966 is shown below.

Aquifer	Decline	Aquifer	Decline
400–600	170 feet (52 m)	1,700	Not known
800	130 feet (40 m)	2,000	345 feet (105 m)
1,000	105 feet (32 m)	2,400	200 feet (61 m)
1,200	220 feet (67 m)	2,800	60 feet (18 m)
1,500	155 feet (47 m)		

The average head decline in the industrial district for all zones approximates 200 feet (61 m), suggesting a ratio of subsidence to head decline of about 0.5 foot (15 cm) subsidence for each 100 feet (30 m) of head decline.

Altough public supply water and some industrial water was pumped (or flowed) from the deeper aquifers the majority of industrial users pumped water only from the "400- and 600-foot" sands prior to about 1940, principally because of the temperature advantage in using cool water from the "shallow" aquifers. By 1940 it was apparent that the "400- and 600-foot" sands alone could no longer continue to supply the increasing industrial demands (Cushing and Jones, 1945).

As a result of the work of Cushing and Jones (1945) and Meyer and Turcan (1955) the water users began to look to the deeper aquifers as sources of water. Essentially all the water-level decline which has taken place in the "400- and 600-foot" sands took place prior to 1940 and much of the decline predated 1929, thus the subsidence in the period

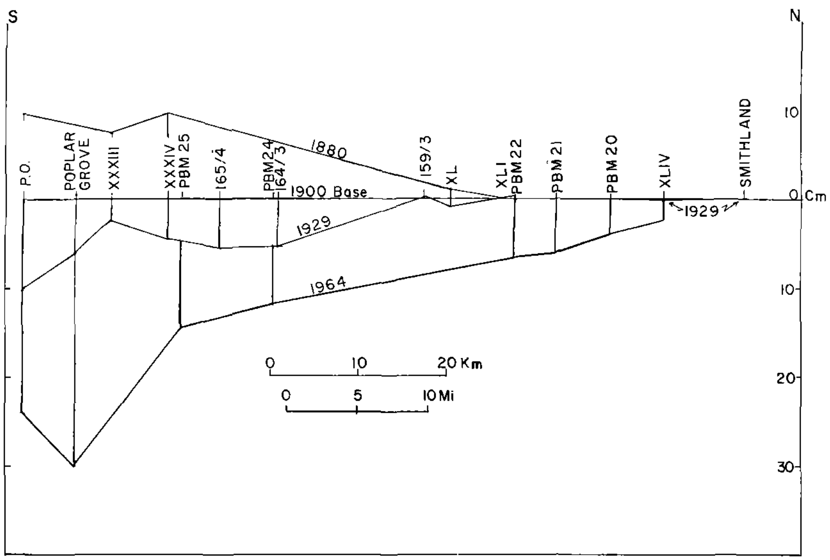

FIGURE 6. *Land subsidence from Baton Rouge north along Mississippi River, composite profiles for 1880-1900-1929-1964*

1880-1929 (fig. 6) must be essentially attributable to the withdrawal of water from those water-bearing zones.

The subsidence between 1929 and 1964-65 is thus related primarily to the increasing withdrawals from the deeper aquifers of the Baton Rouge area. As water use and consequent water-level declines have been accelerating rapidly since the late 1950's, careful attention must be given to potential increase in the rate of subsidence.

SUBSIDENCE

The earliest precise leveling in the Baton Rouge area dates back to 1880 when the US Coast and Geodetic Survey established a level line from Carrollton, Louisiana (New

Orleans) along the east bank of the Mississippi River to Baton Rouge, thence across the river to the west bank and northward upstream along the Mississippi River to Red River Landing (US Coast and Geodetic Survey, 1889). In 1900 the portion on the west bank of the Mississippi upstraem from Baton Rouge was releveled by the Corps of Engineers; however, this line was not tied across the Mississippi to bench marks in Baton Rouge. The portion of the original line from Baton Rouge south to Carrollton was releveled by the Corps of Engineers in 1929.

In addition, the Corps of Engineers in 1929 established a new line of leveling extending northwest from Baton Rouge to the Atchafalaya River, a distributary of the Mississippi, thence upstream along the Atchafalaya to its confluence with the Mississippi and then back downstream along the Mississippi to Baton Rouge via the west bank over lines previously leveled in 1880 and 1900.

Three new lines of leveling were established by the Coast and Geodetic Survey in 1934 and 1935: (1) Extending about 50 mi. (80 km) east from Baton Rouge to Hammond, La., (2) North along the Illinois Central Railroad from Baton Rouge to Vicksburg, Miss., and (3) From Port Allen, on the west bank of the Mississippi across from Baton Rouge, southwestward toward Lafayette, La. In 1938 the Coast and Geodetic Survey releveled the line along the east bank of the Mississippi between Baton Rouge and New Orleans. In 1964-65 the Coast and Geodetic Survey releveled most of the preexisting lines and extended several lines beyond their previous termini.

A preliminary comparison of leveling of 1964-65 with previous levelings (Davis, 1968) indicated that Baton Rouge had declined in altitude relative to bench marks several miles from the city. This implies that land subsidence was taking place in the Baton Rouge area; accordingly, the authors undertook a study based on the 1964-65 releveling and related the land subsidence to the ground-water hydrology of the area.

Because the amount of subsidence indicated was relatively small, unadjusted field data furnished by the US Coast and Geodetic Survey were used to insure that no actual subsidence had been adjusted out of the leveling data. Differences in elevation between successive bench marks leveled in more than one period were compared and the observed variations were recorded. These variations were accumulated for each line of levels. assuming some bench mark distant from Baton Rouge as an origin point of no change. From these data the subsidence profiles shown on figures 6, 7, and 8 were plotted as follows: (1) Bench marks which had been leveled in more than one period were identified and plotted on topographic quandrangles, (2) Profiles were drawn using the straight-line distance between bench marks, (3) By using the earliest leveling as a base line and assuming certain bench marks far from Baton Rouge as stable end points, the computed differences were plotted at the position of each mark, and (4) After obviously unreliable measurements (for example, disturbed bench marks) had been eliminated the profiles were completed by joining the plotted points.

All leveling used in this study was of first-order precision. In such leveling all lines are divided into sections 1 to 2 kilometers in length, and each section is run forward and backward, the two runnings of a section to differ not more than 4 mm $\sqrt{K}$, where K is the length of the section in kilometers (US Coast and Geodetic Survey, 1948, p. 20). In actual practice the accuracy is much better than this minimum acceptable standard. The probable error in modern precise leveling work is generally less than 1 mm per kilometer.

Although precise leveling was done to serve as basic control and not with subsidence in the mind, level lines in the Baton Rouge area lend themselves to an analysis of subsidence because three key bench marks in downtown Baton Rouge (Post Office, XXXI, and PBM 2 have been leveled repeatedly in the various surveys. Their value is further enhanced due to the fact that although 2 of these 3 key bench marks (Post Office and XXXI) are on buildings, they have maintained a consistent relation with time. That is, comparison of PBM 2 at ground level on the grounds of the old State Capitol with BM XXXI on the Capitol Building and BM Post Office on the old Post Office Building indicates that

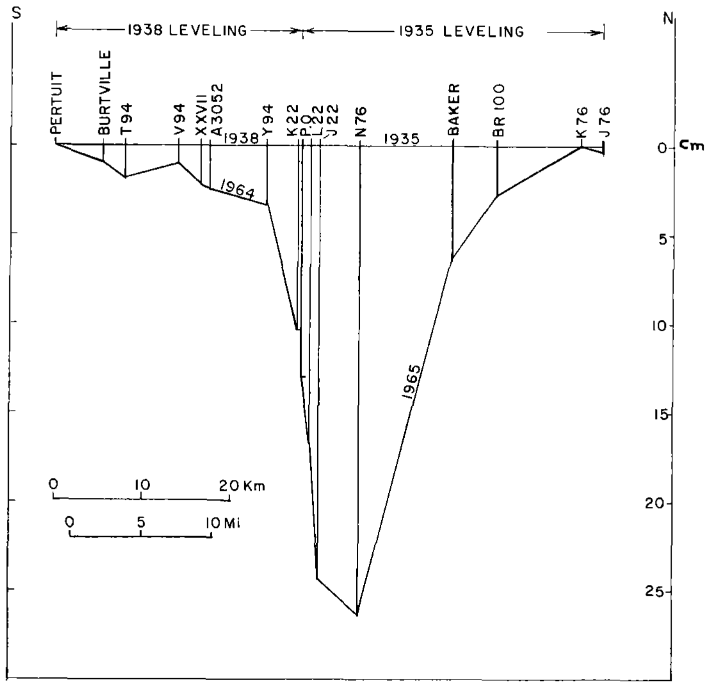

FIGURE 7. *Land subsidence along south-north line through Baton Rouge, 1935-38 to 1964-65*

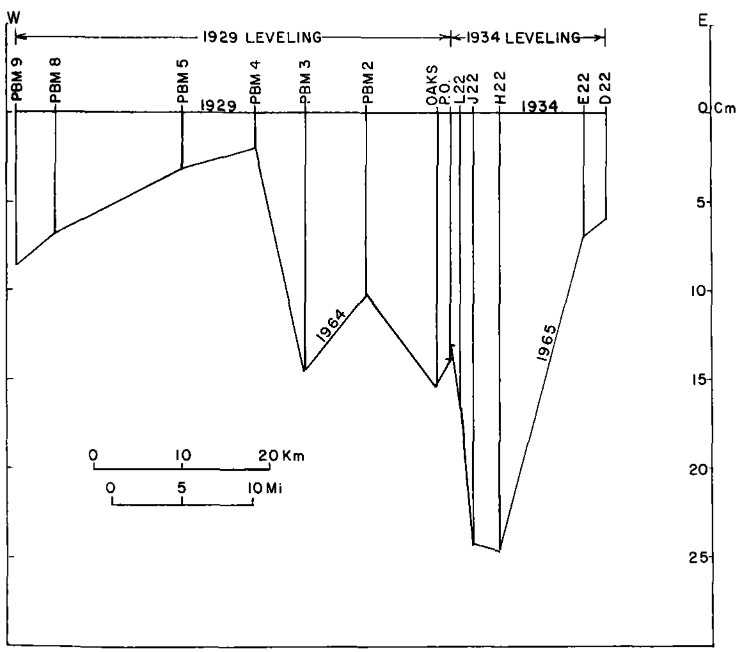

FIGURE 8. *Land subsidence along west-east line through Baton Rouge, 1929-34 to 1964-65*

neither building has been subject to differential subsidence due to the loading of the structure on its foundation.

As indicated on figure 1, subsidence exceeds 5 cm over the period 1929-65 in an area of about 250 sq. ml. (650 sq. km) centering on the Baton Rouge industrial district, where more than 25 cm of subsidence has been recorded in an area of about 2 sq. mi. (5.2 sq.km.) bordering on the Mississippi River.

The maximum change observed at a single bench mark was about 30 cm at BM Poplar Grove for the period 1900-64. This bench mark is directly across the Mississippi River from the area of maximum subsidence during 1929-64, so the record at Popular Grove probably approximates the true maximum for the period. A plot of time versus subsidence for several key bench marks (fig. 2) suggests that the area of maximum subsidence has changed significantly with time. During the period 1900-29, BM Post Office in downtown Baton Rouge subsided significantly faster than did BM Poplar Grove. Only in the post-1929 period has subsidence in the Baton Rouge industrial district exceeded that of the downtown area. This effect is consistent with what is known of the distribution of pumpage and head decline. In the early period the downtown area was affected by pumpage from an old municipal well field, which pumped from the "400- and 600-foot" sands and still has active wells in the "2,000-foot" sand. Very large increases of pumpage and resulting head decline have taken place in the industrial district near BM Poplar Grove. An indicated on figure 2, subsidence at BM N 76 near the north end of the industrial district has been somewhat greater than at Poplar Grove since 1935.

Figures 6, 7, and 8 show profiles of subsidence along principal lines of leveling in the Baton Rouge area. Figure 7 extends from what is apparently stable ground north of Baton Rouge, through the subsidence maximum of the industrial district in the downtown area, to presumably stable ground at BM Pertuit some 20 km south of Baton Rouge. Aside from portraying the maximum subsidence, it also shows the effect of the east-west fault zone which passes through the southern part of Baton Rouge and forms a partial barrier to ground-water movement; the fault zone crosses the profiles between BM K22 and Y94 (fig. 7). The difference in water level and head change associated with the faulting is reflected in the subsidence profile by a sharp decrease in amount of subsidence south of the fault.

The west-east profile (fig. 8) through Baton Rouge is somwhat lrss definitive than the profile on figure 7. This west-east profile does not extend to stable ground at either end because the 1965 leveling was not carried that far.On the west, PBM's 5, 8, and 9 are along an abandoned railroad in the backswamp area between the Mississippi and Atchafalaya Rivers. Moreover, both PBM's 8 and 9 are in oil fields. The lack of stability may be due to surfical settling, or oil production, or both, or possibly other causes; however, in this area subsidence appears to be unrelated to decline in artesian head.

The profile on figure 6 shows subsidence along the original leveling line north of Baton Rouge on the west side of the Mississippi River, which was first leveled in 1880. On this profile the 1900 leveling is used as a base line because the comparison of 1880 leveling was somewhat suspect owing to losses of bench marks and inherent inaccuracies in the early leveling. The computed change shown from 1880-1900 may be real, but as the subsidence shown is based on only three bench marks in the Baton Rouge area little confidence is placed in the 1880-1900 comparison. Later leveling in 1929 and 1964 indicate a growing and deepening cone of subsidence. As of 1929 the area between bench marks 159/3 and Smithland (fig. 6) appeared quite stable, but significant subsidence was registred even as far away as BM XLIV by 1964.

In constructing all the profiles and the map of figure 1, the assumption was made that BM Pertuit, 10 mi. (16 km) south of Baton Rouge, has remained stable throughout the period of leveling control. This assumption is confirmed by the fact that bench marks outside of the area affected by pumpage north, northwest, southwest of Baton Rouge show no significant change relative to BM Pertuit. Furthermore, comparisons from 1938-

65 from Baton Rouge to New Orleans, 75 mi. (120 km) southeast, and 1935-65 from Baton Rouge to Vicksburg, Miss., 130 mi. (120 km) north, show Pertuit to be stable with respect to these distant points. This lends considerable support to the assumption of stability for BM Pertuit, but does not rule out the possibility of slow tectonic subsidence of the entire lower Mississippi Valley area.

CAUSES OF SUBSIDENCE

Subsidence of land surface in other areas has been ascribed to a variety of causes. Among those that are possible in the Baton Rouge area are: (1) Tectonic movement, (2) Decline in pressure due to petroleum production, (3) Loading at land surface, (4) Drying out and shrinkage of surficial deposits, and (5) Decline of pressure head in confined aquifers.

Temporal and spatial relations indicate that decline of artesian head is the major cause of the broad subsidence bowl shown on figure 1. Tectonic movement is not only possible, but likely throughout the Mississippi Embayment; however, it would be expected to occur at a rate much slower than that discussed in this paper. Moreover, such subsidence would likely be of regional extent and not detectable over the distance traversed by the level lines discussed. Decline in pressure due to petroleum production is another possible cause of subsidence in the Baton Rouge district as there are several small oil and gas fields within the area shown on figure 1. This possible effect has been noted in the discussion of subsidence near the west end of the profile of figure 8. However, the oil and gas fields of the Baton Rouge area of small lateral extent, on the order of 1 to 2 sq. mi, and would not be expected to cause of subsidence on the broad scale of figure 1. Loading at land surface by buildings and structures such as levees might be expected to cause subsidence, espacially at bench mark in their immediate vicinity. In this connection, it is noteworthy that bench marks on the old Post Office and old State Capitol Buildings in Baton Rouge have shown no differential sinking over 65 years. Several bench marks located near high levees along the west side of the Mississippi in recent alluvium appear to reflect local loading by the levees, and for that reason were not given credit in the subsidence analysis.

Finally, drying out and shrinkage of surficial deposits, which have caused extensive subsidence in some other areas, is likely in part of the Baton Rouge area. The area west of the natural levees of the Mississippi and extending westward to the Atchafalaya was historically a poorly drained swamp. Drainage and flood-control works constructed in recent decades have resulted in some drying of surficial deposits. Any such effect would be limited to the former swamp, and this may in part explain subsidence on the west end of the profile of figure 8; however, it cannot explain the broad subsidence observed throughout the Baton Rouge area. Compaction due to the decline in artesian head seems an adequate explanation for the broad subsidence cone mapped in the Baton Rouge area. However, the question of where in the geologic section and in which type of deposits the compaction occurs cannot be answered on the basis of available data except on the basis of the temporal distribution of pumping in different zones. In aquifer systems composed of granular sediments, compaction commonly occurs chiefly in fine-grained intercalated materials, such as clay and silty clay having high porosity (Lofgren, 1968). In many areas high rates of compaction have been associated with the presence of montmorillonite (Jones, 1968), presumably derived from the Missouri Basin and other western tributaries (Griffin, 1962; Jacobs and Ewing, 1969).

A detailed picture would require petrographic analysis and physical testing of core materials from different zones and operation of compaction recorders in the principal zones subject to head decline.

Since the rate of subsidence is deduced from widely separated levelings it is problematical whether or not the subsidence observed over the 1929 to 1964-65 period was concentrated near the latter dates. Such concentration is a completely rational possibility which, if true, would suggest that the rate of subsidence can be expected to increase substantially in future years. This possibility exists because it is probable that the last low stand of Pleistocene sea level (400 feet [120 m] below the present level) altered the hydraulics of the aquifer system sufficiently to pre-load the deeper aquifer system. If this is the case little compaction would be expected until this pre-load stress is exceeded.

Thus, most of the subsidence prior to 1929, suggested by the profile, figure 6, probably can be attributed to the decline of water level in the "400- and 600-foot" sands and most of the subsidence since 1929 probably can be attributed to pumpage from the deeper aquifers. Only the installation of subsidence recorders in several sand zones, as previously suggested, would resolve this question.

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TRITIUM DATING OF LAND SUBSIDENCE IN NIIGATA

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ABSTRACT

In order to know the influence of the pumping of groundwater on land subsidence in the Niigata area, an accurate evaluation of the groundwater flow in the area is required. The tritium dating method was applied to the evaluation. The groundwater flow into the area, the flow of such stratum water that has not participated in the natural groundwater flow, and the characteristics of the groundwater flow within the area were clarified. The results have given information for establishing models of the land subsidence from the viewpoint of both hydraulics and soil engineering. This method, when applied to a longer period, was proved to be an appropriate technique for not only a more detailed indication of groundwater flow, but also the quantitative estimation of the changes of the flow in the future, based on the quantitative groundwater flow and the results of hydraulic computation.

RÉSUMÉ

Dans le but d'obtenir l'effet du pompage sur le tassement du sol à Niigata, il est nécessaire de connaître une évaluation de l'écoulement de l'eau souterraine dans la région. La technique de détermination de l'âge par le tritium a été appliquée à l'évaluation de l'écoulment. On a pu déterminer le flot de l'eau souterraine du périmètre vers l'intérieur de la région, ainsi que le mouvement de l'eau dans les couches qui n'ont pas participé au mouvement de l'eau souterraine naturelle, et finalement les caractéristiques de l'écoulement de l'eau particulière de la région. Ces résultats ont donné des suggestions pour l'établissement d'un modèle hydraulique et géotechnique du tassement du sol. Cette méthode de mesure, continuée plus longuement, rendra possible non pas seulement des indications plus détaillées sur l'écoulement mais aussi l'évaluation quantitative des changements des quantités de l'écoulement à l'avenir basée sur l'étude quantitative actuelle et sur les résultats du calcul hydraulique.

I. INTRODUCTION

A major cause of land subsidence in Niigata is the large scale pumping of ground water for the extraction of natural gas. This paper describes a study to determine the relation between land subsidence and the rate of pumping that may be used to estimate subsidence in the future.

Of prime importance in such a study is the clarification of the relation between the quantity of water withdrawn at a center of pumping and the inflow of groundwater from surrounding areas. The conventional hydrogeological methods have failed in correctly describing such inflow of groundwater. Secondly, land subsidence may be started by the movement of water "native" to the aquifer—water that has been in the aquifer a relatively long period of time and that has not participated in the movement of groundwater in the zone normally influenced by recharge-discharge conditions. This phenomenon has not been measured by any conventional method. Thirdly, the development of more precise methods for measurement of complex large scale structural changes that may take place in the land subsidence area is required. If these basic problems were to be solved, land subsidence would be estimated more accurately now and in the future.

In the study in the Niigata area, the tritium dating method was used to confirm the inflow of groundwater to the center of pumping from the surrounding areas, to detect the flow of "native" water that had not previously participated in the local groundwater-flow picture, and to measure the structural changes in land subsidence areas where the subsidence is caused by large scale pumping of groundwater.

II. PRINCIPLE

Tritium is ideal as a groundwater tracer because it is relatively stable chemically, and it does not affect the flow properties of water, as compared with other tracers. It has a half life of about 12.5 years.

Since Libby predicted the existence of tritium in natural precipitation in 1946, the tritium dating method, in which the tritium is considered a tracer, has been used extensively in hydrology. The method is accurate enough to indicate the age of groundwater up to one hundred years. However, the theoretical tritium concentration in natural precipitation in each test area, which is the basis of dating, is difficult to determine, because of the mixing and re-precipitation of water evaporated or transpirated from inland and ocean, and also because, since 1952, the tritium concentrations under "natural conditions" have included that derived from fallout. For these reasons, even if the tritium concentration in the groundwater could be measured accurately, the dated age after precipitation would be only an approximate value. The dated age is a more apparent age also because the groundwater at a well is derived from the confluxed flow of water from various directions and represents a mixing of waters of different ages.

Even with such uncertainty, the flow of groundwater in the land subsidence area in Niigata can be analyzed by the tritium dating method for two reasons.

Firstly, the qualitative flow of groundwater can be evaluated by the variations in time-related distributions of the tritium concentrations in areas where the removal of groundwater directly influenced the land subsidence. In areas where land subsidence is caused by pumping, tritium concentrations in groundwater can be classified into three age groups, i.e.:

1. If the tritium concentration is more than 10 tritium units, the groundwater is constituted of precipitation after 1952. Groundwater of such an age hardly exists in the subsided area. Yet, if such a value is discovered in the area, the subsidence may have destroyed the natural structure of the aquifer.
2. If the tritium concentration of groundwater is less than 10 tritium units, the groundwater is constituted of precipitation before 1952. Groundwater of such an age will occupy the major part of the aquifer in the subsiding area. Because groundwater moves very slowly from surrounding areas to the centers of withdrawals, the slow movement is a contributory factor to land subsidence.
3. If the tritium concentration of groundwater is almost zero, the groundwater is constituted of "native" water—water that has been in the aquifer for a relatively long period of time. Such a small concentration indicates that the "native" water has not previously participated in groundwater flow and is brought into the essential part of the flow under the influence of pumping; this may indicate the initial stage of subsidence.

Large scale pumping will contract rapidly the distribution of the values of different age in the natural flow system in proportion to the difference between the flow velocity of groundwater during pumping and the natural flow velocity. Accordingly, even the above-mentioned rough age classifications will give some indications of the changes in groundwater flow direction, especially in areas of subsidence, if the distribution of tritium concentrations and changes with time are examined.

Secondly, the relative age difference may be analyzed from the difference between tritium concentrations of two samples. As a result of the survey of the surface-water balance, the percolation of surface-water in the test area was known to be nearly zero. An aquifer exists in the depth interval from about 30 to about 130 m. Therefore, the groundwater will flow through the aquifer towards and be concentrated in areas of maximum subsidence; the tritium concentration in the groundwater hardly will disperse, and will move with the water to areas of subsidence. In such a case, if two samples are analyzed for tritium concentrations (two measurements at the same sampling point at different times or measurements at two sampling points at the same time), their difference can be regarded as indicating the relative time between rainfalls, even though the actual lapse of time after a particular rainfall cannot be determined directly from the tritium concentration. Therefore, the tritium dating method will not only permit a more detailed indication of the qualitative flow of groundwater, but also present a possible means for the quantitative judgement of groundwater flow.

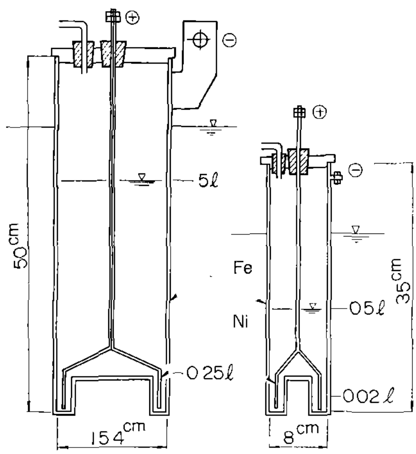

FIGURE 1. *Used electrodes*

If the two principles mentioned above were verified, the tritium dating method would be most suitable for the practical measurement of not only the groundwater flow at present but also the change in groundwater flow resulting from variations in pumping and the constriction of the aquifer.

III. METHOD

A detailed picture of groundwater flow may be obtained by the tritium dating method rather rapidly if the technique is applied in two ways: taking into account (1) geometrical distribution of tritium as affected by geological conditions, and (2) changes in concentration with time, evaluated over an area greater than the subsidence area. In the measurements reported here, stress was placed on the alluvial and upper diluvial aquifers. Sampling was done at 6 wells, 260 m to 1200 m in depth, at which water was being pumped from lower Diluvium and Tertiary for extracting natural gas for industry, as their influence on land subsidence had to be known. Water was sampled at several shallow wells to check the infiltration of precipitation, though a thick clay layer exists in the depth range of 20 to 40 m in the land subsidence area. Samples were collected at 54 wells every 6 months (in April and October). The distribution of the wells are shown in figure 2.

The technique used for tritium concentration determinations was selected because it is relatively fast, simpler than other procedures and gives a uniform accuracy. Groundwater samples, about 10 l each in volume, were distilled and concentrated into about 20 ml by electrolysis. The electrolysis apparatus was assembled as shown in figure 1. Its anode was made of nickel and its cathode was made of pure iron. In the first step, water was concentrated to about 250 ml by the apparatus shown in the lefthand diagram of figure 1. In the second step, two identically concentrated samples (about 500 ml in total) were concentrated further into 250 ml by the apparatus shown in the righthand diagram of figure 1. The current for electrolysis was 0.10 A/cm^2 in both steps, Östlund's electrolysis constant was about 5. The tritium in the samples was concentrated to about 40 times after these steps. The electrolysis method allows simultaneous concentration of many samples because operation of the instruments is simple and they require either operator attention, though the method does require more time than the thermal diffusion method.

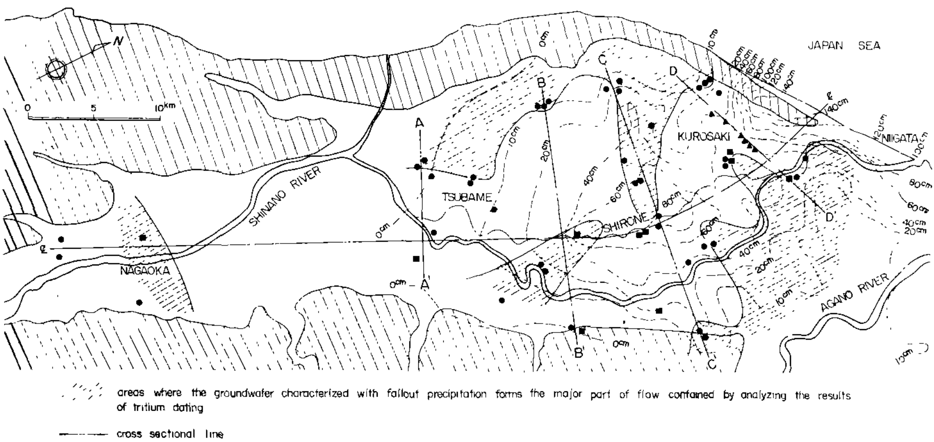

FIGURE 2. *Plan of land subsidence area in Niigata showing groundwater flow*

Three ml of each concentrated water sample was added to 18 ml of liquid scintillator and the radioactive intensity of its tritium was measured by a 2-channel liquid scientillation spectrometer. The scintillator consisted of 0.3 g of POPOP, 6.0 g of PPO and 120 g of naphtalene, diluted into 1 l by dioxane. Measurement was made at 5°C and for 500 to 1,500 minutes depending on the radioactive intensity of each sample so that the error of measurement falls within 1%. The quenching calibration was made by Discriminator Ratio Method to obtain true disintegration. The mean counting efficiency was about 12%. As far as the intensity measurement techniques are concerned, the use of a counter for gasified samples or the measurement of samples converted into benzene has certain advantages but the procedures are too complicated to treat many samples and the reproducability of actual treatments tends to be unstable in each of the two methods. The method that was adopted is simpler and gives more stable results.

The values of tritium concentration obtained by the above procedures proved to fall within the error range of $\pm 5\%$, after being correlated with various standard samples of tritium concentration including the ones of the Radioisotope Research Laboratory, Institute of Physical and Chemical Research, which were adopted as the Japanese National Standard of IAEA.

IV. RESULTS AND DISCUSSION

a) GROUNDWATER FLOW IN ALLUVIUM

The direction of major groundwater flow is considered to be from Nagaoka to Kurosaki, NNE, which is the direction of the dip of the Alluvium. Four sections, A, B, C and D respectively from upstream, were chosen across this direction, as shown in figure 2.

Tritium concentration of groundwater from test wells situated along these sections are tabulated in tritium unit for each measurement period and are presented in figure 3. The hatched portion shows an area in which tritium concentrations are 10 T.U. or more (referred to as concentration range A hereafter) which indicates that most of the groundwater in this area was derived from rain water carrying the influence of fallout. The dotted portion shows an area in which the concentrations ranged from 1 to 9 T.U. (range B) which indicates that most of the groundwater in this area was derived from rain water free from the influence of fallout. The blank portion shows an area in which concentrations were 0.9 T.U. or less (range C) which indicates that most groundwater in this area was "native" water that had not previously participated in natural groundwater flow.

i) main groundwater flow

As shown in figure 3, water containing tritium concentrations within range B occupies most of the pervious Alluvium, 40 to 150 m in depth, along all of the four sections, indicating the tritium concentration of the main groundwater flow. The concentration

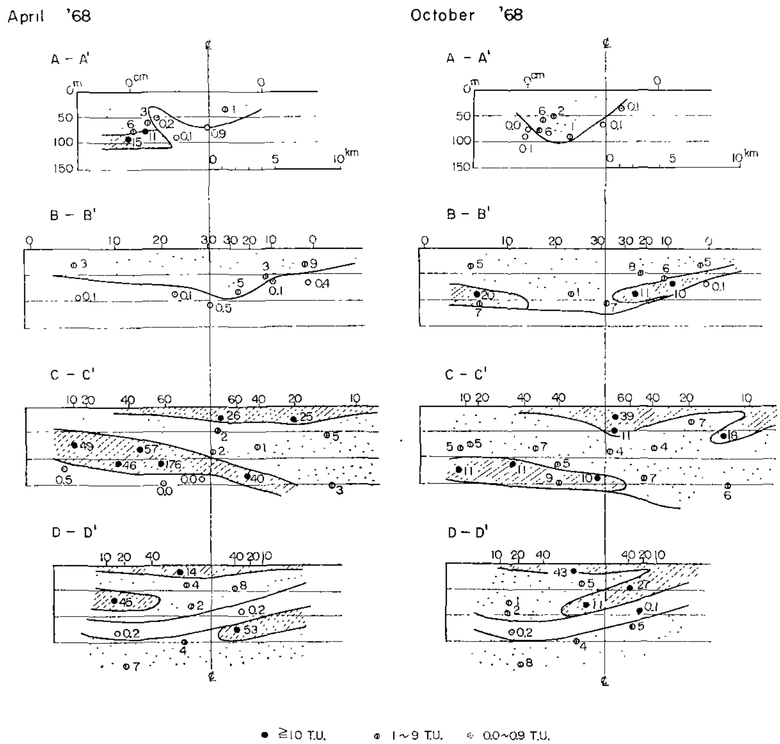

FIGURE 3. *Distribution of tritium concentration*

in this main flow would be 1 to 2 T.U. if the influence of the lateral flow containing concentrations in range A (to be described afterwards) were subtracted. This indicates rain water that fell 30 to 40 years ago. Water of concentration range A exists at Nagaoka, about 30 km upstream, indicating water of about 15 years in age. This shows that the velocity of the major flow is as small as about 2 km/year.

ii) flow of „native" water

Water with tritium concentrations in range C appears at the base of the pervious Alluvium except at a depth range of 100 to 150 m along section D. The contact with water of concentration range B is lower in October than in April, reaching as low as 30 m in the center of section D.

A large quantity of water is pumped in winter for obtaining natural gas. The groundwater flow upstream is slow, as said before, and an impervious clay bed, at a depth of 20 to 40 m, covers the pervious Alluvium. The pressure level of the pervious bed is depressed in winter and water then flows from its underlier because it has a higher pressure. The flow from the underlier reaches a maximum in April, as judged from water level fluctutions. The pumping for gas production decreases in summer and the pressure level in the pervious Alluvium recovers as shown in figure 4. The flow of water from the underlier is minimum in October. The seasonal change in the volume of aquifer occupied by water of concentration range C therefore indicates the annual variation in flow of water from underlying strata. Along section D, a large quantity of water is pumped from Diluvium and Tertiary materials to obtain gas for industry. No change in the position of the upper limit of the "native" water seems to have taken place. The reason is that the pumping from the Alluvium and from its underliers is well balanced and the source of groundwater recharge probably is nearby.

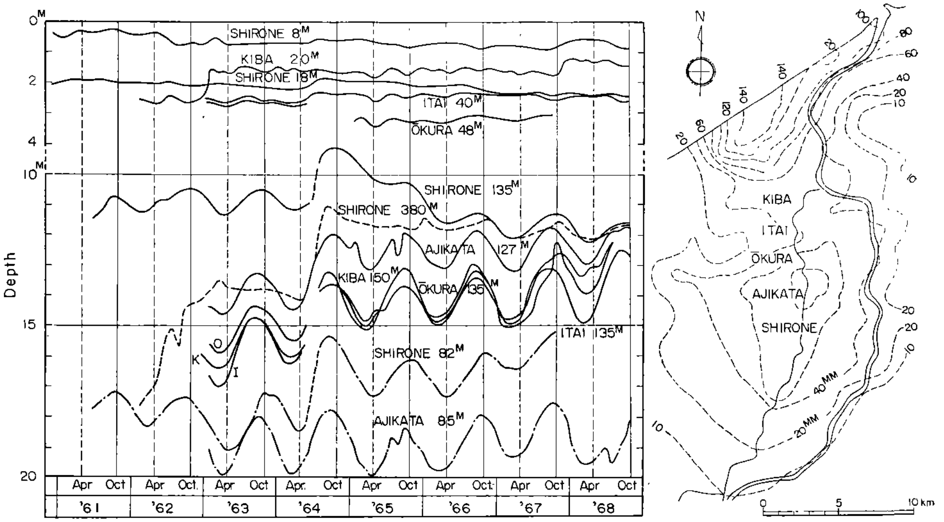

FIGURE 4. *Change of groundwater level at each depth*

iii) lateral flow of groundwater

The tritium concentration range A in water in the pervious Alluvium (fig. 3) probably indicates lateral flow from nearby water sources. The flow variations as indicated by sampling in April or October, is considered to be annual. April flow shows effects of pressure level changes resulting from heavy pumping in winter, and October flow indicates the recharge by precipitation in summer. The April and October samples indicate continuous inflow to the Alluvium from surrounding areas.

These lateral flows, when plotted in plan, are concentrated in areas that coincide with the areas of land subsidence (hatched area in fig. 2). This is the flow path produced by the large quantity of pumping in the past 2 or 3 years rather than the flow path that would occur under natural conditions. This is also verified from the fact that the monthly rate of subsidence of the Alluvium (40 to 140 m in depth; see fig 5) tended to decrease year after year, but it finally started to expand in the summers of these 2 or 3 years of heavy pumping.

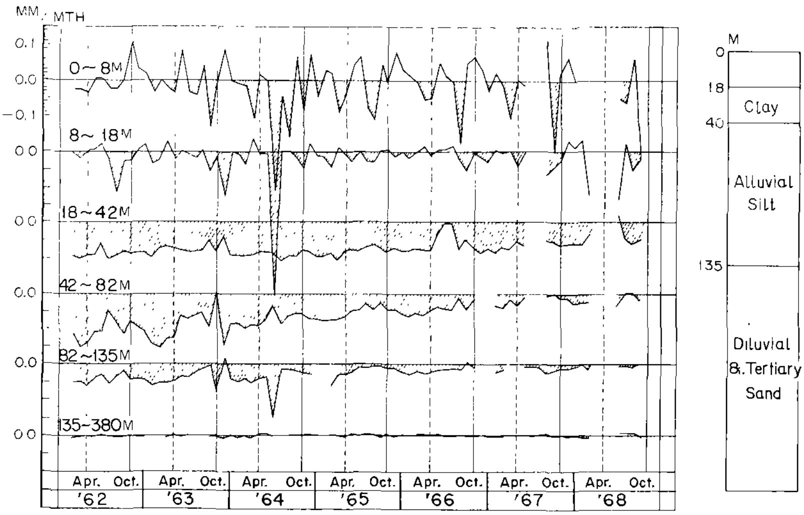

FIGURE 5. *Mounthly Subsidence of Each Stratum*

iv) infiltration of rain water

No infiltration of rain water is detected along upstream sections A or B (fig. 3). In areas such as those indicated by section C and D, where subsidence has been extensive, water of concentration range A which is supplied to the upper, near surface layers, is also found at greater depths. This comes from the dragging of rain water; the natural flow conditions being destroyed by the pressure level fall in the pervious Alluvium in the land subsidence area. The dragging is as slow as about 2 km/year.

b) GROUNDWATER FLOW IN DILUVIUM AND TERTIARY

The results of tritium measurements are shown in table 1. Though the small concentration values include errors as big as ±50%, it is clear that most of the groundwater consists of

TABLE 1. Tritium concentration of groundwater in Diluvium and Tertiary

Depth of strainer	October 1967	April 1968	October 1968
	T.U.	T.U.	T.U.
260 – 320 m	1.1	25	0.02
501 – 550	3.2	2.0	2.6
650 – 680	2.5	1.9	1.2
716 – 744	4.0	1 3	0.03
939 – 999	4.4	3.0	0.4
1620 – 1750	0.05	2.0	–

rain water fallen in recent years regardless of the age of the strata. This indicates the existence of nearby water sources and the possibility of supplying water to the pervious Alluvium. The rapid decrease of tritium concentration, however, indicates the flow of "native" water exceeds that from the nearby sources.

c) CONCLUSION

Though based on only a few measurements, qualitatively the dimension and routes of the groundwater flow in the land subsidence area have been indicated. Natural gas dissolved in water in the Alluvium, 40 to 150 m in depth, enclosed by an impervious bed as thick as 20 m, is removed along with water. The pumped water is derived from (1) the groundwater flow of about 2 km/year from the upstream side, (2) the inflow from its underlier, and (3) lateral flow and infiltration of rain water. The pumping of water has resulted in soil layer compaction in an extensive area where groundwater levels have been lowered. The evaluation of groundwater flow in Diluvium and Tertiary and the quantitative evaluation of each flow element in Alluvium could not be made, because of the lack of measurements both as to areal coverage and time. By continuous tritium measurements, along with the compiled hydrogeological data (pumping test inclusive), the evaluations will become possible. The results, supported by geotechnical considerations, will give information to clarify the causes of land subsidence and make it possible to forecast future subsidence.

DISCUSSION

Intervention of Mr Joseph T. CALLAHAN (U S A)

Question:
Did I understand the speaker to say that the water was dated at ten years?

Answer of Mr. KIMURA
We have dated or divided the water into three age groups : the very old one (tritium unit: almost zero), older than 15 years (tritum unit: 0—10) and younger than 15 years (tritium unit: more than 10). (see fig. 3).

SHRINKAGE OF SUBAQUEOUS SEDIMENTS OF LAKE IJSSEL (THE NETHERLANDS) AFTER RECLAMATION

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(the Netherlands), Molenstraat 28

ABSTRACT

The very soft subaqueous sediments on the bottom of Lake IJssel are subject to a considerable shrinkage after reclamation, brought about by a contraction of the soil, mainly due to high capillary potentials during the growing season. A prediction of this subsidence is necessary in view of the assessment of the waterlevel in the polders and the construction of pumping stations, sluices, etc.

1. Senior Research Officer, Scientific Department, Yssel Lake Development Authority, Kampen, Netherlands.

The prediction is done by means of the method of Hissink by comparing the specific volumes (reciprocal of dry bulk weight) of the original sediments with those of the same sediments, which are reclaimed ± 100 years ago. The applied method is described, including its accuracy and limitations.

Shrinkage increases with increasing clay content and decreases with increasing depth of the layer concerned. A graph shows the relation between the original thickness of the soft layers and 100 years after reclamation at clay contents varying from 5 to 60 % and at intervals of 5 %.

RÉSUMÉ

Les dépôts très mous et flasques sur le fond du Lac d'Yssel ont été soumis après l'endiguement à un tassement considérable, causé par une contraction du sol, qui est surtout la suite des tensions de capillarité pendant la saison de végétation. Une prédiction de ce tassement est nécessaire en rapport avec la fixation du niveau du polder et le projet des stations élévatrices, des écluses etc.

La prédiction est faite au moyen de la méthode "Hissink", qui compare les volumes spécifiques (valeurs réciproques du poids du volume sec) des dépôts originaux avec ceux des mêmes dépôts dans une région endiguée une centaine d'années auparavant. La méthode appliquée est décrite en incluant l'exactitude et les restrictions.

Le tassement augmente avec une teneur en argile croissante et diminue avec la profondeur croissante de la couche considérée. Un graphique fait voir le rapport entre l'épaisseur originale des couches juste après l'endiguement et leur épaisseur cent années après l'endiguement. Les relations sont données pour des teneurs en argile variant de 5 % jusqu'à 60 % avec des intervalles de 5 %.

1. INTRODUCTION

The shallow Lake Yssel, the former Zuiderzee, in the centre of the Netherlands has been separated from the sea by a dam in 1932. It covers an area of about 350,000 ha. Up till now four polders have been pumped dry (166,000 ha). The reclamation of two of them has been finished completely (68,000 ha), whereas that of the third (54,000 ha) is still in progress and the fourth (44,000 ha) has been drained recently (1968). The fifth and last polder (60,000 ha) will be dry about 1980.

The top layers of the bottom of Lake Yssel consist of subaqueous sediments, deposited during the last 20 centuries. Their thickness varies from nearly nil to over 4 m. They are underlain by Pleistocene, predominantly sandy sediments or older Holocene deposits, such as peat and old marine clay deposits.

The subaqueous sediments are very soft and completely impassible shortly after a polder emerges. Hereafter the withdrawal of water from the soil starts, bringing in progress a number of processes (among other things the shrinkage) and together called ripening. Due to evaporation, but mainly to transpiration by plants—in the first few years Reed (Phragmites communis) and after cultivation arable crops—the soil dries out and shrinks. As shrinkage is all-round, a direct subsidence is found, due to the vertical component of this all-round shrinkage. The horizontal component leds to the formation of large cracks, which are filled up partly by crumbs from the ploughlayer afterwards and by that also giving rise to subsidence.

The withdrawal of water from the soil by plant growth does not exceed a depth of 1 m, at the utmost 1.5 m. The layers, situated deeper are compressed by the increased grain tensions, brought about by the reduction of the groundwater table to about 1.2 to 1.5 m below surface. Both processes give rise to subsidence of the surface. As a matter of fact there is a transition layer in which both processes play a part.

Next to the subsidence of the subaqueous sediments, the peat and old marine clay layers, which are situated deeper than 1.5 m below surface in general, also contribute to the total subsidence. Their compaction is, just like that of the deeper situated subaqueous sediments calculated by the methods, usual in soil mechanics and so left out of consideration in this paper. Just for information it should be mentioned here that the subsidence of all Holocene sediments together varies from nil to about 1.5 m. However, in large parts of the polders it varies from 0.5 to 1.0 m. The compaction of the Pleistocene sediments is nil or merely a few centimeters at the utmost and is negligible.

It is very important to predict the subsidence, which may be expected, as exactly as possible, for the level of the polder water is established taking into account the future contour map composed from the elevation of the lake bottom at the moment of emergence and the expected subsidence. The subdivision of the polders in units with a different water level is related to this future elevation, and so are the designs of structures, such as pumping stations, sluices etc. As the period of depreciation of these structures is 80 to 100 years the predictions have to be given for the same period. Moreover, the subsidence is of importance for the design of the widely applied tile drainage systems and the evaluation of soils with a shallow clay cover on a sandy subsoil.

2. DESCRIPTION OF THE SUBAQUEOUS SEDIMENTS

The sedimentation conditions in the area of the present-day Lake Yssel have varied rather widely in course of time. (Wiggers, 1955; de Koning and Wiggers, 1955; Wiggers *et al.*, 1962; Ente and Wiggers, 1963). The oldest sediments have been laid down in lakes, surrounded by bog-peat. Their shores were eroded by wave-action and currents and by that the contents of organic matter suspended in the lakewater, will have been high. Supply of mineral parts was rather small, due to the narrow connection with the North Sea. So gyttja-like sediments were deposited (Detritus) with high contents of organic matter (15-30%). Afterwards the supply of mineral parts increased and at the same time the main part of the bog-peat area was already destroyed. For that reason sediments with lower contents of organic matter were deposited (Almere-deposits, Al). In the Almere deposits different layers can be distinguished, according to differences in contents of organic matter and clay and in the mutual relation between both values. The organic matter contents decrease from the oldest to the youngest layers, brought about by a decreasing erosion of the peaty shores. The supply of mineral parts increased simultaneously, caused by the widening of the connection between the Yssel Lake area and the Norht Sea. From the oldest to the youngest the most important layers are:

(*a*) Al^{c2+3}. rather rich in organic matter (contents of 8 to 15%) and also rather rich in clay (20 to 35%). There is no relation between the contents of organic matter and clay (particles $<2u$);

(*b*) Al^{c1}: poor in clay ($<5\%$) and composed of shallow layers with varying contents of organic matter. Sometimes these layers, rich in organic matter, consist nearly of pure, eroded peat;

(*c*) Al^{a} rich in clay (20-35%) with rather low organic matter contents in comparison with the other Almere-layers. These contents are comparable with those of marine mud deposits and there is a close relation between the contents of organic matter and clay.

The Detritus and the Almere-deposits were sedimented under fresh and afterwards increasing brackish conditions. About 1600 A.D. the salinity of the water increased rather abruptly to over 12,000 mg Cl′/l, brought about among other things by a sharp decrease in the capacity of the river Yssel. Under these saline conditions a marine sediment was deposited, mainly rich in clay. The contents vary from 5 to 50%; in the main part of the polders the range is 10 to 35%. A close relation is found between the contents of organic matter and clay, the first being about 11% of the last. The thickness of these Zuiderzee-deposits (Zu) varies from 0.5 to 0.8 broadly.

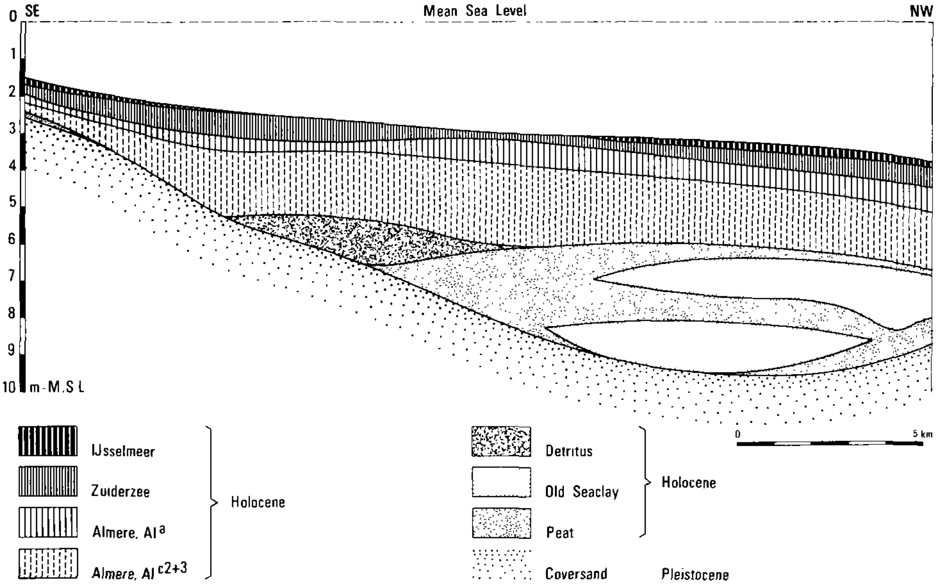

FIGURE 1. *Schematical geological cross-section through Southern Flevoland from the former coast (SE) to the centre of Lake Yssel (NW)*

After the separation from the sea the former Zuiderzee became fresh within a few years. In this fresh water reworked Zu-deposits were laid down, called Ysselmeer-deposits (Ym) which rarely exceed a thickness of 10 cms. The range of clay contents is wide. Organic matter contents are somewhat higher than those of the Zu at comparable clay contents.

A schematic profile of the various sediments is given in figure 1, showing their mutual position. The thickness of the various layers can differ considerably, especially in the Almere and older deposits.

3. METHODS OF INVESTIGATION

The withdrawal of soil moisture from the upper 1.0 to 1.5 m of the profile, brought about by the evapotranspiration, gives rise to widely diverging soil moisture tensions in the course of the year, varying from fieldcapacity or lower in winter to wilting point in the upper 20-40 cms in dry summers. By that soil mechanical formulae cannot be applied in calculating the shrinkage. By Hissink (1935) a method was developed by comparing the specific volumes at the moment of emergence and at the moment hereafter for which the prediction has to be given. The changes in specific volume are linear propertional to the shrinkage. The specific volume (s.v.) is the reciprocal value of the dry bulk weight. It represents the volume taken up by 1 gramme of dry soil under undisturbed conditions.

In aerated soils s.v. is defined by taking samples with a cylindrical sampler (ϕ7.3 cm, h 8.0 cm) with an exactly known content. From this content and the content of dry soil s.v. can be calculated. In non-aerated soils—so saturated with water—s.v. can also be calculated by means of formula (1)

$$\text{s.v.} = \frac{1}{\text{s.w.}} + 0.01\,A \tag{1}$$

in which

s.w. specific weight of the solid parts and 1/s.w. represents the volume of 1 gramme of oven-dry soil;

A water content in % by weight. It represents the volume of the pore space, belonging to 1 gramme of oven-dry soil.

In the non-aerated subaqueous sediments a close relation is found between the contents of clay and organic matter on the one hand and the water content on the other hand. This relation can be represented by formula (2)

$$A = 20 + n(L + bH) \tag{2}$$

in which

L clay content;

H organic matter content;

n and b factors, characteristic for every type of sediment.

On a mean in the subaqueous sediments $n = 2.2$ and $b = 3.0$ (Smits, 1962). The mean specific weight of the mineral parts is 2.66 ± 0.03 g/cm^3 ($n = 256$). The value for organic matter was found to be 1.48 g/cm^3. If L and H are known, s.w. and by that s.v. can be calculated. It will be clear from both formulae, that there is also a relation between L, H and s.v.

In older reclaimed soils also a relation is found between the contents of clay, organic matter and s.v. This relation is represented by formula (3)

$$\text{s.v.} = \frac{58 + m(L + bH)}{100} \tag{3}$$

It is found that s.v increases with increasing contents of clay and organic matter and at equal contents increases with increasing depth in general (see fig. 2), so m increases with increasing depth. For defining the relation of (3) samples have to be taken every 10 cms.

For predicting the shrinkage the s.v.'s of the sediment have to be compared with the s.v.'s of an older reclaimed sediment. The last sediment has to satisfy the following demands:

(*a*) The type of sediment from which the older polder is reclaimed has to be equal to the sediment for which a prediction is wanted. Shrinkage passes with time logarithmically, so the starting point has to be equal, as even after a century differences in s.v. are found in various types of sediments (e.g. mudflads and saltings);

(*b*) The drainage conditions have to be equal to the conditions expected in the area to be reclaimed. Better conditions promote the shrinkage, whereas worse conditions delay it on the contrary;

(*c*) Related to these drainage conditions is the type of land use, e.g. arable or pasture land The depth of the rooted part of the profile and the extent to which the soil is dried out, depends on this land use;

(*d*) The period after reclamation has to be equal to that, for which the prediction is wanted

Though these demands are rather simple, in practice they lead often to difficulties, especially for subaqueous sediments, as such sediments, reclaimed 80 to 100 years ago are rare.

The values of s.v. vary from 0.60 to over 2.00 cm^3/g, dependant on the type of sediment, contents of clay and organic matter, depth below surface and stage of ripening. In aerated soils samples are taken in quadruplicate, whereas in non-aerated soils a mixed sample of 16 cores is usual for analyzing the watercontent. Afterwards the relation between the contents of clay and s.v. is calculated per layer. It is recommended to take at least samples at five clay contents, so per layer 20 couples of observations are available. From a large number of data it appeared, that the standard deviation of s.v. ranges from 0.02 to 0.06 cm^3/g, both in aerated and non-aerated sediments. Only in sediments reclaimed rather recently and by formation of large cracks splitted up into prismatic columns, the standard deviation is higher, ranging from 0.09 to 0.12 cm^3/g. For this stage the method is rather unreliable and can be applied only carefully and at least a larger number of samples is necessary.

The shrinkage is calculated by the following, simple formula (4):

$$d_1 : d_2 = s.v._1 : s.v._2 \tag{4}$$

in which

d_1 thickness of a layer at the starting point;
d_2 thickness of a layer at t years after reclamation;
$s.v._1$ specific volume at the starting point;
$s.v._2$ specific volume at t years after reclamation.

As a rather remarkable increase of $s.v._2$ is found with increasing depth (fig. 2) in older sediments, the thickness of d_2 has to be restricted to 10 cms. In subaqueous sediments the value of *n* (form. 2) does not vary systematically throughout the profile before emergency. Form. 4 can be transformed to (5):

$$d_1 = \frac{s.v._1}{s.v._2} \times d_2 \tag{5}$$

and from the known values of $s.v._1$, $s.v._2$ and d_2 (10 cms) the original thickness of every layer can be calculated. In this way the relation between the original thickness of a layer and its thickness after shrinkage is obtained.

4. RESULTS

The only polders built up of sediments, comparable with those on the bottom of Lake Yssel are the Y-polders, reclaimed in 1872-1876. However, these sediments have high clay contents only (in general over 40%) and such high contents are rare in the sediments in Lake Yssel. For that reason data collected in the well drained Johannes Kerkhovenpolder have been selected. In 1875 this polder has been reclaimed from rather low lying tidal mud flats. The clay content ranges from about 5 to roughly 50%. Sampling has been carried out in 1964. The relation between clay content, depth below surface and s.v. is shown in figure 2. The organic matter content can be left out of consideration, as a close relation is found between the contents of clay and organic matter and moreover the contents of the last are low.

Though no data are available on the value of *n* (form. 2) at the starting point, it is assumed to have been 1.7. This value has been found in present-day mud flads, adjacent to the Johannes Kerkhovenpolder. There is no evidence to suppose that in the presentday mud flats drainage conditions deviate substantially from the conditions of the mud flats in 1875. However, the value is found to be smaller than in the subaqueous sediments (2.2). Nevertheless, the data from the Johannes Kerkhovenpolder can be used, as from a comparison of s.v. in layers corresponding in depth, and comparable in organic matter content in this polder and the Y-polders it appeared that they are about equal. The deviations of the Y-polder data from the mean relation between clay content and s.v. in the Johannes Kerkhovenpolder are the same (fig. 3). For that reason it is supposed that the same will hold at lower clay contents.

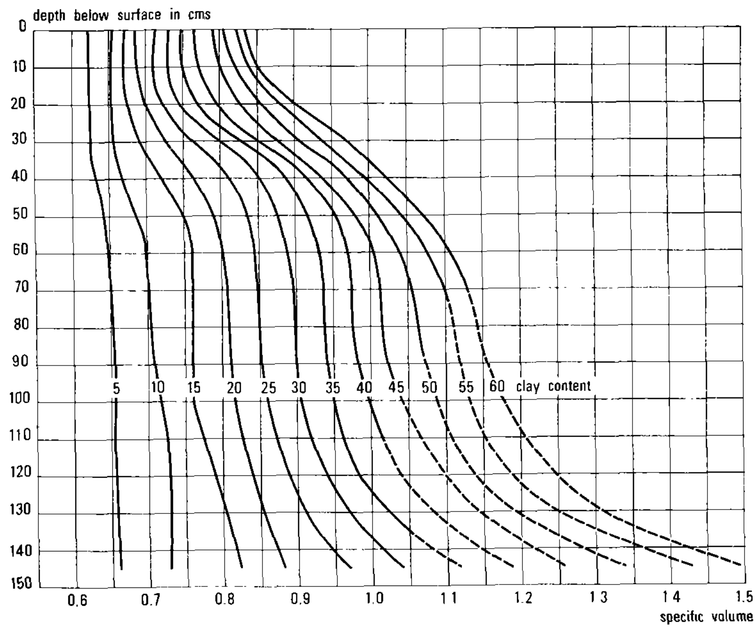

FIGURE 2. *Relation between specific volume and depth below surface at various clay contents in the Johannes Kerkhovenpolder*

At the starting-point the specific volumes of the subaqueous sediments can be calculated with the help of formulae (1) and (2) wherein $n = 2.2$ (Smits, 1962). By means of formula (5) the original thickness of every layer has been calculated at multiples of 5 of the clay content. The results are given in figure 4. By interpolation the shrinkage at other clay contents can be read. The graph as such is valid only for profiles, homogeneous in clay content with depth. However, it can also be used for heterogeneous profiles by applying an auxiliary, transparant graph.

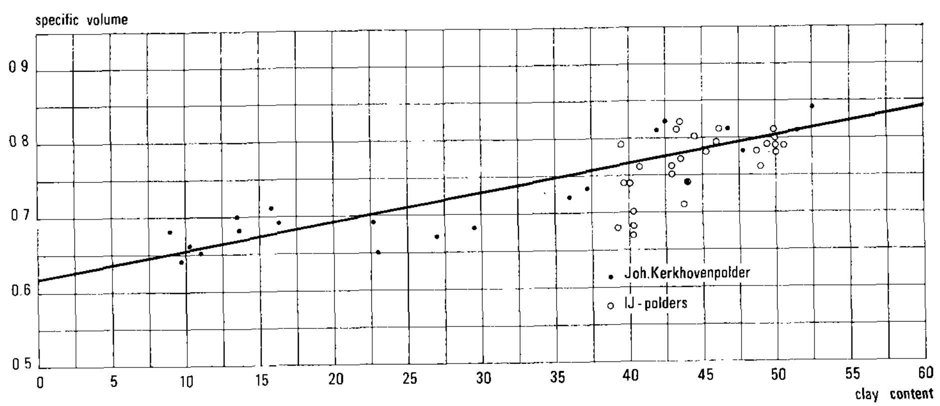

FIGURE 3. *Relation between specific volume and clay content in the plough layer of the Johannes Kerkhovenpolder and the Y-polders*

Supposing at the starting point a profile is build up as follows: 0-40 cms with 40% of clay, 40-70 cms with 25% of clay, 70-130 cms with 15% of clay and overlying sand. The layer 0-40 cms shrinks to 28 cms. Zero of the auxiliary graph is placed in $d_2 = 28$ cms and $L = 25\%$. On this graph the intersection of $d_1 = 30$ cms (70-40 cms) and $L = 25\%$ is fixed and on the d_2-axis read 24 cms. So 40-70 cms shrinks to 24 cms, and hence 0-70 cms shrinks to 0-52 cms (28 + 24 cms). In the same way the shrinkage of the third layer is found by placing zero of the auxiliary graph in $d_2 = 52$ cms and $L = 15\%$. This layer shrinks from 60 cms (130-70 cms) to 54 cms. Hence in all the profile shrinks from 130 cms to $24+28+54 = 106$ cms and the total shrinkage is $130-106 = 24$ cms.

This graph can be applied for Ym-, Zu- and Al^a-deposits, as their organic matter contents are nearly equal or equal to those of the marine sediments of the mud flats. The clay contents of the Al^{c1}-deposits are very low and the layers rich in organic matter already so compact, that hardly any shrinkage occurs. However, it has been at least questionable, if this graph were appropriate for Al^{c2+3}-deposits, brought about by their relatively high organic matter contents. From a comparison of the specific volumes of sediments in the Y-polders, comparable with the Al^{c2+3}-deposits, and such data of the Waard- en Groet- , polder, built up of marine sediments with normal organic matter contents, it appeared that they were about the same. As drainage conditions in both polders are about equal (and worse than in the Johannes Kerhovenpolder and the Ysselmeerpolders), it may be concluded, that in layers, situated deeper in the profile, differences in organic matter contents (naturally to a limited extent, say below 15%) do not influence the shrinkage. Consequently the graph of figure 4 can also be applied for the Al^{c2+3}-deposits.

In general the organic matter contents of the Detritus are too high to apply figure 4. As comparable, older deposits do not exist, only a rough estimate of the shrinkage can be made, being about 10%. However, Detritus is rather rare within a depth of 1.5 after 100 years, except in the North Eastern polder.

From the foregoing it may be concluded that the relation between the thickness of subaqueous sediments before reclamation and 80 to 100 years afterwards can be predicted by the data of figure 4.

5. DISCUSSION

The described method cannot be applied to the prediction of the shrinkage of subaqueous sediments only, but naturally to all sediments, liable to shrinkage after reclamation, such as the sediments of the tidal mudflats and saltings, tidal reed marshes, willow coppices and so on. Though it is the only method to draft such a prediction of the shrinkage of the upper 1.0 to 1.5 m of a sediment, it has as such some considerable restrictions, due to the fact that it is a comparative method.

In practice it is often difficult to satisfy completely the demands, mentioned in 3. Comparable sediments of the required age are not available always and if so their drainage conditions may differ—in general they are or were worse—from those in modern reclamations. Even if such comparable sediments are on hand, the range of the contents of clay and/or organic matter may not be similar or drainage conditions or land use are unequal. In case sediments, satisfying the mentioned demands, do not exist, the shrinkage has to be estimated with the help of data of older or younger sediments or more or less comparable sediments under application of inter- and extrapolation and not always well known relations on the course of shrinkage with time and the mutual relation between the shrinkage of various sediments. It will be clear that such estimated predictions are less reliable than predictions, derived from comparable sediments under the right conditions.

Another disadvantage of this method is that comparisons can be made only under similar climatological conditions. Differences in these conditions bring about differences in the rate of drying out of the soil and so in the extent and variation of the capillary

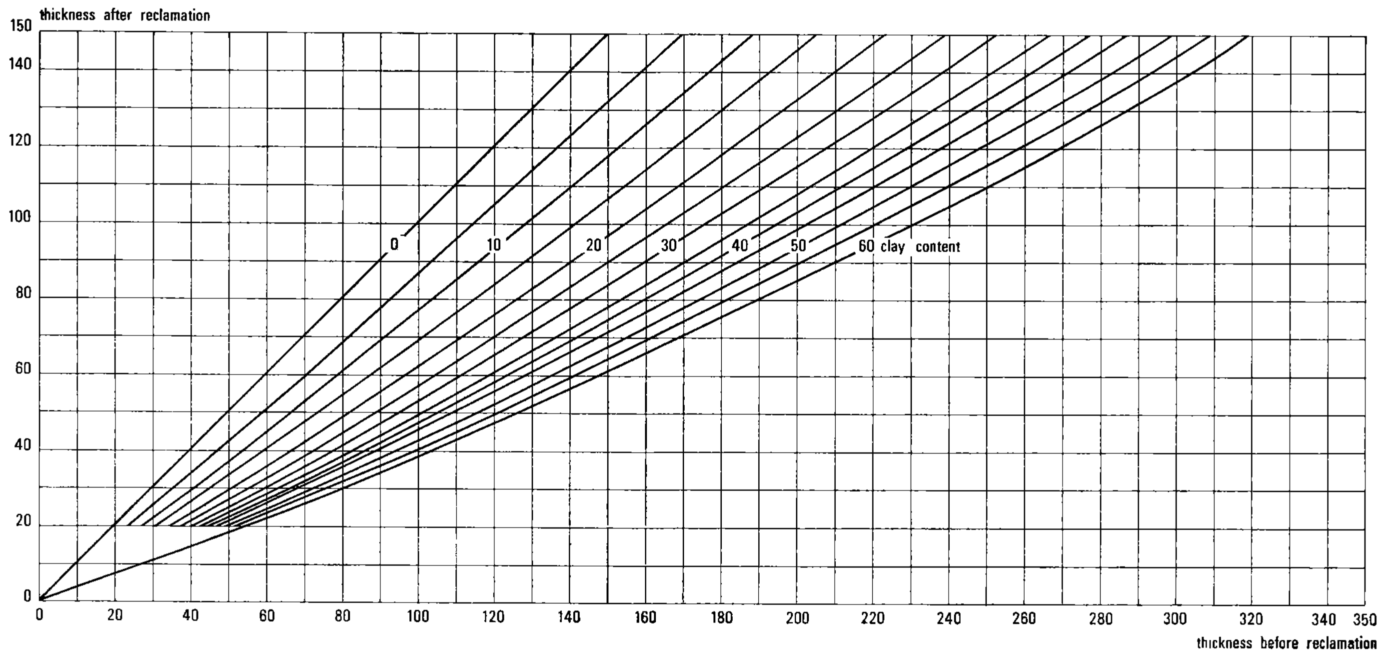

Figure 4. *Relation between the thickness of subaqueous sediments in Lake Yssel before and 100 years after reclamation at various clay contents*

tensions. By that the shrinkage will vary. Of course, slight variations in climate will not affect the shrinkage to a remarkable extent and for instance data valid for Dutch climatological conditions will be valid also for all comparable sediments, bordering the North Sea, without introducing large inaccuracies.

On the other hand for example, in tropical deltaic areas these data should not be applied as the climatological conditions diverge widely from those in the North Sea area. Moreover, the type of landuse deviates considerably, being predominantly irrigated or rain fed paddy. The application of the method will be problematical here as in general in such areas comparable older sediments do not exist and even if they are present their age is often unknown. For such areas merely a rough estimate of the shrinkage to be expected can be made.

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THE HYDROLOGIC BALANCE IN THE LAND SUBSIDENCE PHENOMENA

Tatsuo SHIBASAKI and Shizuo SHINDO

ABSTRACT

The hydrologic balance in confined ground water basins was studied for certain cases in the land subsidence areas in Japan. The equation for hydrologic equilibrium in a particular basin is given by $Qr = R + L = AS\ \mathrm{d}h/\mathrm{d}t + Qd$, where Qr is the recharge to the basin per unit time, R is the recharge through lateral seepage flow, L is the recharge through leakage from semiconfining strata, A is the area of the basin, and S is the average storage coefficient of the basin, $\mathrm{d}h/\mathrm{d}t$ is the average change in the height of ground water level in the basin per unit time, Qd is the discharge per unit time from the whole area of the basin. The results of computations show that the leakage is estimated to be about 60 to 70 % of the total recharge to the basin. These values of leakage ratios will be useful in solving the problems in the land subsidence area.

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RÉSUMÉ

Le bilan hydrologique a été étudié dans certains cas de bassins artésiens au Japon. L'équation de l'équilibre hydrologique dans un bassin particulier est donné par $Qr = R + L = AS\, dh/dt + Qd$ où Qr est la recharge du bassin par unité de temps, R est la recharge par filtration latérale, L est la recharge par pertes de couches voisines semi-artésiennes, A est la superficie du bassin, S est un coefficient d'emmagasinement moyen et dh/dt est la variation moyenne de la hauteur d'eau souterraine par unité de temps; Qd est le débit sortant par unité de temps de l'étendue totale du bassin. Le résultat de certaines recherches donne pour la recharge provenant des pertes des couches voisines, 60 à 70 % de la recharge totale. Ces valeurs peuvent être utiles pour résoudre le problème dans les cas d'affaissements.

INTRODUCTION

In several areas in Japan, land subsidence has been observed to accompany extensive ground-water drawdowns by excessive pumping from confined aquifers, but no atention has been paid to the hydrologic balance in connection with land subsidence phenomena. The changes in height of ground-water levels, of course, depend upon the quantities of water recharged and discharged. The quantitative prediction of the drawdowns is the first step to the prediction of the land subsidence.

Our team have commenced an investigation of the hydrologic balance in a confined ground water basin according to the flow chart shown in figure 1. Although our investigation is still incomplete, we will discuss the results of a few of our recent studies.

In preparing this presentation, our cordial thanks go to our many colleagues for assistance in various ways.

MATHEMATICAL MODELS FOR THE HYDROLOGIC BALANCE IN CONFINED GROUND WATER

For hydrologic systems analysis, the water complex described previously is replaced by a simplified model. Two models are given to describe the hydrologic balance in a confined

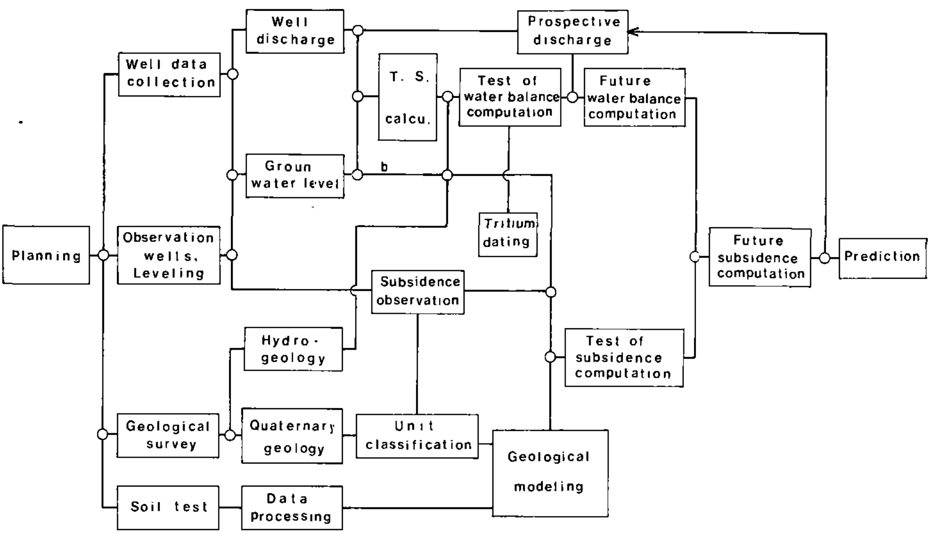

FIGURE 1. *Simplified flow chart for prediction of land subsidence*

ground-water system, one is the equation for hydrologic equilibrium in a unit confined ground-water basin, another is for dimensional zones in it.

The equation for hydrologic balance in a unit basin, which was demonstrated by Shibasaki and Kumai (1968), is given by

$$Qr = R+L = AS\frac{dh}{dt} + Qd \tag{1}$$

in which, Qr = recharge to the basin per unit time, R = recharge through lateral seepage flow, L = recharge through leakage from semiconfining strata, A = area of the basin, S = average storage coefficient of basin, dh/dt = average change in the height of ground water level in the basin per unit time, and Qd = discharge per unit time from the whole area of basin.

MacNeal (1953) proposed that the ground-water basin model was divided into small polygonal zones, a typical node point, its neighbors, and the associated polygonal zone are shown in figure 2. In such an asymmetric network of node points, the difference-differential equation for ground water flow is given by

$$\sum_i (h_i - h_B)\, Y_{i,B} = A_B S_B \frac{dh}{dt} + A_\beta Q_\beta \tag{2}$$

where

$$Y_{i,B} = \frac{J_{i,B} T_{i,B}}{I_{i,B}}$$

and, A_B = area associated with node B, $Y_{i,B}$ = conductance of the path between nodes, i and B, S_B = values of the storage coefficient of polygonal zone centered at node B, Q_B = volumetric flow rate per unit area at node B, $T_{i,B}$ = value of the transmissibility at the midpoint between nodes i and B, $I_{i,B}$ = distance between nodes i and B, and $J_{i,B}$ = length of the perpendicular bisector associated with nodes i and B.

The left-hand side of equation (2) is the summation of subsurface flows between a given area and its surrounding areas. The rate of change of storage is given by the first term on the right-hand side, and the second term represents the surface flow rate out of the zone of saturation of the given unit area. Replacing the terms of equation (2) as follows; $A_B = A$, $\Sigma\ (h_i - h_B)Y_{i,B} = R$, $A_B Q_B = \mathrm{Qd} - L$, and $S_B = S$, equation (2) is completely the same as equation (1).

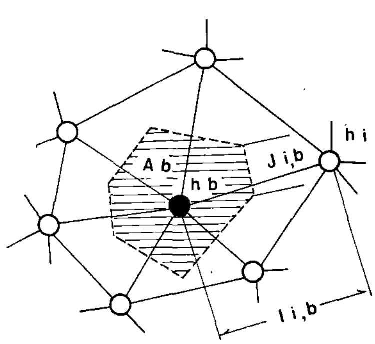

FIGURE 2. *Polygonal geometry*

Equation (1) is convenient for computing the hydrologic balance in the whole confined ground-water basin. The elevation of the water table at a certain point in the basin is computed by equation (2). The solution of equation (2) on the analog or digital computer was reported by Tyson and Weber (1962).

FORMULATION OF THE HYDROLOGIC BALANCE COMPUTATION

Before the computations of ground-water balance are made, it is necessary to collect the basic data of the basin parameters which define the magnitude of the basin behavior. The steps of data processing taken in this approach are shown schematically in the upper part of figure 1, which indicates the sequence and relationship of the several contributing investigations. For the basis of unit classification, geologic studies should be completed in an early stage in the investigation.

The computional procedure is divided into two phases; in the first phase the values of basin parameters, especially the storage coefficient, are identified by the historical hydrologic data, and in the second phase the values of basin parameters determined in the first phase are used to predict the elevation of future ground water levels. The results of prediction, of course, should be checked for further reference.

DETERMINATION OF BASIN PARAMETERS

1. Transmissibility coefficient (T)

Usually the value of transmissibility is obtained by pumping tests from a pumped-well and observation-well system. In a case where it is hard to get many pumping-test wells, the following approximate solution may be adopted.

In the more general case of a well penetrating a confined aquifer, the well discharge is given by

$$Q = 2.73\ kMs/(\log R - \log r) \tag{3}$$

where, Q = well discharge, k = permeability coefficient, M = the thickness of aquifer s = drawdown, R = the radius of the cone of pressure relief, and r = well radius. Equation (3) is well known as the equilibrium formula, or Thiem equation. Substituting in equation (3) gives

$$T = kM \tag{4}$$

$$R = 2s\sqrt{T} \tag{5}$$

where, T is the transmissibility coefficient in cu. m per day; therefore equation (3) reduces to

$$\log T = \frac{5.46}{Q}sT - 2\log\frac{2s}{r} \tag{6}$$

To obtain the formation constant from pumping test data, Klimentov (1961) suggested an approximate solution based on a graphical method. When many well data are collected

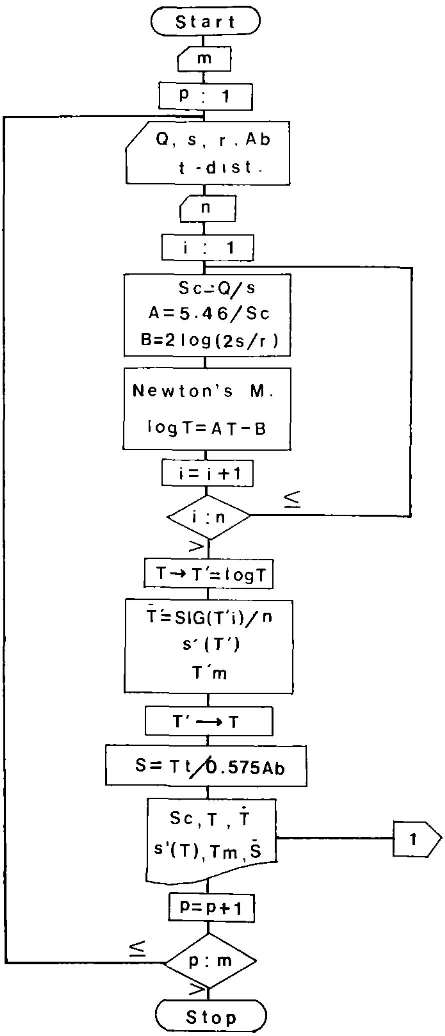

FIGURE 3. *Simplified flow chart for digital computer solution of formation constants*

in a polygonal zone, the average formation constants can be computed statistically by computer. A portion of the flow chart that was used is shown in figure 3. An example of the result of the formation constant calculation is given in figure 4.

2. STORAGE COEFFICIENT (S)

The value of storage coefficient of a confined aquifer is also obtained by a pumping test. In the case where it is hard to get many pumping test data, the storage coefficient can be calculated by the next formula, which is based upon the essential definition

$$S = \Delta Qd/\Delta h \tag{8}$$

where, ΔQd = the increasing volume of water released from the unit surface area of confined aquifer, and Δh = the corresponding decline of ground-water level.

The average storage coefficient of the polygonal zone at node B may be approximately calculated by

$$S_B = \frac{1.784\, t}{A_B} T_B \tag{9}$$

where, t = time of pumping period, T_B = average transmissibility coefficient at node B, and A_B = area associated with node B. Equation (9) was presented by Robinson and Skibitzke (1962).

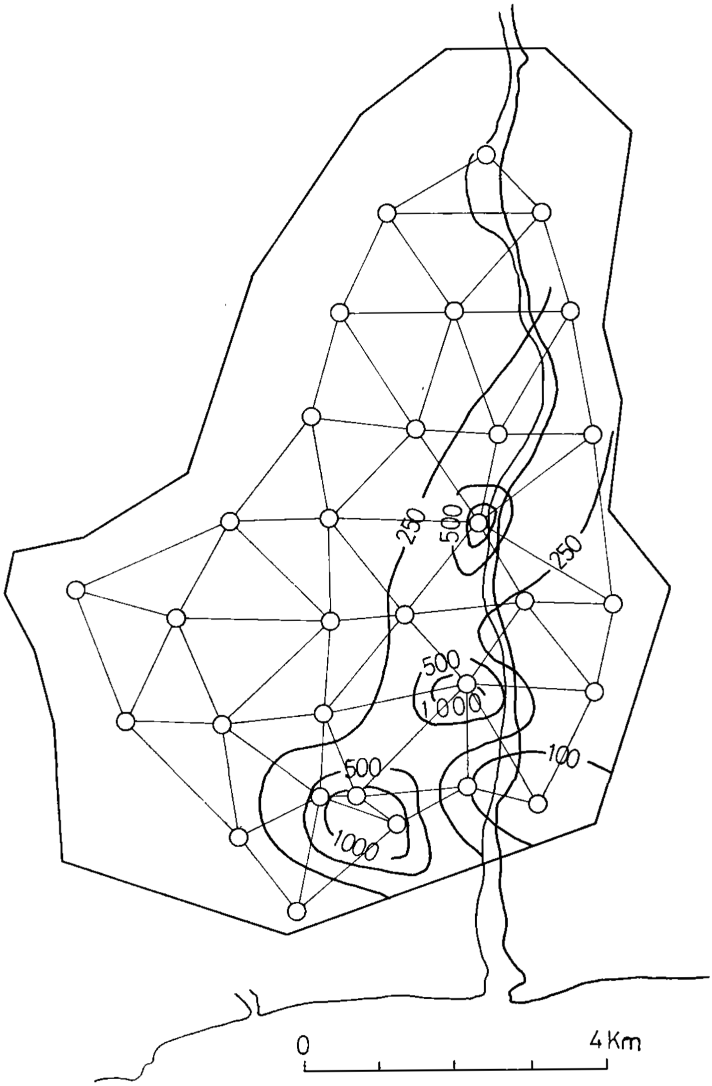

FIGURE 4. *Transmissibility distribution in Hiratsuka basin, Kanagawa Pref*

3. Well discharge (Qd)

This item is the most important parameter to influence the hydrologic balance computation, because it is directly affected by human activities related to irrigation, industries, and municipalities. This discharge must be estimated as accurately as possible. Although tedious it is accurate to make the measurements and sum all of them.

In practice, well discharge estimates are made from samples of representative wells, multiplying the unit well discharge by corresponding number of wells gives the pumping discharge for each zone. The sum of these becomes the total discharge over the basin.

The estimation of future pumping discharge is an important problem for a future hydrologic balance computation. Time series analysis and regression analysis are used, which have been developed in the branch of econometrics recently.

EXAMPLE OF THE HYDROLOGIC BALANCE COMPUTATION IN A UNIT BASIN

Recharge into the basin is the most difficult to evaluate, because it cannot be directly measured. When well systems are pumped, ground-water levels are lowered and the

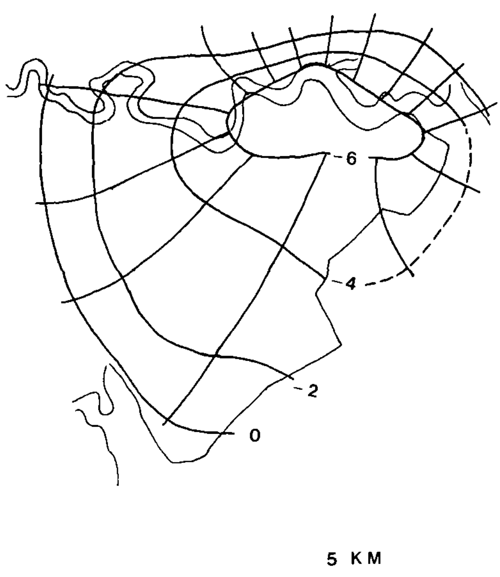

Figure 5. *Ground-water flow net of Shiroishi basin, Saga Pref. (14th Mar. 1966)*

supplemental recharge flows are removed from the surroundings. Taking the case of the Shiroishi Plain, Saga Prefecture, a concrete example will be given to explain the estimation of supplemental recharge to the basin.

Ten sheets of the flow-net maps were finished from August 1962 to March 1967. An example of the map is given in figure 5. The volumetric recharge flow through lateral seepage can be estimated from Darcy's law

$$R = \frac{nf}{nd} T \cdot H \tag{10}$$

in which, nf = number of stream channels in flow nets, nd = the number of equipotential drops, T = transmissibility coefficient, and H = total head loss.

For these cases, the average transmissibility of the basin as obtained by pumping tests is $3.0 \times 10^{-3} m^2/sec$. Thus the volumetric recharge flow, R, could be calculated by

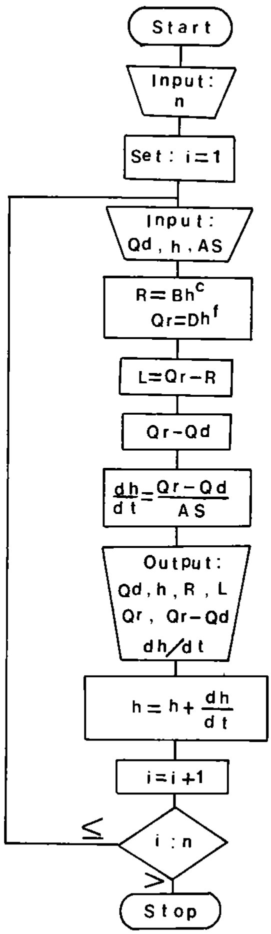

FIGURE 6. *Simplified flow chart for digital computer solution of hydrologic balance in a confined ground-water basin*

equation (10) for each time. The closed relation was observed between R and the average height of water levels in the basin, which is given by

$$R = 3.5 \times 10^4 h^{0.887} \tag{11}$$

where, R = recharge through lateral seepage, in cu. m per 10 days, and h = the average height of water levels below the sea level, in m.

Giving actual field values to equation (1), the computed dh/dt may be a larger value than the actual value, because the leakage from the semiconfining strata, L, was neglected. The item Qr is repeatedly ajusted until the computed water levels match the known water level records. Thus, Qr is given by

$$Qr = 6.4 \times 10^4 h^{1.175} \tag{12}$$

The relationship among R, L, and Qr is approximately expressed by

$$R = 0.33\, Qr \tag{13}$$

and

$$L = 0.67\, Qr \tag{14}$$

After the relation between the recharge and h is obtained, it is easy to estimate the hydrologic balance computation by computer. The flow chart that was used is shown in figure 6. The result which was obtained in the way described above is shown in figure 7.

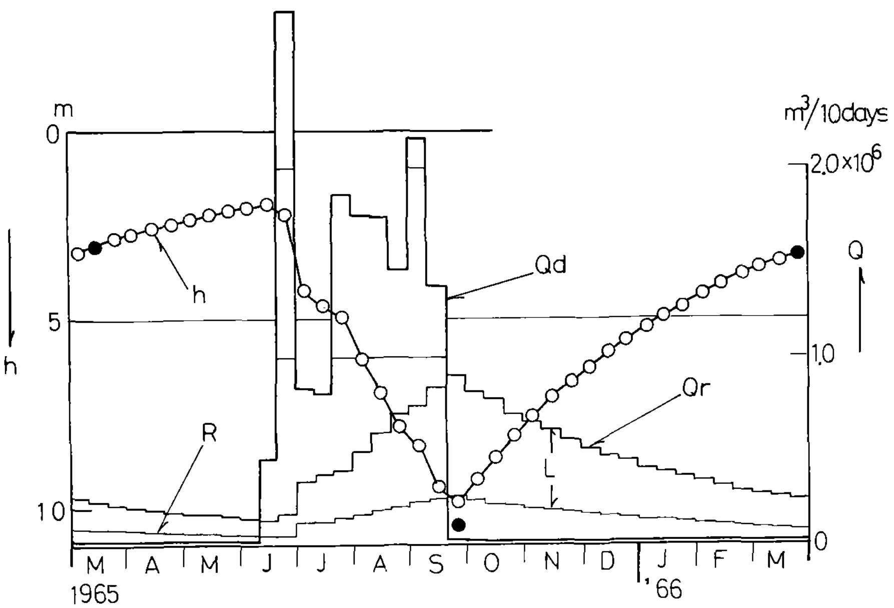

FIGURE 7. *Hydrologic balance diagram of Shiroishi basin*

EXAMPLE OF THE HYDROLOGIC BALANCE COMPUTATION IN POLYGONAL ZONES

The hydrologic balance in polygonal zones is computed by equation (2). Recently, numerical analysis by the digital computer has developed rapidly. Several reports of this type of study were published (e.g. Tyson and Weber, 1962; Vemuri and Dracup, 1967).

The size of the polygonal zones is dependent on the variation in replenishment, extraction, transmission, storage, and water level data. For purposes of testing the model against historical water level data, provision is made for the extraction of time-varying flow rates from each of the zones.

The general flow chart that was used is shown in figure 8. The left-hand side of the flow is the identification phase and the right-hand side is the computational phase, respectively. In the identification phase, the parameter S is repeatedly ajusted until the computed

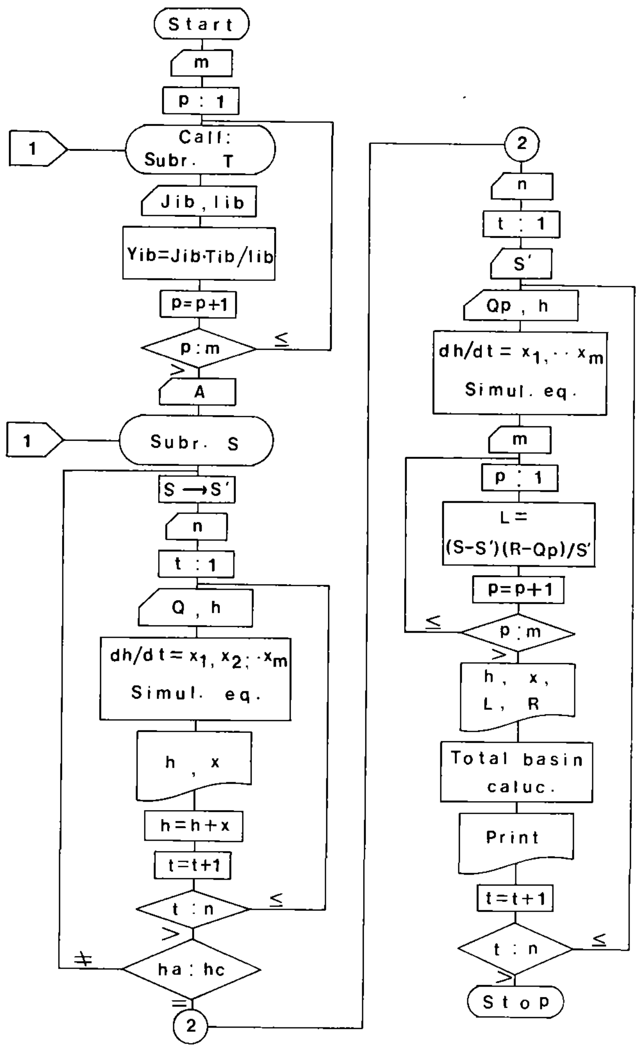

FIGURE 8. *Simplified flow chart for digital computer solution of hydrologic balance in polygonal zones*

water levels match the historical water-level data by a trial and error method. The fixed values of S at each node are stored in the digital computer for future use in the next phase, and the mathematical model of the basin is subjected to various operating conditions corresponding to future pumping discharge or any other pertinent situation to predict the future water-level trends.

The rate of leakage per unit area in each of zones can be calculated by the following equation

$$L_B = (S_B - S'_B)(R_B - A_B Q_B)/A_B S_B \tag{15}$$

where

$$R_B = \sum_i (h_i - h_B) Y_{i,B}$$

and S'_B = fixed S_B in the identification phase.

Figure 9 shows an example which was computed by this method under the prospective pumpage of 12.5×10^4 cu. m per day over the basin.

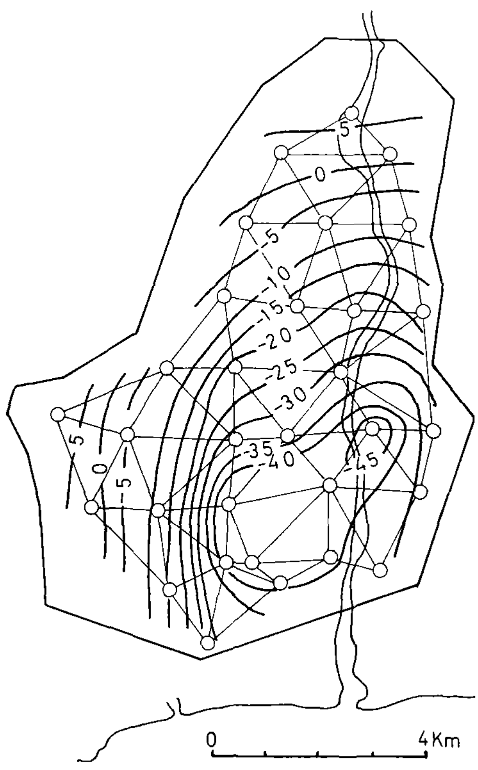

FIGURE 9. *An example of computed water-table map of Hiratsuka basin*

ITEMS OF THE HYDROLOGIC BALANCE IN CONFINED GROUND WATER BASIN

The items of the hydrologic balance in the Hiratsuka basin, Kanagawa Prefecture, are given in figure 10. The data covered the nine-year period from 1960 to 1968. The total supplemental recharge is about 80 percent of the total discharge from the basin, the lowering annual water level results from minus 20 percent in unbalanced water budget, and the leakage is estimated to be about 60 percent of the total recharge to the basin. It should be noted that these values of leakage ratios, about 60 to 70 percent, have been estimated by our during investigations of other basins.

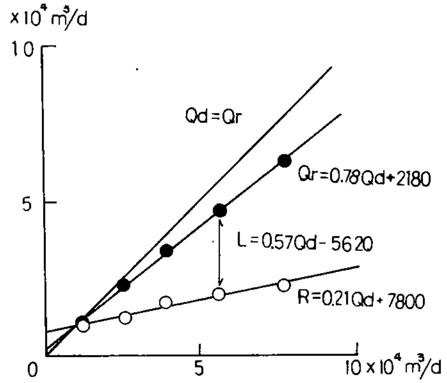

FIGURE 10. *Simple relationship among items of hydrologic balance in Hiratsuka basin*

Figure 11 shows an estimate of the distribution of leakage rate per unit area for the Santama basin, in the suburbs of Tokyo. The basin consists of a system of eight major water-producing aquifers which are the late Tertiary and Quaternary sands and gravels,

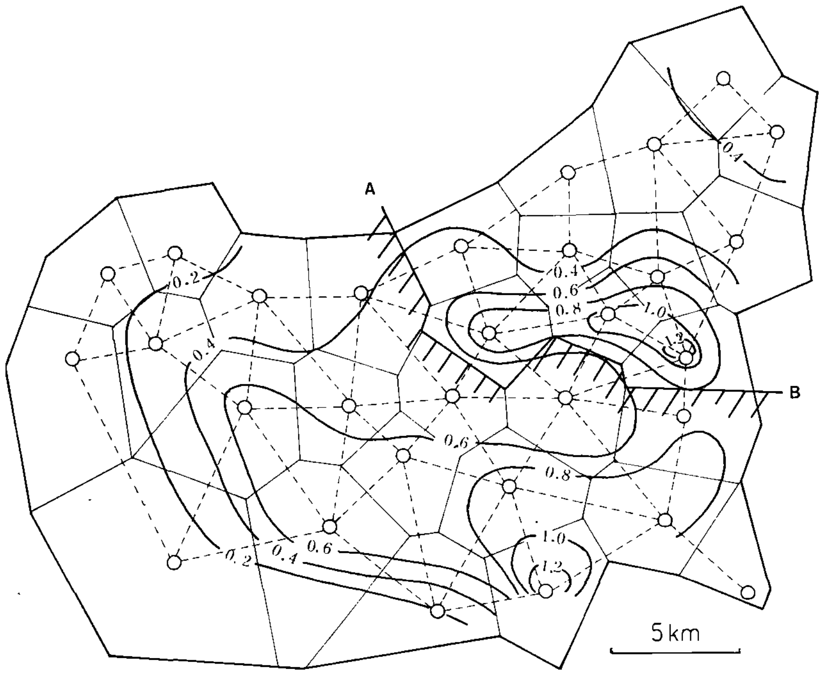

FIGURE 11. *Computed leakage rates per unit area, in m per yr, of Santama Basin, Tokyo, in 1967*

with the monoclinal structure toward the north-east direction (Shindo, 1968). The recharge zones is show as the west side of the A-B line, distributing a high rate of leakage. Influent stream seepage and unconfined ground water in the recharge zone usually may be the chief supplemental source of the confined ground water, because, the total of volumetric leakage flow rate in the recharge zone in 1967 as measured by stream and shallow-well gaging, was about 57×10^4 cu. m per day and the corresponding computed rate was 49×10^4 cu. m per day.

A VIEW ON THE HYDROLOGIC BALANCE FOR THE LAND SUBSIDENCE PHENOMENA

In an investigation in the land subsided area, the Shiroishi Plain, samples of confined ground water were taken for tritium analysis. The tritium content of samples ranged from 0.2 to 0.3 T.U., indicated apparently a poor recharge from the outcrop area to the basin. Those results were also obtained in other land subsided areas (Kimura, *et al.*, MS 1969).

On the steps of the results, it is satisfactory to consider that the main replenishment of water to the aquifer is the leakage through semi-confining strata or underlying semi-pervious beds. It does not contradict the results of leakage ratios, which have been computed in the preceding section.

Figure 12 shows the changes in the computed leakage rate, L, and the measured volumetric land subsidence rate Vs over the Shiroishi Plain. Here, over the three-year

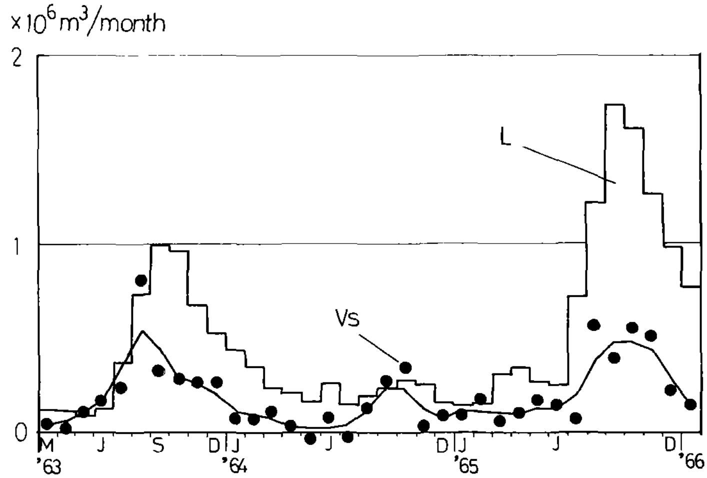

FIGURE 12. *Changes of leakage rate and volumetric land subsidence rate, both in cu. m per month, over Shiroishi basin*

period from 1963 to 1965, the simple relationship between L and Vs is shown in figure 13 and is given by

$$Vs = 0.27\,L + 0.25 \tag{17}$$

where each unit is in $\times 10^7$ cu. m per month.

The mechanism of the phenomena is as yet incompletely understood. However, it is believed that the relation between them will be useful in solving problems related to land subsidence phenomena.

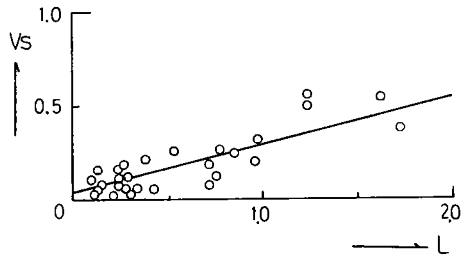

FIGURE 13. *Simple relationship between leakage rate and volumetric subsidence rate over Shiroishi basin*

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DISCUSSION

Intervention of Dr. Naomi MIYABE:

I think the recharge amount, Q_2, is dependent on the recharge pressure. But in your formula, which term represents the effect of recharge pressure?"

Answer of Mr. SHIBASAKI:

"Yes, Q_2 is dependent on the pressure or on the difference in level, however, it is not included in the terms of our formula. It was derived experimentally."

Question of Dr. MIYABE:

"Is your formula applicable to the recharge into deeper aquifers?"

Answer of Mr. SHIBASAKI:

"As a matter of fact, it will be difficult to apply the equation "2". It can be calculated by distinguishing the contents of Q_2 of our equation 1."

Intervention of Mr. Ben E. LOFGREN (USA):

"How dependent is your solution on the pattern of the nodes selected? If you took twice as many nodes, or half as many nodes, would you get the same results?".

Answer of Mr. SHIBASAKI:

"Do you mean "polygon nodes"?". We are using about 20 nodes. Or do you mean the samples contained in the nodes?".

Question of Dr. LOFGREN:

Suppose you use, instead of 30 nodes, 10 nodes to cover the same area, or you use 100 nodes for the same area. Would you come up with the same results?

Answer of Dr. SHIBASAKI:

I do not think the results would be the same. As we have to consider the gross error in solving this simultaneous equation, which becomes larger with the increase in the number of equations. This is a point that requires further consideration.

Intervention of Mr. Ben LOFGREN (U S A):

Comments:

I have two comments. First, on your third or fourth slide, you showed several unexplained loops in the relationship between rate of subsidence and rate of water-level decline. We plot these data in a little different way, but get the same general hysteresis loop. You will be interested in paper No. 45 by Mr. Riley scheduled for tomorrow.

The second comment relates to the time rate of subsidence. You attribute any delay of compaction to secondary consolidation. We are finding in our studies that a good deal of this delay is due to the slow drainage of water from the fine-grained beds of the aquifer system. Sometimes slow drainage continues for many months or even years after the water level is pulled down. Much of the time delay is related to the slow escape of pore water from the very slow draining beds and not to secondary consolidation. Frequently, delayed compaction is a significant part of the total compaction.

Answer of Dr. SAYAMA:

Thank you very much for your comments. I would very much like to hear the paper you mentioned. And as for your second comment, the time lag, due to slow drainage, would have, I am sure, much to do with it. These soil mechanics factors have, so far, not been given full consideration. These must be studied in the future.

SIMULATION OF GROUNDWATER BALANCE AS A BASIS OF CONSIDERING LAND SUBSIDENCE IN THE KOTO DELTA, TOKYO

Soki YAMAMOTO[1], Isamu KAYANE[1], Shigeru AOKI[2] and Seietsu FUJI

ABSTRACT

The major cause of land subsidence in the Koto Delta area, Tokyo is believed to be the heavy withdrawal of groundwater. The rate of subsidence is generally in accord with the rate of change in piezometric surface. The purposes of this paper are: 1) to make clear the secular changes in regional distribution of groundwater surfaces with the heavy withdrawal of groundwater, and 2) to test the applicability of the computer simulation technique for groundwater balance to this area. To simulate the secular trend of ground-

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water levels, a two-dimensional diffusion model of a groundwater basin is adopted. Extractiontion data and water level histories are collected for the delta area and coefficients of storage and transmissibility are determined by the method of least squares using a digital computer. Based on the obtained results, the behavior of the groundwater basin is discussed.

RÉSUMÉ

La cause principale de l'affaissement de terrain dans la région du delta Koto de Tokyo est supposée être le pompage intensif des eaux souterraines. Le taux d'affaissement s'accorde généralement avec le taux de modification de la surface piézométrique. Les buts de cet article sont: 1) clarifier le changement séculaire de la distribution régionale de la surface des eaux souterraines par le pompage intensif des eaux souterraines, 2) examiner l'applicabilité de la technique de simulation par le calculateur électronique pour la balance des eaux souterraines de cette région. Pour simuler la marche séculaire de niveau des eaux souterraines, le modèle de la diffusion à deux dimensions du bassin des eaux souterraines est adopté. Les données d'extraction et les variations de niveau des eaux sont rassemblées sur l'étendue du delta et les coefficients d'emmagasinement et de transmissibilité sont déterminés par la méthode des moindres carrés à l'aide du calculateur digital. Avec les résultats obtenus, l'allure des eaux souterraines du bassin est discutée.

INTRODUCTION

The major cause of land subsidence in the Tokyo lowland plain, known as the Koto Delta, is considered to be the heavy withdrawal of groundwater. Generally speaking, the secular trend of repeated leveling records coincides well with that of groundwater levels. The rate of land subsidence markedly decreased during the World War II, when the economic activity depressed, compared with those observed both before and after the war. The water level history at the University of Tokyo shown in figure 1 indicates the corresponding recovery of groundwater level during the war time. Analyses have been made of the relationship between the rate of subsidence and the groundwater level, Miyabe [1] bore one, for the area of our concern. However, few studies are made from the standpoint of groundwater balance. The purposes of this paper are to make clear the change of groundwater level in time and space due to pumping by using available data as much as possible, and to test the applicability of the computer simulation technique of groundwater balance to this area.

Geomorphologically, Tokyo is divided into two areas; one is the lowland of our concern and another is the upland areas. The surface parts of the former area consists mainly of Holocene sand and clay deposits, those of the latter comprising the Kanto loam formation. These deposits cover the Pleistocene and Pliocene formations which are widely distributed in the deeper part of this region. The strata consisting of subsurface geology of Tokyo are subdivided into several formations. Among these, the main system of the aquifer is considered to be the Pleistocene Edogawa formation of 100 to 400 meters in thickness, which lies over the Pliocene Kazusa Group with unconformity increasing its thickness towards the center of the Kanto Plain. The formation is composed of alternating layers of brackish sand, clay and gravel in the upper part and of marine sandy strata in the lower.

INPUT DATA

Three kinds of data are used to construct past groundwater levels: (1) Records of observation wells, whose locations are shown in figure 2. Representative records of them are reproduced in figure 1 [2, 3]. The depth of wells range from 380 meters at the University of Tokyo to 47 meters at Azuma-A; (2) Groundwater levels observed at the time of well

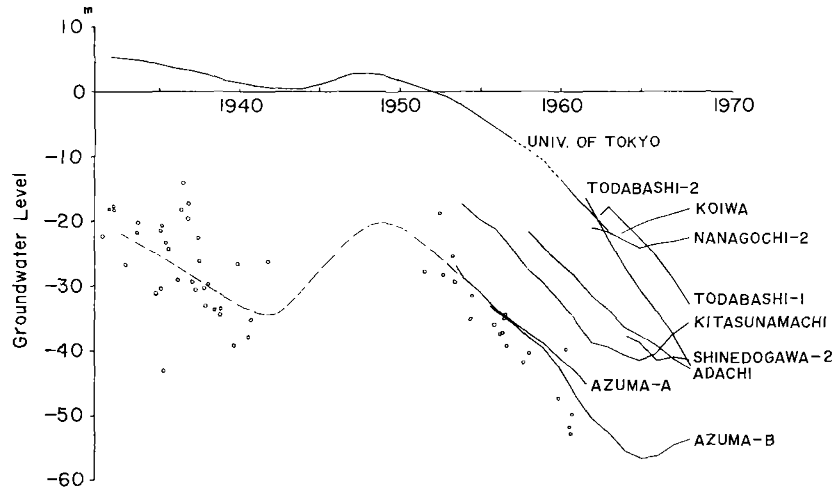

FIGURE 1. *Secular trend of groundwater levels. Small circles indicate groundwater levels at the time of bore drilling near AZUMA-A and B (node 4)*

drilling. These data are collected from the driller's field notes. Small circles in figure 1 represent a part of such data within node 4 (fig. 3) in which observation wells of Azuma-A (115 meters in depth) and Azuma-B are located. A dashed line is drawn by eye-fitting to show a presumed water level change at node 4; (3) Static groundwater level of wells for industrial or building use entered on "Groundwater Investigation Card". Investigations have been tried several times by organizations of Tokyo Metropolis. Figure 2 is one example, which is constructed by us from such data, indicating the distribution of groundwater level in and around the Koto Delta at the end of August in 1962.

By using the three kinds of data stated above, the groundwater level histories shown by a solid line in Figure 4 are constructed, one for each nodal area. The method of constructing these histories is described elsewhere in detail [4].

The amount of water pumped from a groundwater basin is of fundamental importance in studying the groundwater balance. Several data from different sources are available in this area but the accuracy of them are dubious in many respects unless the data are based on direct measurement by the trained investigator. Five source data used in our study are those obtained by the 'enquête' method and great discrepancies are found between them. Some adjustments were necessary to construct the past records of groundwater pumpage shown in table 1 from these source data, although we have no confidence in that the adjusted pumpage data has enough accuracy. Lateral groundwater inflows across the outer boundary are subtracted from the pumpage for nodes 2, 8, 9 and 11. The amount ranges from 3×10^3 to 7×10^3 cubic meters per day. Detailed explanation on the method employed for adjustment is described alsewhere [4]. Table 1 indicates that the pumpage from eleven nodal areas is about 280,000 cubic meters per day in 1952 with increasing trend up to the maximum of 610,000 cubic meters per day in 1963 followed by a slight decreasing trend resulted from a groundwater regulation in the southern half of the area concerned. Pumpage data of the same accuracy are not available for nodes 12 and 13, so these nodes are omitted from the present computation.

ASYMMETRIC POLYGONAL NETWORK

An asymmetric polygonal network adapted by us are shown in figure 3. It covers a area of about 300 square kilometers. It is desirable that a network covers an entire groundwater basin and the network is denser where greater hydraulic gradient exists. Judging from the hydrogeological conditions, a network consisting 100 nodes covering about

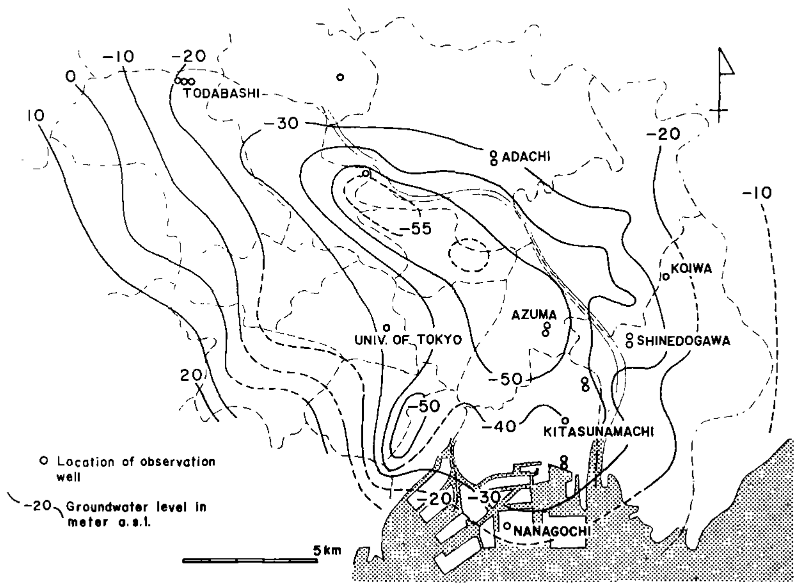

Figure 2. *Horizontal distribution of groundwater level at the end of August, 1962*

Table 1. Amount of pumped groundwater from each node ($A_B Q_B$)

Year \ Node Nos	1	2	3	4	5	6	7	8	9	10	11
1950	16	16	9	27	28	18	35	4	1	61	20
1951	16	16	9	27	28	18	35	5	2	61	20
1952	17	16	9	28	30	19	36	6	4	63	21
1953	20	17	10	33	37	21	37	6	5	69	27
1954	22	19	15	41	41	23	40	7	8	74	30
1955	27	21	19	48	46	26	42	8	9	80	35
1956	37	35	22	56	56	29	44	10	11	86	39
1957	47	42	25	58	63	32	48	11	12	91	40
1958	53	38	27	57	61	36	53	12	12	94	38
1959	58	33	30	59	58	40	56	13	13	100	34
1960	61	34	33	61	58	43	59	15	13	106	38
1961	62	37	34	64	61	43	60	15	15	112	46
1962	62	34	33	61	64	44	60	16	15	115	49
1963	60	34	33	62	68	45	60	16	16	120	57
1964	52	33	30	59	71	43	55	16	16	121	62

unit: 1000 m^3/dây

1000 square kilometers, including surrounding areas, may be more appropriate for the groundwater simulation in the Southern Kanto. But the limitation is imposed by the lack of available input data. Past pumpage data are available only by ward (*ku*) and only a few for the upland area. No observation well is installed in the upland area yet.

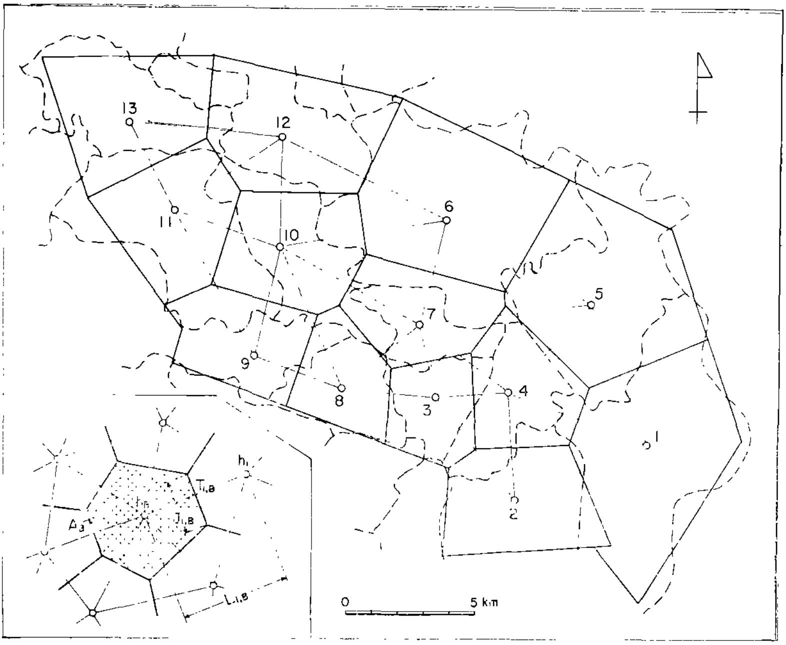

FIGURE 3. *Polygonal geometry and Tokyo Koto Delta asymmetric network. Figures in polygons indicate node number*

SIMULATION

Tyson and Weber [5] proposed a method of digital computer solution for a two-dimensional diffusion model of groundwater flow in an unconfined aquifer. Under approximate assumptions the dynamics of flow in the aquifer is described by the following equation:

$$\nabla T \nabla h - S\frac{\partial h}{\partial t} - Q = 0 \tag{1}$$

where h refers to the hydraulic head or groundwater level, T and S are, respectively, the transmissibility and storage coefficients, and Q indicates the time dependent flow rate per unit area. This flow is the sum of pumpage and replenishment flows. Equation (1) is solved on the general purpose digital computer by an implicit numerical integration technique. Using this technique, the equation is replaced by

$$\left.\begin{aligned} \sum_i (h_i^{j+1} - h_B^{j+1})\, Y_{i,B} &= \frac{A_B S_B}{\Delta t}(h_B^{j+1} - h_B^j) + A_B Q_B^{j+1} \\ Y_{i,B} &= \frac{J_{i,B} T_{i,B}}{L_{i,B}} \end{aligned}\right\} \tag{2}$$

in which the superscripts j denote points along the time coordinate, A_B express the area associated with node B and $Y_{i,B}$ is the conductance of path between nodes i and B as shown in figure 3. More detailed explanation for equation (1) and (2) is given in the original paper by Tyson and Weber.

Assuming that the equation (2) is valid for the groundwater balance in the Koto Delta, Tokyo, we take two steps for simulating groundwater level histories. In step 1, the storage coefficient S_B and the conductance $Y_{i,B}$ are calculated by the method of least squares based on the data of past water level and flow rates Q_B. In step 2, the coefficients obtained in step 1 are used to simulated water level histories. If the simulated water level histories in step 2 coincide well with those observed, the forecast of the future water level will become possible in the next step by giving planned data of groundwater extraction. If simulated levels do not coincide, the accuracy of the input data or the applicability of the method should be examined.

Explanation of steps 1 and 2 is as follows.

Step 1: The purpose is to calculate S_B and $Y_{i,B}$ from $K.N$ equations of (2) for pas K time steps (year) and N nodes. A equation (2) is valid for each node and each year.

1. The number of pairs of $Y_{i,B} = Y_{B,i}$ is counted and expressed by M.
2. Construction is made of an augmented matrix Z of ($K{\cdot}N$, $N+M+1$) in which the coefficients for S_B, i.e.
$$\left(-A_B\,\frac{h_B^{j+1}-h_B^j}{\Delta t}\right),$$
are put in $1\sim N$ colums; the coefficients for $Y_{i,B}$, i.e. $(h^{j+1}-h_j^{B+1})$, are put in $N+1\sim N+M$ th columns; a constant term, i.e. $(A_B Q_B^{j+1})$, is put in $N+M+1$ th column.
3. Product of $Z^T{\cdot}Z$ is calculated to give a square symmetric matrix of ($N+M+1$, $N+M+1$). Z^T refers to a transposed matrix of Z.
4. Assuming the $N+M+1$ th column of the square summetric matrix as a constant term and $1\sim N+M$ th columns as coefficients of simultaneous equations, these linear equations are simultaneously solved to give S_B and $Y_{i,B}$.

Step 2: The purpose is to calculate the groundwater level by using a given set of coefficients such as those calculated in step 1. By rearranging the equation (2), the following one is obtained.

$$\sum_i h_i^{j+1}\,Y_{i,B}-h_B^{j+1}\left(\sum_i Y_{i,B}+\frac{A_B S_B}{\Delta t}\right)=A_B Q_B^{j+1}-\frac{A_B S_B}{\Delta t}h_B^j \tag{3}$$

As the equation (3) is valid for $B = 1, 2, \ldots N$, the following matrix operation is formed. Given the groundwater levels of one time step before, h_j^B, the present levels h_j^{B+1} are obtained by the simultaneous solution of equation (4). The inverse method is used for the simultaneous solution in steps 1 and 2.

Input data for the computational scheme stated above are water level histories and pumpage data described in the preceeding section. In order to test a computer program written for steps 1 and 2, arbitrarily selected sets of S_B and $Y_{i,B}$ shown in table 2 under the heading of "Test-1" are, at first, given in step 2 together with pumpage data, and groundwater levels are arbitrarily constructed as an output of step 2. Then, these arbitrarily constructed groundwater levels shown by broken line in figure 4 are given in step 1 as the input data together with pumpage data, and sets of S_B and $Y_{i,B}$ are calculated by the method of least squares. As a result, the same sets of S_B and $Y_{i,B}$ given in step 2 are obtained as an output of step 1 and the computer program is proved to be effective so long as the input

$$\begin{bmatrix} -\left(\sum Y_{i1} + \dfrac{A_1 S_1}{\Delta t}\right) & Y_{21} & Y_{31} & \cdots & Y_{N1} \\ Y_{12} & -\left(\sum\limits_i Y_{i2} + \dfrac{A_2 S_2}{\Delta t}\right) & Y_{32} & \cdots & Y_{N2} \\ Y_{13} & Y_{23} & -\left(\sum\limits_i Y_{i3} + \dfrac{A_3 S_3}{\Delta t}\right) & \cdots & Y_{N3} \\ \vdots & \vdots & \vdots & & \vdots \\ Y_{1N} & Y_{2N} & Y_{3N} & \cdots & -\left(\sum\limits_i Y_{iN} + \dfrac{A_N S_N}{\Delta t}\right) \end{bmatrix} \begin{bmatrix} h_1^{j+1} \\ h_2^{j+1} \\ h_3^{j+1} \\ \vdots \\ h_N^{j+1} \end{bmatrix} = \begin{bmatrix} A_1 Q_1^{j+1} - \dfrac{A_1 S_1}{\Delta t} h_1^j \\ A_2 Q_2^{j+1} - \dfrac{A_2 S_2}{\Delta t} h_2^j \\ A_3 Q_3^{j+1} - \dfrac{A_3 S_3}{\Delta t} h_3^j \\ \vdots \\ A_N Q_N^{j+1} - \dfrac{A_N S_N}{\Delta t} h_N^j \end{bmatrix} \qquad (4)$$

data are accurate enough and the groundwater behaves in the manner as expressed by equation (1).

Having tested the computer program, the data of actual groundwater level shown by solid lines in figure 4 and the pumpage data in table 1 are given in step 1. The calculated set of S_B and $Y_{i,B}$ are shown in table 2 under the heading of 'L.S.', for least squares. Groundwater levels are simulated, next, in step 2 by using these coefficients and the results are shown in figure 4 by a series of small circles. Computation is made for eleven nodes ($N = 11$) and fifteen years from 1950 to 1964 ($K = 15$).

As can be seen from the figure 4, the results are not satisfactory. The deficiency is that the negative sign appeared on eight sets of transmissibility coefficient out of twenty one. This makes the results physically meaningless and negates the value of the simulated groundwater levels. Two main causes are considered for this deficiency. One is inaccuracy of pumpage data and the other is the instability of the solution affected by a slight error involved in the input data.

After comparing the simulated groundwater levels from the coefficients of 'Test-1' and 'L.S.', a value of about 0.2 seems to be appropriate for S_B. The groundwater levels

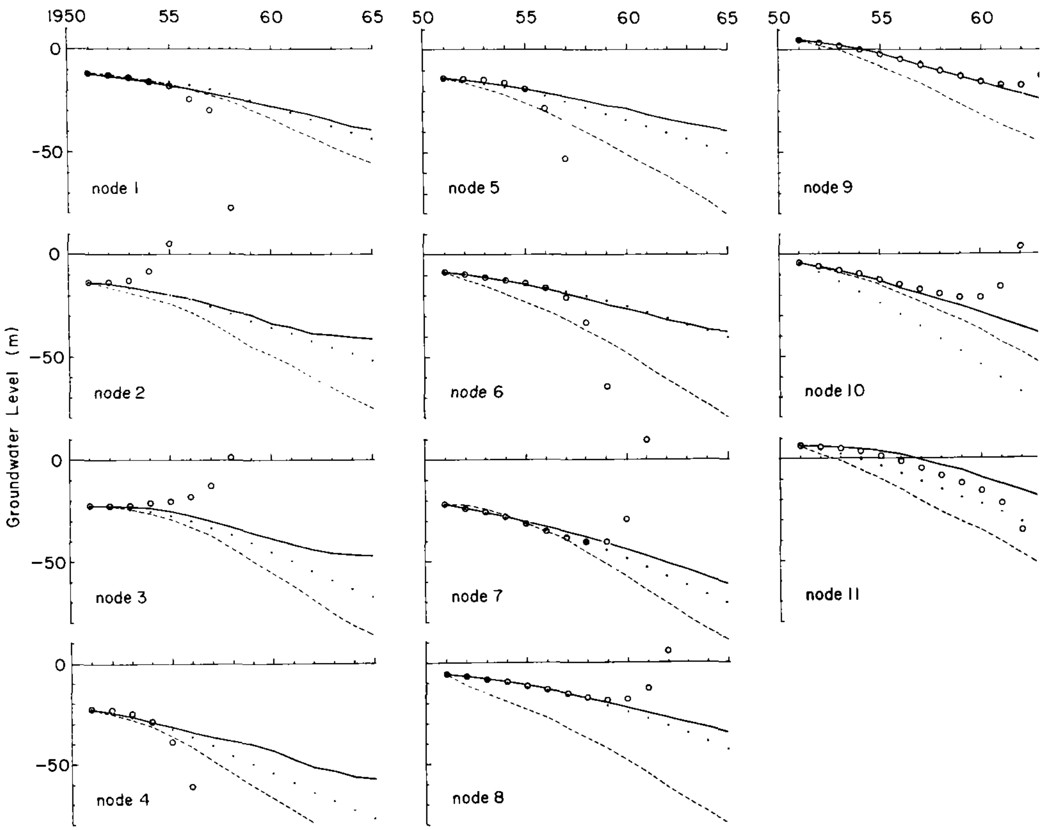

FIGURE 4. *Observed and simulated groundwater levels*
Solid line: *Observed groundwater levels*
Broken line: *Results of Test-1 in table 2*
Circles: *Results of L.S. in table 2*
Dots: *Results of Test-2 in table 2*

TABLE 2. Values of S_B and $T_{i,B}$ used for simulating groundwater levels (see fig. 4)

Storage Coefficient

S_B	Test-1	L.S.	Test-2
1	0.201	0.237	0.200
2	0.100	0.115	0.200
3	0.154	0.169	0.200
4	0.103	0.011	0.200
5	0.114	0.143	0.200
6	0.065	0.084	0.200
7	0.074	0.873	0.200
8	0.066	0.818	0.200
9	0.117	0.152	0.200
10	0.412	0.520	0.200
11	0.124	0.121	0.200

Test-1: arbitrarily given sets of S_B and $T_{i,B}$.
L.S. $T_{i,B}$ and S_B obtained by the method of least squares.

Coefficient of Transmissibility (m²/*day*)

$T_{i,B} = T_{i,B}$	Test-1	L.S.	Test-2
1– 2	808	–1910	300
1– 4	1240	1341	300
1– 5	330	2960	300
2– 3	605	1112	300
2– 4	413	–2194	300
3– 4	245	196	300
3– 7	998	–1203	300
3– 8	362	402	300
4– 5	417	4761	300
4– 7	455	4421	300
5– 6	741	1027	300
5– 7	6855	–76693	300
6– 7	554	–710	300
6–10	2136	–2156	300
7– 8	1133	2644	300
7–10	1930	–2566	300
8– 9	178	720	300
8–10	3667	6784	300
9–10	1061	289	300
9–11	2635	1872	300
10–11	1179	–1410	300

shown by a series of dots in figure 4 are those calculated under the assumption that all nodes have $S = 0.2$ and $T = 300$ (m²/day) in order to see regional differences in the response of groundwater levels.

DISCUSSION

A flow rate of Q in the equations used are the sum of extraction and replenishment flows. Although the amount of lateral inflow into some of the nodal areas are adjusted as stated before, a vertical recharge from the surface is assumed to be negligible. This is based on the fact that the tritium concentrations in groundwater sampled from a 65-meter-depth well located in node 8 in 1962 and 1963 are 1.0 and 1.1 in Tritium Unit, respectively [6] Another tritium analysis also revealed that the groundwater in this area seems to have recharged at least before 1954 [7].

As shown in figure 4, simulated water levels coincide fairly well with those observed for marginal nodes 1, 2, 6, 8 and 9, but not for central nodes 3, 4 and 7. This may be corrected either by increasing the storage coefficient of the latter nodes or by increasing the transmissibility coefficient of all areas. The storage coefficient of 0.2 is considered equivalent to the specific yield of fine sand, or sand and gravel, so that the adjustment on transmissibility is more realistic than that on storage coefficient. Greatest deviation from the observed water level are found for nodes 10 and 11. Since the basin is not closed at the boundaries of nodes between 10-12 and 11-13, and no lateral inflow is taken into account for node 10, these descrepancies may be interpreted as a result of neglected lateral inflow. As a whole, it seems important to land subsidence problems that most of the pumped groundwater in this area has been supplied not from the recharged groundwater but from that stored in the aquifer.

The following suggestions may be to the future simulation work of the groundwater balance: (1) The necessary procedure should be taken for obtaining accurate data on groundwater level and extraction, especially for the latter; (2) Given the accurate data, a trial and error method on the digital computer will be used to find hydraulic coefficients and (3) Hydrogeological information, either on storage coefficient or transmissibility, will make a trial and error procedure much easier.

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WATER BALANCE INVESTIGATION BASED UPON MEASUREMENTS OF LAND SUBSIDENCE CAUSED BY GROUND WATER WITHDRAWAL

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Abstract

The total water demand of the town Debrecen is supplied from wells, tapping a 20-30 m thick coarse sand aquifer of the lower Quaternary at 100-200 m depth. The aquifer is overlain by a fluvial series topped by loess and sand. The first wells were drilled some 70 years ago. The yearly water withdrawal now exceeds 12 million cu. m. The piezometric level of this aquifer in the vicinity of the wells has subsided 15-20 m since 1913.

It can be assumed that owing to the water withdrawal of greater quantity—respective to the depression produced—the alluvial layer will consolidate and the land subside. For determining the amount of this land subsidence the former town survey data could be used. By comparing with more recent measurements a consolidation data series extending over about 40 years was obtained. The maximum subsidence of land amounts over 40 mm since 1927.

The water balance of the aquifer can be characterized by parameters of the compaction and of the dynamic resource determined by land subsidence data. According to present investigation the maximum degree of compaction (which is equal to the pore volume variation produced by possible consolidation) can be estimated at 4-5 per cent of the pore volume of the layer.

RÉSUMÉ

L'ensemble de la demande en eau de la ville de Debrecen est couvert par des puits prenant leur eau dans une couche de 20 à 30 m de sable grossier du Quaternaire inférieur à une profondeur de 100 à 200 m. L'aquifère en question est recouvert de dépôts fluviaux couronnés par du loess et du sable. Les premiers puits furent forés il y a quelque 70 ans. Le pompage annuel dépasse 12 millions de m³. Depuis 1913, le niveau piézométrique au voisinage des puits a baissé de 15 à 20 m.

Il faut bien admettre que le pompage de plus grandes quantités d'eau et sous la dépression ainsi produite, le sol alluvial va se consolider et provoquer des affaissements. Pour déterminer le montant de cet affaissement, les données de la dernière enquête de la ville peuvent être utilisées. En les complétant par de plus récentes mesures encore, une série de chiffres de consolidation s'étendant sur près de 40 ans a pu être établie. L'affaissement maximum depuis 1927 s'élève à 40 mm.

Le bilan d'eau de l'aquifère peut être caractérisé par des paramètres de la compaction et par les ressources dynamiques déterminées par les données d'affaissements. D'après les recherches présentes, le degré maximum de compaction (qui est égal à la variation du volume des pores produite par la consolidation possible) peut être évalué à 4-5 % du volume des pores de la couche.

1. THEORETICAL FUNDAMENTALS OF WATER BALANCE INVESTIGATION

The water resources of porous, sedimentary strata are usually composed of two parts. The part stored in the voids is called the statical resource, whilst the part arriving from elsewhere to the point of use, that is, the part supplied by seepage, is called dynamic resource.

When tapping an aquifer, together with the reduction of neutral stresses there is an increase in the effective stress, resulting eventually in consolidation. When extending the drawdown caused by water production to the top stratum, this can result in a maximum consolidation of the aquifer, or, with the propagation of reduced stresses, of the whole profile, determined by local geological conditions. Obviously, consolidation can only occur if the volume of water corresponding to the change in porosity is withdrawn from the aquifer. Denoting the porosity of the fully consolidated stratum by n_0 and the one pertaining to natural conditions (preceding the production of water) by n_n, then the value

$$n_n - n_0 = n_k \tag{1.a}$$

will be a measure of *specific water resources* related to the *consolidation* of the aquifer. The volume of water pertaining to n_0 (or at least its part not bound by molecular forces) can also be exploited, without however causing any structural change in the aquifer. It seems thus reasonable to divide static resources into two parts characterized by the values n_0 and n_k. Due to compressive properties of the soils, the reversibility of this process is (under practical conditions) unlikely. Accordingly, the discontinuation of withdrawal followed by the restoration of the groundwater table and neutral stress to their original value, cannot be expected to result in an essential change of porosity. The exploitation of *consolidation water resources* is thus an irretrievable consumption of resources, contrary to the one pertaining to n_0, that can be replaced by a repeated replenishment of the voids.

In the range of stresses common in cold-water aquifers, the soil compression curve can be replaced by a straight line and thus the expected change in porosity, due to the increment Δs in drawdown, can be expressed as follows:

$$\Delta n_k = m \cdot \gamma \cdot \Delta s,$$

with Δs being the decrease in the piezometric level, γ the specific weight of water and m the compression factor of the layer. If one assumes the average value of a short-time change in the drawtown to be proportional over the whole drawdown area F to a value

Δs_0 observed at a representative point, and on the other hand it is allowed to take m, γ, and the aquifer thickness h into account by their mean values, then the change in the consolidation resources (ΔV_k) will be

$$\Delta V_k = \alpha \cdot m \cdot \gamma \cdot F \cdot h \cdot \Delta s_0 ,$$

where

$$\alpha = \frac{\int_F \Delta s \, \mathrm{d}F}{\Delta s_0 F}$$

to be interpreted in accordance with Figure 1.

By supposing the order of magnitude of the drawdown area F to be constant, and introducing thus the notation $\alpha.m.\gamma.F.h = B$, the consumptive use during unit time of the consolidation resources can be characterized by the following relationship:

$$Q_k = B \frac{\mathrm{d}s_c}{dt} \tag{1}$$

where Q_k is the part of the exploited discharge originating in the *irretrievable consolidation resources*, B is a factor assumed to be constant under the conditions described above, and which is equal to the consolidation resources pertaining to a drawdown change in unit time.

Water inflow to the aquifer, induced by drawdown, may occur either as infiltration across the perimeter, or from neigh bouring aquifers. In both cases the validity of the Darcy law can be assumed for both the horizontal and the vertical seepage flow. If so, it is logical to assume that the *inflow discharge* is a linear function of drawdown, that is,

$$O_P = A \cdot s_0 \tag{2}$$

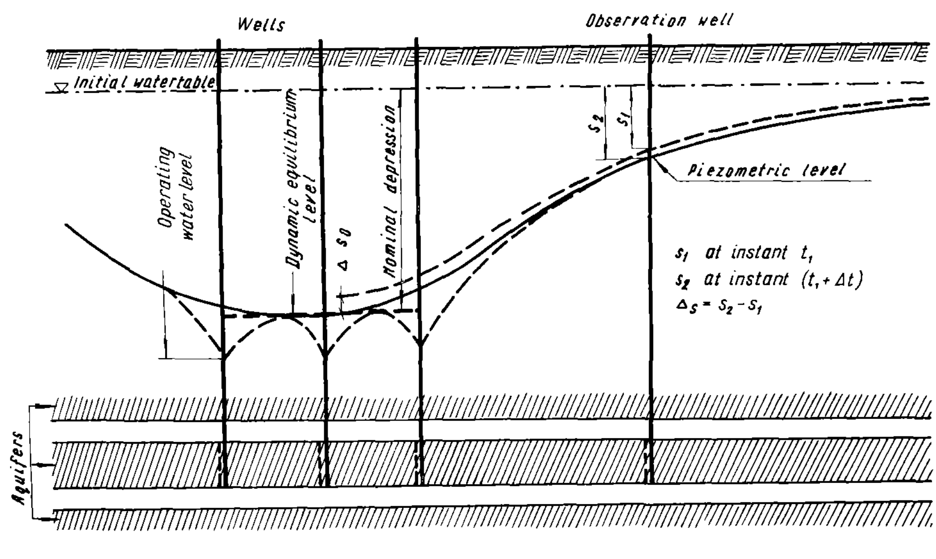

FIGURE 1. *Definition of the water levels*

where:

Q_P the part of the exploited discharge coming from the dynamic resources;
A the product of factors to be considered constant for given hydrogeological and withdrawal conditions, and
s_0 the drawdown measured at a representative point.

In Quaternary formations and generally, in such of alluvial origin, the underlying and the overlying bed can only be considered impervious for values of the hydraulic gradient lower than a given value. In such cases, the hydraulic relations between the prospected aquifer and the adjacent ones, developing through incidental sand pockets, is determined by the cone of depression and the amount of drawdown.

In the area around the well or group of wells, where there are hydraulic gradients higher than the limit, due to the drawdown, general seepage is started across the sublayer and the cover from the neighbouring aquifers. At such times, there is a close relationship between depression surface and amount of drawdown and thus, the inflow discharge can be defined in principle by a single independent variable, the drawdown appearing at the representative point.

In accordance with the propagation of the pressure drop, all aquifers involved become parts of an interconnected hydraulic system, and a consolidation proportional to the amount of water removed may occur in them. Thus, an apparently dynamic yield from the pumped aquifer may be drawn actually from the static resources of neighbouring strata. By considering the possible character and the influencing factors it can be assumed that the relationships (1) and (2) are acceptable also in the case of complex systems, and that they can be applied to groups of wells. In these cases however, the values of the factors A and B are also influenced by the position of the wells.

* * *

Neglecting among the possible components of water resources the change of the elastic expansion and of the vapour phase of water (factors not giving rise to the formation of substantial resources at depths of 150 to 250 metres), the equation of the full yield withdrawn, and at the same time of the *water balance of the aquifer*, can be written as $Q_T = Q_p + Q_k$, or, by substituting (1) and (2):

$$Q_T = As_0(t) + B\frac{ds_0}{dt} \tag{3}$$

with Q_T being the available resource of the well or group of wells. The value of the drawdown s should be related to the original piezometric level of the aquifer, before pumping. The stationary level which depends on hydrogeological conditions and natural factors, may be constant or variable. Due to the irreversibility of resources consumption, when determining the consolidation resources, the deepest position of the stationary level below the terrain, should be taken into account. In the case of well groups, the characterization of drawdown s by a single value can be achieved by applying the method of the "fictitious equivalent well". The average of piezometric levels of wells temporarily out of action within the group of wells, is assumed directly proportionate to the central drawdown, characterizing the inflow to the "fictitious well". Accordingly, the position of the piezometric level, i.e., the level of dynamic equilibrium within the well group area, can be defined by the distance s_0 measured from the original stationary level, and called nominal drawdown (Fig. 1).

2. DETERMINATION OF CONSUMPTION FROM CONSOLIDATION RESOURCES

From the record of withdrawal and drawdown, the water balance equation can be solved and applied, if the factors A and B, assumed to be constant, have been previously determined. It can be seen immediately that, under the conditions already described, the factor B of the change of consolidation resources depends in principle solely on the soil physical properties of the aquifer. However, the determination of such data by core-samples, is very expensive and questionable, even if the area influenced by the waterworks is small. Thus, another approach should be adopted. In the wake of the consolidation of the aquifer, the ground must subside over the whole area affected by withdrawal. If this is perceptible, then the deformation of the terrain due to withdrawal can be followed by levelling. Obviously, there can be a number of other reasons for the movement of the ground and the bench marks and these should be discovered and evaluated when one endeavours to determine the subsidence coming from consolidation.

In mining, and especially in the excavation of coal beds, the extent of surface subsidence depends under otherwise identical conditions of geology and mining upon the position and extension of the terrain. By considering pertinent experiences, it is reasonable to assume that if water is exploited from a porous stratum lying 100 to 200 m deep, and the ground subsidence extends over several sq. km, then no arching or loosening can occur. Thus, obviously the amount of ground subsidence is equal to the consolidation of the strata. Consequently, the volume of the subsidence trough is equal to the volume of consumption from consolidation resources from the hydraulically interconnected formations. Accordingly, the equation

$$\int_T Q_k \,\mathrm{d}t = B \int_T \left(\frac{\mathrm{d}s_0}{\mathrm{d}t}\right) \mathrm{d}t = V_k \tag{4}$$

can be written, expressing the fact that the consolidation water resources withdrawn during the interval T are equal to the volume V_k of ground subsidence.

From this relationship the constant B is

$$B = \frac{V_k}{\int_T \mathrm{d}s_0} \tag{5}$$

whilst the constant A is

$$A = \frac{\left(\int_T Q_T \,\mathrm{d}t\right) - V_k}{\int_T s_0 \,\mathrm{d}t} = \frac{V_T - V_k}{\int_T s_0 \,\mathrm{d}t} \tag{6}$$

By this method, the parametric characterization of the water balance of a sedimentary aquifer defined by water exploitation can be boiled down to a simple problem of surveying.

3. A PRACTICAL APPLICATION OF THE GROUND-SUBSIDENCE METHOD

Due to demands for the determination of available water resources and to relatively favourable conditions, the water balance investigation described above has been carried out first in the area of Debrecen, in the years 1964-1966. The full water demand of

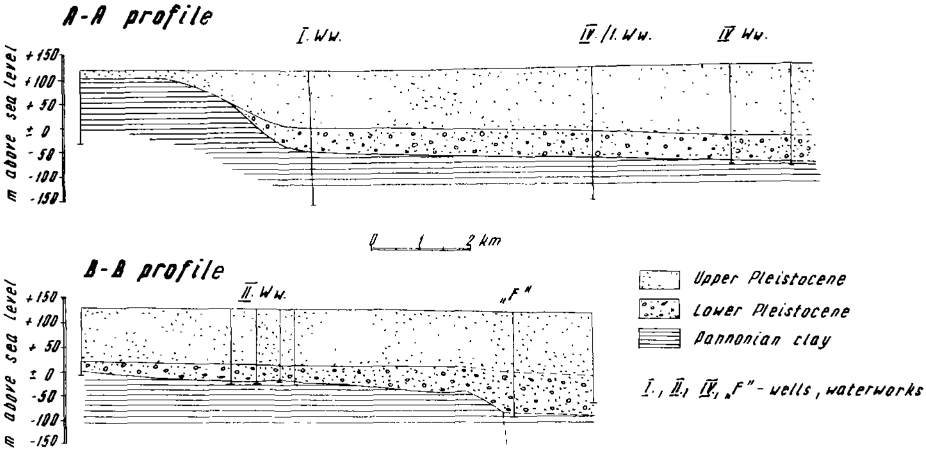

FIGURE 2. *Geological profile in Debrecen*

Debrecen, a city of 140,000 inhabitants, is covered by artesian water. The geological profiles are shown diagrammatically in Figure 2.

In the Pleistocene alluvial deposits reaching down to about 200 metres, there are several good aquifers, the most important one, from the view of water supply, being between 110 and 170 metres.

The municipal waterworks were commissioned in 1913. Actually, there are three plants of a daily output of 12-14 thousand cu. metres each, feeding a common distribution network. The "waterworks"-aquifer 110-170 m deep is tapped by about 100 wells, 75 of which belonging to the municipal waterworks and another 25 maintained by various institutions and companies. Whilst the first ones are grouped in the three waterworks, the latter are scattered all over the city area.

60 to 70 years ago, the nonpumping piezometric surface of the "waterworks" aquifer was near the surface, practically over the whole city area, its sudden lowering has been observed only since 1952, as shown by curve s_0 in Figure 3.

3.1 Determination of ground subsidence

In order to determine ground subsidence and the value V_k in equation (4), data of former municipal surveys have been tracked down and evaluated. For the sake of this particular investigation, new levelling has been carried out in 1965-1966.

From among the surveys performed at different dates and in different systems, the ones made in 1927-1937, 1954-1959 and 1965-1966 have been arranged successfully for the purpose of these studies. Subsidences have been determined from measured level differences of the bench-marks by using a point of reference assumed (and proved) to be immobile. Data were processed also for determining: to what extent subsidence has been influenced by factors unrelated to withdrawal. Omitting details, it was found that generally such effects are negligible.

By the extra- and interpolation of observed data, subsidence values were determined for three periods. In Figure 4, the lines of equal ground subsidence for the periods 1927-1937, 1927-1955 and 1927-1966 are shown.

The accuracy of measurements used for the construction of these lines is generally satisfying the requirement of precise levelling (the relative error being of the magnitude of 0.4 to 0.5 mm). When plotting the isobase lines, difficulties arose chiefly in the outskirts of the city, where the points were rather scarce. Here, uncertainties were compensated by

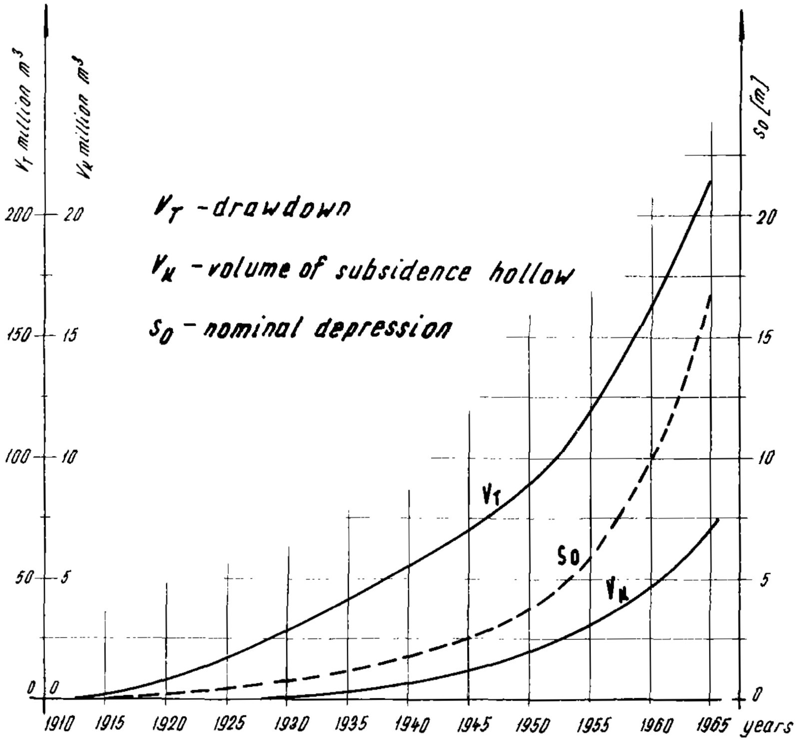

FIGURE 3. *Parameters of water balance equation*

a careful weighing of factors causing subsidence. Nevertheless, it should be pointed out that especially the lines indicating a low value of subsidence are informative only. But since the volume of the subsidence trough is hardly influenced by probable errors occuring with the isobase lines of 0 and 5 mm, the figures are considered suitable for further investigations.

Subsidence troughs are made rather irregular by the indentation of the eastern side and by the widening-out in the southeastern and southwestern direction. A gradual eastward shift of lines of equal subsidence can unequivocally be explained by the starting of new water producing wells (Waterworks No. II were put into operation in 1952).

3.2 EVALUATION OF WATER BALANCE

Volume changes of the subsidence trough are shown in Figure 3, whilst the changes of the factors A and B in function of the nominal depression s_0 are illustrated by Figure 5. In case of relatively low drawdown values ($s_0 < 4$ m), that is, practically up to 1952, these factors were gradually decreasing; thus, as far as the first period is concerned, assumptions referring to the constancy of the factors A and B have not been verified.

On the other hand, values of A showed little variation between 1952 and 1965; when compared with the mean value of 0,9 referring to 14 years, the deviation is less than $\pm 10\%$ The value of B is during this period practically constant and the deviation from the mean of 0.36 remains within $\pm 5\%$. When compared with earlier periods, the one of the last 14 years was characterized by a substantial increase and concentration (to the surroundings of the well groups) of withdrawal, and consequently, by a rapid decline of dynamic

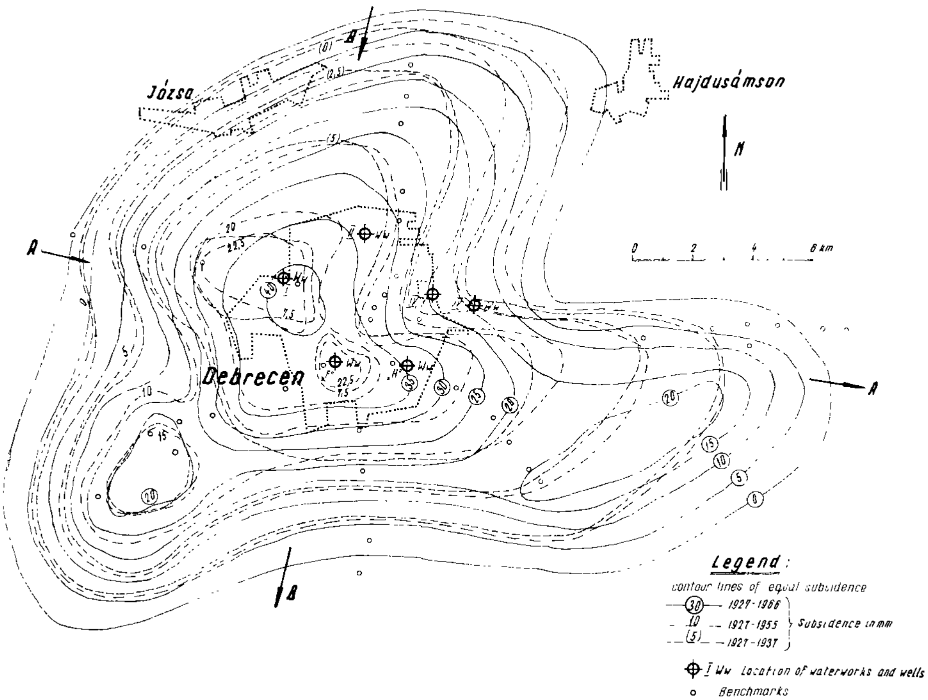

FIGURE 4. *The relative subsidence of landsurface in Debrecen*

equilibrium level. It seems reasonable to assume that results referring to the period 1927-1952 are materially affected by the wide spacing of withdrawal points, which is confirmed also by the double centres of ground-subsidence maps. Attention should also be paid to the fact that basic data originating in times prior to 1952 are less reliable and less in numbers than those referring to the last decade. In addition to these effects, one can be sure of the prevalence of other factors as well. It is mostly likely that changes in the arrangement of well groups, the putting into operation of new points of exploration (like Waterworks No. II), the variation of aquifer thickness, the propagation of drawdown area (through the inhomogeneity of the top layer) also influence the value of the constants.

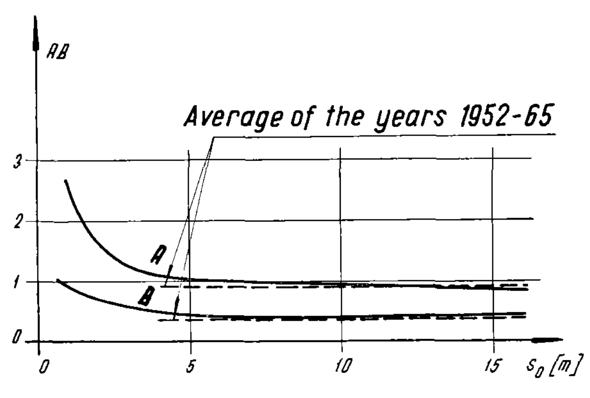

FIGURE 5. *Value of parameters A-B in terms of* s_0

It can be assumed that the conditions corresponding to the hydraulic assumptions made earlier have been developed around 1952, and presently, the relationships established are suitable for characterizing the order of magnitude of the water balance. When solving the differential equation (3) under assumption of a constant rate of yield Q_T, one obtains the relationship

$$s_0 = \frac{Q_T}{A}\left[1-\exp\left(-\frac{A}{B}t\right)\right] \tag{7}$$

indicating that only a discharge equal to the external supply Q_p can be withdrawn lastingly, if the drawdown is kept constant.

According to these investigations, actual withdrawal is in excess of the aquifer's external water supply corresponding to given conditions. Hence, the static level is dropping indicating the using-up of consolidation resources.

The knowledge of the factors A and B – 0,9 × 10^6sq.m/year and 0,36 × 10^6sq.m respectively,—is rendering a useful help in planning engineering installations and an optimum operation of withdrawal.

GEOLOGICAL AND GEOHYDROLOGICAL PROPERTIES OF THE LAND SUBSIDED AREAS

Case of the Niigata Low Land

S. TAKEUCHI, S. KIMOTO, M. WADA, H. SHIINA and K. MUKAI

ABSTRACT

The authors tried to trace the origin of the land subsidence which had struck low-land area in Niigata Prefecture. In order to form an estimate of downward movement of the land and also to plan countermeasures to cope with it, they thought it essential to delve into geological and geohydrological properties of the subsided area, and carried out the following:

— Micropaleontological analysis;
— Absolute chronology by the radio carbon method;
— Tritium dating method;
— Penetration test;
— Exploratory boring.

The results showed that the land subsidence had a close relationship with the chronology and depositional environment of alluvium and that the underground water was rather old.

On the basis of those findings, they have made an approach to the land subsidence in terms of hydraulic balance of the underground water.

RÉSUMÉ

Les auteurs essayent de déterminer l'origine de l'affaissement du sol qui a affecté les pays bas de la préfecture de Niigata. Dans le but de se former une idée sur la valeur de l'affaissement du pays et aussi pour présenter les contre-mesures pour s'y opposer, ils ont cru essentiel d'approfondir la connaissance de la géologie et de l'hydrogéologie de la région affaissée et de réaliser :

1. The Sinano River Basin Agricultural Water Resources Development Survey Office.
2. Agricultural Land Bureau, Ministry of Agriculture and Forestry.

— une analyse micropaléontologique ;
— une chronologie absolue à l'aide du carbone radioactif ;
— un datage tritium ;
— un essais de pénétrate ;
— un forage exploratoire.

Les résultats ont montré que l'affaissement du sol est en relation étroite avec la chronologie et les conditions de dépôt de l'alluvium et que l'eau souterraine est assez ancienne. Sur la base de ces données, les auteurs ont essayé d'exprimer l'affaissement en fonction du bilan hydraulique de l'eau souterraine.

1. INTRODUCTION

Being located in the central part of Japan proper and facing the Japan Sea, the so-called Niigata Lowland is an alluvial plain stretching along the Shinano, the longest river in this country (fig. 1). The subsidence area is situated in the northern part of the middle of this lowland, and extends over about 60,000 hectares. The subsidence, having been accelerating since around 1956, wrought a lot of damage on urban communities, harbors, and facilities for drainage and irrigation (fig. 2).

The subsidence area can be divided into two parts, the seaside and the inland (fig. 3). The subsidence of the inland section was investigated by the authors from the viewpoint of agricultural land conservation.

The subsidence phenomenon was observed by means of leveling and subsidence recorders, and exploratory drilling was made for clarifying the mechanism of the phenomenon. The locations of the subsidence recorders and exploratory boreholes are given in figure 3.

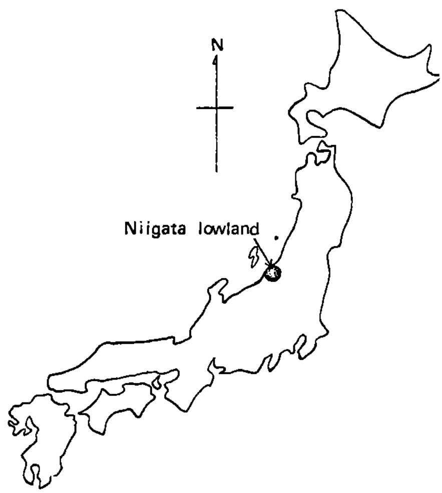

FIGURE 1. *Location of Niigata lowland*

2. THE FEATURES OF LAND SUBSIDENCE IN NIIGATA LOWLAND

Leveling was conducted on September 1st every year from 1959 to 1968. The results are shown by contour lines in figure 3. The maximum value of cumulative subsidence in the seaside area amounts to 1,944 mm at Terao, and that of the inland area amounts to 963 mm at Ajikata. The volume of yearly subsidence is shown in figure 4.

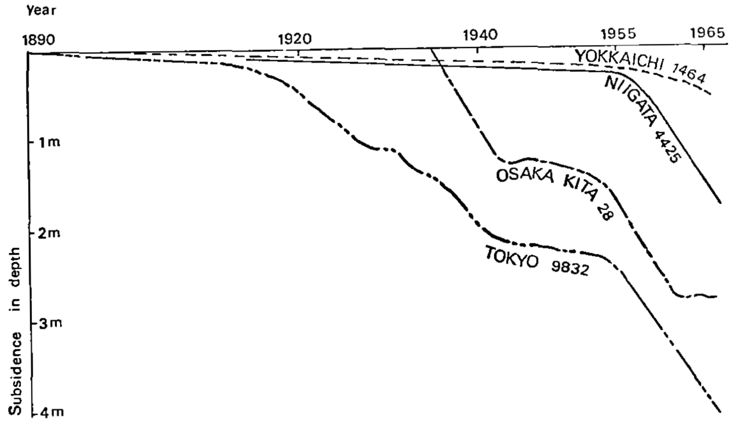

FIGURE 2. *Cumulative subsidence of bench marks in several subsided areas of Japan*

From 1961 to 1963, three successive regulations were imposed on the pumping of natural gas for industrial use. This contributed to the deceleration of the subsidence, except when the big earthquake struck Niigata City in the early summer of 1963.

Listed in figure 5 are the findings of those subsidence recorders which were set up at the Shirone Observatory in the central part of the inland as well as at the Niigo Observatory in the seaside.

The subsidence recorders, set up for the observation of vertical distribution of subsidence, reflected the land sinking movement in a decrement curve except for a sudden drop, possibly caused by the Niigata earthquake.

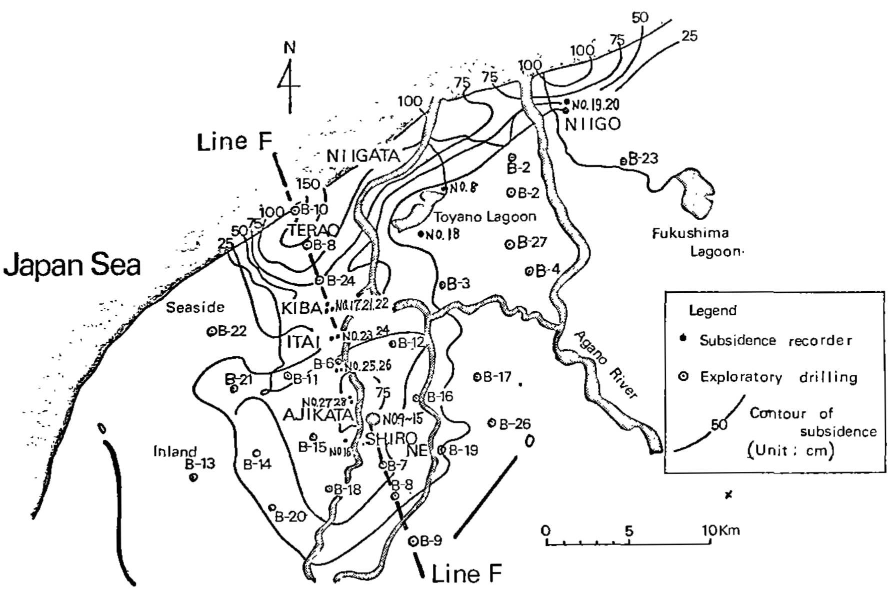

FIGURE 3. *Contours of subsidence from 1959 to 1968 and location of subsidence recorders and exploratory drillings*

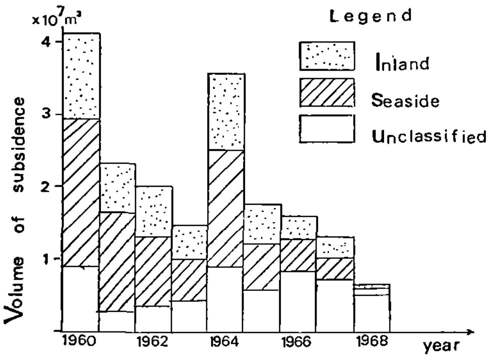

FIGURE 4. *Volume of yearly subsidence*

The comparison of the results of leveling and the record of subsidence recorders reveals that the subsidence in the seaside area resulted chiefly from the contraction of strata more than 150 meters deep and that in the inland area primarily from the compaction of layers less than 150 meters below the surface. As shown in figure 6, the ratio of compaction below and above the 150 meter depth tends to change from the seaside to the inland.

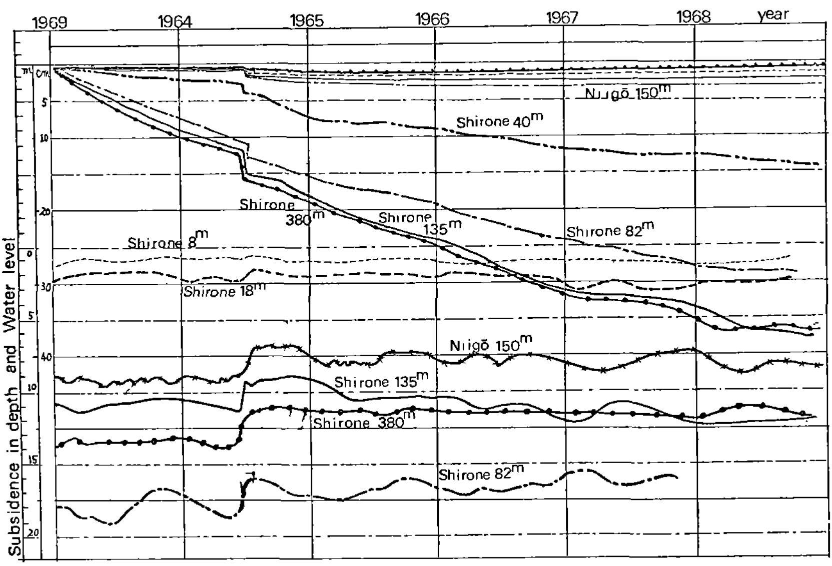

FIGURE 5. *Cumulative subsidence records at different depths and the water level in corresponding layers*

3. THE LATE QUATERNARY GEOLOGY OF NIIGATA SUBSIDENCE AREA

In order to clarify the mechanism of the land subsidence, it seems to be indispensable to know the geological conditions of the subsidence area and its environs, as the ground sinking probably was caused by the compaction of the underlying strata. As far as Niigata Lowland is concerned, it is conceivable that the strata deposited in the same age have

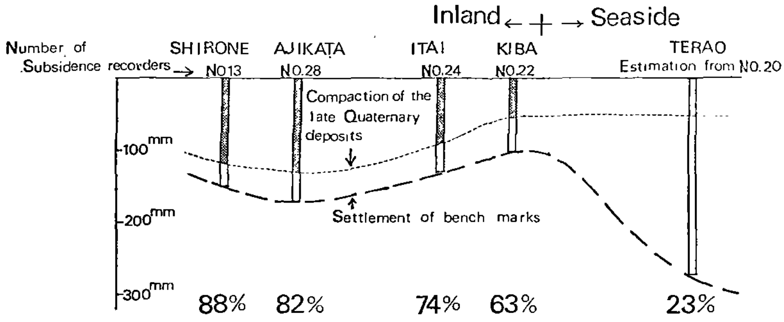

FIGURE 6. *Relation between compaction of the late Quaternary deposits and settlement of bench marks, in the same place from 1965 to 1968*

experienced an identical load. Different lithofacies such as clay, silt or sand, which originated from different sedimentary environments, have different degrees of consolidation even if they were subject to an identical load. Also, different sedimentary environments such as inland bay, lagoon, or lacustrine show different types of aquifers bearing natural gas. The classification of the land subsidence area into such geological units is instrumental in analyzing the land subsidence in terms of the hydrological balance of ground water and soil mechanics.

Presuming that the regional differences of the land subsidence depend on geological and hydrogeological variations, the authors investigated the geology of the groundwater basin. They chiefly investigated the late Quaternary deposits in view of the fact that 80-90 percent of the subsidence in the inland area comes from the compaction of the said late Quaternary deposits, regardless of the true causes of the subsidence.

In their investigation, they resorted to exploratory drilling, penetration tests, micropalaeontological analysis, and absolute chronology by the radio-carbon and tritum dating methods for knowing the groundwater circulation. In this connection, pollen analysis proved to be very useful for the correlation of each formation and diatom analysis for knowing the sedimentary environments.

Most of the strata beneath the several subsided area belong to the Quaternary System, and the late Quaternary deposits can be divided into five layers, I, II, III, IV, and V, respectively. Figure 7 represents a typical geological section along the so-called Line F which runs from north-northwest to south-southeast in the central part of the land subsidence area. The materials obtained by the exploratory drilling were put to micro- palaeontological analysis, the results of which are given in table 1.

Geological characteristics of each of the afore-mentioned five layers are as follows

Layer I

Deposits in humid plains like the swampy land in the regression stage of the latest Holocene with a climate as mild as the present. Its general depth is less than 20 meters below the ground surface. It consists of sand, silt and clay containing fossil wood, and is very little compacted in general.

Layer II

Lagoon or inland bay sediments of Holocene age deposited under a stable water level and a mild climate, the temperature being a little lower than that of the present. Its general depth is from 20 to 40 meters below the ground surface. The said layer consists of very soft clay at Shirone, and of sand or silt at Kurotori which is situated around the boundary line between the inland and the seaside areas. The extent of the clay bed coincides with the area where a considerable subsidence is observed in the inland.

Layer III

Lagoon or inland bay deposits in the process of gradual stabilization of the sea level with a mild climate. Its general depth is from 40 meters to 80 meters below the ground surface. The inland part of this layer consists chiefly of clay, and the seaside part consists of silt.

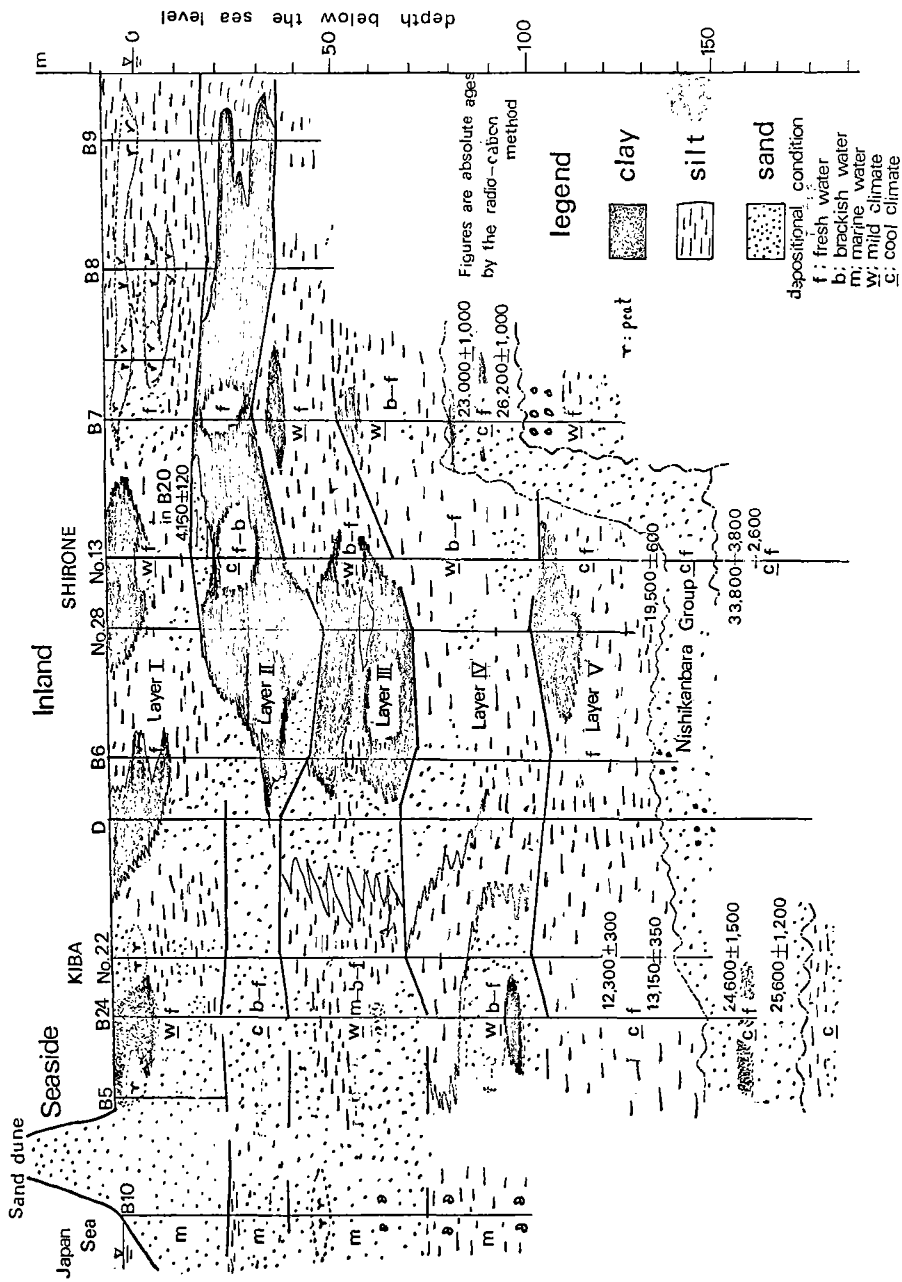

FIGURE 7. *Geological section of line F*

Layer IV

Lagoon or lacustrine deposits in the process of a radically changing water level and a mild climate. Its general depth is between 80 and 100 meters below the surface.

Layer V

Lacustrine sediments deposited less than 20,000 years ago under a stable water level and a cool climate. This layer is generally distributed between 100 and 150 meters below the surface. Beneath it, there are terrace-like deposits called "Nishikambra Group", which were formed more than 25,000 years ago.

TABLE 1. The result of micro palaeontological analysis of No 13 and B.24

drilling / items / layer	B. 24					NO. 13				
	depth	thickness	lithofacies	main pollen	main diatom	depth	thickness	lithofacies	main pollen	main diatom
I	0 m ~ 19.0 m	19.0 m	clay peat silt sand fossil wood	Fagus Quercus	Fragilaria Construens	0 m ~ 18.5 m	18.5 m	clay peat silt fossil. wood	Alnus Cryptomeria	Eumatiapecimolis Tabellaria fenestrata
II	19.0 m ~ 40.5 m	21.5 m	sand silt	Abies Picea Tsuga	Diatom rare	18.5 m ~ 39.0	20.5 m	fossil Shells clay	Pinus Quercus Gramineae Abies Picea	Diploneis ovalis Nitaschia granulata Nitaschia punotata
III	40.5 m ~ 75.5 m	35.0 m	silt sand clay	Fagus Quercus	Melosira sulrata Coscinodiscus lacustris	39.0 m ~ 68.0 m	29.0 m	Siltysand clay silt sand	Fagus Quercus Gramineae	Gomphonema angustatum Gomphonema olivaieum
IV	75.5 m ~ 131.0 m	55.5 m	sand silt clay (alternation)	Fagus Quercus	Tubellaria fcenestrata	68.0 m ~ 104.0	36.0 m	clay Silt Sand (alternation)	Fagus Pinus Alnus Pteridophyta	Epithemia febra Stauroneis phoeniceteron Pinnularia gibba
V	131.0 m ~ 152.0 m	31.0 m	clay peat silt	Pinus Quercus		104.0 m ~ 135.0 m	31.0 m	clay silt Sand Siltysand peat	Pinus Abies Picea Fagus	Melosira italica Navicula americana

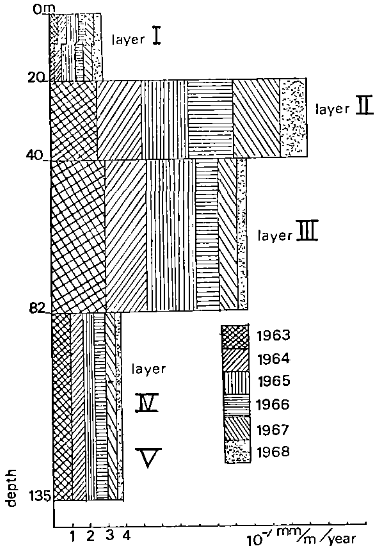

FIGURE 8. *Yearly compaction ratio of each layer*

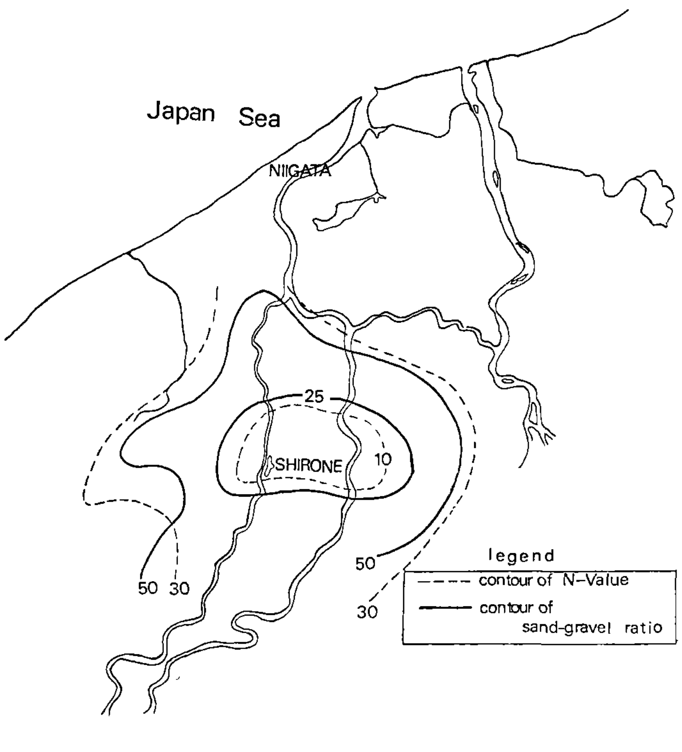

FIGURE 9. *Contours of N-Value and sand-gravel ratio*

As shown in figure 8, most of the soil compaction occurs in the Layers II and III. The distribution of the mean N-value and the sand-gravel ratio in Layers II and III coincides with that of land subsidence in the inland, indicating regional differences of lithofacies (fig. 9).

From those findings, it is to be surmised that the Niigata lowland as a whole has grown in a lagoon-like environment, though temporarily it may have been severely affected by sea water or may have been transformed into an inland bay or a delta.

In view of the fact that most of the subsidence in the inland resulted from the compaction of Layers II and III, and that the distribution of N-value and sand-gravel ratio are similar to that of the land subsidence, the sedimentary lagoon-like environment seems to have been closely related to the land subsidence.

4. GROUNDWATER IN LAND-SUBSIDENCE AREA

The groundwater in the land-subsidence area in the Niigata lowland can be divided into two types, namely, unconfined and confined groundwater. The unconfined aquifer consists of the afore-mentioned Layer I; sometimes its water table fluctuates under the influence of rainfall or irrigation water for paddy fields, but has remained little changed in the long run. Natural gas is dissolved in all the groundwater confined in the afore-mentioned Layer II and in underlying layers in Quaternary deposits. In order to utilize the gas, which is separated from the groundwater at the surface for fuel and for producing chemical products, the confined groundwater has been pumped out of the aquifer.

There are two kinds of uses for the gas, that is, industrial use on a larger scale and domestic use on a smaller scale. The gas for industrial use is obtained from the aquifer in early Quaternary deposits and that for domestic use from late Quaternary deposits. These yearly quantity of pumped groundwater are given in figure 10.

Both figure 4 and figure 10 indicate close relationships between the volume of land subsidence and quantity of the groundwater brought up to the surface. On an average, proportions of subsidence as against pumped-up groundwater generally range from 15 to 19 percent.

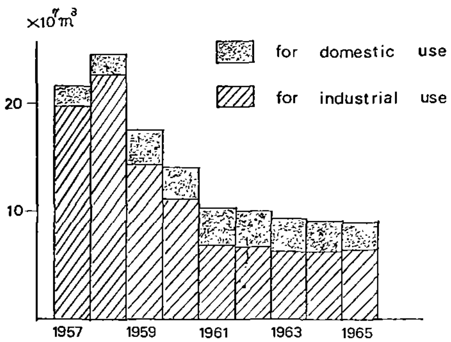

FIGURE 10. *Yearly quantity of pumped groundwater*

The aquifer in the later Quaternary deposits generally has a coefficient of permeability of between 1×10^{-2} and 1×10^{-3} centimeter per second. The aquifer in the early Quaternary deposits has a coefficient of permeability ranging from 2×10^{-2} to 3×10^{-2} centimeters per second.

The apparent age of the confined groundwater was made known by means of the tritium dating method, which revealed that the hydrological circulation was slow. Mr. Kimura's report deals with this matter in detail. It is conceivable that imbalance of pumping-up and supply of the groundwater sent the land sinking down.

5. RELATION OF COMPACTION TO GEOLOGICAL CHARACTERISTICS

The subsidence recorders at Shirone Observatory showed different movements of each layer. Rate of compaction per meter per year is large in the Layers II and III. The velocity of compaction underneath Layer III has been decreasing year by year, but there can hardly be recognized any sign of reduced compaction in Layer II which is the most uncompacted. The amount of confined water pumped up from Layer II was small. This shows that pumping-up of groundwater from the strata below the Layer II brings down on the most uncompacted stratum.

Anyway, the land subsidence in the inland area is closely related to the physical character of the late Quaternary deposits.

6. COUNTERMEASURES FOR PROTECTING DRAINAGE AND IRRIGATION FACILITIES

The northern part of the so-called Niigata Lowland was originally a low and swampy flood plain along the Shinano River, which was often hit by flood-water. The history of development of this area is adorned by a series of drainage system improvement works. In old times, "short-cut" methods were employed for free drainage of the river water to the Japan Sea.

In 1878, the Okozu Diversion Canal was dug so as to divert some of the water of the Shinano River into the sea, and meanwhile, drainage pumps were introduced. In consequence, this area gradually transformed itself into a stabilized rural district.

But, the abrupt land subsidence has brought about shrinkage of draining capacity of those pumps and partial disarray of the canal network in this area. Total cost for reconstruction of those damaged facilities is estimated at 20 billion yen (about US $ 56,000,000).

7. APPENDIX

As the abrupt land subsidence, which has directly affected our living environment, was caused by abstraction of confined groundwater, it seems reasonable to make an approach to this phenomena in terms of the groundwater hydrologic balance. On the basis of those findings they have obtained so far, the authors are trying further researches on the hydrologic balance of the water confined in aquifers in the Quaternary deposits, coupled with study of soil mechanics of the subsidence.

They owe a great deal to Drs. T. Shibazaki, S. Yamamoto, T. Utashiro, M. Minato and Mrs. S. Kimura and H. Negishi, who have helped them prepare this paper. And also, they are grateful for the kind cooperation and assistance extended by their senior officials of the Shinano River Basin Agricultural Water Resources Development Survey Office.

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INFLUENCE DES AFFAISSEMENTS SUR L'HYDROLOGIE TANT DE SURFACE QUE SOUTERRAINE

Prof. L. J. TISON
Université de Gand

RÉSUMÉ

1. L'auteur considère d'abord l'aspect purement hydraulique du problème et il envisage l'influence des affaissements sur les niveaux d'eau d'une rivière libre successivement, en temps ordinaire et ensuite en temps de crue.

a) Temps ordinaire. — Nous appelons ainsi les écoulements dont le débit n'est pas influencé par les affaissements. L'influence de ceux-ci se détermine donc en comparant, avant et après affaissements, des écoulements avec le même débit. L'auteur rappelle le théorème de la pente superficielle qui lui permet d'établir que dans les conditions d'égalité des débits, les niveaux d'eau absolus sont diminués dans la zone affaissée et à l'amont de celle-ci. A l'amont, il y a donc assèchement des terrains voisins, mais dans la zone affaissée, l'auteur peut encore montrer que la réduction des hauteurs d'eau absolues est inférieure à la valeur de l'affaissement ce qui amène donc au relèvement des niveaux par rapport aux terrains voisins.

b) Temps de crue. — L'auteur montre qu'à conditions hydrologiques égales (alimentation identique de la rivière), le débit est augmenté par l'affaissement avec les conséquences suivantes :

1° relèvement du niveau des eaux d'une crue donnée à l'aval de la région affaissée ;
2° relèvement absolu de ce niveau dans la partie aval de la zone affaissée, mais abaissement absolu dans la partie amont de cette zone. Toutefois, même dans cette partie, il y a relèvement relativement aux terrains voisins ;
3° abaissement du niveau de la crue à l'amont de la zone affaissée.

2. Ce qui précède suppose que la rivière n'a pas de débit solide.
Les dépôts seront certainement plus aisés dans la partie affaissée mais ils le seront moins à l'amont de cette région.

3. Il se peut que le relèvement relatif des niveaux d'eau dans la région affaissée (dont il a été question au chiffre 1) soit tel qu'il produise des inondations permanentes dans cette région. Ces nouveaux champs d'inondation agiront à leur tour sur les crues pour réduire le débit à l'aval de la zone affaissée, ce qui amène dans ce cas une réduction des hauteurs d'eau à l'aval de l'affaissement, contrairement à ce qui a été dit sous le chiffre 1.

ABSTRACT

1. First of all the hydraulic aspect of the problems is considered and the influence of land subsidence on a river is determined.

a) Usual Conditions. — In this case, the discharge of the river does not change under the influence of the land subsidence. This influence can therefore be deduced from the comparison of the situations, before and after the subsidence, with the same discharge. Author applies what he calls the theorem of the "surface slope" and shows that the absolute water levels are reduced in the zone of the subsidence and upstream. However, the reduction of the absolute waterlevel in the zone of the subsidence is smaller than the amount of the subsidence and there is therefore a relative rising of the waterlevel.

b) Floods. — It is shown that, for the same hydrological conditions, the discharge of the river increases under the influence of the subsidence with the following consequences:

1° Increase of the waterlevels downwards in the region of subsidence.
2° Increase of the absolute waterlevels in the downwards part of the subsidence but decrease of these levels in the upwards part of the subsidence. However, this decrease is always less than the amount of the subsidence with the consequence of a relative increase of the waterlevels for the riverbanks.
3° Decrease of the waterlevel upwards of the subsidence.

2. There is no question in these first considerations of a movement of sediments. Debris and silt will certainly have more opportunity to deposit in the region of the subsidence but not in the region upwards of this subsidence.

3. It may be that the relative rising of the waterlevels in the region of the subsidence (see 1) will provoke permanent inundations of the neighbouring grounds. In this case, the inundation will reduce the flood discharge downwards with the consequences that the waterlevels will decrease in this region (in opposition with what was told under 1).

1. La production d'affaissements à la surface du sol amène des perturbations tant dans l'écoulement des eaux de surface que dans celui des eaux souterraines.

Le but de cette étude est de rechercher quelle peut être cette influence, particulièrement dans les régions où les rivières ne disposent que de pentes faibles ou modérées et où les perturbations en question peuvent avoir des effets particulièrement désastreux du point de vue hydraulique.

2. L'étude théorique de l'action des affaissements est rendue aisée par l'établissement d'un théorème que nous avons appelé de la pente superficielle, théorème que nous établissons en annexe ainsi que ses conséquences les plus importantes.

On étudiera deux situations hydrauliques nettement différentes : tout d'abord ce qu'on appellera la situation ordinaire, caractérisée par le fait que la production d'affaissements ne modifie pas le débit et ensuite on considérera le temps dit de crue caractérisé par l'existence de champs d'inondation qui peuvent être à l'origine de modifications de débits sous l'action des affaissements.

3. Situation ordinaire (il n'y a pas de champs d'inondation). Soit *A B C D E* (fig. 1) le fond d'une rivière naturelle avant les affaissements et soit *A B C′ D E* ce que devient ce fond après la production des affaissements du sol. Soit encore *a b c d e* la ligne d'eau avant les affaissements. La ligne d'eau étant déterminée (dans les conditions normales)

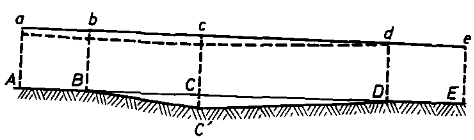

FIGURE 1.

par son point aval *a*, il est évident qu'elle ne sera pas modifiée par les affaissements entre *a* et *b*, puisque le lit entre ces sections n'est pas modifié. À partir de *b*, dans la zone affaissée, trois possibilités sont à envisager à priori :

(*a*) La ligne d'eau n'est pas modifiée par les affaissements. L'application du théorème de Bernoulli

$$z_b + \frac{v_b^2}{2g} = z_d + \frac{v_d^2}{2g} + \int_B^D \frac{\chi}{\omega^3}\, bq^2\, \mathrm{d}s$$

montre que cette supposition est inacceptable. En effet, les quatre premiers termes de l'équation seraient inchangés, alors que le cinquième serait réduit du fait de l'augmentation de ω.

z cotes à partir d'un plan de repère horizontal;
v vitesses moyennes;
g est l'accélération de la pesanteur ;
$\varkappa$ est le périmètre mouillé;
ω la section mouillée;
b le coefficient introduisant la rugosité;
ds un élément de longueur.

(*b*) La ligne d'eau est relevée par les affaissements. Un raisonnement analogue au précédent montre que c'est encore plus impossible.

(*c*) La ligne d'eau est rabattue par les affaissements. C'est la seule possibilité non exclue par l'équation de Bernoulli.

On peut se demander si le rabattement de la ligne d'eau est égal, supérieur ou inférieur à l'affaissement.

La même équation de Bernoulli appliquée entre les sections *C* et *D* montre que les deux premières suppositions sont erronées et que c'est la troisième qui se produira.

Ainsi est établi un premier point important : en temps ordinaire, l'affaissement du lit d'une rivière naturelle et des régions qui la bordent, produit un *relèvement relatif* du niveau de l'eau par rapport aux terrains voisins mais ce relèvement est inférieur à l'affaissement, cette dernière conclusion apportant un correctif favorable à la première.

La question est évidemment de savoir si ce correctif favorable est réellement à prendre en considération. Chaque problème est un cas d'espèce. Dans certains cas, le relèvement relatif du niveau d'eau peut être très fortement réduit et la correction signalée ici est loin d'être négligeable.

Autre conséquence de l'affaissement : on a vu que l'abaissement du plan d'eau dû à l'affaissement s'étend jusqu'en *A* (il en résulte que dans cette région le plan d'eau est rabattu par rapport aux terrains voisins). Ce rabattement s'étend à l'amont de *A* comme on le voit en appliquant le premier corollaire du théorème de la pente superficielle : en effet, en amont de *A*, l'affaissement n'a pas modifié le lit et si le plan d'eau aptès affaissement coupait l'ancien pour provoquer des hauteurs d'eau plus grandes, le débit après affaissement serait supérieur au débit initial, ce qui est exclu dans le cas étudié.

4. Temps de crue caractérisé par l'existence de champs d'inondation, notamment à l'amont de *A* (fig. 2).

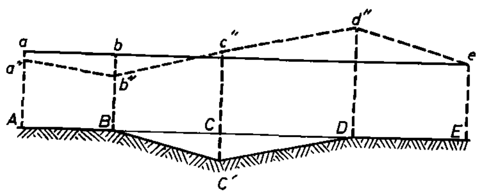

FIGURE 2.

Il est clair que dans ce cas, la production d'affaissements augmentera le débit instantané du cours d'eau dans la zone *A B C D E* et à l'aval de celle-ci. En effet, si le débit restait non influencé par l'affaissement, on se trouverait dans un cas analogue à celui traité ci-dessus et le plan d'eau, après affaissements, à l'amont de *A* serait inférieur au plan d'eau primitif, avec la conséquence qu'à la sortie du champ d'inondation, quelque part à l'amont de *A*, la chute serait augmentée (la hauteur d'eau ne variant guère dans le champ d'inondation du fait de son étendue). Cette augmentation de chute correspondrait à une augmentation de débit, ce qui est contraire à l'hypothèse.

On démontrerait qu'à fortiori, de la même façon, l'affaissement ne peut réduire le débit. Il doit donc l'augmenter.

L'affaissement augmentant le débit de crue, la ligne d'eau à l'aval de la région affaissée s'établira au-dessus de la ligne d'eau d'origine, comme le montre l'application du corollaire du théorème de la pente superficielle.

D'autre part, à l'amont de la zone en question, à la sortie des champs d'inondation, l'affaissement aura pour effet, en augmentant le débit, d'augmenter la chute et par conséquent de rabattre le niveau du plan d'eau sous celui se produisant pour la même crue avant les affaissements. De plus, à l'aval de la sortie des champs d'inondation, le plan d'eau, après affaissements, ne pourra pas couper le plan d'eau primitif car cela n'est possible, d'après le corollaire du théorème de la pente superficielle, que si le débit après affaissements était inférieur à l'autre : or, on a vu que c'est l'inverse qui se produit.

La ligne d'eau après affaissements prendra donc la forme *a b″ c″ d″ e″* avec aggravation de la situation dans la partie aval et amélioration vers l'amont.

Les conséquences au point de vue de l'inondation directe par débordement de la rivière, se déduisent aisément de ce qui précède.

5. Mais, très souvent, l'inondation ou l'humidification des terres se produit par la nappe aquifère. En effet, cette nappe est en relation avec le cours d'eau et à tout relèvement relatif du niveau de ce dernier, correspond un relèvement relatif de la nappe. Ce relèvement relatif, pour un affaissement uniforme de la plaine d'alluvions, sera maximum le long de la rivière quand les eaux souterraines alimentent la rivière. En effet, l'épaisseur de la couche alluvionnaire filtrante est augmentée par les affaissements, ce qui réduit la pente superficielle de la nappe aquifère. Le relèvement relatif de la nappe phréatique par rapport au niveau de sol ne sera donc toujours qu'une fraction du montant de l'affaissement, fraction qui sera toujours un cas d'espèce mais dont l'évaluation sera soumise aux considérations développées ci-dessus.

6. Les principes qui ont servi à l'étude qui précède sont applicables aux rivières non libres, avec barrages-écluses. On remarquera cependant que les barrages créent des profondeurs d'eau plus grandes que si la rivière était libre. Il en résulte qu'à affaissement égal, l'augmentation de section provoquée par l'affaissement constitue une plus faible proportion de la section avec barrage qu'il ne le fait sur la rivière libre. Il en résulte que sur la rivière barrée la réduction de pente superficielle est relativement moins forte que sur la rivière libre. La conséquence en est que sur la rivière barrée une plus faible fraction de l'affaissement est regagnée dans le relèvement relatif de la ligne d'eau du fait de l'affraissement.

7. On a parfois écrit que les profondeurs d'eau, accrues par les affaissements, facilitaient le dépôt des matériaux transportés par la rivière ramenant le fond de la rivière à sa cote absolue primitive. S'il en était ainsi la grandeur de relèvement du niveau de la rivière tendrait vers la valeur de l'affaissement avec toutes les conséquences désastreuses qui en dériveraient.

L'étude de ce qui se passe sur certaines rivières belges dont des tronçons sont soumis à des affaissements miniers, montre, qu'après un siècle, le remblayage dont il vient d'être question, ne s'était pas encore produit ou simplement à une échelle qui permettait le maintien des surprofondeurs créées par les travaux normaux d'entretien. C'est le cas de la Sambre comme l'a signalé Caulier dans ses études sur cette rivière.

Le fait peut sans doute s'expliquer en raison du faible charriage sur le fond de ces rivières, ne se produisant que lors des crues assez importantes et aussi en faisant appel aux considérations ci-dessus suivant lesquelles la surprofondeur n'est qu'une fraction, parfois très réduite, de la valeur de l'affaissement.

8. CONSÉQUENCES ET REMÈDES

Les conséquences du relèvement relatif, même fractionnaire, du niveau de la rivière d'une part et de la nappe phréatique d'autre part, se déduisent aisément : débordements plus fréquents et plus longs de la rivière et submersion plus ou moins complète des terrains de la vallée alluvionnaire par la nappe phréatique avec difficultés d'égouttage des zones habitées, noyade des caves et transformations des terrains en régions plus ou moins marécageuses. A remarquer cependant que ces considérations ne sont pas générales: comme on l'a vu, d'une part la région amont voit la situation s'améliorer et même pour le reste de la région affaissée le relèvement relatif du niveau de l'eau par rapport aux berges est inférieur, et parfois de beaucoup, à la hauteur de l'affaissement.

Dans la région liégeoise en Belgique, des affaissements ont porté sur une région très industrialisée et très habitée, s'étendant sur de nombreux kilomètres.

Il ne peut être question de décrire les travaux entrepris pour sauver cette région dont certaines parties se trouvent plus de cinq mètres en contrebas des hautes crues. Donnons simplement les principes appliqués :

(*a*) Endiguement de la rivière Meuse de façon à empêcher tout débordement.
(*b*) Dérivation des eaux des hauteurs par des canalisations fonctionnant en conduite forcée dans la traversée de la plaine d'alluvions (exutoires des hauteurs).
(*c*) Réseau d'égouts normaux pour l'évacuation des eaux usées, le déversement des eaux à la rivière pouvant parfois se faire par pesanteur en temps ordinaire, mais se faisant à l'aide de stations de pompages dites principales dès que le niveau de la rivière l'exige.
(*d*) Établissement d'un second réseau d'égouts profonds reliés aux caves qui jouent le rôle de dispositif drainant la nappe pour empêcher l'inondation de ces caves. Les canalisations de ce réseau profond dirigent toujours leurs eaux de drainage vers des stations de pompage, dites secondaires, qui rejettent leurs eaux dans le réseau d'égouts supérieurs.

ANNEXE

THÉORÈME DE LA PENTE SUPERFICIELLE

Nous n'envisagerons que le cas des cours d'eau prismatiques. L'équation fondamentale du mouvement permanent :

$$\frac{\mathrm{d}h}{\mathrm{d}s} = \frac{i - \dfrac{\chi}{\omega^3} bq^2}{\sqrt{1-i^2} - \dfrac{q^2}{g}\dfrac{l}{\omega^3}}$$

peut s'écrire :

$$-i + \frac{\mathrm{d}h}{ds}\sqrt{1-i^2} = -\frac{\chi}{\omega^3} bq^2 + \frac{q^2}{g} \cdot \frac{l}{\omega^3} \cdot \frac{\mathrm{d}h}{\mathrm{d}s} \tag{1}$$

h est la hauteur d'eau prise perpendiculairement au fond;
s est l'abscisse mesurée suivant la droite de fond;
i est la pente de fond;
χ est le périmètre mouillé et
ω est la section mouillée;
q est le débit;
b est un coefficient de frottement;
l est la largeur du cours d'eau en surface;
g est l'accélération de la pesanteur.

La fonction

$$-i + \frac{\mathrm{d}h}{\mathrm{d}s}\sqrt{1-i^2}$$

est linéaire en $\mathrm{d}h/\mathrm{d}s$ et est représentée en fonction de $\mathrm{d}h/\mathrm{d}s$ par une droite passant par les points

$$(0, -i) \text{ et } \left(\frac{i}{\sqrt{1-i^2}}, 0\right).$$

On appellera cette droite la droite I (fig. 3).

Le second membre est aussi linéaire en dh/ds et est donc représenté par une droite passant par les points

$$\left(0,\ -\frac{\chi}{\omega^3}\,bq^2\right) \quad \text{et} \quad \left(gb\,\frac{\chi}{l},\ 0\right)$$

c'est la droite II.

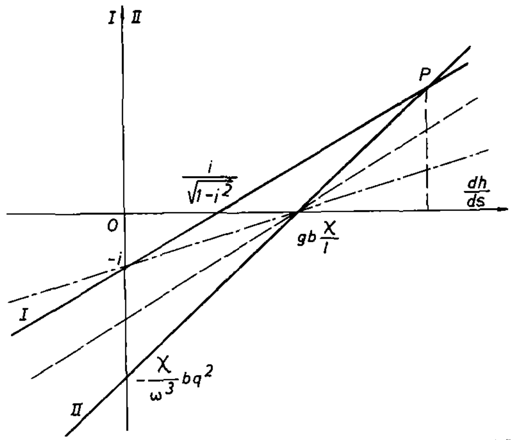

FIGURE 3.

Ces droites I et II se coupent en un point dont l'abscisse dh/ds rend égaux les deux membres de l'équation (1) et satisfait par conséquent à l'équation du mouvement permanent.

On se propose de rechercher la variation de dh/ds quand le débit varie.

VARIATION DE dh/ds AVEC q; i ET h RESTANT CONSTANTS

Dans ce cas, les points

$$(0,\ -i) \quad \text{et} \quad \left(\frac{i}{\sqrt{1-i^2}},\ 0\right)$$

sont fixes et il en est de même de la droite I. D'autre part, h étant constant, (gb (χ/l, 0) est un point également fixe, tandis que $-(\chi/\omega^3)\,bq^2$ varie avec q. La droite II pivote donc autour de gb (χ/l).

Nous n'envisagerons que les cas des rivières (en opposition aux torrents dont la pente est forte).

$$\frac{i}{\sqrt{1-i^2}} < gb\,\frac{\chi}{l}\,.$$

Si $q = 0$, le point (0, $-(\chi/\omega^3)bq^2$) se confond avec l'origine et le point d'intersection P vient en ($i/\sqrt{1-i^2}$, 0) : (horizontalité).

Si q croît, le point d'intersection se déplace vers la gauche, c'est-à-dire que dh/ds diminue, tout en restant positif tant que le point (0, $-(\chi/\omega^3)\,bq^2$) reste au-dessus de la droite I.

Lorsque $(\chi/\omega^3)bq^2$ devient égal à i, dh/ds devient nul (mouvement uniforme).

Le débit q continuant à croître, le point $(0, -(\chi/\omega^3)\,bq^2)$ passe sous la droite I et dh/ds devient négatif, continuant à décroître.

Le point $(0, -(\chi/\omega^3)\,bq^2)$ prend une position telle que les droites I et II deviennent parallèles : l'augmentation du débit a donc provoqué une telle diminution de dh/ds que celui-ci devient égal à $-\infty$.

La continuation de l'augmentation du débit fait passer brusquement le point d'intersection des droites I et II de la gauche vers la droite : dh/ds passe donc brusquement de $-\infty$ à $+\infty$ pour la valeur de q qui assure le parallélisme des droites I et II, c'est-à-dire pour :

$$\frac{i}{\dfrac{\chi}{\omega^3}\,bq^2} = \frac{\dfrac{i}{\sqrt{1-i^2}}}{gb\,\dfrac{\chi}{l}} \quad \text{ou} \quad \sqrt{1-i^2} = \frac{q^2}{g}\,\frac{l}{\omega^3}\,(\text{hauteur } H_1).$$

Si q continue à croître, le point $(0, -(\chi/\omega^3)\,bq^2)$ descend de plus en plus et le point d'intersection des droites I et II se déplace encore une fois vers la gauche : dh/ds décroît encore quand q croît, pour devenir égal à $gb\,(\chi/l)$ pour $q = \infty$.

En résumé, si $i/\sqrt{l-i^2} < gb\,.(\chi/l)$, le dh/ds décroît quand q croît, sauf cependant pour la valeur du débit telle que $h = H_1$ pour laquelle dh/ds passe brusquement de ∞ – à $+\infty$.

CONSÉQUENCES : Corollaires

(*a*) Dans le cas de faible pente envisagé, le théorème ci-dessus permet de voir qu'en un point, la ligne d'eau correspondant au plus fort débit se place au-dessus de celle du plus faible débit, quand on regarde vers l'amont. C'est le corollaire appliqué au cours de la discussion sur l'action des affaissements.

Supposons d'abord dh/ds positif. En vertu de ce qui a été établie dans le cas de faible pente envisagé, l'axe l qui a le plus petit dh/ds doit avoir le plus grand débit, ce qui établit la propriété (fig. 4).

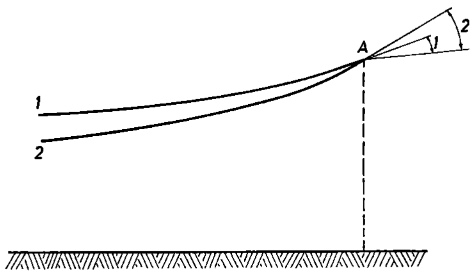

FIGURE 4.

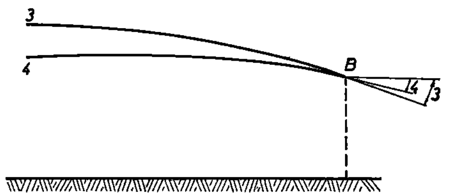

FIGURE 5.

Supposons maintenant dh/ds négatif. L'axe 3 a un dh/ds en B qui est en valeur absolue plus grand que le dh/ds de l'axe 4 : il en résulte que le dh/ds de l'axe 3 est inférieur à celui de l'axe 4 : l'axe 3 a par conséquent le plus fort débit (fig. 5).

(*a*) Deux axes hydrauliques ne peuvent se couper en plus d'un point. C'est une conséquence des propriétés précédentes, car s'il y avait un second point d'intersection, l'axe avec le plus fort débit, à l'amont de A, deviendrait l'axe avec le plus faibledébit à l'amont de B (fig. 6).

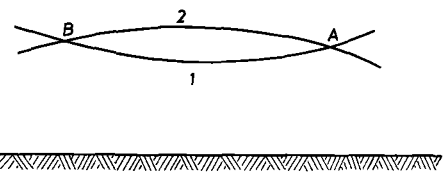

FIGURE 6.

SOME PROBLEMS AND RESULTS OF LABORATORY AND FIELD INVESTIGATIONS INTO ROCK MOVEMENTS CAUSED BY WATER MIGRATION IN LOOSE GRANULAR GROUNDS IN HUNGARY

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ABSTRACT

Some observations about the effect of water migration in loose granular rocks upon the state of equilibrium in reservoirs based upon recent theoretical research work performed in Hungary.

Some possible test methods for investigating our theoretical assumptions and the results of experiments:

Measuring and estimating rock movements caused by pore water pressure drop in the vicinity of Gyöngyösvisonta open pit mine, especially forecasting rock movement in the neighbourhood of a thermal power-station and determining the constants necessary for calculating the degree of reduction in porosity caused by secondary consolidation.

Possibilities and limitations of laboratory experiments and some new methods used in Hungary for investigating rock movements caused by water migration in loose granular sedimentary rocks. The applicable methods are: magnetic photoelastic analogous modelling and special Sectormodel (triaxial filter cell) for investigating the basic phenomena of rock equilibrium.

RÉSUMÉ

Quelques observations concernant l'action de l'eau en mouvement dans un milieu granulaire meuble sur l'état d'équilibre de l'ensemble ont été faites en Hongrie, en se basant sur des recherches théoriques récentes.

L'auteur présente quelques méthodes d'essais pour rechercher la valeur de ses hypothèses ainsi que les résultats d'expérience :

Mesure et estimation des mouvements du sol causés par la chute rapide de la pression de l'eau au voisinage de la mine de Gyöngyösvisonta et spécialement prévision du mouvement du sol dans le voisinage de la centrale thermique et détermination des constantes nécessaires pour calculer le degré de réduction de porosité causé par la consolidation secondaire.
Possibilités et limites des essais en laboratoire et de quelques nouvelles méthodes utilisées en Hongrie pour étudier le mouvement des sols dus au mouvement de l'eau dans les milieux granulaires meubles. Les méthodes utilisables sont : utilisation de modèles analogues magnétiques photoélastiques et emploi d'un modèle en secteur (cellule filtrante triaxiale) pour rechercher les phénomènes de base de l'équilibre des sols.

The flow of liquids in grained, loose rocks influences, as well known, also the equilibrium of water bearing, "reservoir" rocks. The deformations resulting from equilibrium disturbances of the reservoir rock are either elastic, that is, reversible or non-elastic, that is, irreversible. Elastic deformations are mostly of a negligible extent with respect to both the involved surface subsidence and the reserve of the stored liquid, being only of importance in respect to the variation of the rate of flow of the same.

The extent of the irreversible deformations are due either to the consolidation or the migration of the rock material. The former, that is, the extent of consolidation depends besides the properties and bedding characteristics of the rocks also on the extent of the pore-liquid pressure drop, the extent of migration on the other hand depending on the pressure gradient (i.e. the flow velocity of the liquid). The consolidation results first of all in regional subsidence, while the migration of the grains will but cause local displacements, both phenomena being dependent upon the characteristics of the reservoir rock and the extent of its drainage.

Hungary is for more than one reason interested in the investigations concerned with rock displacements due to consolidation following the pressure drop of the pore-liquid. Previous investigations have shown that the effect of the irreversible deformations are not to be neglected when determining the exploitable water reserve of the reservoir rocks in the Hungarian Basin with their highly limited water supply possibilities [2, 3, 7]. When intensive drainage operations are being carried out over large areas (as is the case with the large open pit lignite mining at the Northern boundary of the Hungarian Basin), informations must be given as to the expected extent of surface subsidences in order that suitable measures might be taken for the protection of prospective big industrial structures otherwise exposed to damages in the area influenced by drainage. As the damages in such structures are not primarily caused by a simultaneous and uniform subsidence of the surface points but rather by their relative displacements (i.e. the deformation of the surface), not only the consolidation phenomena but also those of internal suffusion[1] have had to be investigated into.

According to Hungarian researchers, the non-elastic rock deformations caused by increased effective stresses involved by the pore-water pressure drop can be interpreted as a secondary consolidation due to an additional loading of the ground or the rock. Consequently, the known soil mechanical relations concerning the secondary consolidations with certain neglections (such as the relations given by *Florin*, *Terzaghi*, *Kézdi*, *Gibson* and *alii*) can be taken as adaptable also for the description of consolidations occurring at the points of the pore-water pressure drop. In Hungary, J. *Juhàsz* gave a solution for this problem for various initial and boundary conditions [2, 3], mainly for purposes of water reserve estimations. Juhász, calculating the pore-volume reduction caused by increased effective stresses for given pore-volume variations with depth, which stress increase corresponds in value to that of the pore-liquid pressure drop, made the assumption that

1. (Migration of rock particles e.g. mostly of sands into gravel layers, effected by the flow of water).

the affected layer will descend to a deeper level with the increase of the effective stresses. As in every analytical solution some neglections are necessarily involved, the accuracy of these analytical relations must be checked up by field measurements, if applied for the determination of exploitable consolidation water reserves.

The area of a large open-pit lignite mining seemed to be the most suitable for investigation purposes, as the bedding conditions of both the water bearing and impermeable (sealing) layers, as well as the variations of the water level were well known from borehole data, so that only the surface subsidences have had to be periodically measured. The water level varies rapidly and its variations only extend over the upper layers of an overall thickness of 100 to 150 m. Surface subsidences will thus appear in a relatively short time so that the tectonic displacements of slow spedd can be left out of consideration. A geological profile of the experimental area taken along a surface survey lineis given in figure 1. together with the observed subsidences and the pertaining pore-water pressure changes obtained for a rather important water reservoir and relating to an observation period of one year.

The experimental area covering a considerable part of the area influenced by the lovering of the ground-water surface,and in knowledge of the surface subsidences, the total quantity of the consolidation water reserve raised during the respective observation period can—as a first step—be caculated in the known way [2, 7]:

$$Q_c = \sum \Delta z \cdot \Delta F$$

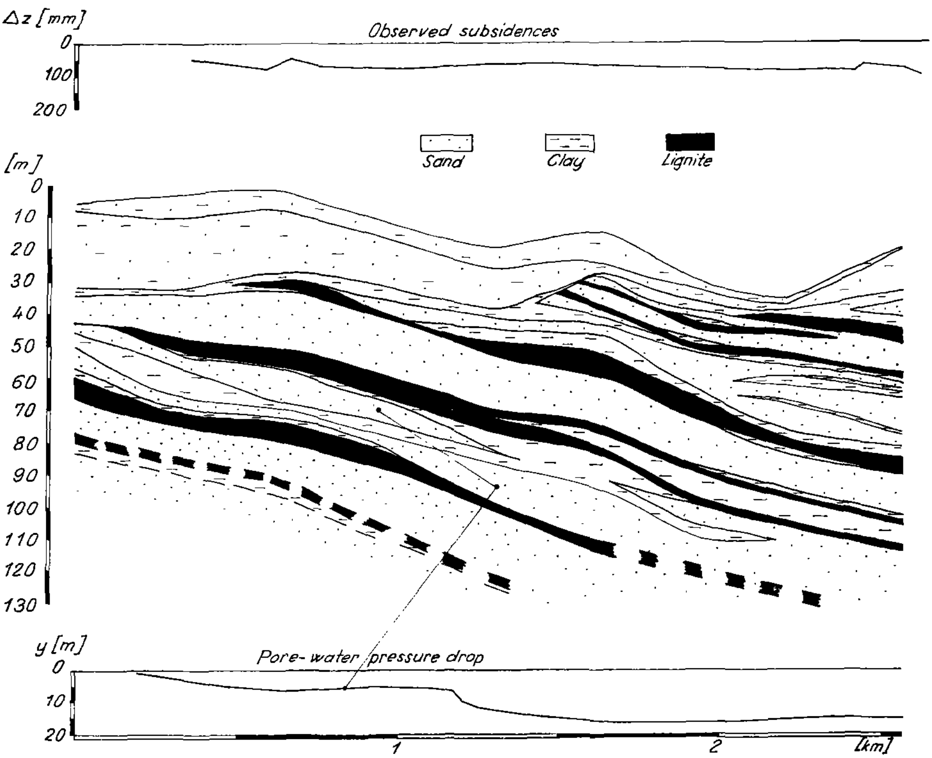

FIGURE 1. *Geological profile of the area of the open-pit lignite mining at Gyöngyösvisonta, along one of the survey lines of subsidence measurements*

where:

ΔF a surface element and

Δz the mean subsidence of the same surface element.

The value thus obtained must be compared with the consolidation water reserve calculated from the elements of the water balance, reliably known from previous investigations [8]:

$$Q_c = Q - (Q_d + Q_e + Q_s)$$

where:

Q the actual quantity of water exploited in the area during the same period;
Q_a the dynamic water supply (inflow) in the same period;
Q_s the water quantity corresponding to the depleted pore-volume;
Q_e the water quantity corresponding to the elastic change in the pore-volume.

If the difference between the values obtained for the consolidation water reserve (Q_c and Q_e) is found to be a considerably large one, the reliability of the subsidence measurements and the water balance must both be controlled. If this difference is found to be of an acceptable value, the functions

$$Q_c = f(t) \quad \text{and} \quad Q_c^* = f^*(t)$$

must be compared in knowledge of the variation in time of both the surface subsidence and the quantity of the water delivered, in order to draw conclusions from the character of the curves as to a possible delay between the occurrence of the underground and surface subsidences. In case this delay is found to be of a negligible value according to our previous assumptions [3], the component of the resultant surface subsidence can be examined and assessed for each layer along the vertical lines in each survey point:

$$\Delta z_i = \int_{z_1}^{z_2} [e_z - e_{(z+y/\gamma_g)}]\,dz$$

$e(z)$ the porosity of the rock at the depth z;
y the pore-water pressure drop;
j_g the mean specific gravity of the rock;
$e(z \pm j/j_g$ the porosity of the rock at the depth $z+y/\gamma g$.

As the reduction of the pore-volume with depth is, as well known, of a decreasing, it is often approached according to its character by an exponential function [2, 4] of the form:

$$e = d - c \log \frac{z+a}{b}$$

(if the change of the compressibility factor with depth is being considered as linear), or by a hyperbola of the form:

$$e = \frac{A}{A \cdot z^n + 1} + B$$

(if no assumption is made as to the change of the compressibility factor)

where:

A = e_0, that is the value of e at the point z = 0,and
B = e_∞, that is the value of e in the most consolidated state of the respective rock.

As

$$\Delta_Z = \sum_{i=1}^{n} \Delta_{Zi}$$

for every surface survey point, and in possession of some 50 survey points, the characteristic values of a, b, c and d or A, B and n, respectively, can be determined from excessive data for each rock layer.

By comparing the standard deviations obtained for the characteristic values, the relationship best satisfying the change of the pore-volume in the given case can also be determined.

Though the numeric examinations are already in progress, their results will only be published at a later date.

In cases when the displacement and migration of the grains caused by the s.c. seepage pressure cannot be neglected, the analytical solution of the process will meet with difficulties which cannot be overcome [5, 9]. The character and extent of both the displacement and migration of the grains depend on the one hand upon the state of stress of the rock (the latter again being dependent upon the boundary conditions, the properties of the rock, the pore-water pressure and the seepage pressure) and the fracture conditions characteristic for the rock on the other hand. The analytical determination of the resultant state of stress for intricate boundary conditions is a problem not easy to solve even when a homogeneous and isotropic rock material and elastic deformations are being assumed,

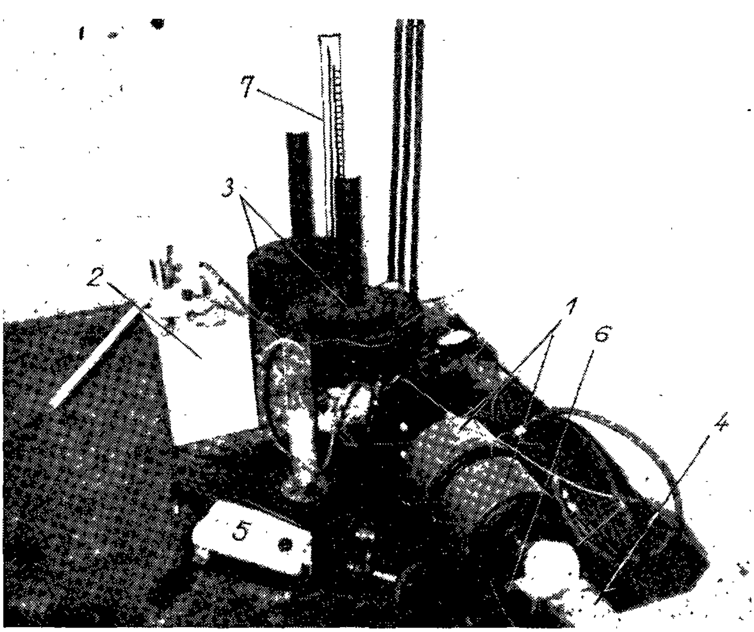

FIGURE 2. *Triaxial filtration pressure cell for the investigation of fundamental phenomena in grained rocks saturated with seeping water*

the problem being practically unsolvable if there are not limiting conditions; the fracture condition, furthermore, is also a material characteristic which can be only determined by experiments. It follows from the above that in the investigation of such cases experimental methods have a prominent part so that the research work done in Hungary was first of all aimed at the determination of the most suitable experimental methods.

According to our previous theoretical examinations, the equilibrium of a water bearing grained rock cannot be simulated by a s.c. physical model without falsifying the very essence of the phenomenon [5]. Consequently, only the s.c. sector model examinations can be reckoned with, that is, forces, similar to those transferred to the rock sample by its natural surroundings previous to its having been taken, must be made to act without reduction on the sample, while, at the same time, also seepage conditions approaching the original ones must be produced in the same. An apparatus satisfying the above conditions was designed and completed in our Mining Research Institute (fig. 2). A prismatic rock sample test piece, $130 \times 130 \times 130$ mm in size, can be placed into cell 1, to be loaded by an evenly distributed loading which again can be varied in pairs, independently from each other, between the limits of 0 to 50 kp/sq.cm and 0 to 150 kp/sq.cm, respectively. This loading acts in directions perpendicular to the surface of the test piece and its value is independent of the deformations. At the same time also a water seepage of a uniformly distributed flow rate can be generated in the direction parallel to the longer axis of symmetry of the test piece, by adjusting a hydraulic gradient and a pore-water pressure of any chosen value. Accordingly, the lateral faces of this test piece and also the plane parallel to the same are to be considered as planes of principal stresses. The two pressure pads placed perpendicularly to the direction of seepage can be filled by feed pump 2. The loads can be set to a constant value or be varied at liberty, independent of possible deformations of the test piece, by adjusting or, respectively, varying the weight loading of the hydro-accumulators 3. The loading parallel to the flow of the liquid is generated by a "Lucas" type hydraulic plunger 4 and can be varied by the hand operated pump 5. Both the gradient of the hydraulic pressure determining the rate of seepage and the pore-water pressure can be controlled by the valve battery 6. The fall of the hydraulic pressure across the test piece can be measured with the multiposition precision mercury piezometer 7.

The apparatus in its present form can be suitably applied for the investigation of the fundamental phenomena of equilibrium, such as the internal suffusion of grained rocks saturated by seeping water, and the determination of the fracture limit curve of such rocks. It can be investigated further whether, when inhomogeneous layers are being drained, any migration (internal suffusion) is likely to occur under natural pressure and seepage conditions due to the liquid's cross flow from a poorly permeable, finer grained layer into a highly permeable, coarser grained one, while also conclusions can be drawn as to the approximate extent of this internal suffusion, based upon which also the character and extent of the expected surface subsidence can be estimated.

The described triaxial pressure filtration cell apparatus is used at the present for the investigation of equilibrium problems in rocks surrounding a drainage installation, but a study into the problems of internal suffusion is also being contemplated by its aid.

A frequent preliminary condition for the investigations made with the above described triaxial pressure cell apparatus is the knowledge of the state of stress of the respective rock. The difficulties met with in both the analytical and model test determinations of the same have already been outlined. At the present not even an analogue model suitable for the true simulation of the phenomenon is known to exist. However, if—for a first approximation—we assume the validity of Hooke's law, the distribution of stresses under the combined effect of own weight, pore-water and seepage pressure can be determined for any intricate boundary condition by a special magnetic photo-elastic model. The essence of this method is that a fine grained ferro-magnetic material is admixed to an optically active gelatine (photo-gelatine) widely applied in photo-elastic examinations. In this photo-gelatine of adequate consistency deformations are induced under its own weight (or by an external loading), which deformations are characteristic of the state of stress of the gelatine and can be made visible in the known way. If now a magnetic field of suitable intensity and distribution is made to act on the test piece, the latter will be loaded by an additional field or fields of forces, similar in character to inertia forces, which fields of forces can be considered to correspond to the neutral stress and the seepage pressure. By varying the shape and material properties of the test piece, the distribution of the ferro-magnetic dust and the character and magnitude of the magnetic field, a great number of variations of the boundary conditions can theoretically be obtained with which the natural conditions of a given problem can be closely approximated. The applicability of the method has already been proved by several fundamental experiments [6]. The completion of the model i.e. the choice of its suitable size as well as the accomplishment of the variability of the boundary conditions is just a question of not too high expenses.

According to the practice followed in soil mechanical investigations, the state of elastic stresses produced and made observable by the magnetic photo-elastic method can be taken in the first approximation as valid also for non-elastic materials. (This negligence is mostly advantageous with respect to satefy.) As a next step the character and extent of the expected material suffusion can be examined by the triaxial filtration pressure cell apparatus for the most characteristic points on actual rock samples.

Though not in possession of final results, we described some of our special test methods in the hope that the eventual contributions to our paper will be of help in their improvement.

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ON THE VARIATION OF ARTESIAN HEAD AND LAND-SURFACE SUBSIDENCE DUE TO GROUND-WATER WITHDRAWAL

Shauzow KOMAKI

ABSTRACT

In and near Yokkaichi, the observation of artesian head and subsidence has been carried out since August 1963. The secular variation of ground-water and land-surface levels is disturbed by oceanic tides, meteorological effects and other factors with periods of about one day or less. In order to obtain the secular variation, these disturbances were eliminated from observed data by Pertzev's method.

After various considerations of the results, it becomes evident that the secular variation of both the ground-water and the land-surface levels is subject to the influence of ground-water withdrawal except oceanic tides, meteorological and other factors. Assuming that the discharge in the regulated area starts at 06.00 hours and ends at 22.00 every day, and taking account of discharge after-effect on the ground-water level, we obtained the effective amount of discharge Q by means of the Theis recovery method.

There is a good correlation between Q of one day and the secular value of the ground-water level at 06.00 hours of each following day. As for the layers of depth range from which in fact ground water is withdrawn, there is also a good correlation between Q and the amount of expansion-compaction from 06.00 hours to the same hour the next day.

1. INTRODUCTION

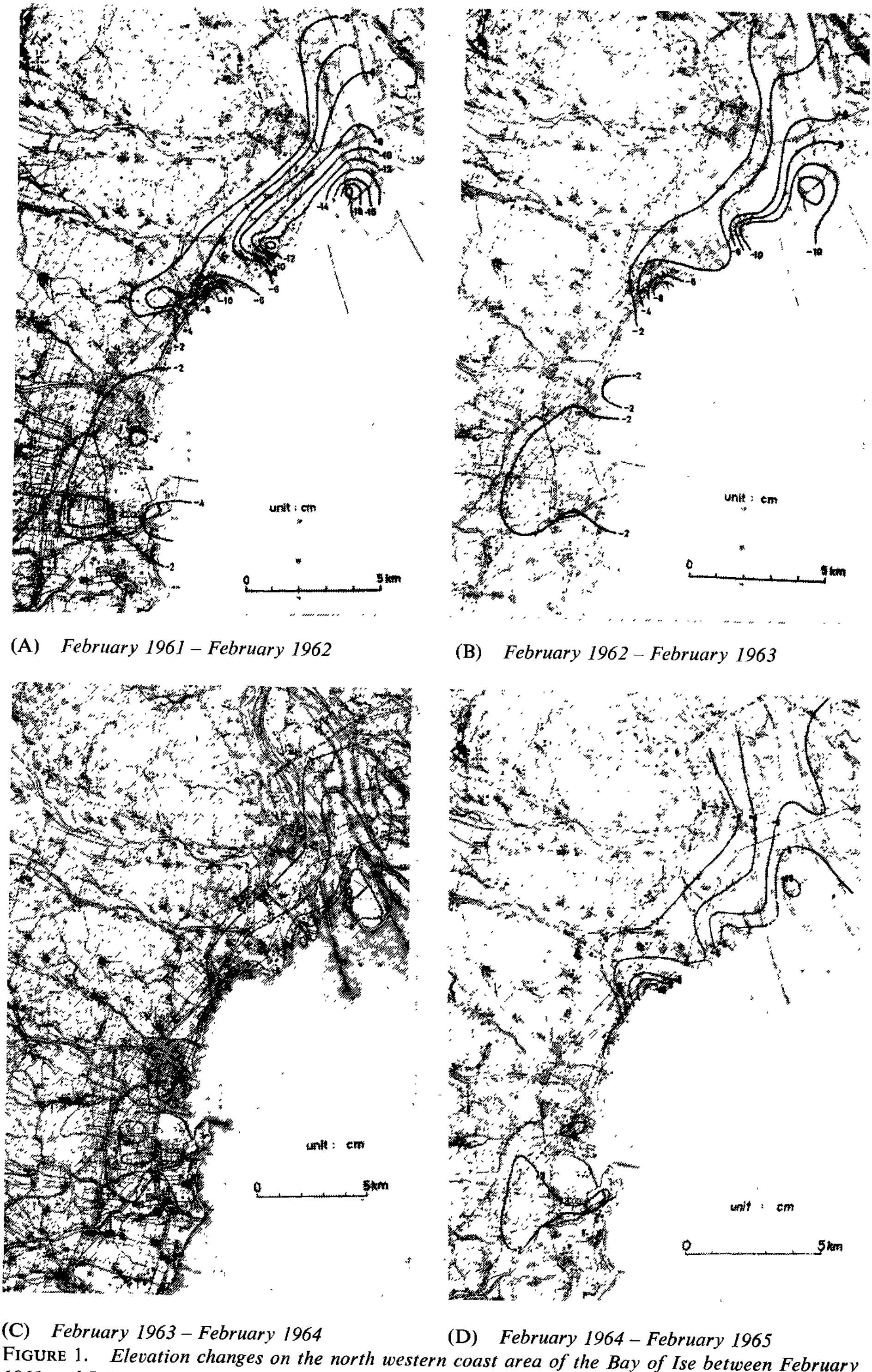
For many years the subsidence of land surface has been known to occur at several places in Japan, e.g., Niigata, Kawaguchi, Tokyo, Kawasaki, Nagoya, Osaka and Amagasaki, due mainly to ground-water withdrawal for industrial use. This phenomenon occurs also in many areas in other countries. One of the most outstanding subsidence area is the harbour area of Los Angeles and Long Beach in the United States.

In Yokkaichi and its vicinity, situates about 30 kilometres southwest of Nogoya, the subsidence of land surface has received considerable attention recently. Since July 1963, the discharge of ground water (by industrial plants). has been regulated by law. The elevation changes on the north-western coast of the Ise Bay, including the regulated area (see figure 2) – during each one-year period from February 1961 to February 1962, February 1962 to February 1963, February 1963 to Fevruary 1964 and February 1964 to February 1965, are shown in figure 1 (A), (B), (C) and (D) respectively.

2. OBSERVATIONS

As one of the methods for investigating the substance and causes of land-surface subsidence, observation wells were installed at three stations as shown in figure 2 and the observation of artesian head and subsidence has been carried out since August 1963. The geological profiles of the area are given in figure 3 [3]. At each observation station, the 1st and 2nd wells have their bottom fixed at the upper (alluvial) and the lower (diluvial) sand and gravel layer respectively and also their strainer at the same layer functioning as an aquifer. The 3rd well reaches the sand and gravel layer underlying the lower clay and silt layer, the 4th well extending to a deeper aquifer, as shown in figure 4.

The form of the observation well is illustrated in figures 5 and 6. A measuring apparatus of artesian head, namely, confined ground-water level, is set on each well, and that of land subsidence on each well except the first. With such an apparatus, the variation of both confined ground-water level and compaction of the surface layers, i.e., the amount of protrusion of the tubing pipe to the steel beam fixed on a land surface, can be observed. The casing pipe protects the *tubing pipe* from yielding and other effects due to surrounding soil pressure.



(A) *February 1961 – February 1962* (B) *February 1962 – February 1963*

(C) *February 1963 – February 1964* (D) *February 1964 – February 1965*

FIGURE 1. *Elevation changes on the north western coast area of the Bay of Ise between February 1961 and February 1965*

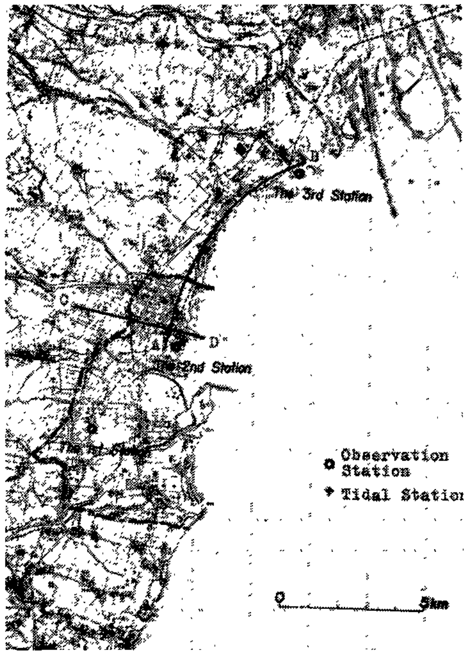


FIGURE 2. *Distribution of observation stations and tidal stations. Area surroundnig a chain line is the regulated one of ground-water withdrawal*

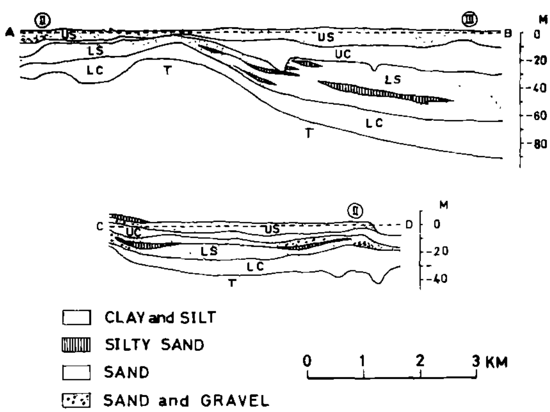


FIGURE . *Geological profiles of section*

A – B and C – D shown in figure 2.

US	Upper sand layer	Alluvium
UC	Upper clay & silt layer	
UC	Upper clay & silt layer	Diluvium
LS	Lower sand layer	
LC	Lower clay & silt layer	
T	Tertiary	

II The 2nd observation station

III The 3rd observation station

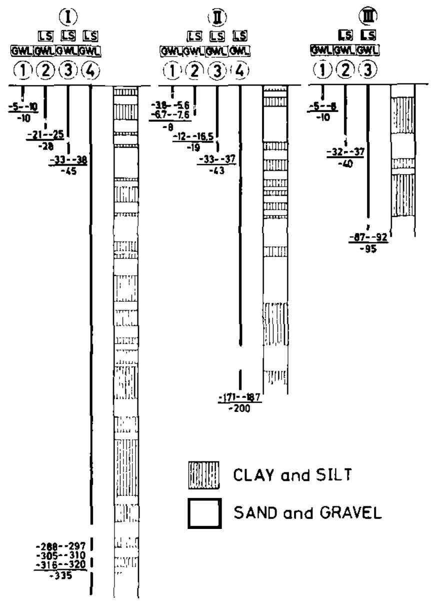


FIGURE 4. *Diagram of the observation wells at three stations*

(I) *The 1st station;* (II) *The 2nd station;* (III) *The 3rd station.*

(1) *The 1st observation well;* (2) *The 2nd observation well;* (3) *The 3rd observation well;*

(4) *The 4th observation well.*

GWL *Measuring apparatus of confined ground-water level equipped on a well;*

LS *Measuring apparatus of land subsidence equipped on a well;*

a—b / *c* — *Depth of strainer* / *Depth of well bottom* — *unit : metre*

3. OBSERVED RESULTS

Examples of the records are shown in figure 7. They are related to the ground-water level and the land subsidence at the 2nd and the 3rd observation stations near the coast. To these records, the tide curve of the same observational period (December 7-14, 1964) is affixed, so that its time scale coincides with that of the ground-water level record. The tidal station is about 200 metres away from the 2nd observation station and is controlled by the Yokkaichi Harbour Bureau (see figure 2). By careful examination of these records, we find that oceanic tides exert an influence on the variation of the land-surface level as well as on the ground-water level.

In order to understand the influence of oceanic tides more clearly, the values per hour of the ground-water and the land-surface levels in February 1964 and August of the same year read from the records at the 4th well of the 2nd observation station are indicated in figures 8 and 9. In a similar way, the values per hour of the tide curve for the same periods are shown on these figures. By glancing at these observed curves, we find that oceanic tides have a great effect upon the variation both of ground-water and of land-surface levels. For the purpose of examining the other effects—except for oceanic tides— the hourly values of the ground-water and the land-surface levels at the 2nd, 3rd and 4th wells of the 1st observation station which are less influenced by oceanic tides because of their situation away from the coast, and the hourly values of the air temperature, the atmospheric pressure and the sunshine periods at the Tsu Meteorological Observatory,

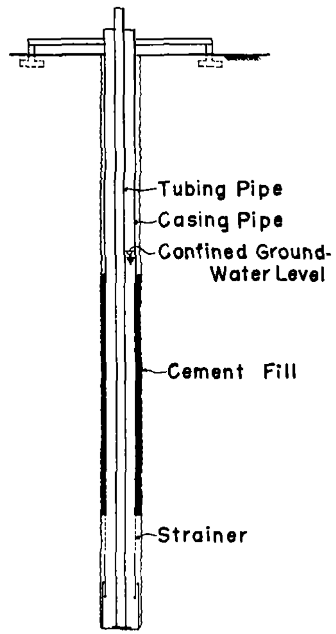


FIGURE 5. *Sketch of the observation well*

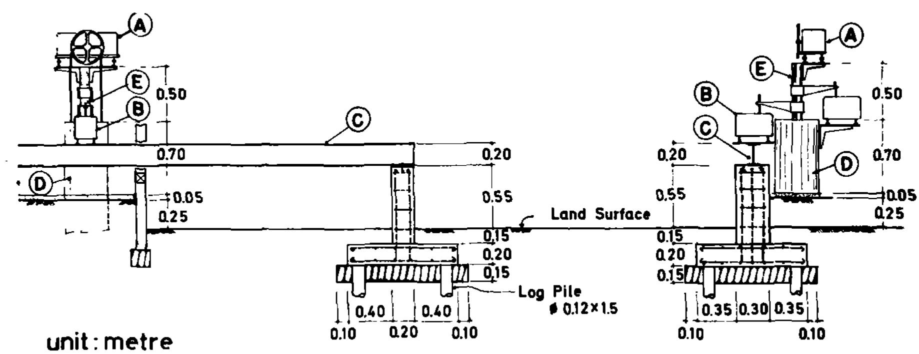


FIGURE 6. *Upper construction of the observation well*

(A) *Measuring apparatus of confined ground—water level;*

(B) *Measuring apparatus of land subsidence;*

(C) *Steel beam, I − 200 × 100 × 7 mm, L = 6.0 m;*

(D) *Casing pipe, EX. DIA. 406.4 mm, 7.9 mm* thick;

(E) *Tubing pipe, EX. DIA. 101.6 mm, 4.2 mm thick.*

situated about 30 kilometres southwest of Yokkaichi, are also shown in figures 8 and 9. Furthermore, in these figures, the daily amount of precipitations observed at the Kita Hamada Primary School, situated in the regulated area for removal of ground water, and the daily amount of discharge of the industrial using wells in the regulated area are given.

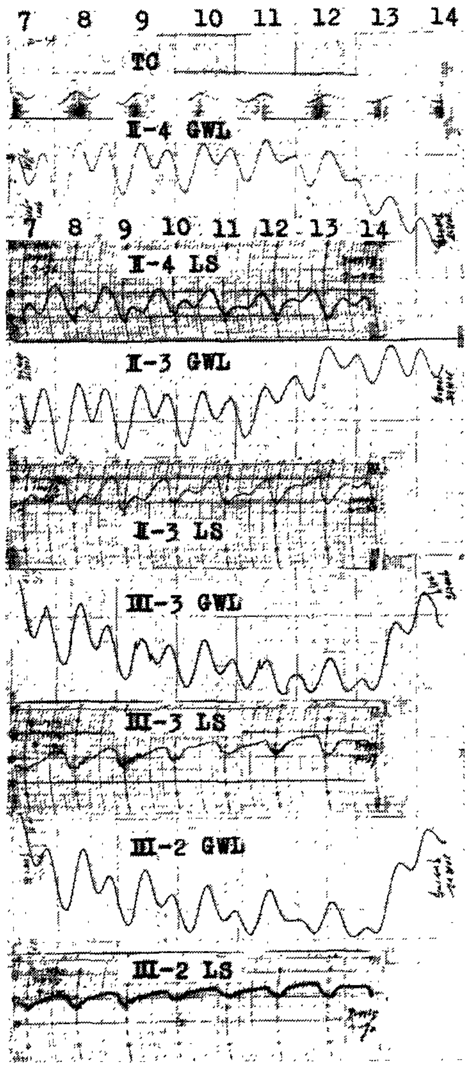


FIGURE 7. *Example of records for the period 7-14 December, 1964*

By considering these figures, the conclusion is reached that the atmospheric pressure and the amount precipitations has hardly any direct influence upon the variation of the land-surface as well as of the ground-water levels. However, as shown in figure 10, the daily amount of precipitations has an influence upon the ground-water level in the first well, the shallowest one, at the first observation station. Furthermore, in figures 8 and 9, the land surface shows a fluctuation during a period of about one day, which seems to be affected by the air temperature and concerned by the sunshine duration, or the solar radiation.

The variation both of the land-surface level at the 4th well and of the ground-water level at the 3rd well depends particularly on the amount of discharge, with periods of about one day and one week.

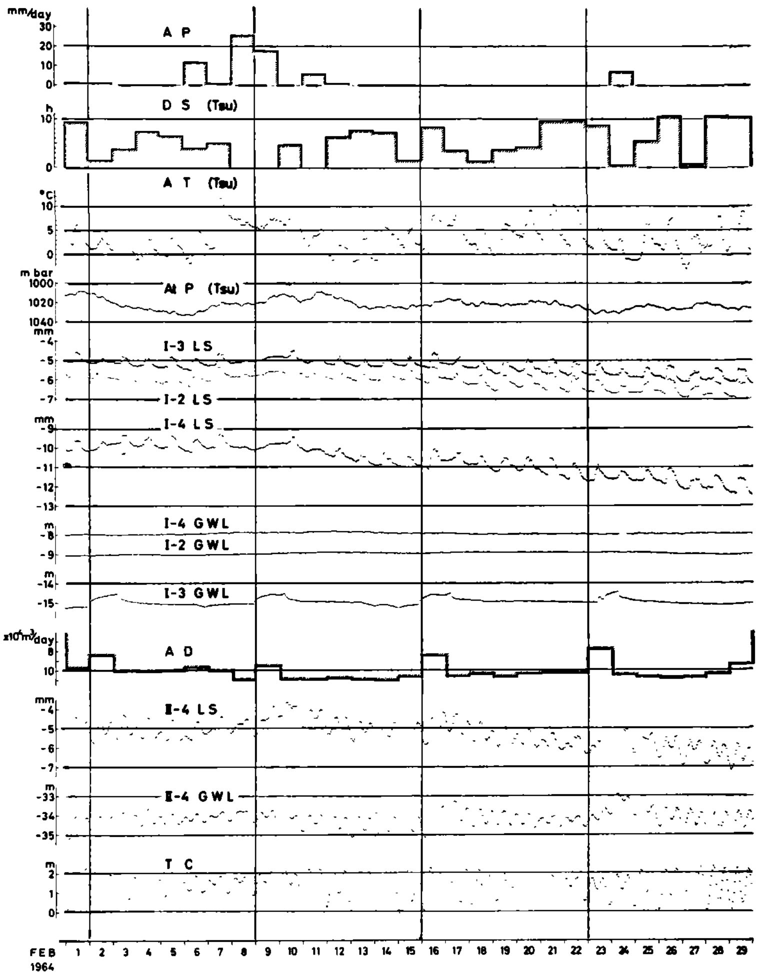


FIGURE 8. *Observed variation during February 1964 concerning ground-water level* GWL *and land subsidence* LS *at the 2nd, 3rd and 4th observation wells of the 1st station and at the 4th observation well of the 2nd station, daily amount of precipitation* AP, *duration of sunshine at Tsu* DS (Tsu), *air temperature at* Tsu AT (Tsu), *atmospheric pressure at Tsu* At P (Tsu), *daily amount of discharge of industrial using wells* AD, *and tide curve* TC

4. ANALYSIS

As above mentioned, the observed variations of ground-water and land-surface levels are on the whole made up of two parts. One is a long secular variation period probably due to removal of ground water only, and the other is fluctuations disturbed by oceanic tides, meteorological effects and other factors with periods of about one day or less. In order to obtain the secular variation of ground-water and land-surface levels, these disturbances must be eliminated from the observed data. For the reason given in detail in another article [1], the secular variation curve was determined from all the hourly values by the method proposed B. P. Pertzev [5], which has been found to be higher satisfactory by many investigators [2-4].

Reading of records was done every hour for about one year at 0000 hours on January 1964 to 0700 hours January 4, 1965. Therefore for each observation 8864 hourly values were used in calculations, as far as there was no omission in recording. Calculations were carried out by an IBM 7090 electronic computer.

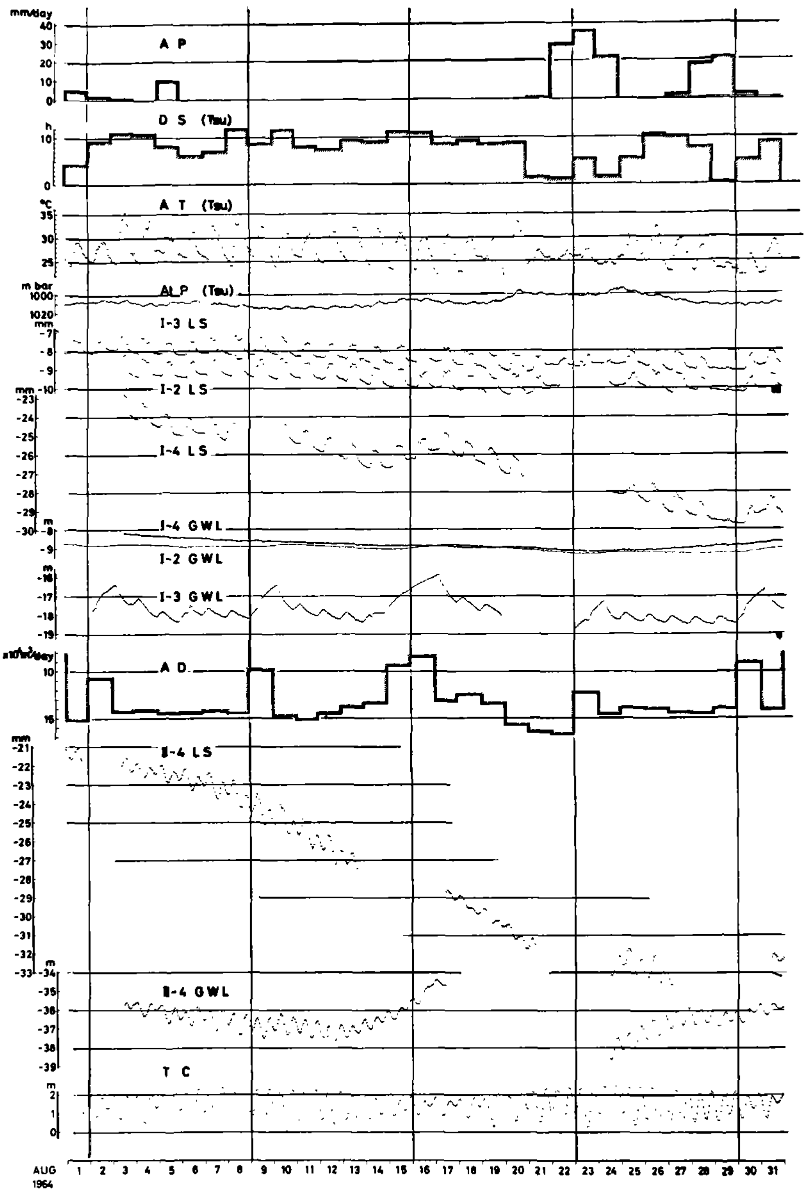


FIGURE 9. *Observed variation during August 1964*

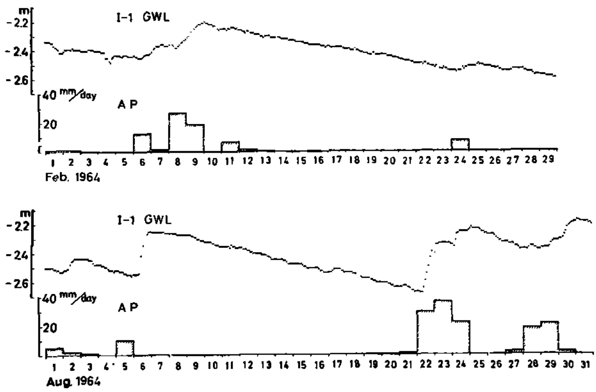


FIGURE 10. *Variation of the ground-water level in the 1st observation well at the 1st station and daily amount of precipitations*

5. DISCUSSION

Using the above-mentioned Pertzev method we obtained secular (hourly) values of both ground-water and land-surface levels for the period 1800 hours on 1 January, 1964 to 1300 hours on 3 January 1965. The secular variation curve of each observation was plotted by values at every 6 and 18 hours of these calculated ones. The secular variation curves concerning the individual well at the 1st, 2nd and 3rd observation stations are shown in

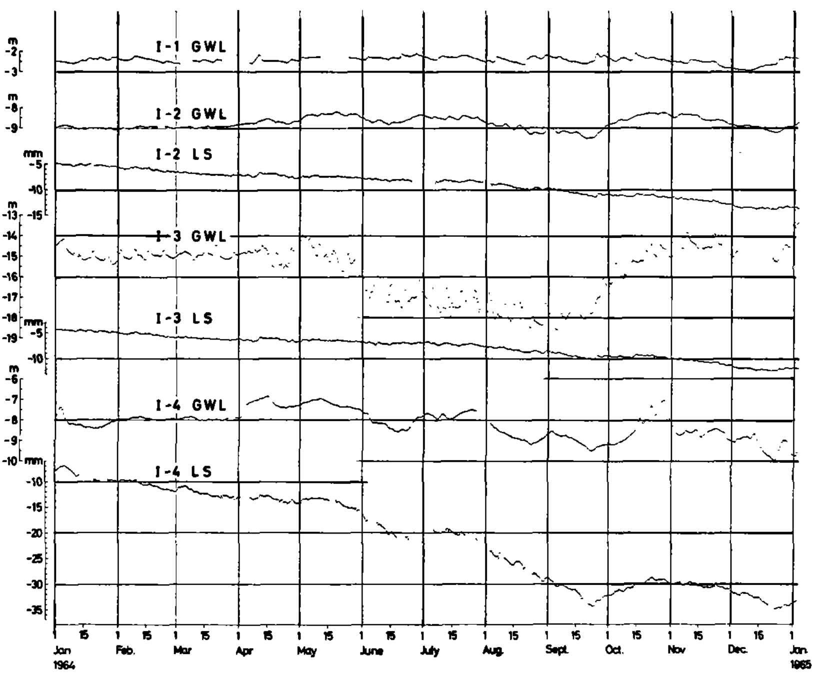


FIGURE 11. *Secular variations of both the ground-water level and the land surface at each observation well of the 1st station*

figures (11-13) respectively. Similarly, using hourly values of the tide curve, hourly values of its zero line, namely, of the mean sea level, were determined by the Pertzev method, the mean sea level being plotted by the values at every 6 and 18 hours, as shown in figure14. The other variation curves shown in this figure were drawn from the daily and weekly amount of discharge of industrial using wells in the regulated area, the atmospheric pressure values at every 3 and 15 hours, and the air temperature values at every 6 and 18 hours observed at the Tsu Meteorological Observaory. Moreover, the daily amount of precipitations observed at the Kita-Hamada Primary School, situated in the regulated area for removal of ground water, is given in the figure.

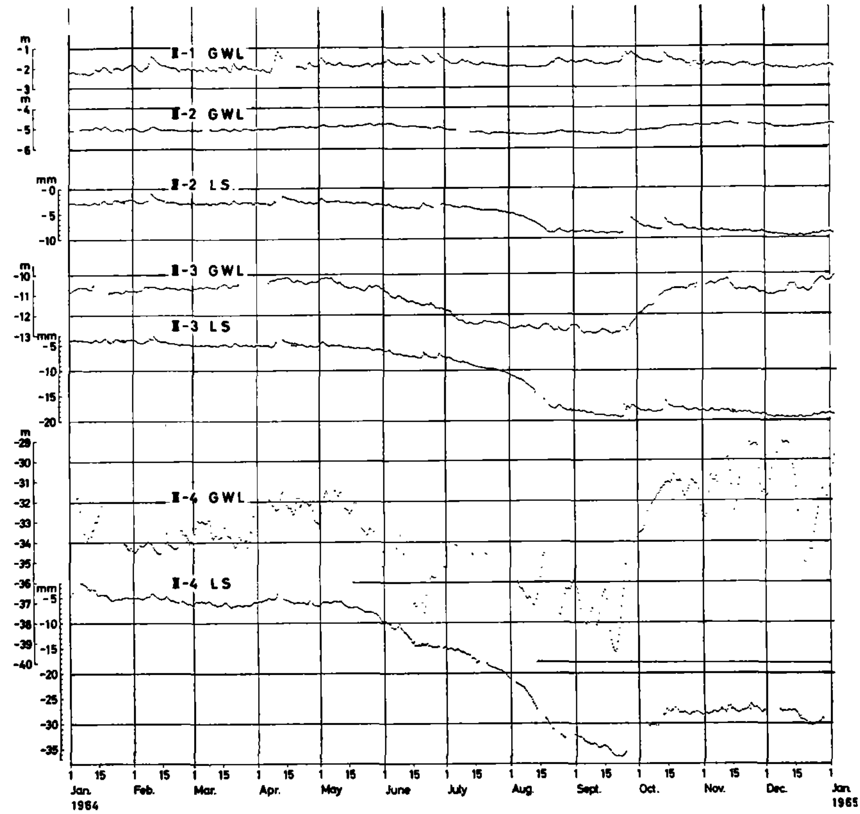


FIGURE 12. *Secular variations of both the ground-water level and the land surface at each observation well of the 2nd station*

The following can be mentioned as conclusions from the present investigations, the results of which are shown in figures 11-14.

1. It is likely that secular variations of both the ground-water and the land-surface levels have hardly any relations with the mean sea level. The mean sea level almost reverts to a former state for a one-year period in spite of the effects of atmospheric pressure, this seeming to have no influence on the secular variations of the land-surface and the ground-water levels.

2. It seems that the air temperature hardly has any direct influence upon the ground-water level or similarly upon the land subsidence, On the other hand, the air temperature

has a close relation with the amount of discharge of industrail using wells. For instance, there was a close relation between them when the air temperature suddenly dropped in the last week or so of September. At first sight, therefore, the air temperature seems to be concerned with the secular variation, but the variation of amount of discharge in fact intervenes between them. There is presumably no substantial relation between the air temperature and the secular variation.

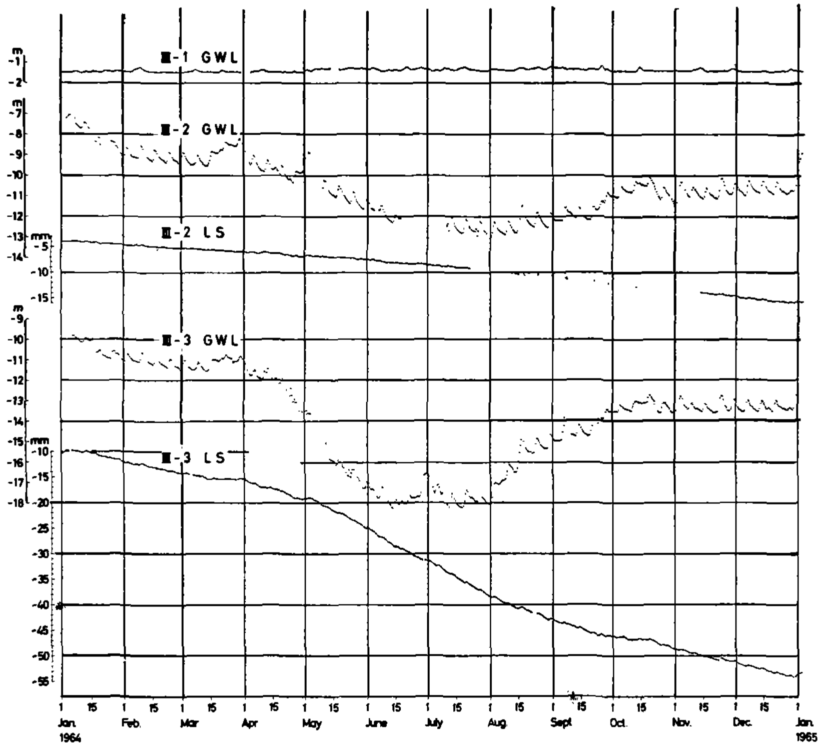


FIGURE 13. *Secular variations of both the ground-water level and the land surface at each observation well of the 3rd station*

3. The relation between the amount of precipitations and the variation of both the ground-water and the land-surface levels is more clearly seen by eliminating the effects of oceanic tides and other factors within a short period from the observed data. Nevertheless, the amount of precipitations has no influence upon the ground-water level in the deeper observation wells excepts for the first one. When we have heavy precipitations, the secular variation of the ground-water level usually rises in the first observation well at each station, especially at the second observation station, situated near the observation point of precipitation. Moreover, the precipitation has a slight influence upon the expansion of the surface layer above the bottom of the second observation well.

4. After various considerations of the results, it can be concluded that the amount of discharge of industrial wells exerts a great influence upon the secular variation of the ground-water and the land-surface levels. As can be easily seen from figures 11-14, the secular variation of the land-surface as well as of the ground-water levels may completely follow the amount of discharge, especially in the deeper wells.

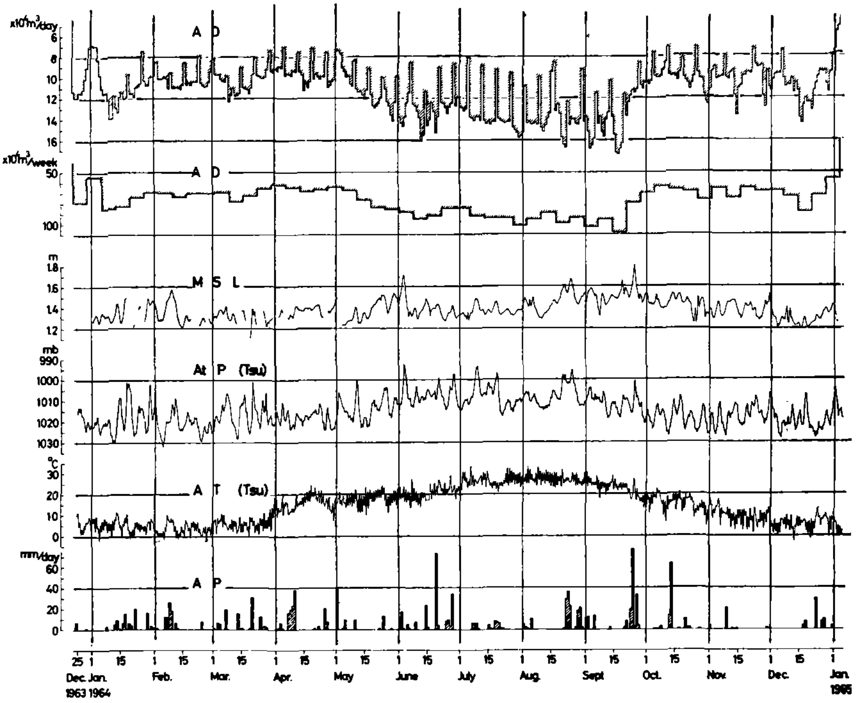


FIGURE 14. *Variations concerning daily and weekly amounts of discharge* AD, *mean sea level* MSL, *atmospheric pressure at Tsu* AT P (Tsu), *air temperature at Tsu* AT (Tsu), *and daily amount of precipitations* AP

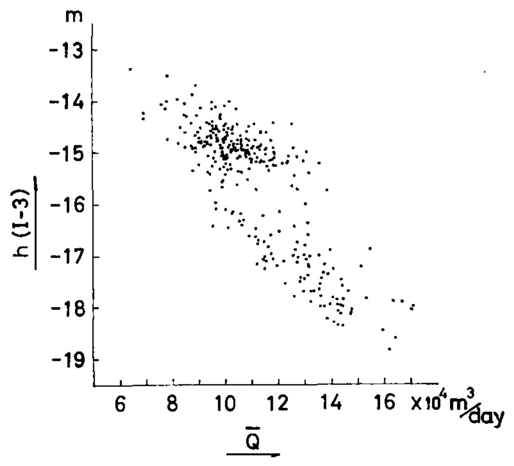


FIGURE 15. *Relation between the effective amount of discharge Q and the ground-water level h(I-3) in the 3rd observation well at the 1st station*

However, we find that a time lag of about two or three days on the influence of the amount of discharge upon the ground-water level. Judging from the fact that the observation record of the ground-water level is the highest at about 0600 hours and the lowest at about 2200 hours every day (not including) holidays, for a year, we assume that the

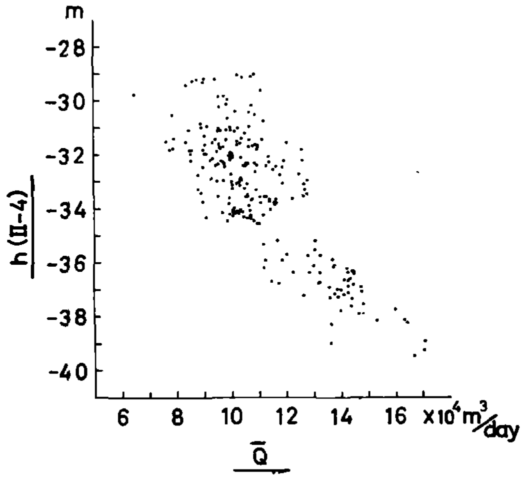


FIGURE 16. *Relation between the effective amount of discharge Q and the ground-water level h(II-4) in the 4th observation well at the 2nd station*

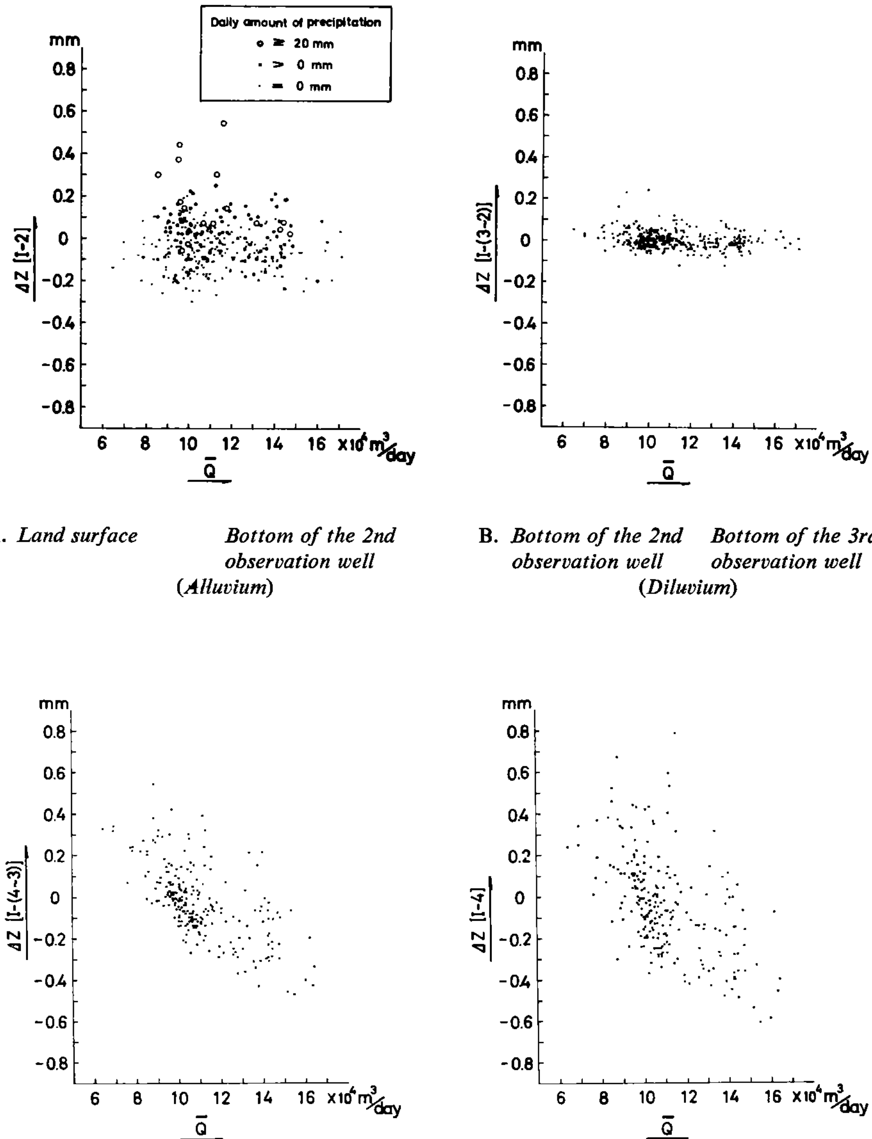
discharge in the regulated area starts at about 0600 hours and ends at about 2200 hours each day. Then, applying the Theis recovery method, we obtain the effective amount of discharge Q for one day, which is given by

$$\bar{Q} = 0.626\ Q + 0.231\ Q_{-1} + 0.143\ Q_{-2}$$

where Q, Q_{-1}, and Q_{-2} are the amount of discharge of the day treated, of the previous day and of the day before the previous one respectively. Figures 15 and 16 show the relations between the effective amount of discharge on one day and the secular value of the ground-water level at 0600 hours of the following day for a year, with regard to the third well of the first observation station and the fourth well of the second station. From these figures it becomes evident that there is a good correlation between the effective amount of discharge and the secular value of the ground-water level.

Similarly, with regard to the first and second observation stations, the relation between the effective amount of discharge of one day and the amount of expansion-compaction from 0600 hours to the same hour of the next day of each depth-range are plotted in figures 17 and 18. From a glance at these figures, we find that there is also a good correlation between the effective amount of discharge and the amount of expansion-compaction for the layers of the deeper depth-range, in which there are many strainers of industrial wells in the regulated area, as shown in figure 19. Furthermore, the surface layers above the bottom of the second observation well are influenced by precipitation.

On the basis of the results, it follows that the subsidence of land surface is likely to be caused by compaction of surface layers from which in fact ground water is withdrawn.



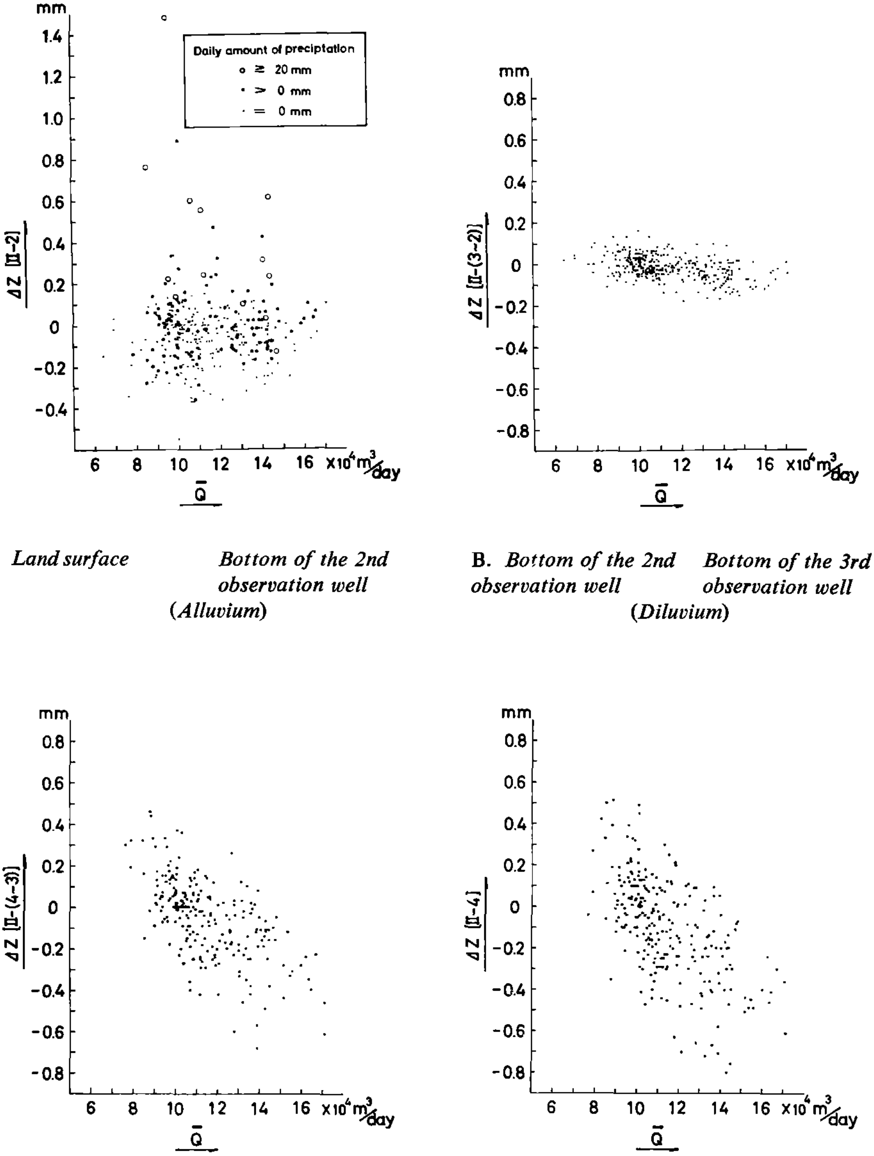
A. *Land surface* *Bottom of the 2nd observation well* (*Alluvium*)

B. *Bottom of the 2nd observation well* *Bottom of the 3rd observation well* (*Diluvium*)

C. *Bottom of the 3rd observation well* *Bottom of the 4th observation well* (*Tertiary*)

D. *Land surface* *Bottom of the 4th observation well*

FIGURE 17. *Relation between the effective amount of discharge $\bar{Q}$ and the amount of expansion-compaction of surface layers at the 1st observation station*



A. *Land surface* — *Bottom of the 2nd observation well* (*Alluvium*)

B. *Bottom of the 2nd observation well* — *Bottom of the 3rd observation well* (*Diluvium*)

C. *Bottom of the 3rd observation well* — *Bottom of the 4th observation well* (*Tertiary*)

D. *Land surface* — *Bottom of the 4th observation well*

FIGURE 18. *Relation between the effective amount of discharge $\bar{Q}$ and the amount of expansion-compaction of surface layers at the 1st observation station*

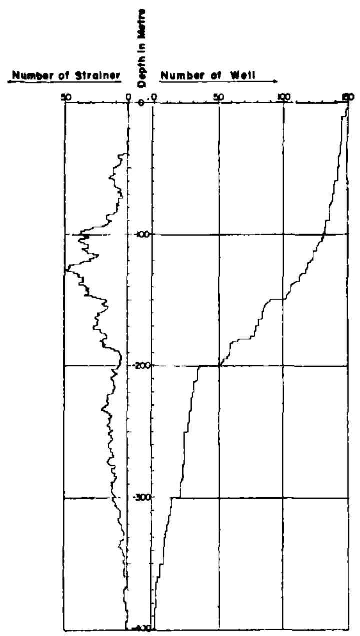


FIGURE 19. *Depth distributions both of industrial using well and of its strainers in the regulated area of ground-water withdrawal*

ACKNOWLEDGEMENTS

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FIELD MEASUREMENT OF AQUIFER-SYSTEM COMPACTION, SAN JOAQUIN VALLEY, CALIFORNIA, USA[1]

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ABSTRACT

Two types of field measurements have been successfully used to monitor aquifer-system compaction, and accompanying land subsidence in California: (1) periodic releveling of a network of surface bench marks, referenced to distant stable bedrock, and (2) continuous recording of vertical shortening of the water-bearing deposits, using extensometers installed in cased wells at selected locations.

The compacting deposits of the aquifer system are readily compressible and sensitive to change in overburden stress. Imposed hydraulic stresses, caused by the extracting of fluids, results in immediate strain in the aquifer system. Both elastic compression and inelastic rearrangement of the intergranular structure are caused by a stress increase; however, the elastic expansion during a stress decrease is small compared to ultimate plastic compaction for a comparable stress increase. The subsidence/stress-change ratio ranges from 0.5 to 10 $\times$ 10^{-2} feet of subsidence per foot of hydraulic stress change during the first prolonged cycle of water-level decline.

RÉSUMÉ

Deux types de mesure sur place ont été utilisés avec succès pour contrôler la compaction des nappes aquifères accompagnant les affaissements de terrain en Californie : (1) renivellement périodique d'un réseau de bornes-repères, par rapport à un lit de rochers éloigné et stable, et (2) enregistrement constant de la diminution verticale des alluvions détenteurs d'eau, en utilisant des extensomètres installés dans les trous de forages tubés, à des endroits déterminés.

Les alluvions compressibles des nappes aquifères sont facilement sensibles à tout changement de pression. Les pressions hydrauliques imposées, résultant de l'extraction des fluides, provoquent une tension immédiate dans les nappes aquifères. La compression élastique et la reconstitution non-élastique de la structure granulaire des couches sont causées par une augmentation de pression ; cependant, l'expansion élastique, lors d'une diminution de pression, est faible si on la compare à la compaction plastique définitive pour une augmentation équivalente de pression. La proportion affaissement/changement de pression varie de 0,5 à 10 $\times$ 10^{-2} pieds d'affaissement par mètre de changement de niveau de pression hydraulique pendant le premier cycle prolongé de diminution du niveau d'eau.

INTRODUCTION

Land subsidence affects 3,500 square miles of productive farm land in the San Joaquin Valley, California, as the intensive pumping of ground water continues. Locally, 28 feet of subsidence has occurred, and subsidence rates have exceeded 1.2 feet per year. Nowhere has man produced more extensive subsidence of this magnitude.

Subsidence was first recognized in the valley in 1935, when surveys discovered differential settlements in areas of intensive pumping. With the accelerated use of ground water for agriculture, particularly since World War II, subsidence has continued to the present. Today one-third of the entire valley is subsiding, and damage costs and remedial expenditures represent many millions of dollars. Damage caused by subsidence has been restricted principally to significant changes in gradients of canals, aqueducts and drainage systems, and the breakage of thousands of deep water-well casings.

The San Joaquin Valley, the southern half of the Great Valley of California (fig. 1) is bounded on the east by the lofty Sierra Nevada, on the west by the Coast Ranges, on the

1. Publication authorized by the Director, U.S. Geological Survey.

south by the Tehachapi Mountains, and on the north it opens into the Sacramento Valley. The valley is a northwest-trending structural trough that has accumulated thick sedimentary sequences from the surrounding mountains. Although basement rocks underlie the valley floor at great depth, the effects of ground-water pumping are limited to the upper few thousand feet of unconsolidated alluvial, lacustrine, and marine deposits.

AREA AND NATURE OF SUBSIDENCE

Within the San Joaquin Valley (fig. 1), subsidence is concentrated in the southern part and west side of the valley where annual rainfall is sparse and ground-water recharge is minimal.

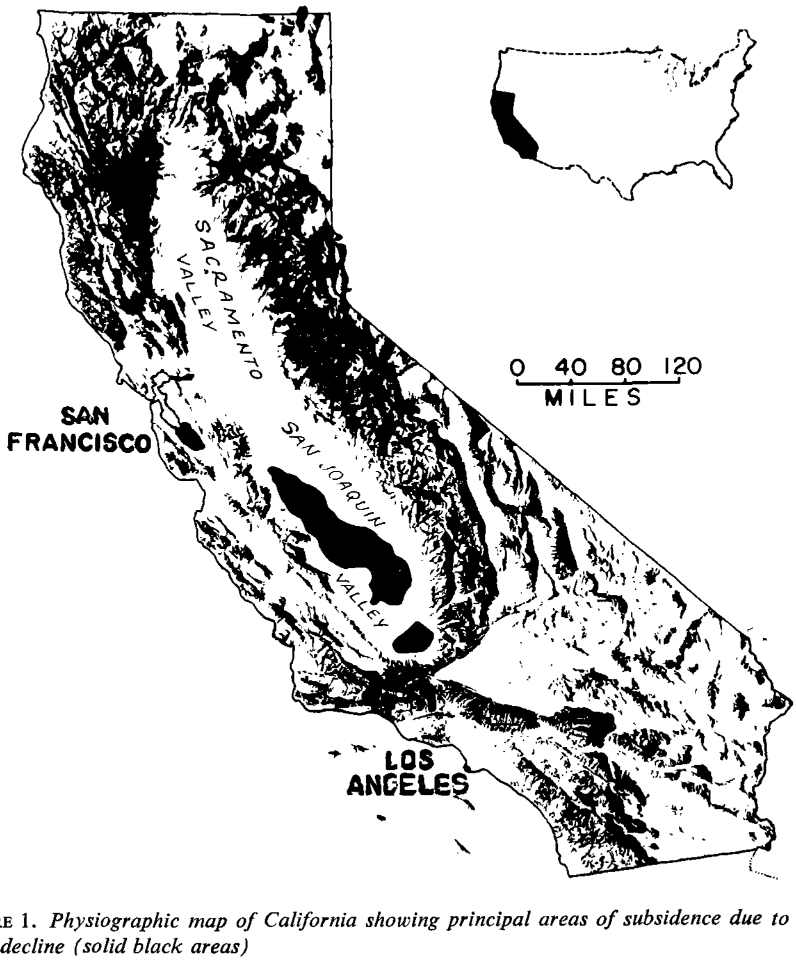


FIGURE 1. *Physiographic map of California showing principal areas of subsidence due to ground-water decline (solid black areas)*

Figure 2 shows the magnitude and areal extent of subsidence during extended periods of available record. The subsidence has been greatest in three areas. Maximum subsidence is an elongate trough close to the mountains west of Fresno, where more than 20 feet of subsidence occurred between 1920 and 1963 and total subsidence to 1969 is about 28 feet. A second center is 30 miles south of Tulare, where more than 12 feet of subsidence has occurred. The third center, south of Bakersfield, has subsided more than 8 feet. Although subsidence rates vary greatly areally and from year to year, subsidence continues in all areas except south of Tulare where surface-water imports have reversed the downward trends of water levels.

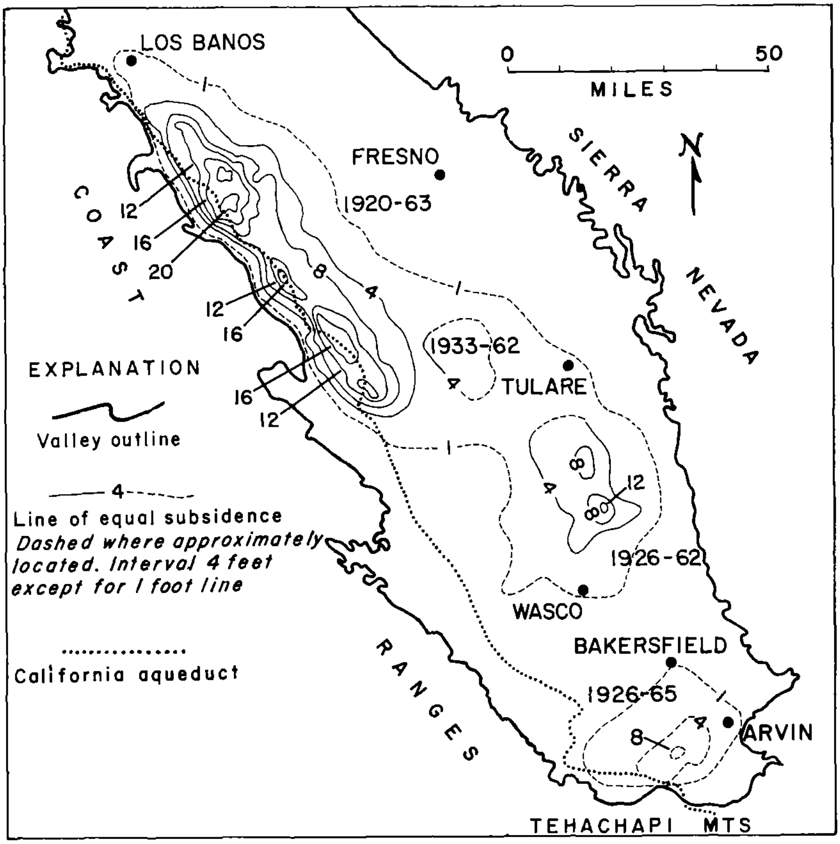


FIGURE 2. *Map showing the magnitude and areal extent of subsidence in southern San Joaquin Valley*

Most of the irrigation water within the subsidence areas is pumped from the ground-water reservoirs. Thousands of deep irrigation wells, many 1,000 to 2,500 feet deep and each pumping in excess of 1,500 acre-feet of water each year, have caused water levels to decline as much as 450 feet.

The relationship between subsidence and water-level decline near the center of subsidence west of Fresno is shown in figure 3. There, the excessive pumping of ground water began about 1940, and since that time water levels have declined about 300 feet. In this area, 1 foot of subsidence has occurred for each 16 feet of head decline. Throughout the area, this ratio varies from about 8 feet of water-level decline in areas of maximum subsidence to more than 25 feet of decline per foot of subsidence in perimeter areas.

Figure 4 shows a correlation of water-level fluctuations and subsidence 30 miles south of Tulare (fig. 2) since 1930. Ground water was intensively pumped in this area until 1951 at which time the importation of surface canal water for irrigation reversed the previous declining water-level trend. Pumping levels declined 235 feet from 1930 to 1951, causing nearly 9 feet of subsidence, for an overall rate of 1 foot of subsidence for 26 feet of water-level decline. Since 1951, water levels have risen to their pre-1930 levels, and land subsidence has virtually stopped. This is one of the few localities where water levels have recovered appreciably.

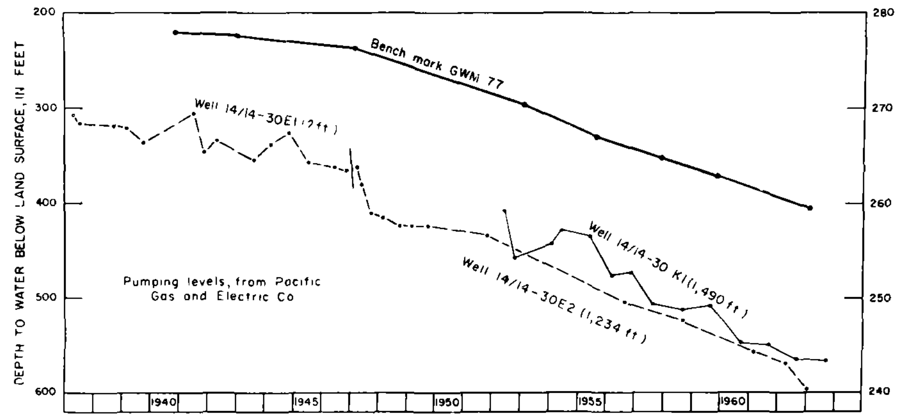


FIGURE 3. *Correlation between subsidence and change in artesian head near center of subsidence west of Fresno*

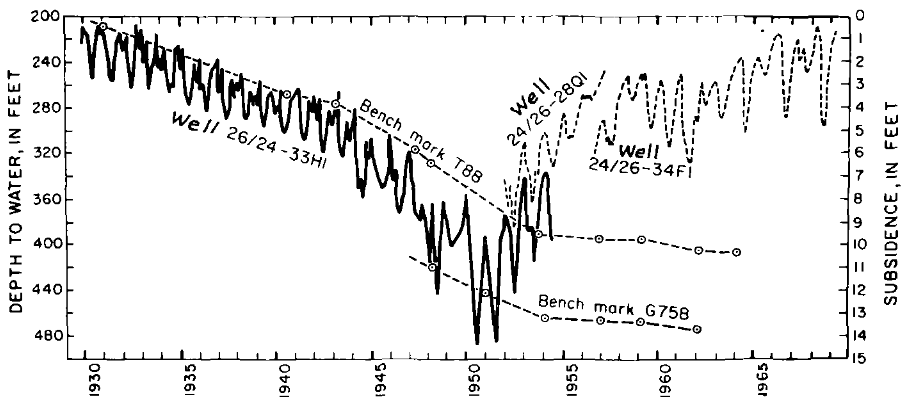


FIGURE 4. *Correlation of water-level fluctuations and subsidence near center of subsidence south of Tulare*

THEORY

Subsidence is attributed to the compaction of the compressible deposits of the aquifer system as intergranular stresses are increased by water-level changes. Effective stresses are changed in two principal ways (Lofgren, 1968): (1) Water-table fluctuations change the buoyant support of grains in the zone of the change, and (2) a change of the water table or of the piezometric head, or both, may induce hydraulic gradients and seepage stresses in the deposits. These stress changes are additive in their effect, and together cause compaction. In most of the subsidence areas, the changes in effective stress are roughly equal to changes in head in the principal aquifers of the confined aquifer system.

Compaction results from the slow escape of pore water from the stressed deposits, accompanied by a gradual transfer of stress from the pore water to the granular structure of the deposits. Effective stress increases and compaction occurs in a water-bearing bed only as rapidly as water can move out. A slowly draining aquitard may take weeks or years to adjust to an applied stress increase, whereas a coarse-grained aquifer will adjust quickly. In either case, one-dimensional consolidation results, which is directly related to the change in effective grain-to-grain stress.

Depending on the nature of the deposits, compaction may be (1) largely elastic, in which case stress and strain are proportional, independent of time, and reversible, or (2) principally nonelastic, resulting from a rearrangement of the granular structure in such a way that the volume of the deposits is permanently decreased. In general, if the deposits are coarse sand and gravel, the compaction will be small and chiefly elastic and reversible, whereas if they contain fine-grained clayey beds, the compaction will be much greater and chiefly inelastic and permanent. In either case, a one-directional compression of the deposits occurs which results in a subsidence of the land surface.

FIELD MEASUREMENTS

Two types of field measurements were initiated early in the subsidence study to monitor the effects of continuing changes in ground-water levels: (1) Periodic releveling of a network of surface bench marks, and (2) continuous record of compaction as obtained from specially designed compaction recorders. In addition, continuous water-level recorders were installed in selected observation wells to determine changes in applied stress causing subsidence. These field measurements are the most direct approach to obtaining the "stress-strain" parameters for subsidence occurring in the San Joaquin Valley.

RELEVELING OF SURFACE BENCH MARKS

Figure 5 shows the network of level lines laid out to traverse the subsidence areas of the San Joaquin Valley. Surface bench marks at about 1-mile spacing along these lines have been releveled at 2- to 6-year intervals by the Coast and Geodetic Survey, US Department of Commerce.

Bench marks within the subsidence areas are considered floating, and are necessarily referenced to distant unchanging bench marks. Primary circuits in the San Joaquin Valley are tied to stable bench marks set in bedrock around the perimeter of the valley (fig. 5). Insofar as possible, leveling has been done during the winter months of least subsidence.

From the comparative elevations of bench marks, the rate of subsidence of the various bench marks (figs. 3 and 4) and the areal extent and magnitude of subsidence between successive periods of leveling (fig. 2) are obtained. Also, profiles of subsidence along selected lines across the subsidence area (fig. 6) are an effective means of illustrating the magnitude and distribution of subsidence.

COMPACTION RECORDERS

Subsidence is caused by the compaction of the water-bearing deposits at depth, and continuous measurements of the magnitude and rate of subsidence at a location can be obtained by a specially designed bore-hole extensometer (fig. 7). About 30 of these recorders are now operating in the San Joaquin Valley. The equipment is simple in design and yet has proved highly successful and accurate in measuring the vertical shortening of the aquifer system (Lofgren, 1961).

The compaction recorder consists of a heavy anchor weight emplaced in the formation below the bottom of a well casing, with an attached cable stretched upward in the casing, and counterweighted at the land surface to maintain constant tension. A modified hydrographic recorder mounted over the open casing measures directly the amount of cable that appears above the casing as subsidence occurs. At the land surface, it appears as if the bottom-hole weight is rising; actually, the land surface is settling with respect to the bottom weight.

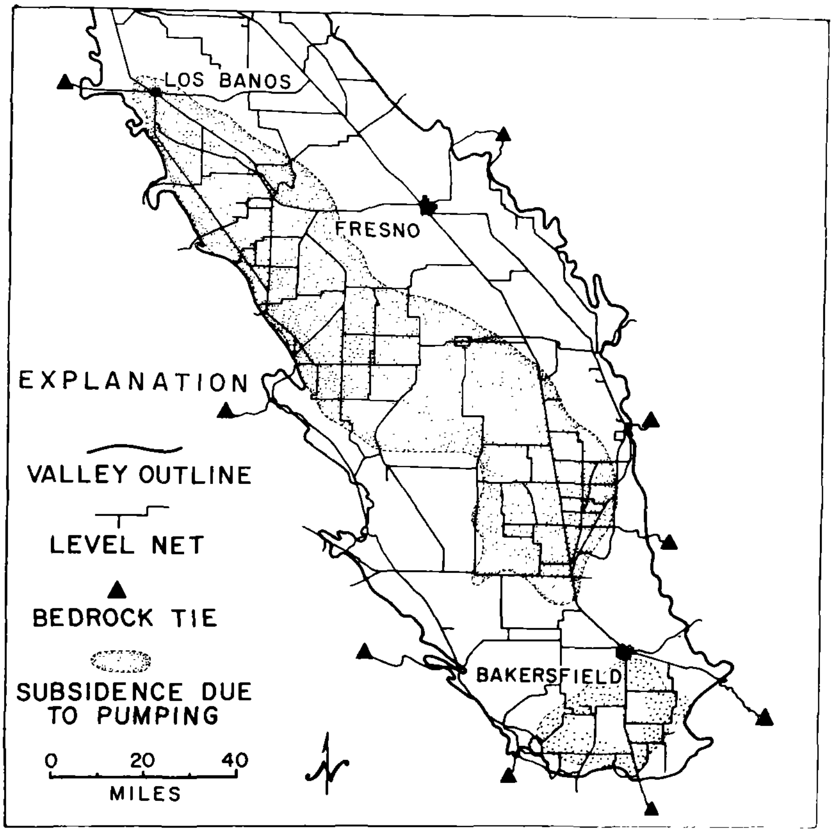


FIGURE 5. *Subsidence areas and level net in southern San Joaquin Valley*

The success of this recorder depends largely on the durability and stretch characteristics of the down-hole cable and on minimizing down-hole friction between the cable and the well casing. The cable must remain at constant length during the period of record. If the cable changes in length, due to temperature changes, fatigue elongation, or untwisting, this change is indistinguishable from the record of compaction. After considerable experimentation, a 1/8-inch, stainless steel, 1 × 19 stranded, reverse-lay cable was selected which meets the rigorous demands of the installation very well. Compaction recorders have been installed in unused irrigation wells, and also in specially drilled wells to depths as great as 2,200 feet.

In order to minimize frictional drag of the surface sheaves, a "teeter-bar" on a knife-edge fulcrum (fig. 8) has been designed. Changes as small as a few ten thousandths of a foot in thickness of the aquifer system can be detected with this equipment during both the compaction and expansion phases of the annual loading cycle.

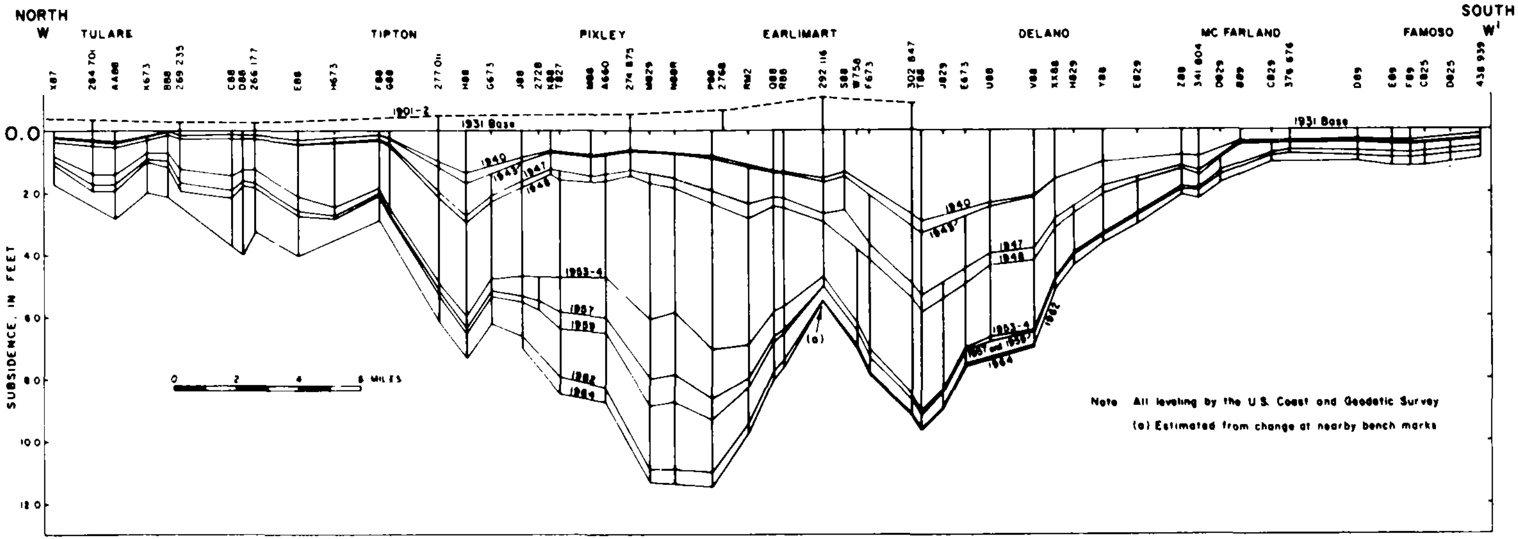


FIGURE 6. *Land-subsidence profiles along US highway 99 south from Tulare, 1901-2 to 1964*

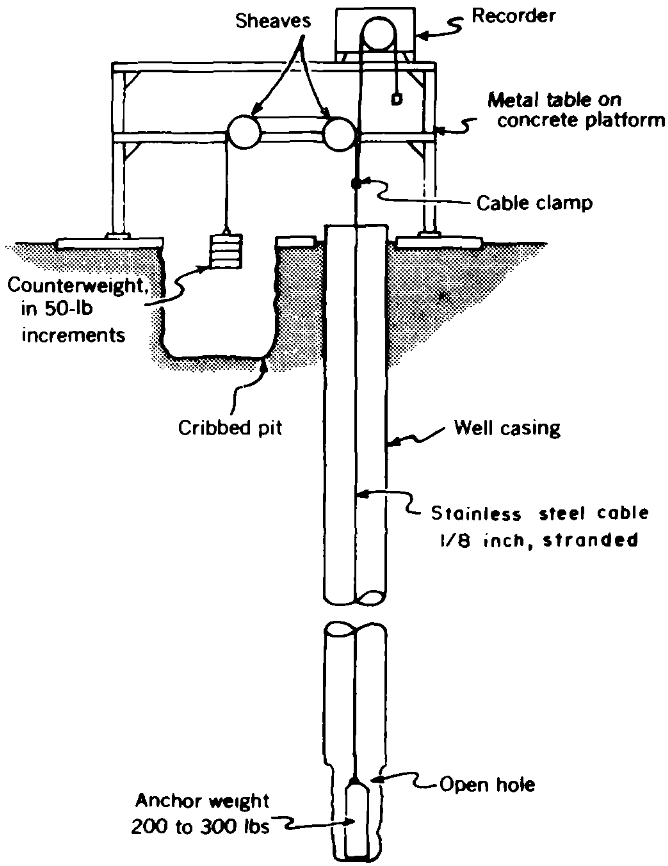


FIGURE 7. *Diagram of compaction-recorder installation*

COMPRESSIBILITIES OF DEPOSITS

Figure 9 shows the depth relationship of four compaction recorders installed at different depths in four wells near Pixley, at the center of the subsidence area south of Tulare. Each recorder measures the vertical shortening between the land surface and the subsurface anchor. By comparing the records of the four recorders, the magnitude and rate of compaction in each of four depth zones can be computed. Also, by knowing the total subsidence of the land surface, the amount of compaction occurring below the deepest anchor can be determined. The annual specific unit compaction rate in these deposits varied from 0.7 to 2.5×10^{-5} foot of compaction per foot of thickness per foot of water-level decline from 1960 to 1964 (Lofgren and Klausing, 1969).

As shown in figure 10, the deposits to a depth of 760 feet at the Pixley site south of Tulare (fig. 9) are intimately reponsive to water-level changes. The upper graph shows the water-level fluctuations in the 430-foot observation well tapping the confined aquifer system. The center graph is a 1:1 record of measured compaction to a depth of 760 feet, and the lower graph is a 24:1 amplification of this same compaction record. Of particular interest are the excellent correlation between compaction and water-level changes, and, also, the magnitudes of compaction and expansion that accompany changes in water

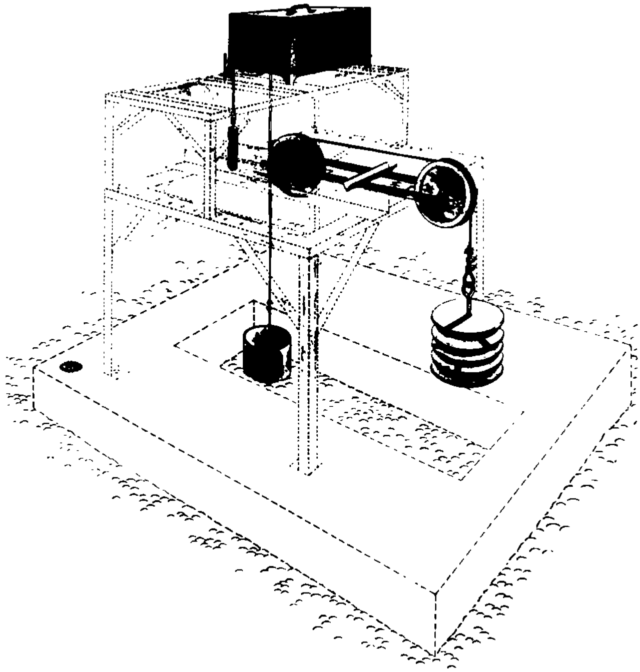
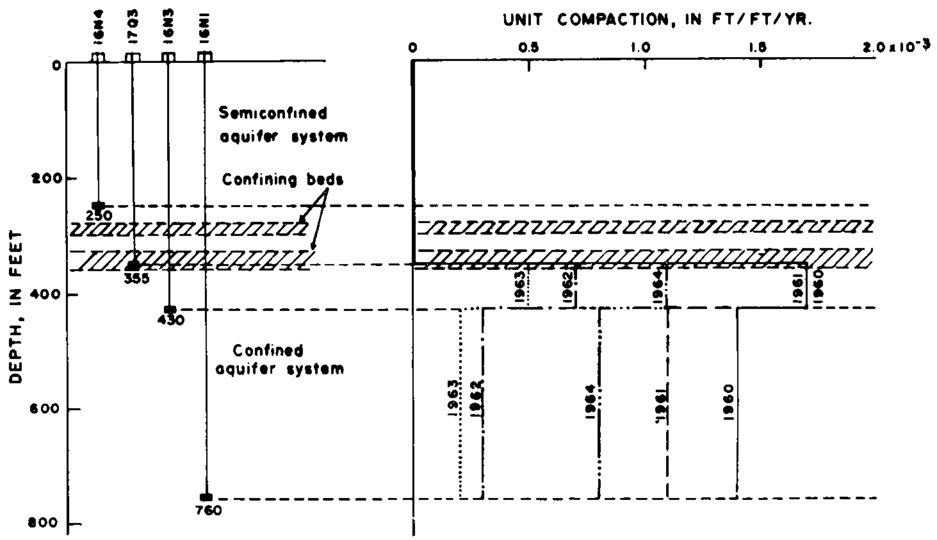


FIGURE 8. *Compaction-recorder assembly*



A. Relation of multiple recorders to the hydrologic units

B. Measured compaction rate in four depth zones for 5 years of record

FIGURE 9. *Annual rate of compaction of deposits in four depth zones in center of subsidence south of Tulare, 1960-1964*

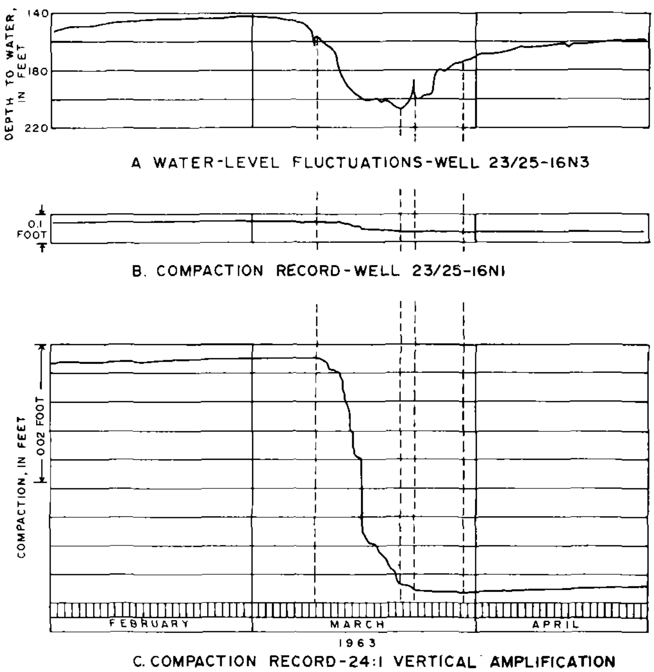


FIGURE 10. *Observed compaction and expansion south of Tulare*

level. During February, for example, a water-level rise of 11 feet resulted in a measured expansion of about 8×10^{-4} foot or about 7.5×10^{-5} foot per foot of water-level rise. From March 9 to March 21, a water-level decline of roughly 50 feet produced an abrupt compaction of 0.034 foot or 0.68×10^{-3} foot of compaction per foot of water-level decline. The delicate sensitivity of the aquifer system to changes in effective stress is readily apparent. Compaction frequently begins within minutes after pumping effects are observed in the observation well. Less than 5 percent of the compaction is elastic in nature, and results in rebound when stresses are decreased. Most of the compaction is nonelastic, representing a permanent decrease in the volume of the stressed deposits.

Figure 11 shows the interrelationship between compaction, water-level fluctuations in the semiconfined (hydrograph, 16N4) and confined (hydrograph, 16N3) aquifer systems, and changes in applied stress at this same recorder site near Pixley for 9 years of record. During this period, measured compaction to a depth of 760 feet was 75 percent of the total subsidence. Thus, 25 percent of the vertical shortening was due to compaction below 760 feet.

Comparison of the compaction rate with fluctuations in water level in the semiconfined and confined aquifer systems indicates that compaction began each year during the period of rapid head decline in the aquifer system, continued through the pumping season, and ceased during the early stages of head recovery. Although the correlation between compaction and artesian-head change in the confined aquifer system is good, the correlation with changes in effective stress in the confined aquifer system is closer.

A special computer program has been written whereby the relationships of figure 11 are calculated and graphed by computer. Field data from the deep and shallow water-level and compaction recorders are supplied to the computer in digital form, and from these various stress-strain parameters are obtained.

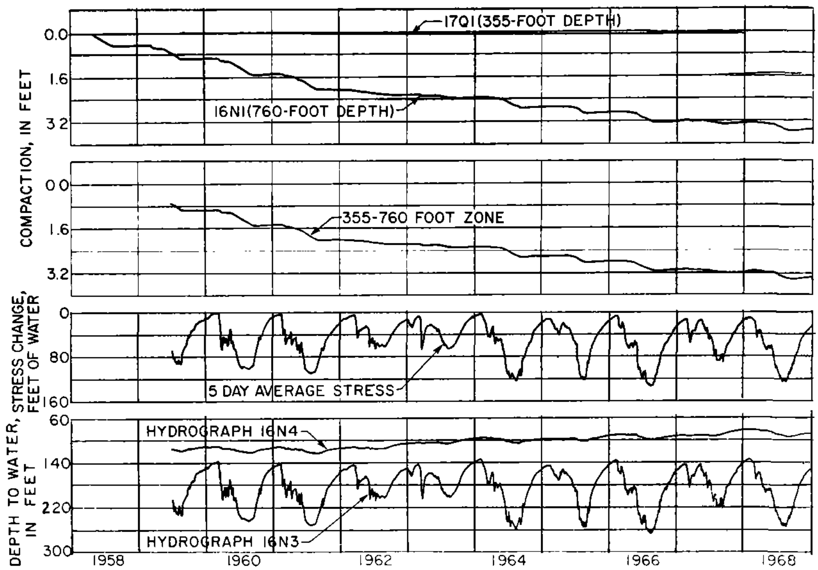


FIGURE 11. *Correlation of water-level fluctuations, change in effective stress, and compaction at the center of subsidence south of Tulare*

CONCLUSIONS

1. The extent and magnitude of subsidence in the San Joaquin Valley has been defined by repeated leveling over an extensive network of surface bench marks. Direct evidence that aquifer-system compaction is the cause of subsidence is obtained from compaction recorders, which give a continuous record of vertical compression and expansion of the water-producing strata.
2. Compaction recorders that span the full compacting section furnish a continuous record of land subsidence, and serve at any time as bench marks of known elevation for control in running local surveys.
3. Where several compaction recorders span different depth intervals at the same location, the compression characteristics of specific intervals are obtained. If changes of water level (stress changes) are also available, the stress-strain characteristics of the deposits, both the elastic and the plastic parameters, can often be derived (see Riley, 1969).
4. Stress-strain relationship derived from compaction and water-level recorders are the most specific parameters for estimating future subsidence.

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DISCUSSION

Intervention of Prof. George V. CHILINGAR (U.S.A.)

Question:

In your excellent paper you have been continuously using the term "compaction". Yet, many civil engineers violently object against using this term; they prefer the term "consolidation". To them, "compaction" has a different meaning. Would you please comment on this?

Answer of Mr. LOFGREN:

We use "compaction" as the decrease in thickness of sediments as a result of increase in vertical compressive stress. It is synonomous with "one-dimensional consolidation" as used by engineers. The term "compaction" is applied both to the process and to the measured change in thickness.

Intervention of Mr. Dennis R. ALLEN (U.S.A.)

Comment:

Regarding the definition and use of the terms "compaction" and "consolidation": the commonly used term in the oil business is "compaction", meaning either or both processes. The terms are sometimes confusing.

Intervention of Mr. Herbert H. SCHUSMANN (U.S.A.)

Question:

Have you observed surface fissures in the subsiding areas of California of the type associated with subsidence in the adjacent State of Arizona?

Answer of Mr. LOFGREN:

One fissure has opened up in the San Joaquin Valley similar to the Arizona fissures. As of six months ago, no surface fissuring was recognized in our subsidence areas. During March 1969, a half-mile long earth fissure opened up in a flooded area. The fissure represents a small graben in which the surface dropped 6 to 10 feet. The dropped block is roughly 10 feet wide. This is a very interesting feature and we are studying the hydrogeologic environment in detail.

Intervention of Dr. Naomi MIYABE (Japan)
Question:

Is there any change or difference in responses due to periodic or cyclical changes in the groundwater level?

Answer of Mr. LOFGREN:

Very good. There will be a paper tomorrow that discusses this question in detail. Paper No. 45 by Mr. Riley will be presented by Mr. Poland, and should be of interest in answering this question.

We find that the first time the water level is drawn down, rapid subsidence occurs because the stresses are great. If the water level recovers and then is drawn down a second time, the second application of that same stress results in much less compaction or subsidence. For the third and fourth stress applications the compaction is further decreased, and eventually all of the compaction for that load change will have come out of the system. This process is demonstrated for nine years of record of cyclic loading at Pixley. During the first cycle of loading, the compaction is largely nonelastic, nonrecoverable. On the last cycle, as we will see on tomorrow's slide, the compaction is mostly elastic. The delay for full compaction to occur is attributed to the slow drainage of the fine-grained deposits.

Intervention by Dr. Joseph F. ENSLIN (Republic of South Africa)

Question:

Is there a change in the specific yield of the ground-water reservoir caused by the subsidence?

Answer of Mr. LOFGREN:

Most of the compaction occurs in the fine-grained beds; very little occurs in coarse-grained aquifers. The amount of change in the overall system is quite small. The total storage characteristic of the system are not appreciably changed by subsidence.

Question of Dr. ENSLIN:

What is the initial storage coefficient and how much is it altered?

Answer of Mr. LOFGREN:

The coefficient of storage varies considerably throughout the valley. At Pixley, we have good confinement. We have a low value of coefficient of storage–in the general range of 0.003, as determined by short-term pumping tests. But during long periods of water-level decline, much of the water pumped comes from the permanent compaction of the beds. The long-term inelastic component may be 100 times the elastic coefficient. In other words, the water derived from the elastic response of the beds represents only 1 to 5 percent of the total water derived during a long-term drawdown cycle.

Question of Dr. ENSLIN:

But it does seem your subsidence is a function of the depth of the water-bearing deposits. You said the specific yield varies from 1/8 to 1/28. That is larger than your storage coefficient.

Answer of Mr. LOFGREN:

The thickness of the water-bearing section is an important element in the subsidence equation. Two factors must be considered. One is the compressibility of the material; the other is the total thickness undergoing a stress change. A system twice as thick as another, but with the same compressibility value, would yield twice the water of compaction, and result in twice as much subsidence, for a given water-level change. These values vary considerably throughout the valley.

LAND SUBSIDENCE AND AQUIFER-SYSTEM COMPACTION, SANTA CLARA VALLEY, CALIFORNIA, USA[1]

J. F. POLAND
U.S. Geological Survey, Sacramento, California

ABSTRACT

Intensive withdrawal of ground water from the confined aquifer system, 800 feet (240 m) thick, in the San Jose area of Santa Clara Valley, California, has drawn down the artesian head as much as 150 feet (75 m) since 1912. Resulting land subsidence, which began about 1915, was 12.7 feet (3.9 m) in 1967.

Periodic releveling of bench marks, core-hole data, and continuous measurements of water-level change and aquifer-system compaction have furnished quantitative evidence on the response of the system to change in applied stress. The adjusted subsidence-head decline ratio in San Jose for 1920-38 was about 1:10; this ratio can be utilized to estimate ultimate subsidence from subsequent head declines. Compaction records at some sites define the magnitude of excess pore pressures in aquitards; at one site, increasing artesian head by 46 feet (14 m) should stop the subsidence.

RÉSUMÉ

Une extraction considérable d'eau souterraine d'une nappe aquifère artésienne dont l'épaisseur atteint 800 pieds (240 m) dans la région de San Jose, Santa Clara Valley, California, a fait descendre le niveau de la pression artésienne de plus de 250 pieds (75 m) depuis 1912. L'affaissement du sol, qui en résulte et qui commença aux environs de 1915, était de 12,7 pieds (3,9 m) en 1967.

Des renivellements périodiques de bornes-repères, des données fournies par des carottes perforées de sondage et des mesures constantes du changement du niveau d'eau et de la compaction des nappes aquifères ont fourni des résultats quantitatifs sur la réaction des nappes à tout changement de pression appliquée. Le taux ajusté d'affaissement/diminution de pression à San Jose entre 1920-38 était d'environ 1:10 ; cette proportion peut être utilisée pour évaluer un affaissement définitif postérieur dû à des diminutions de pression. Des enregistrements de la compaction, à différents endroits, indiquent l'ampleur des pressions en excédent dans les pores des couches légèrement perméables ; à un endroit, un accroissement de la pression artésienne de 46 pieds (14 m) devrait arrêter l'affaissement.

INTRODUCTION

The Santa Clara Valley, south of San Francisco, Calif., has the distinction of being the first area in the United States where subsidence due to ground-water withdrawal was recognized and described (Rappleye, 1933). It was first noted when releveling in 1932-1933 of a line of first-order levels established by the US Coast and Geodetic Survey in 1912 showed about 4 feet of subsidence at San Jose. Land subsidence occurs in the central reach of the valley in an area of intensive ground-water development. Discussion in this paper pertains to this central reach, which extends southeastward about 30 miles from Redwood City and Niles to Coyote (fig. 1).

The alluvium-filled valley is a large structural trough lying between the San Andreas fault and the Santa Cruz Mountains on the southwest and the Hayward fault and the Diablo Range on the northeast. (See fig. 1). The alluvial fill includes the semiconsolidated Santa Clara Formation of Pliocene and Pleistocene age and the overlying unconsolidated alluvial and bay deposits of Pleistocene and Holocene age. This fill is as much as 1,500 feet thick; it is tapped by many hundreds of water wells to depths of 500-1,000 feet, and by a few wells as much as 1,200 feet deep.

1. Publication authorized by the Director, U.S. Geological Survey.

Fine-grained materials such as clay, silt, and sandy clay, which retard the movement of ground water, constitute the major part of the valley fill. Sand and gravel aquifers predominate near the valley margins where the stream gradients characteristically are steeper. A well-log section from Campbell north to Alviso (Tolman and Poland, 1940, fig. 3) indicates that, to a depth of 500 feet, the deposits at Campbell are about 75 percent gravel and 25 percent clay, but between Agnew and Alviso near the Bay, about 80 percent is clay. Between depths of 600 and 1,000 feet at Agnew, the deposits are about half clay and half gravel.

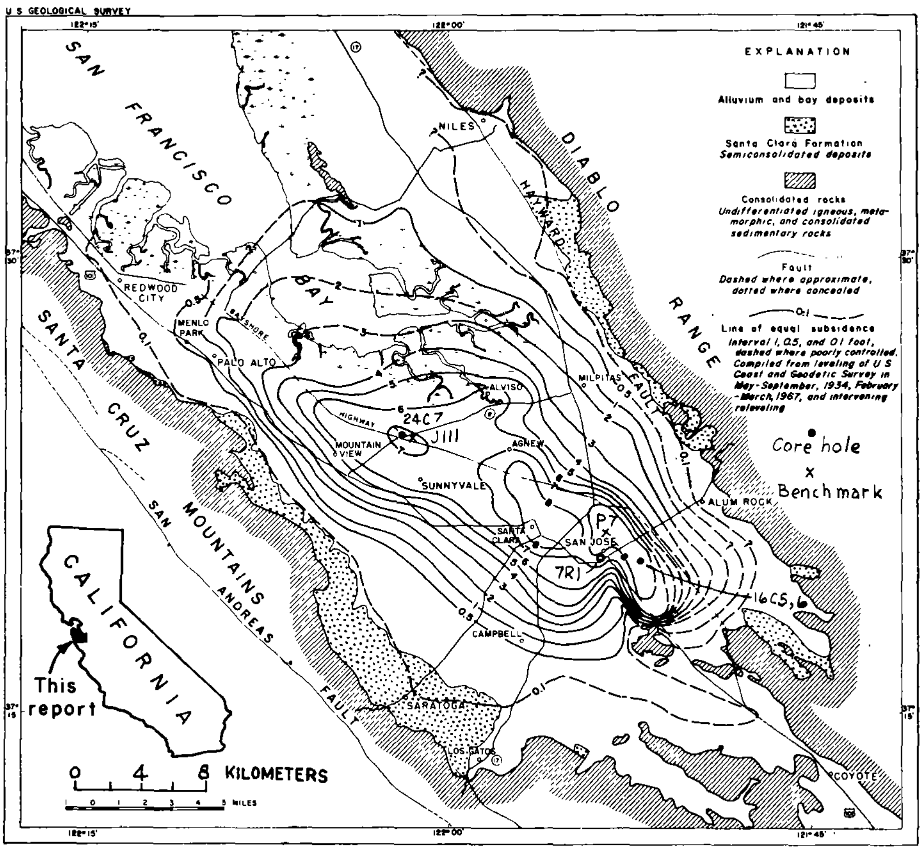

FIGURE 5. *Land subsidence from 1934 to 1967, Santa Clara Valley, California*

Below a depth of about 200 feet, ground water is confined by the clay layers, except near the margin of the valley where most of the recharge occurs. Initially, wells flowed as far south as San Jose. Within nearly two-thirds of the valley in the northern part of Santa Clara County, the principal aquifer system is confined.

Well yields in the valley commonly range from 500 to 2,500 gallons a minute. In the triangle between Campbell, Santa Clara, and San Jose, where the aquifer system has the highest transmissivity, the specific capacity of most wells exceeds 100 gpm per foot of drawdown (Calif. Dept. Water Resources, 1967, pl. 6).

DECLINE OF ARTESIAN HEAD

Extraction of ground water from the valley, in acre-feet per year, increased from about 40,000 in 1915 to about 180,000 in the 1960's. Until the middle 1930's, about 90 percent of the extraction was for agricultural use. However, rapid urban expansion since World War II has radically changed the pattern of water use. As a result, by 1967, pumpage for agricultural use had decreased to about 18 percent of the total, and 65 percent was used for municipal supply.

This increasing draft of ground water caused a fairly continuous lowering of the artesian head, which in 1915 was at or above land surface from San Francisco Bay south to San Jose. By the summer of 1965, the artesian head had been drawn down 150-200 feet below the land surface within most of the confined area.

The hydrograph for well 7S/1E-7RI, 840 feet deep, in San Jose (fig. 2) is a representative example of the decline of artesian head from 1915 to 1967. Except for the 75-foot recovery in 1936-1943, during a period of above-normal precipitation, the trend has been generally down, with an overall decline of 185 feet in the 52 years.

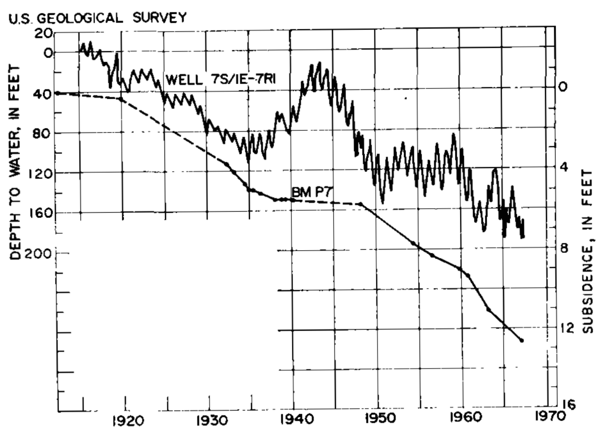

FIGURE 2. *Subsidence at bench mark P7 in San Jose and change in artesian head in nearby well*

LAND SUBSIDENCE

As a result of the excitement caused by the discovery early in 1933 of 4 feet of subsidence at San Jose, a network of level lines was established by the Coast and Geodetic Survey, in collaboration with C. F. Tolman and the writer, to span the area of known and suspected subsidence. This network consisted of a main Y extending north from Coyote to San Jose, with the two arms branching northward from San Jose to Redwood City and to Niles (Poland and Green, 1962, fig. 3). Bedrock ties were established at the terminals of the Y and also at the ends of several transverse lines crossing the valley and the two principal faults. The total length of this level net is about 350 miles. The net has been releveled 12 times; the latest relevelings were in 1960 and 1967.

Subsidence from 1934 to 1967 is shown in figure 1. The map was made by computing changes in altitude from 1934 to 1967, at several hundred bench marks, as determined by precise leveling of the Coast and Geodetic Survey, Department of Commerce. Subsidence in the 33-year period exceeded 8 feet in San Jose, and about 100 square miles subsided more than 3 feet. The volume of subsidence from 1934 to 1967 was about half a million acre-feet, equivalent to nearly 3 years of gross pumpage in the 1960's, and about 10 percent of estimated gross pumpage 1934-1967. Thus, about 10 percent of ground-water pumpage in the 33-year period was obtained from compaction of the confined aquifer system. This represents reduction in the pore volume of the ground-water reservoir but the reduction has been principally in the fine-grained compressible aquitards, and therefore should not affect appreciably the storage capacity or the other hydrologic properties of the sand and gravel aquifers.

The subsidence record for bench mark P7 in San Jose (fig. 2) reveals that subsidence in San Jose began before 1920 and increased to 5 feet by 1935. It virtually ceased from 1938 to 1947 during a period of artesian-head recovery. This recovery was the result of above-normal rainfall and recharge; furthermore, natural replenishment was augmented by controlled percolation releases from detention reservoirs constructed in 1935-1936 (Hunt, 1940). By 1948, artesian head had once more declined to the low levels of the middle 1930's and subsidence had recommenced. It reached its most rapid rate of 0.72 foot per year in 1960-1963 in response to the most rapid historic head decline of 1959-1962. By February 1967, subsidence at P7 was 12.7 feet.

Subsidence problems

This subsidence has created problems. Lands adjacent to San Francisco Bay have sunk 2 to 8 feet since 1912, requiring construction and raising of levees to restrain the saline Bay water, and flood-control levees near the bayward ends of the valley streams. From Palo Alto around the south end of the Bay are about 30 square miles of evaporation ponds for salt production. Behind the landward chain of dikes bordering these ponds, at least 17 square miles of land lie below the highest tide level of 1967. These lands currently are protected by the dikes and stream-channel levees, but the public cost to 1967 of levee construction due to subsidence was about $6 million, according to Lloyd Fowler, Chief Engineer of the Santa Clara County Flood Control and Water District. The subsidence has affected stream channels in two ways: Bay water has moved upstream and channel grades crossing the subsidence bowl have been downwarped. Both of these changes tend to cause channel deposition near the Bay and reduce channel capacity, thus creating the need for higher levees. Even though levee heights have been raised, flooding behind the Bay levees occurs at times of excessive runoff.

When the sediments in the confined aquifer compact to produce the subsidence, well casings are compressed and many have been ruptured. Protrusion of casings as much as 2-3 feet above land surface also has been observed (Tolman, 1937, p. 344). Several hundred well casings have been repaired and many new wells have been drilled to replace wells destroyed by compaction. Roll (1967) estimated the cost of this well repair and replacement as at least $4 million.

Core holes

In 1960, the Geological Survey drilled two core holes at the two centers of subsidence in San Jose and Sunnyvale (fig. 1), to a depth of 1,000 feet—the maximum depth tapped by nearby water wells. Cores were tested in the laboratory for particle-size distribution, dry unit weight, specific gravity of solids, porosity, permeability (vertical and horizontal), Atterberg limits, and consolidation and rebound (Johnson, Moston, and Morris, 1968). Meade (1967) determined the clay-mineral assemblage for 20 core samples by X-ray diffraction methods and found the average composition to be 70 percent montmorillonite, 20 percent chlorite, and 5-10 percent illite.

Compressibility of the fine-grained sediments

The compressibility of the fine-grained clayey sediments is a basic parameter in determining how much compaction and subsidence would occur ultimately in response to a given decline in artesian head. The compressibilities of 10 cores from the Sunnyvale hole, ranging in depth from 191 to 865 feet, have been computed from one-dimensional consolidation tests and are plotted in figure 3. This graph shows that at an effectivestress of 260 psi (18.3 kg per cm^2), the native stress at mid-depth of the aquifer system, the range in compressibility of the 10 samples is 2.3 to $3.4 \times 10^{-4} psi^{-1}$ (3.3 to $4.8 \times 10^{-3} cm^2$ per kg). The compressibility of these samples is about 3 times as great at 160 psi (11.3 kg per cm^2) (1960 effective stress at top of aquifer system) as at 490 psi (34 kg per cm^2) (stress at base of aquifer system). For the compressibility-effective stress log-log plots for the eight cores that are closely grouped in figure 3, the equation of the average compressibility-effective stress line is

$$m_v = 0.053\, p'^{-0.93}$$

where:

m_v is the coefficient of volume compressibility in psi^{-1}, and
p' is effective stress in psi.

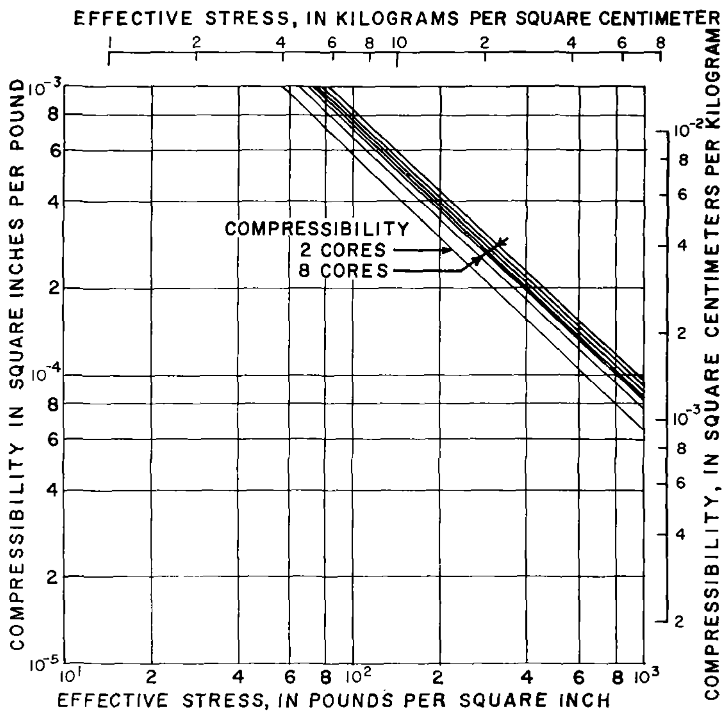

FIGURE 3. *Compressibility of fine-grained samples, Sunnyvale core hole*

When the compressibility curves of the fine-grained aquitards or aquicludes are as closely bunched as those at Sunnyvale, the approximate ultimate subsidence (Δz) for a given increase in stress can be computed, utilizing the equation

$$\Delta z = m_v m \Delta p'$$

where:

m_v is average compressibility,
m is aggregate thickness of compacting beds, and
$\Delta p'$ is change in effective stress.

However, because compressibility does not decrease linearly with increasing stress, and the fine-grained beds are not uniformly distributed, the well section should be divided into zones not more than 200 feet thick, and the average compressibility for each zone should be read from the plot, for the mean effective stress.

$$\frac{2p'_0+\Delta p'}{2}$$

where:

p'_0 is the initial effective stress at the midpoint of the zone, and
$\Delta p'$ is the increase in stress induced by artesian-head decline.

Compaction of the aquifer system

Compaction recorders of the type described by Lofgren (1970) have been operated in the core holes since 1960 to measure the magnitude and rate of compaction. Figure 4 shows the record of compaction at Sunnyvale through 1968, including compaction in the 1,000-foot core hole (well 24C7) and in two satellite wells 250 and 550 feet deep. It also shows artesian-head fluctuation in well 24C7 (casing perforated only in aquifers below a depth of 800 feet) and in irrigation well 25C1 (depth 500 feet). Subsidence of the land surface at nearby bench mark J111 from 1933 to 1960 was 5.73 feet, and estimated subsidence 1915-1960 was 8.7 feet. Subsidence of this bench mark from October 1960 toFebruary 1967,

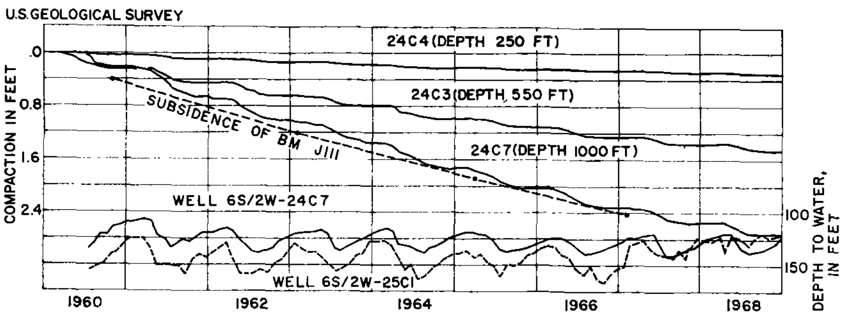

FIGURE 4. *Measured compaction, water-level change, and subsidence in Sunnyvale*

as measured by releveling of the Coast and Geodetic Survey, was 2.03 feet. Compaction of the aquifer system to the 1,000-foot depth as measured in 24C7 in the same time interval was 2.13 feet. The 5 percent excess measured compaction is attributed to instrumental error in the earlier years of operation. Surface releveling and measured compaction from April 1965 to February 1967 agreed within 0.01 foot. Therefore it is concluded that land subsidence at this site is due entirely to aquifer-system compaction to the 1,000-foot depth.

The unit compaction of the fine-grained beds at the Sunnyvale site, 1961-1968 inclusive, is tabulated below.

It is assumed that the measured compaction occurred entirely in the fine-grained clayey beds, which here constitute 88 percent of the confined aquifer system. The unit compaction, 1961-1968, in the deepest interval was about 60 percent of that in the uppermost interval, 168-250 feet below the land surface.

Figure 5 shows the measured compaction in the San Jose core hole (well 16C6) and the compaction and artesian-head fluctuation in nearby unused well 16C5 (depth 908 feet)

Well	Depth (feet)	Total Compaction 1/61-12/68 (feet)	Depth interval (feet)	Compaction (feet)	Fine-grained beds mickness in interval (feet)	Fine-grained beds unit compaction (feet)
24C7	1,000	2.50	550-1,000	1.32	432	3.05×10^{-d}
24C3	550	1.18	250- 550	0.92	244	3.75×10^{-d}
24C4	250	0.26	*168- 250	0.26	52	5.00×10^{-d}

* Midpoint of upper confining bed is 168 feet.

through 1968. Total measured compaction of the aquifer system from July 1, 1960, to December 31, 1968, was 4.42 feet. The rapid decline of artesian head from 1959 to 1962 (60 feet) caused rapid compaction of the aquifer system in those years but the rate decreased during the relatively consistent head fluctuation from 1962 to 1967. The 30-foot recovery of head in the spring of 1968 above the 1967 high caused a net expansion of the aquifer system (0.06 foot) for the first time since the compaction recorders were established in 1960.

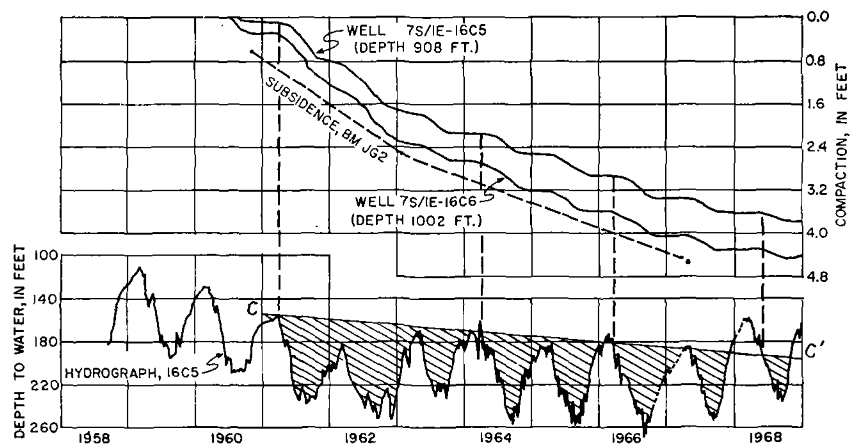

FIGURE 5. *Measured compaction, water-level change, and subsidence in San Jose*

A stress-strain plot of head versus compaction (not shown) indicates that response was entirely elastic for head fluctuations in the 160-190-foot depth range. The slope of the elastic-response line indicates that the component of the storage coefficient attributable to elastic response of the confined aquifer system skeleton (depth interval 200-1,000 feet) is 1.25×10^{-3}. Thus, the component of specific storage attributable to elastic response of the confined system skeleton is $1.6 \times 10^{-6} ft^{-1}$ ($5.25 \times 10^{-6} m^{-1}$) and the gross elastic compressibility is $3.7 \times 10^{-6} psi^{-1}$ ($5.25 \times 10^{-5} cm^2$ per kg).

Response of the system was wholly elastic for artesian-head change above the 190-foot depth to water in 1968. Therefore, maximum excess pore pressures in the aquitards must have been completely eliminated when the head in the aquifers rose to 190 feet below the land surface. Utilizing the compaction records for 1961, 1964, and 1966, when compaction stopped at peak winter rise, a line has been drawn on the hydrograph of well 16C5 to define the approximate depth to water at which excess pore pressures were eliminated (line C-C′). The shaded area beneath this line defines the variation in magnitude of maximum excess pore pressures in the aquitards from 1961 to 1968. As of 1968, a net rise of

46 feet in the summer low water level would eliminate all permanent compaction and stop land subsidence.

The response of the system shown by the long-term subsidence graph of bench mark P7 and the head change in nearby well 7Rl (fig. 2) can be utilized to estimate ultimate subsidence from head change since 1938. From 1920 to 1938, when subsidence stopped, 50 feet of net head decline from about 30 to 80 feet below land surface caused 5 feet of subsidence. The gross plastic-plus-elastic compressibility of the 800-foot thickness of the compacting aquifer system derived from this stress-strain relation is 5 ft/800 ft $\times$ 21.6 psi or $2.9 \times 10^{-4}\mathrm{psi}^{-1}$ ($4.1 \times 10^{-3}\mathrm{cm}^2$ per kg). The subsidence/head decline ratio for the period from 1920 to the steady-state conditions of 1938 (nocom paction) was 5.0/50 or 1:10. The subsequent head decline of 100 feet below 1938 levels by 1967 had produced 7.3 feet of additional subsidence, but decay of excess pore pressures in the aquitards was causing continued subsidence. The product of the gross compressibility (derived from the1920-38 step change), the aquifer-system thickness, and the change in applied stress (100 feet of decline) is 10 feet, suggesting that roughly 2.7 feet of residual compaction and subsidence would occur at this site if artesian head remained at a depth of 180 feet. Obviously, an estimate so derived is only a rough approximation, because the assumption is made that gross compressibility would remain constant under the additional stress of 43 psi. At mid-depth of the system (600 feet), the reduction in plastic compressibility due to this increase in stress would be about 10 percent. It is of interest to note that the computed gross plastic-plus-elastic compressibility at bench mark P7 is 78 times as large as the gross elastic compressibility at well 16C6.

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DISCIUSSION

Intervention of Prof. George CHILINGAR (USA):

Question:

I believe that your contribution to our knowledge on subsidence and compressibilities of clays is indeed great. I wish that you would describe the equipment used and calibration techniques in your paper. This will enable us to better evaluate your compressibility values. As you know, friction presents a big problem. Listening to your excellent paper, I got the following idea: Compressibility formulas (as a function of pressure) can be determined directly in the laboratory. One can also determine compressibility formulas from void ratio versus pressure curves on assuming a density for a clay. Does this present a possibility of determining density of clays indirectly, which defies direct determination?

Answer of Mr. POLAND:

All the consolidation tests were made by the Bureau of Reclamation, Department of the Interior, at their Earth Laboratory in Denver, Colorado. Their test prodecures are des cribed in their Earth Manual (1960). The tests were made on cores trimmed to fit 2-inch rings. Results of the tests are reported in U S Geological Survey Professional Paper 497-A (1968), with a brief description of how they were made.

The log-log plots of compressibility vs. effective stress shown in figure 3 were computed from the curves of void ratio vs. log of pressure obtained from the one-dimensional time-consolidation tests made in the laboratory. Therefore, they are subjects to the same assumptions and limitations as the e-log *p* plots.

Question of Prof. CHILINGAR:

How do our laboratory results on compaction of API standard clays compare with yours?

Answer of Mr. POLAND:

As I have indicated, each one of these compressibility curves, the straight lines on the log-log plot (fig. 3), was obtained from the plot of void ratio versus the log of pressure that is, the results of the consolidation tests. Your laboratory results for kaolinite and illite fall below these lines (those standard clays are less compressible), but for montmorillonite they plot above (standard montmorillonite is more compressible).

Intervention of Dr. Joseph E. ENSLIN:

Question:

Are the annual rises of the water table effected by artificial recharge or natural recharge?

Answer of Mr. POLAND:

The rise of the artesian head in the winters of 1967 and 1968, which is shown in that last graph, was due in part to above-normal precipitation and runoff. In addition, watcr is now being imported from the Central Valley, which decreases the pumping demand. Also, the local agencies have constructed detention reservoirs which store run-off for slow release down stream channels, in order to put more water underground. Thus, the winter increase in storage is due to both natural and artificial recharge.

Question of Dr. ENSLIN:

What percentage of the total subsidence is caused by the first, second, or any subsequent lowering of the water table through a specific depth zone?

Answer of Mr. POLAND:

Very little subsidence has occurred near the valley margins where a water table exists at the upper surface of an unconfined water body. The subsidence occurs in the area where the aquifer system is confined, and is due chiefly to the seepage stresses developed by the decline of artesian head. The artesian head (piezometric surface) in San Jose has been lowered through the same depth range in two periods. The subsidence per foot of head decline was about 40 times as great during the first drawdown phase (1915-1934) as it was during the second (1943-1948), showing that inelastic consolidation of the clayey aquitards in response to the increased applied stress was almost completed during the first drawdown phase.

Question of Dr. ENSLIN:

I would like to know what part of your aquifer storage is lost by compaction.

Answer of Mr. POLAND:

What has been lost is primarily due to compaction of the clay interbeds.

Question of Dr. ENSLIN:

How much is that? 20% or 30% of your storage? If you used it over and over again.

Answer of Mr. POLAND:

A reduction of about 2% in the porosity of the confined aquifer system, say from 40 to 38 percent, would account for the full subsidence.

Question of Dr. ENSLIN:

That is, if you in future are to recharge all these aquifers by outside water, then you will still have a very appreciable aquifer left.

Answer of Mr. POLAND:

Oh yes, so far as the aquifers themselves are concerned (the permeable beds), there has been very little compaction, so that the usable storage capacity of the aquifer system is affected very little by this actual subsidence.
Intervention by Dr. Manuel N. Mayuga (USA).

Question:

How much rebound did you observe as the aquifer were recharged by heavy rainfall as shown in Slide No. 2?

Answer of Mr. POLAND:

During the rapid artesian-head recovery of the early 1940's shown in figure 2, no levelling was done because it was a war period. Therefore, how much rebound occurred at that time is not known.

In the past two years, about a tenth of a foot maximum rebound has been observed but this was in response to about 30 feet of head recovery above prior winter levels. The stress-strain measurements we have obtained in wells in the elastic range of response (stress less than preconsolidation stress) indicate that if artesian head was restored to its original level, the land surface would rebound a few tenths of a foot.

LAND SUBSIDENCE, EARTH FISSURES AND GROUNDWATER WITHDRAWAL IN SOUTH-CENTRAL ARIZONA, U.S.A.[1]

H. H. SCHUMANN and J. F. POLAND[2]

ABSTRACT

Land subsidence in western Pinal County, south-central Arizona, is related to groundwater withdrawal and resultant water level declines. Differential subsidence and earth fissures have damaged irrigation systems, interstate highways, and railroads and have necessitated rerouting of a proposed aqueduct. Subsidence was first detected in 1948. The maximum documented subsidence from 1948 to 1967 is 2.30 meters.

Large-scale pumping for irrigation from wells that penetrate as much as 700 meters of permeable sediments has lowered water levels as much as 61 meters. Net water-level declines correspond with subsidence and water-level fluctuations correlate with sediment compaction and expansion. Measured compaction, 1965-67, in the upper 253 meters accounts for only 65 percent of measured subsidence.

Numerous earth fissures, as much as 12.8 kilometers long, occur in the alluvium and transect natural drainageways. Some of the fissures appear on the periphery of the subsiding areas and may be tensional breaks.

RÉSUMÉ

La subsidence du terrain, dans la partie occidentale du département de Pinal, dans l'Arizona sud-central, est liée au prélèvement des eaux souterraines et à l'abaissement du niveau d'eau qui l'a suivi. La subsidence différentielle et les fissures du sol ont endommagé les systèmes d'irrigation, les autoroutes et les voies des chemins de fer et ont amené la nécessité d'un nouveau tracé pour un aqueduc. La subsidence a été détectée pour la première fois en 1948. La subsidence maximum enregistrée entre 1948 et 1967 est de 2.30 mètres.

Les grandes quantités d'eau pour l'irrigation, qui ont été pompées de puits qui pénètrent les dépôts sédimentaires perméables à une profondeur aussi grande que 700 mètres, ont abaissé le niveau d'eau de 61 mètres en certains points. L'abaissement net du niveau d'eau correspond à la subsidence, et les fluctuations du niveau d'eau sont en corrélation avec la compression et l'expansion de la couche aquifère. Entre 1965-67, la compression, mesurée dans les 253 mètres supérieurs, n'explique que 65 pour-cent de la subsidence mesurée.

De nombreuses fissures du sol, ayant parfois une longeur de 12.8 kilomètres, se trouvent dans l'alluvion et traversent les drainages naturels. Certaines de ces fissures apparaissent dans la périphérie de la région de subsidence et peuvent être des fractures de tension.

INTRODUCTION

Land subsidence and earth fissures in western Pinal County, south-central Arizona, are related to water-level declines caused by large-scale ground-water withdrawal for irrigation. The area is a northwest-sloping alluvial plain bounded by mountains on the east and west (fig. 1). The central part, which is extensively irrigated, is underlain by as much 2,300 feet (700 m) of permeable alluvial deposits that constitute the principal aquifer. The upper 50-600 feet (15-180 m) of the alluvial deposits is silty sand and gravel, which is underlain by a silt and clay layer that is as much as 2,000 feet (610 m) thick in the central part of the area (Hardt and Cattany, 1965).

The withdrawal of ground water for irrigation began about 1914, when shallow wells were drilled near Toltec, and large-scale withdrawal began in 1936. From 1940 to 1967, more than 16 million acre-feet of water (20 km^3) was withdrawn (fig. 2). The draft on the aquifer system greatly exceeded the inflow, and most of the water pumped was removed from storage. Pumping has caused a steady decline in water levels throughout the area

1. Publication authorized by the Director, U.S. Geological Survey.
2. U.S. Geological Survey, Phoenix, Arizona and Sacramento, California.

(fig. 1). The cumulative net change in the average depth to water from 1940 to 1967 was about 141 feet (43 m) (fig. 2), and the maximum decline was more than 200 feet (61 m).

Differential subsidence and earth fissures have damaged irrigation systems,interstate highways, and railroads and have necessitated rerouting of a proposed major aqueduct. Earth fissures have damaged Picacho Reservoir (fig. 1) (Robinson and Peterson, 1962). Well-casing collapse and casing protrusion have been reported in several places. Differential settlement along Interstate Highway 10 near Picacho (fig. 1) necessitates continued highway maintenance (Winikka, 1964).

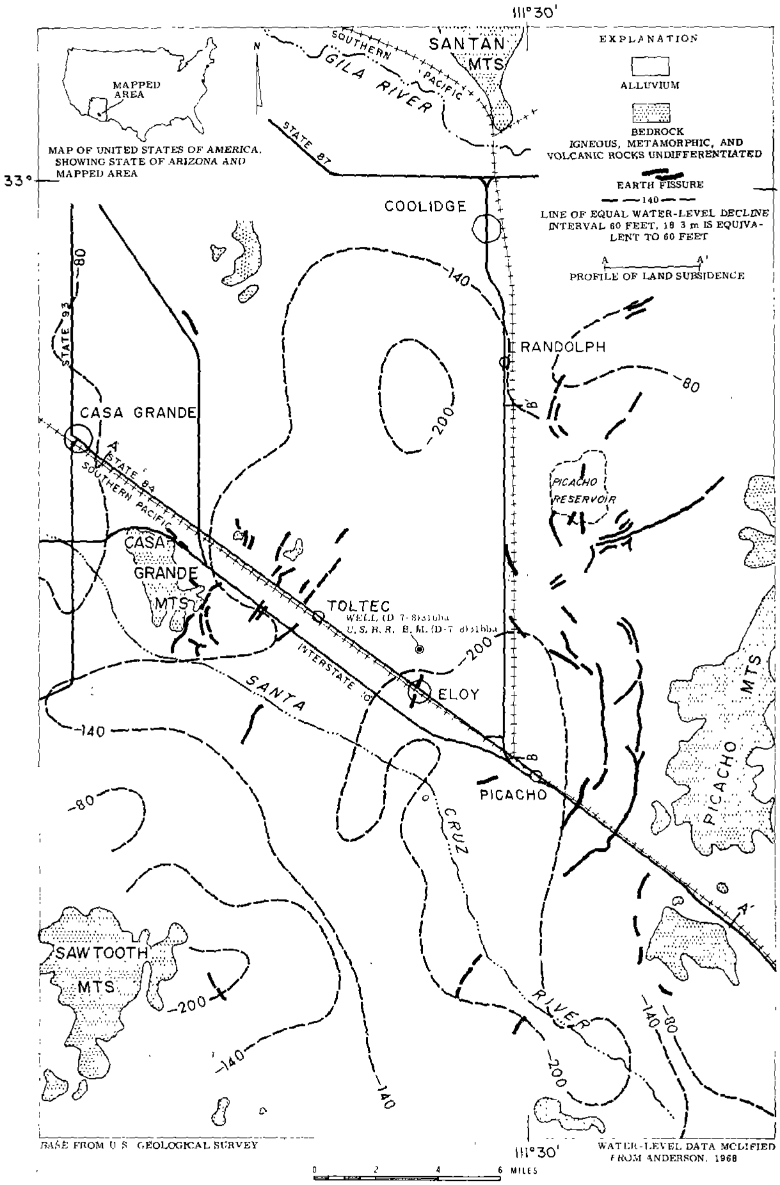

FIGURE 1. *Location of earth fissures, 1969, and water-level declines, 1923-64, in western final County*

LAND SUBSIDENCE

Land subsidence was first detected through releveling of the 1905 U.S. Geological Survey first-order level lines by the U.S. Coast and Geodetic Survey in 1948; the land surface near Eloy had subsided about 0.1 foot (0.03 m). Releveling in 1952 indicated that subsidence was continuing (Robinson and Peterson, 1962), and from 1948 to 1967 as much as 7.54 feet (2.30 m) of subsidence was recorded northeast of Eloy (figs. 1 and 3). During the same period, the average water level declined about 110 feet (34 m) (fig. 2). The maximum water-level decline was more than 150 feet (46 m).

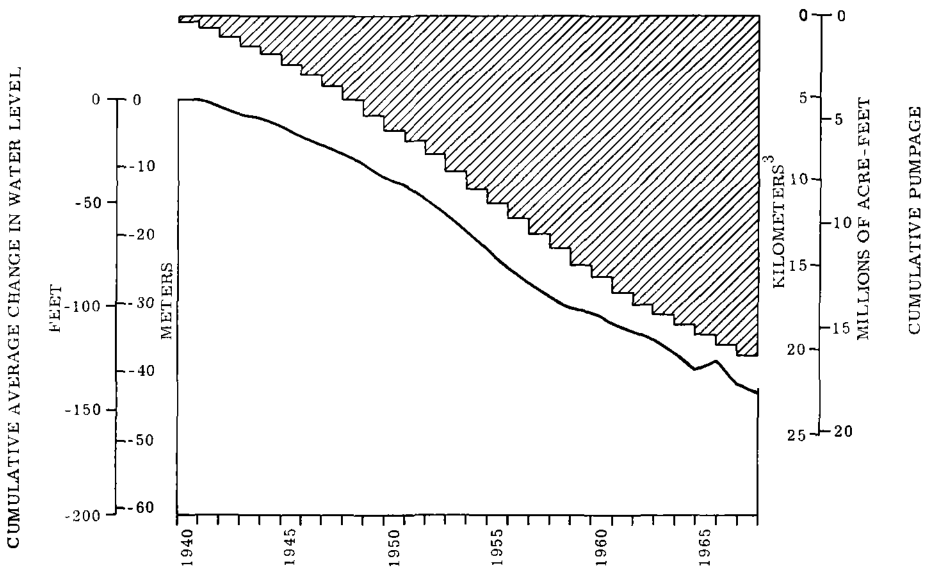

FIGURE 2. *Cumulative average change in water levels and cumulative pumpage, 1940-67, in western Pinal County.*

Most of the area along subsidence profile A-A′ (figs. 1 and 3) has settled more than 2 feet (0.61 m). Along subsidence profile B-B′, much of the land surface has settled more than 4 feet (1.22 m). Comparison of areas of maximum subsidence and long-term water-level declines indicates areas of maximum subsidence occur where water-level declines have been greatest (fig. 3).

The point of maximum documented subsidence northwest of Eloy settled 1.29 feet (0.39 m) from 1948 to 1952 at an average rate of 0.325 foot (0.099 m) per year (fig. 3, profile A-A′). From 1952 to 1964, the rate of subsidence increased to 0.560 foot (0.171 m) per year in response to continued and accelerated water-level declines (fig. 2). Incomplete releveling data for 1967 indicates continued subsidence, but at the greatly reduced rate of 0.113 foot (0.034 m) per year. The reduced subsidence rate probably reflects the 1966 water-level rises throughout the area (fig. 2). Similar rates of subsidence occurred northeast of Eloy along subsidence profile B-B′ (fig. 3). From 1963 to 1964, subsidence along this line slightly exceeded that from 1964 to 1967.

A compaction recorder was installed on well (D-7-831) bba near Eloy (fig. 1) in March 1965 to determine the relation between water-level changes, compaction, and land subsidence. Aquifer compaction and expansion are seasonal and correlate with the trend in water-level fluctuations (fig. 4). Compaction occurs during the summer period of water-level decline, and substantial rebound has been recorded during the winter period of water-level recovery (Poland, 1967). Periods of net water-level decline result in land subsidence (fig. 4). Some of the land subsidence is due to compaction of sediments below the 830-foot depth of the compaction well. From March 1965 to October 1967, the land surface subsided 0.69 foot (0.21 m), and the compaction of the sediments between the land surface and the bottom of the well was 0.45 foot (0.14 m). Thus, only about 65 percent of the total measured subsidence is due to compaction of the upper 830 feet (253 m) of sediment.

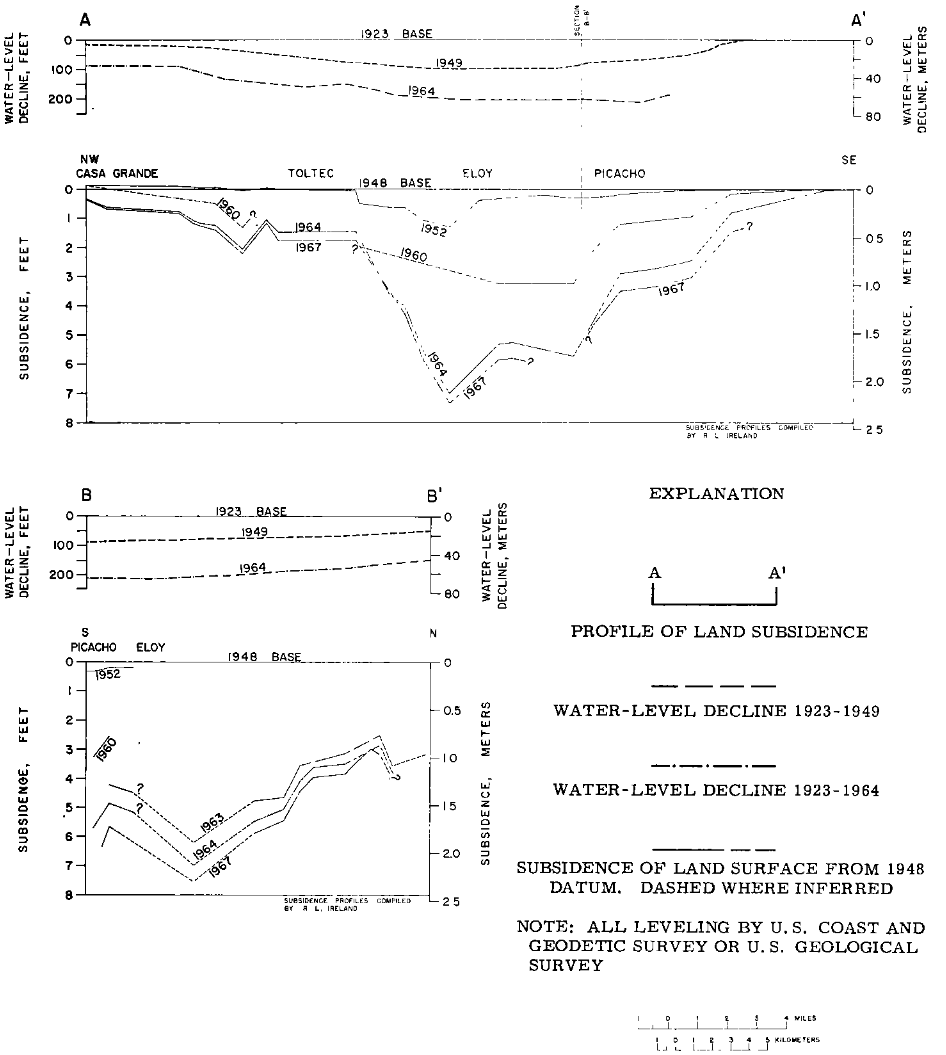

FIGURE 3. *Profiles of land subsidence, 1948-67, and water-level declines, 1923-64, in western Pinal County.*

H.C. Starkey of the U.S. Geological Survey made clay-mineral analyses of 14 core samples from depths of 504 to 1,675 feet below the land surface in the alluvial deposits penetrated by core hole (D-7-8)31bba (fig. 1) and by a nearby deeper core hole. He reports that about 6 parts in 10 of the $<2\mu$ fraction is montmorillonite, which is about equal to the proportion in fine-grained sediments in subsiding areas in central California and at Clear Lake, Texas. Compared with the other common clay minerals, montmorillonite is particularly sensitive to changes in effective stress – it compacts the most under increased stress (Poland, 1967, pp. 57-58).

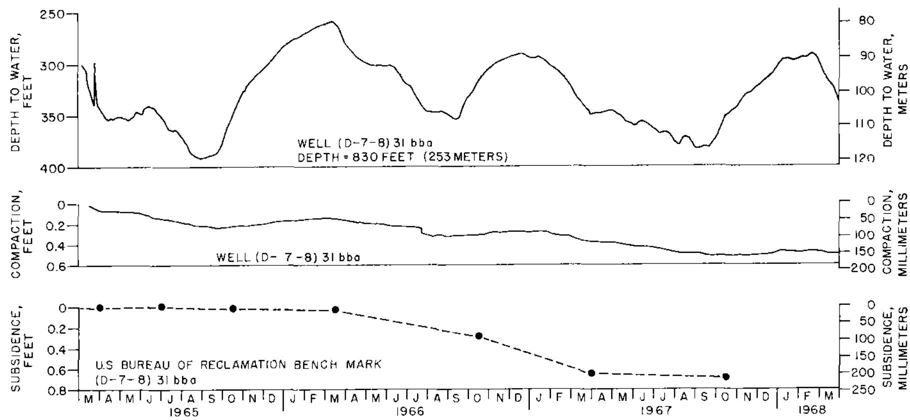

FIGURE 4. *Depth to water, compaction, and subsidence near Eloy.*

EARTH FISSURES

The first fissure or "earth crack" of record occurred 3 miles (4.8 km) southeast of Picacho in 1927 after a severe rainstorm (Leonard, 1929). Subsequent reopening of this fissure and the location of other fissures in the area have been described in several reports (Heindl and Feth, 1955; Pashley, 1961; Robinson and Peterson, 1962; Winikka, 1964). The fissures are confined to the alluvial deposits and have not been observed to continue into adjacent bedrock outcrops (figs. 1 and 5).

The fissures first appear as long narrow linear features, usually less than 1 inch (2.5 cm) wide and as much as 1 mile (1.6 km) long. The fresh fissures have sharp edges and exhibit no evidence of lateral movement. The movement appears to be simple horizontal separation of the landblocks on either side of the break; thus, as suggested by Heindl and Feth (1955), the fissures are believed to be tensional breaks.

The fissures roughly parallel the surface contours and transect natural drainage patterns. Upon application of irrigation water or following high-intensity rainstorms, the fissures intercept overland flow and act as drains. The water moves downward into the fissures causing them to increase rapidly in width – as much as several feet in places. The fissures widen partly by slumping but mainly by erosion of the sides. Gullying often occurs on the upstream side of the fissure. The fissures tend to connect and to form fissure systems that are as much as 8 miles (12.8 km) long.

The trends of many of the fissures roughly conform with linear zones of steep gravity gradients (fig.5). Most of the steep gravity gradients are adjacent to the mountain masses and may reflect buried fault scarps along the periphery of the subsiding basin. If this is true, the buried fault scarps probably are sites of maximum tensile stress. Thus, the most likely sites for new fissures would be along these zones.

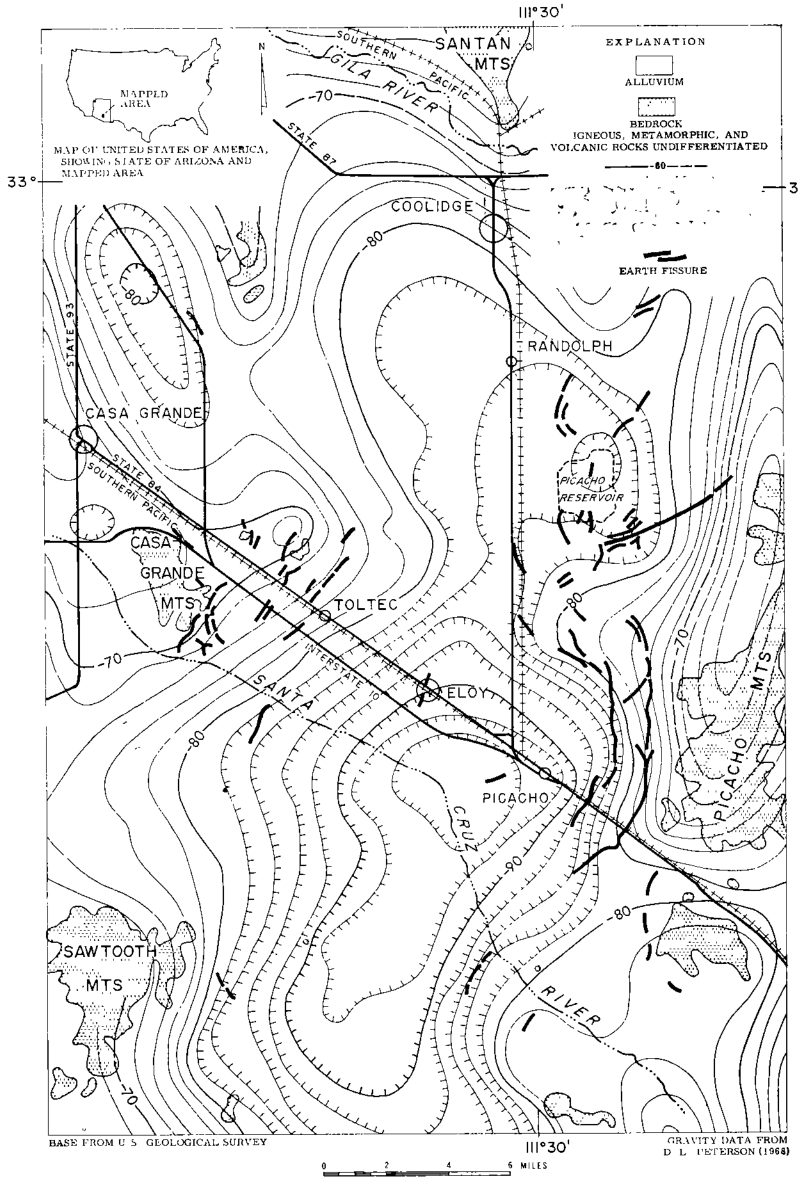

FIGURE 5. *Bouguer gravity map showing earth fissures, western Pinal County.*

Gravity conrour dashed where approximately located. Contour interval 2 milligals. Hachuraded contours indicate areas of low gravity closure.

CONCLUSIONS

Land subsidence and earth fissures in the area are related to the large-scale withdrawal of ground water and the resultant water-level declines. Compaction rates in the upper 830 feet (253 m) correlate with the trend of seasonal water-level fluctuations – substantial winter rebound of the sediments accompanying water-level recovery has been recorded. In general, measured land subsidence is proportional to net water-level declines.

The land surface has subsided as much as 7.54 feet (2.30 m) where the water-level decline is greater than 200 feet (61 m). Earth fissures occur in the alluvial deposits on the periphery of the subsiding area, appear to be tensional breaks, and may overlie buried fault scarps.

If rates of ground-water withdrawal from the alluvial deposits continue to exceed inflow, continued land subsidence can be expected.Additional earth fissures associated with land subsidence probably will occur, and existing fissures will enlarge and extend. Structural damage to existing and proposed engineering structures by differential subsidence and (or) earth fissures can be anticipated.

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DISCUSSION

Intervention by Prof. Kenneth L. LEE (USA):

Question:

Can you give us any information on the depth of the cracks?

Answer of Mr. SCHUMANN:

I do not know the total depth of cracks. However, I have measured depths up to about 60 feet or 20 m. The volumes are rather large because they do accept large amounts of water and sediments. They (the cracks) are doubly terminated, and appear to grow by erosion as water pours into them. Perhaps they may go to the water table. They have to be rather deep to accomodate the large volumes of water they capture.

Question by Prof. LEE:

Have you made an analysis of range in depth of cracks to the deformation within your subsidence area?

Answer of Mr. SCHUMANN:

In answer to your question, no, we have not done this. We have only very recently organized an interagency committee to study land subsidence in Arizona. We hope to work on this in the future.

Intervention by Mr. Owen G. INGLES (Australia):

Question:

Could you please indicate the climatic environment, especially precipitation and evaporation, in the vicinity of these tensional cracks?

Answer of Mr. SCHUMANN:

The annual precipitation of the area is about 10 inches or 25 cm per year, and the potential evaporation is about 100 inches or 250 cm per year. The ratio is 10 to 1 of evaporation to precipitation. It is a desert.

Question by Mr. INGLES:

Thus in the upper 60 feet or so where cracks occur, there would normally be considerable moisture tension. They would occur in subsaturated soils. How deep is the water table below the cracks? Can you tell me something of the clay mineralogy of the upper horizons?

Answer of Mr. SCHUMANN:

The soils are not saturated. Please recall the area where I showed the two comparative slides—of the fissure first opening, and three years later. In that area, the water table was about 500 feet below the land surface and there has been about 350 feet water-level decline. The clays are a mixture of montmorillonite and kaolinite. These soils often have a high calcium carbonate content.

Intervention by Prof. George V. CHILINGAR (USA):

Question:

As pointed out by Dr. Miyabe in his introductory lecture, there is a great need for research on compaction at greater depths. We are doing research on this subject at the Petroleum Engineering Department of the University of Southern California. In line with this, I would like to ask the following question because of your extensive experience with subsidence problems: What is the maximum recorded depth compaction at which was reflected at the surface as subsidence? (Mr. Schumann referred the question to his co-author, Mr. Joseph F. Poland).

Answer of Mr. POLAND:

The maximum depth at which compaction has been measured in water wells in the United States is 2,200 feet, but compaction in oil wells has been measured at depths exceeding 4,000 feet by the casing collar logs at the Wilmington oil field near Los Angeles, California.

SOME PROBLEMS OF TIME-SOIL COMPACTION IN PUMPING LIQUID FROM A BED

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ABSTRACT

This report concerns one-dimensional and plane axi-symmetrical problems of bed consolidation from which a liquid is pumped affected by the weight of a overlying formation in different schemes of pumping.

The problems are solved for the most general case when the bed, from which a liquid (oil, water) is pumped, is a deformed elastic-creeping porous medium filled with a compressible liquid. Here, the mixture of liquid and gas is taken as a compressible liquid. A spatial problem of soil compaction due to the formation in them of a cone of depression of the liquid pumped is also discussed.

RÉSUMÉ

Ce rapport concerne des problèmes de consolidation d'un milieu à une dimension et avec symétrie-axiale, duquel on extrait un liquide soumis au poids des formations supérieures, avec différents schémas de pompage.

Les problèmes sont résolus dans le cas le plus général quand l'aquifère dont le liquide (eau, pétrole) est pompé, est un milieu poreux élastico-fluant imprégné d'un liquide compressible. Dans notre cas, le mélange de liquide et de gaz est considéré comme compressible.

Un problème spatial de compaction due à la formation d'un cône de dépression du liquide pompé est discuté.

Intensive use of ground water, pumping of oil and gas from beds result in large settlements of the earth surface. Observations of settlements of the earth surface in the regions of exploitation of water intakes and oil and gas deposits show [3, 11, 12, 13, 14] that these settlements reach the values from some tens of centimeters to some meters. Owing to this the normal working conditions of engineering structures are disturbed, submergence and swampiness of large territories are observed.

In the work [3] a complete review of the studies published on the problem of settlement of the earth surface as result of intensive ground water pumping is given and some problems of the forecast theory of settlement of the earth surface are treated.

In recent years a significant development of the soil consolidation theory [4, 7, 8], how ever, enables these problems to be posed and solved with full consideration of the physical-mechanical properties of individual components of porous media. It should be noted that in this direction in hydrogeology a number of problems has been solved in which the elastic properties of a porous medium and liquid were taken into account [5, 6]. These problems are however restricted to determination only of the function of fall in head with the constant discharge of a well.

For reasons given above the authors of the present paper are considering some problems of time-porous medium compaction in the light of the new theory of soil consolidation. In the author's opinion it will partly fill up this gap.

At first, let us consider the generalized properties of individual components of a porous deformed medium filled with a compressible liquid.

1. ORIGINAL EQUATIONS

A pore medium may be represented as a deformed elastic-creeping isotropic body subject to the equation of hereditary creep [1, 4, 7, 8] which for the case of uniaxial compression without any lateral expansion is written in the form

$$\varepsilon(t) = \varepsilon(\tau_1) - \sigma(t)\, a(t, \tau) + \int_{\tau_1}^{t} \sigma(\tau) \frac{\partial}{\partial \tau} a(t, \tau)\, \mathrm{d}\tau \tag{1.1}$$

where:

$\varepsilon(\tau_1)$, $\varepsilon(t)$ initial and variable in time void ratios;
$\sigma(t)$ stress in soil skeleton variable in time (effective stress;
τ_1 time of applied loading;
$a(t, \tau)$ coefficient of consolidation characterizing the void ratio change to the moment "t" affected by loading of the unit stress applied at the moment τ_2 and expressing by the formula

$$a(t, \tau_1) = a_m + a_e\left[1 - \mathrm{e}^{-\eta(t-\tau_1)}\right] \tag{1.2}$$

where:

a_m coefficient of transient (elastic) consolidation;
a_e coefficient of continuous (viscous) consolidation;
η creep parameter.

It can be noted that if the medium creep is absent, this is, when $\eta \to \infty$ equation (1.1) comes to the form of a common compression relation [9], thus we have

$$\varepsilon(t) = \varepsilon(\tau_1) - a\sigma(t)$$

According to L. S. Leibenzon [5], liquid and gases filling voids of the medium are represented as a frictionless compressible liquid. We have shown [8] that for practical calculations and a change of pressures over a narrow range the equation of such a liquid can be approximately expressed as

$$-\frac{1}{V} \cdot \frac{\mathrm{d}V}{\mathrm{d}P} = \frac{1 - J_f}{P_a} = a_f \tag{1.3}$$

where:

V liquid volume under consideration;
J_f coefficient of *filling voids* with liquid;
P_a atmospheric pressure;
a_f coefficient of volume compressibility of liquid.

The differential equation of the deformed porous medium compaction filled with a compressible liquid, as known [7, 8], can be given in the form

$$\frac{\partial \varepsilon}{\partial t} + \bar{\varepsilon} a_f \frac{\partial P}{\partial t} = \frac{1 + \bar{\varepsilon}}{\gamma_f} K \nabla^2 \mathrm{P} \tag{1.4}$$

in which:

$\bar{\varepsilon}$ average void ratio;
γ_f unit weight of liquid;
K coefficient of liquid leaking;
∇ the Laplace operator.

In compaction of a porous medium the equilibrium condition is believed to be observed

$$q = \sigma(t) + P(t) \tag{1.5}$$

where:

q total stress;

$\sigma(t)$ stress in soil skeleton (effective stress);
$P(t)$ pressure in pore liquid.

The unit weight of a pore medium which was lessened by the weight of a liquid pushed out by it is known to be determined by the formula [9]

$$\gamma'_f = \frac{\gamma_s - \gamma_f}{1+\varepsilon} \tag{1.6}$$

in which:

γ_s specific weight of the pore medium material;
γ_f specific weight of liquid.

Stress intensity arising in a unit volume of the medium free itself from the suspended effect of a liquid can be determined by the formula

$$\gamma^* = (\gamma - \gamma')K' \tag{1.7}$$

in which:

γ unit weight of the medium.

Appropriate initial and boundary conditions should be selected for solving specific problems.

2. ONE-DIMENSIONAL PROBLEMS OF COMPACTION

In practice, there are often found instances when pumping is carried out simultaneously from numerous wells located in large territories. In this case a lowering of the liquid level in a large territory may be regarded as the process satisfying the conditions of a one-dimensional problem.

The problems of this type are equivalent to one-dimensional problems of consolidation of the pore medium upon which a uniform loading has been applied (fig. 1). In this case, depending on the rate of liquid level lowering, two problems may be considered.

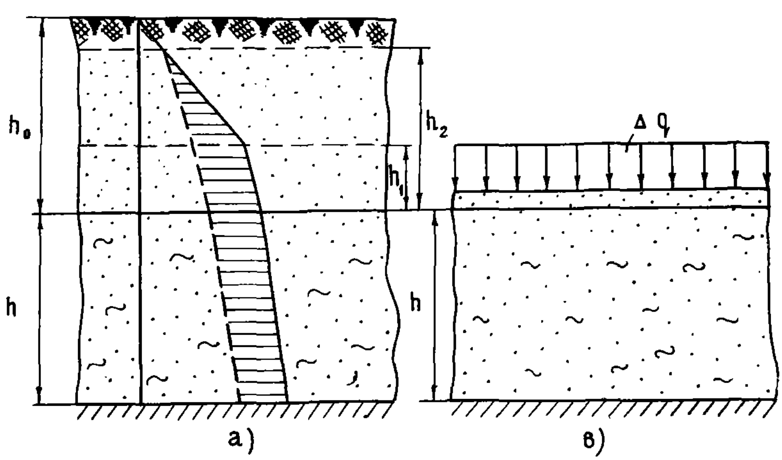

FIGURE 1. *A design scheme of the one-dimensional problem of soil consolidation:*
(a) *graph of natural pressure in soil skeleton before and after liquid level lowering;*
(b) *equivalent scheme*

2.1. THE CASE OF RAPID LEVEL LOWERING

If the liquid level lowering rate is rather high in comparison with the rate (h) of a bed consolidation it can be neglected. One may consider that the level lowered instantaneously.

Then the solution of the one-dimensional problem of compaction in time reduces to the solution of differentional compaction equation (1.4) in conjunction with integral equation (1.1), taking into account equilibrium equation (1.5). The value equal to

$$q = (\gamma - \gamma')(h_2 - h_1) \tag{2.1}$$

is taken as a loading applied to the upper limit of a bed, where h_2 and h_1 are intial and final levels of liquid.

In the case of one-dimensional compaction the solution of differentional equation (2.2) obtained by Z.G. Ter-Martirosyan [7] takes the form

$$P(t, Z) = \frac{4q}{\pi} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m} (C_m \cdot e^{-M_m t} - D_m \cdot e^{-N_m t}) \sin \frac{m\pi z}{2h} \tag{2.2}$$

whereı

$$\left\{ \begin{aligned} C_m &= \frac{\eta a_e - \left[(a_m + \bar{\varepsilon} a_f) N'_m + \eta a_e + \dfrac{(1+\bar{\varepsilon}) K \cdot \alpha_m^2}{\gamma_f} \right] A_0}{2(a_m + \bar{\varepsilon} a_f)\sqrt{Q_m^2 - R_m}} \\ D_m &= \frac{\eta a_e - \left[(a_m + \bar{\varepsilon} a_f) M'_m + \eta a_e + \dfrac{(1+\bar{\varepsilon}) K \cdot \alpha_m^2}{\gamma_f} \right] A_0}{2(a_m + \bar{\varepsilon} a_f)\sqrt{Q_m^2 - R_m}} \end{aligned} \right. \tag{2.3}$$

$$\left\{ \begin{aligned} M'_m &= -M_m = -Q_m + \sqrt{Q_m^2 - R_m} \\ N'_m &= -N_m = -Q_m - \sqrt{Q_m^2 - R_m} \end{aligned} \right. \tag{2.4}$$

where:

$$Q_m = \frac{1}{2}\left[\eta \frac{a_m + \bar{\varepsilon} a_f + a_e}{a_m + \bar{\varepsilon} a_f} + \frac{(1+\bar{\varepsilon}) K \alpha_m^2 .}{\gamma_f (a_m + \bar{\varepsilon} a_f)} \right] \tag{2.5}$$

$$R_m = \eta \frac{(1+\bar{\varepsilon}) K \alpha_m^2}{\gamma_f (a_m + \bar{\varepsilon} a_f)} \tag{2.6}$$

$$\alpha_m = \frac{m \cdot \pi}{2n}; \tag{2.7}$$

$$A_0 = \frac{a_m}{a_m + \varepsilon(\tau_1) a_f}$$

In this instance the settlement is determined by the formula [7, 8]

$$S(t) = \frac{qh}{1+\varepsilon(\tau_1)} [a_m U_1(t) + a_e U_2(t)] \tag{2.8}$$

where U_1 (t), U_2 (t) are degrees of the medium compaction from elastic and viscous constituents of deformation which are defined by the expressions

$$U_1(t) = 1 - \frac{16}{\pi} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2} (C_m \cdot e^{-M_m t} - D_m \cdot e^{-N_m t})$$

$$U_2(t) = 1 - \frac{e^{-t\eta}16\eta}{\pi^2} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2} \left[\frac{C_m}{-M_m+\eta} (e^{-M_m t} - e^{-rt}) - \frac{D_m}{-N_m+\eta} (e^{-N_m t} - e^{rt}) \right]$$

These values vary from 0 to 1 with $t \to \infty$. For a given magnitude of settlement it is possible to determine the time required for its attainment or to forecast a magnitude of settlement for the given period of time.

2.2 THE CASE OF THE GIVEN REGIME OF LIQUID LEVEL LOWERING

Using the above solution the formulae can be determined for forecasting bed settlement "h" in time at the given regimes of pumping. Actually, if a loading increases progressively on the given law $q = q$ (t) conforming with the progressive level lowering a settlement for any given moment of time may be found by means of integration of equation (2.8) in which its differential instead of "q" is taken

$$\mathrm{d}q = \frac{\partial q}{\partial \tau} \mathrm{d}t ; \tag{2.10}$$

Let us consider two regimes of liquid level lowering when

$$q_I = \beta\tau ; \tag{2.11}$$

$$q_{II} = q_\infty (1 - e^{-\lambda\tau}) \tag{2.11}$$

Substituting the differentials of these values in equation (2.8) after integration we obtain

$$S_I(t) = \frac{h}{1+\varepsilon(\tau_1)} [a_m u_1^I(t) + a_e u_2^I(t) \tag{2.12}$$

$$S_{II}(t) = \frac{h}{1+\varepsilon(\tau_1)} [a_m u_1^{II}(t) + a_e u_2^I(t)] \tag{2.12}$$

$$\left\{ \begin{aligned} & u_1^I(t) = \beta t - \frac{8\beta}{\pi^2} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2} \left[c_m(1-e^{-M_m t}) - \frac{D_m}{N_m}(1-e^{-N_m t}) \right] \\ & u_2^I(t) = \\ & \eta \left\{ \beta t^2 - \frac{8\beta}{\pi^2} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2} \left[\frac{c_m}{M_m} \left(t - \frac{1}{M_m}(1-e^{-M_m t}) \right) - \frac{D_m}{N_m} \left(t - \frac{1}{N_m}(1-e^{-N_m t}) \right) \right] \right\} \end{aligned} \right. \tag{2.13}$$

$$u_1^{II}(t) =$$

$$t + \frac{1-e^{-\lambda t}}{\lambda} - \frac{8\lambda}{\pi^2} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2}\left(\frac{c_m}{M_m + \lambda}\left[1-e^{-(M_m+\lambda)t}\right] - \frac{D_m}{N_m + \lambda}\left[1-e^{-(N_m+\lambda)t}\right]\right)$$

$$u_2^{II}(t) =$$

$$\eta\left\{t + \frac{1-e^{-\lambda t}}{\lambda} - \frac{8\lambda}{\pi^2} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m^2}\left[\frac{c_m}{M_m + \lambda}\left(\frac{1-e^{-\eta t}}{\eta} - \frac{1-e^{-(M_m+\lambda+\eta)t}}{M_m+\eta+\lambda}\right) - \right.\right.$$
$$\left.\left. - \frac{D_m}{N_m+\lambda}\left(\frac{1-e^{-\lambda t}}{\eta} - \frac{1-e^{-(N_m+\eta+\lambda)t}}{N_m+\lambda+\eta}\right)\right]\right\} \tag{2.14}$$

These solutions can be used for forecasting a settlement of the earth surface at the above regimes of liquid pumping. The medium parameters coming into these equations may be readily determined on the results of compression tests (7, 8).

3. THE AXI-SYMMETRICAL COMPACTION PROBLEM

Let us consider the compaction problem of a confined bed "h" from which liquid pumping is simultaneously carried out from several wells having the diameter "$2R_0$" and the distance between them "$2R$" (fig. 2).

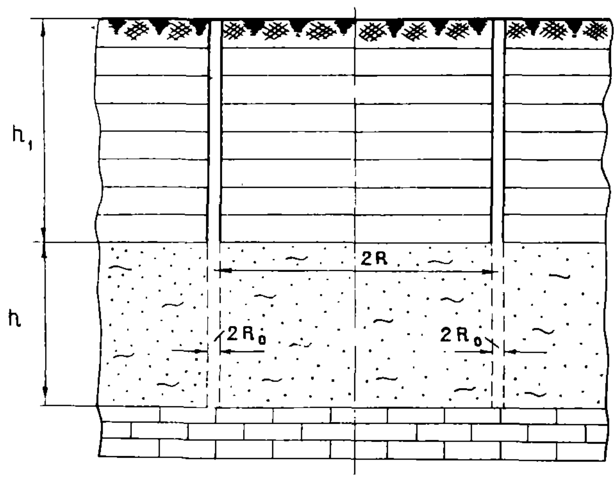

FIGURE 2. *A design scheme of soil compaction in group pumping*

In the case of the axi-symmetrical compaction problem equation (1.4) takes the form

$$\frac{\partial \varepsilon}{\partial t} + \bar{\varepsilon} a_f \frac{\partial P}{\partial t} = \frac{1+\bar{\varepsilon}}{\gamma_f} K\left(\frac{\partial^2 P}{\partial r^2} + \frac{1}{r}\frac{\partial P}{\partial r}\right) \tag{3.1}$$

Solving it in conjunction with equation (1.1) and taking into account equilibrium equation (1.5) we come to the differential equation of the form

$$(a_m+\bar{\varepsilon}a_f)\frac{\partial^2 P}{\partial t^2}+\eta(a_m+a_e+\bar{\varepsilon}a_f)\frac{\partial P}{\partial t}=$$
$$=\frac{(1+\bar{\varepsilon})}{\gamma_f}K\left[\left(\frac{\partial^3 P}{\partial t\,\partial r^2}+\frac{1}{r}\frac{\partial^2 P}{\partial r\,\partial t}\right)+\eta\left(\frac{\partial^2 P}{\partial r^2}+\frac{1}{r}\frac{\partial P}{\partial r}\right)\right] \tag{3.2}$$

Designating the intensity of loading from overlying beds by q and initial pressure in pore liquid and soil skeleton by $P(\tau_1)$ and $\sigma(\tau_1)$ respectively, we obtain the equilibrium condition in the form

$$q=\gamma h_1=P(\tau_1)+\sigma(\tau_1) \tag{3.3}$$

The second initial condition can be found in solving equations (1.1) and (3.1) together, this is, we have

$$(a_m+\bar{\varepsilon}a_f)\frac{\partial P(\tau_1)}{\partial t}+[q-P(\tau_1)]\eta a_e=\frac{(1+\bar{\varepsilon})}{\gamma_f}k\left(\frac{\partial P}{\partial r^2}+\frac{1}{r}\frac{\partial P}{\partial r}\right) \tag{3.4}$$

The boundary conditions of the problem under consideration will be

$$P(t,R_0)=0;\quad \frac{\partial P(t,R)}{\partial r}=0; \tag{3.5}$$

Then the solution of equation (3.2) will be expressed by

$$P(\tau_1,t)=\sum_{i=1}^{\infty}[E_iJ_0(n_ir)+F_iY_0(n_ir)](c_ie^{\beta_{i,1}t}+D_i^{\beta_{i,2}t}) \tag{3.6}$$

where $J_0(n_i,r)$ and $Y_0(n_i,r)$ are the Bessel functions,

$$\left.\begin{matrix}\beta_{i,1}\\ \\ \beta_{i,2}\end{matrix}\right\}=A_i\pm\sqrt{A_i^2+B_i} \tag{3.7}$$

$$A_i=\frac{1}{2}\left[\eta\cdot\frac{a_m+a_e+\bar{\varepsilon}a_f}{a_m+\bar{\varepsilon}a_f}+\frac{(1+\bar{\varepsilon})Kn^2}{\gamma_f(a_m+\bar{\varepsilon}a_f)}\right]$$
$$B_i=\eta\cdot\frac{(1+\bar{\varepsilon})Kn^2}{\gamma_f(a_m+\bar{\varepsilon}a_f)} \tag{3.8}$$

The coefficients E_i, F_i, C_1, D_i are defined by the initial and boundary conditions. Boundary conditions (3.5) give the characteristic equation

$$J_1(n_iR)\,Y_0(n_iR_0)-J_0(n_iR_0)\,Y_1(n_iR)=0 \tag{3.9}$$

After some transformations equation (3.6) reduces to the form

$$P(r,t)=\sum_{i=1}^{\infty}G_i\frac{V_0(n_ir)}{Y_0(n_iR_0)}(H_ie^{\beta_{i,1}t}+e^{\beta_{i,2}t}) \tag{3.10}$$

where

$$V_0(n_i r) = J_0(n_i r)\, Y_0(n_i R_0) - J_0(n_i R_0)\, Y_0(n_i r) \tag{3.11}$$

Determining the constants G_i and H_i from initial conditions (3.3) and (3.4) and substituting them in equation (3.11) after some transformations we finally obtain

$$P(r,\, t) = \sum_{i=1}^{\infty} \frac{J_1^2(n_i R)\, Y_0(n_i R_0)\, V_0(n_i r)}{J_1^2(n_i R) - J_0^2(n_i R_0)} \times$$

$$\cdot \frac{P(\tau_1)\left[\eta\, a_e - (a_m + \bar{\varepsilon}\, a_f)\beta_{i,1} - \dfrac{1+\bar{\varepsilon}}{\gamma_f} K n_i^2\right] - q}{(a_m + \bar{\varepsilon}\, a_f)\,(\beta_{i,2} - \beta_{i,1})} \times$$

$$\times \left(\left\{ \frac{P(\tau_1)\left[(a_m + \bar{\varepsilon}\, a_f)\,\beta_{i,2} - \eta\, a_e + \dfrac{1+\bar{\varepsilon}}{\gamma_f} K n_i^2\right] + q}{P(\tau_1)\left[\eta\, a_e - (a_m + \bar{\varepsilon}\, a_f)\,\beta_{i,1} - \dfrac{1+\bar{\varepsilon}}{\gamma_f} K n_i^2\right]} \right\} e^{\beta_{i,1} t} + e^{\beta_{i,2} t} \right); \tag{3.12}$$

In defining a settlement of the bed "h" compression stress is assumed to occur. In this case to evaluate the degree of bed compaction the value of the medium pressure in pore liquid within the limits of "R_0" to "R" can be used. The value of the medium pressure in pore liquid depending on the variable "r" may be determined by the known formula of the average value of the function

$$\bar{P}(r,\, t) = \frac{1}{R - R_0} \int_{R_0}^{R} P(r,\, t)\, \mathrm{d}r \tag{3.13}$$

Substituting the value of this integral in equation (3.11) we get

$$\bar{P}(r,\, t) = \frac{1}{R - R_0} \sum_{i=1}^{\infty} \frac{G_i}{Y_0(n_i R_0)} \int_{R0}^{R} V_0(n_i r)\, \mathrm{d}r (H_i e^{\beta_{i,1} t} + e^{\beta_{i,2} t}) \tag{3.14}$$

The average value of settlement in time is found by the compression formula (9)

$$\bar{S}(t) = \frac{\varepsilon(\tau_1) - \varepsilon(t)}{1 + \bar{\varepsilon}} \cdot h \tag{3.15}$$

Substituting the values $\varepsilon\,(\tau_1) - \varepsilon\,(t)$ from equation (1.1) and using equilibrium equation (1.5) we get the following expression

$$S(t) = \frac{h}{1 + \bar{\varepsilon}(\tau)_1} \left[\left(q - \bar{P}(t) + \eta\, a_e \int_{\tau_1}^{t} \left(q - \bar{P}(\tau) \right) e^{-\eta(t-\tau)}\, \mathrm{d}\tau \right] \tag{3.16}$$

After integration taking into account the value of $\bar{P}\,(t)$ from equation (3.13) we obtain

$$\bar{S}(t) = \frac{hq}{1 + \varepsilon(\tau_1)} \left[a_m \bar{U}_1(t) + a_e \bar{U}_2(t) \right], \tag{3.17}$$

where

$$\bar{U}_1(t) = q - \frac{1}{R-R_0}\sum_{i=1}^{\infty}\frac{G_i}{Y_0(n_iR_0)}\int_{R_0}^{R} V_0(n_ir)\,dr\,[H_i e^{\beta_{i,1}t} + e^{\beta_{i,2}t}]\,;$$

$$\bar{U}_2(t) = q(1-e^{-\eta t}) - \frac{1}{R-R_0}\sum_{i=1}^{\infty}\frac{G_i}{Y_0(n_iR_0)}\int_{R_0}^{R} V_0(n_ir)\,dr\,\times$$

$$\times\left\{\frac{H_i}{\beta_{i,1}-\eta}[\exp(\beta_{i,1}-\eta-1)t - \exp(\beta_{i,1}-\eta)t] + \right. \tag{3.18}$$

$$\left. + \frac{1}{\beta_{i,2}-\eta}[\exp(\beta_{i,2}-\eta-1)t - \exp(\beta_{i,2}-\eta)t]\right\}.$$

Here the functions of $\bar{U}_1\,(t)$, $\bar{U}_2\,(t)$ determine the degree of bedcompaction

Introduce designations in equation (3.18)

$$n_iR = x_i\,; \qquad \mu = \frac{R_0}{R}. \tag{3.19}$$

Then, we obtain $V_0\,(x_i\,\frac{r}{R})$ and $Y_0\,(x_i)$ instead of $V_0\,(n_ir)$ and $Y_0\,(n_iR_0)$ and characteristic equation (3.11) takes the form

$$Y_1(x_i)\,Y_0(x_i\mu) - J_0(x_i\mu)\,Y_1(x_i) = 0\,, \tag{3.20}$$

in which X_i roots of this equation.

Thus, the solution of the problem can be considered to be complete since it was reduced to finding the Bessel functions of an entire order for which detailed tables were made.

4. THE AXI-SYMMETRICAL PROBLEM OF THE MEDIUM COMPACTION AT STEADY INCONFINED LIQUID FLOW

Let us have an inconfined horizontal layer with free surface in which a cone of depression was formed due to liquid pumping from a well (fig. 3). In this case a complicated spatial stress condition arises which produces a settlement of the earth surface.

Here the problem of determining the stress-strain condition of a bed may be put as follows. A change of the unit weight of the medium occurring inside a cone of depression is replaced with a uniform loading of "q" intensity. Then, the solution of this problem reduces to finding the functions of stresses and strains in semispace due to uniform loading over the volume of the internal part of a cone of depression. To do this, we make use of the Mindlin solution [10] on the point force applied inside elastic semispace (fig. 4a). To determine vertical displacements the following expression was attained by Mindlin

$$W = \frac{P}{16\pi G(1-\gamma)}\left[\frac{3-4v}{R_1} + \frac{\varepsilon(1-v^2)-(3-4v)}{R_2} + \frac{Z-C}{R_1^3} + \right.$$

$$\left. + \frac{(3-4v)(Z+C^2)-2CZ}{R_2^2} + \frac{6CZ(Z+C^2)}{R_2^5}\right] \tag{4.1}$$

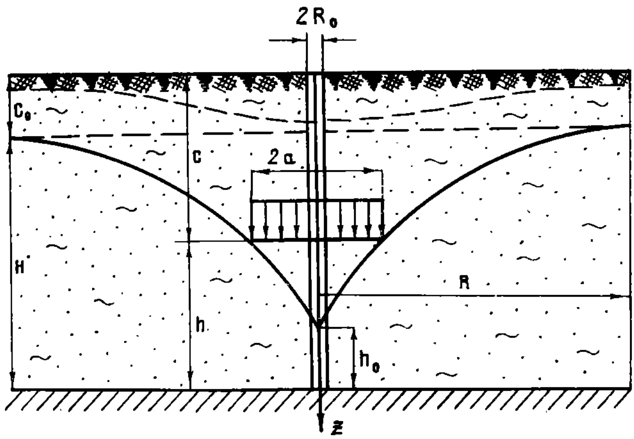

FIGURE 3. *A design scheme of soil compaction taking into account a cone of depression*

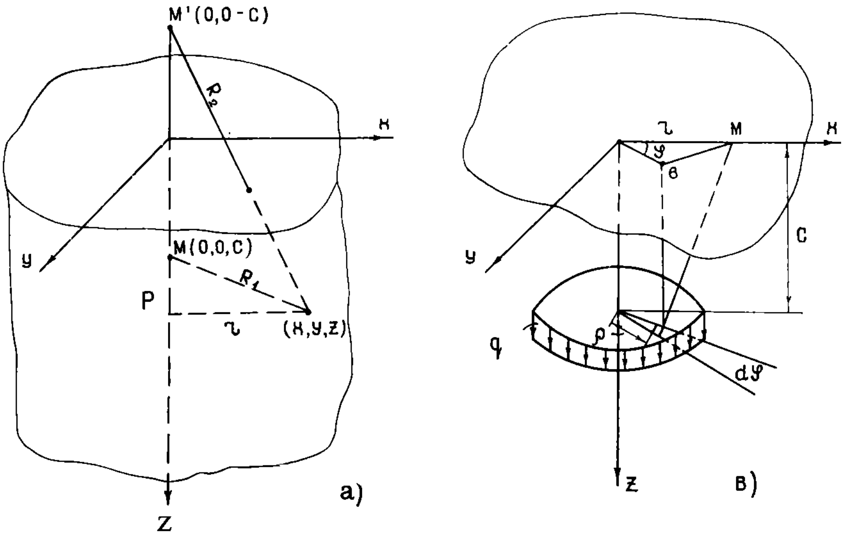

FIGURE 4. *Design schemes of the axi-symmetrical problem of compaction*

With $Z = 0$ we have $R_1 = R_2 = R$ and obtain the more simple relation

$$W = \frac{P}{8\pi E}\left[\frac{8(1-\nu^2)}{R} + \frac{C^2(3-4\nu)}{R^3}\right] \tag{4.2}$$

Let us determine a settlement at the point M (fig. 4c) under the effect of uniform loading of "q" intensity over a circle with a radius of "a" at a depth of "c". From the geometrical considerations it is possible to write that

$$R^2 = t^2 + C^2 = r^2 + \rho^2 + 2r\rho \cdot \cos\varphi + C^2 \tag{4.3}$$

In this case the settlement at the point of M based on (4.2) will be equal to

$$W_M = \frac{q}{8\pi E}\iint_F \left[\frac{8(1-\nu^2)}{R} + \frac{C^2(3-4\nu)}{R^3}\right]\rho \, \mathrm{d}\varphi \, \mathrm{d}\rho \tag{4.4}$$

The solution of this equation comes to determination of elliptic integrals. In the case of determination of the maximum settlement at a point with the coordinates of 0, 0, 0 we come to the expression

$$W_0 = \frac{q}{4E}\left[8(1-\nu^2)\left(\sqrt{C^2+a^2}-\sqrt{C^2+R_0^2}+\frac{C^2(3-4\nu)}{5}\right)[C^2+a^2)-(C^2+R_0^2)\;]^{\frac{5}{2}}\right] \tag{4.5}$$

For determination of a settlement at the point of (0, 0, 0) under the effect of loading all over the volume, integration should be also done in the direction of "C_0" to "h_0" taking "q", "a", "c" depending on "Z".

Since in most cases the curves of depression cones $h = f(C, r)$ are complicated [2, 6] the solution of elliptic integrals fails to be avoided. Therefore integration is appropriate to substitute by summation, dividing the whole formation into elementary beds.

In this case we get

$$W_0 = \frac{(\gamma-\gamma')}{4E}\Delta C\sum_{i=1}^{i=n}\left\{8(1-\nu^2)\left(\sqrt{C_i^2+a_i^2}+\sqrt{C_i^2+R_0^2}\right)+\right.$$
$$\left.+\frac{C_i^2(3-4\nu)}{5}\left[(C_i^2+a_i^2)^{\frac{5}{2}}-(C_i^2+R_0^2)^{\frac{5}{2}}\right]\right] \tag{4.6}$$

where is the thickness of an elementary bed.

When the medium is characterized by hereditary creep of the type (I) a settlement in time is found as follows:

$$W_0(t) = \frac{(\gamma-\gamma')\Delta C}{4E(t)}\sum_{i=1}^{i=n}\left\{8(1-\nu^2)\left(\sqrt{C_i^2+a_i^2}-\sqrt{C_i^2+R_0^2}\right)+\right.$$
$$\left.+\frac{C_i^2(3-4\nu)}{5}\left[(C_i^2+a_i^2)^{\frac{5}{2}}-(C_i^2+R_0^2)^{\frac{5}{2}}\right]\right\}, \tag{4.7}$$

where

$$\frac{1}{E(t)} = \frac{1}{E_m} + \frac{1}{E_e}\left[1-\mathrm{e}^{-\eta(t-\tau_1)}\right], \tag{4.8}$$

E_m the modulus of transient deformation;
E_e the modulus of continuous deformation. The remaining designations are the previous ones.

Here, the following ratio is observed between $E(t,\tau_1)$ and $a(t,\tau_1)$:

$$\frac{1}{E(t, \tau_1)} = [1+\varepsilon(\tau_1)]\,\beta\cdot a(t, \tau_1), \tag{4.9}$$

where

$$\beta = 1 - \frac{2\nu^2}{1-\upsilon},$$

ν is the Poisson coefficient.

In conclusion, it should be noted that in correct use of the successes achieved in the soil mechanics theory studying the regularities of compaction of deformed pore media filled with liquids a strong theoretical base can be created for forecasting a settlement of the earth surface due to pumping of ground water, oil and gas in exploitation of their deposits.

Naturally, the solutions obtained can not be used for all instances as they correspond to definite hydrogeological conditions. However, the statement and solution of the problems given show that at modern development of the theory of pore media consolidation the problem of forecasting a settlement of the earth surface based on theoretical solutions can be successfully solved.

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CONSIDERATION ABOUT THE COMPACTION MECHANISM OF STRATUM LYING AT THE DEEPER HORIZON IN TOKYO LOWLAND

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ABSTRACT

The writer brought forward a conception "Detour Travelling" which regulates the behavior of ground-water and intergranular water, that is, the water being retained within pores of semi-pervious strata or aquacludes. The Quaternary sediments in the Tokyo Lowland, especially the sediments at deeper horizon are heterogeneous and frequently discontinuous, and the situation strongly regulates "Detour Traveling" effect.

According to the study of the writer, it is presumed that the deeper strata retaining intergranular water accounts for the greater part of the compaction rather than that by the overburden sediments. The change of ground-water pressure causes ground subsidence. It may be attributed to the compaction of a little loose clayey material composing the strata and also " Detour Traveling" which expresses the form of the water flow sucked out from the strata and moreover the pressure propagation through the extending "Arterial Network" consisted of more porous materials.

RÉSUMÉ

L'auteur présente une conception qui règle le comportement de l'eau souterraine et de l'eau interstitielle, c'est-à-dire l'eau retenue par les pores des couches semi-perméables ou imperméables. Les sédiments quaternaires dans la région basse de Tokyo, spécialement les horizons les plus bas, sont constitués de couches hétérogènes et souvent discontinues. D'après l'étude de l'auteur, il est à présumer que ces couches profondes retenant l'eau prennent sur elles une bonne part de la compaction. Le changement de la pression de l'eau souterraine cause l'affaissement. Il doit être attribué à la compaction du matériel quelque peu argileux composant ces couches et aussi au " Detour travelling " qui exprime la forme de l'écoulement de l'eau extraite de ces couches.

INTRODUCTION

Vertical displacement between buildings and ground surface in the lowland and highland areas of Tokyo Metropolis has been observed since about 1930. It has been reported in the shore line belt area of Osaka, Kobe and Amagasaki cities in Hyogo Prefecture. The centers of industrial, commercial and official buildings are most affected in Tokyo. This movement is rapid and can be recognized yearly.

The displacement between a road and adjacent buildings corresponds to the scale of two or three steps of a staircase. N. Miyabe, K. Wadachi and others studied the phenomena and concluded that it was due to compaction brought about by heavy pumping of ground water for cooling and air conditioning purposes. They stated that the pumping lowered the water level and caused the young saturated and layered strata of clay and silt to lose buoyancy inherent to the aquifer, and thereby were subjected to a load corresponding to the weight of the overlying sediments. This resulted in compaction or change in thickness of the strata. In other words the heavy pumping decreased the water pressure in the aquifer and caused squeezing of water of higher pressure from the impermeable strata into the aquifer. This phenomenon has been observed in the cities of Niigata, Nagoya, Hiroshima and Shiroishi.

GEOLOGY OF THE TOKYO AREA

The Tokyo area is roughly divided into the Yamate-highland, Tokyo-lowland and the narrow alluvial lowland along Tokyo Bay.

The Pre-Tertiary igneous and metamorphic rocks underlie the Tertiary and Quaternary sediments in the Kanto Tectonic Basin, about 1800 m beneath Tokyo Bay. The thicknesses of the different sediments are: Holocene Yurakucho Formation 50 m$\pm$; Pleistocene Tokyo Group, Narita Group 350 m$\pm$; Pleistocene to Pliocene Miura Group 1500 m$\pm$; and, Miocene Tokigawa Group 300 m$\pm$.

FORMATION CHARACTERISTICS

The "Kanto Basin", named by H. Yabe and R. Aoki, began development in earliest Neogene and was completed in late Miocene or Pliocene. It is now buried by Pleistocene and younger sediments. The Tokyo Group or Narita Group unconformably underlies the Holocene Yurakucho Formation and other alluvial deposits of the Tokyo lowland, Tokyo Bay and its environs. These areas were subjected to eustatic changes several times during latest geological history. The character of their works is as follows: (1) Pre-Tertiary rocks are very hard and are of igneous and metamorphic origin; (2) Tokigawa Group is composed mainly of tuff, tuff breccia and sandstone; (3) Miura Group is composed of thick and rather homogeneous fine sandstone to siltstone. It is intercalated with a few sandstone layers (100-200 m thick in total) in the upper and middle to middle-lower parts; (4) Narita Group and Tokyo Group contain sand facies with gravel lenses and beds of fine-grained materials, and rather heterogeneous coarse grain size components; (5) Yurakucho Formation and alluvial sediments contain clay and sand (lower), sandy silt intercalated with laminal sand and silty materials (middle), and a sand facies (upper). The thickness of each formation, except the Pre-Tertiary and Holocene sediments, increases towards the center of the Basin. The greatest thickness of sediments is almost in the center of the basin.

Ground subsidence is related to the young upper sediments, especially when they suffer notable change in their thickness by compaction or other reasons.

TOPOGRAPHY

Tokyo Bay is situated in the southeastern part of the Kanto Plain. Its deeper part is 10-20 m at the inner bayment. The bottom is covered with soft muddy sediments. Pleistocene and Plio—Miocene sediments occur in the Yamate-highland and hilly land, and under the Tokyo-lowland. The Boso and Miura Peninsulas (NNE-SSW) are presumed to have been once connected as a range and were separated at the mouth of Tokyo Bay.

TIDES IN TOKYO BAY

In Tokyo Bay there are about two tidal changes per day. The tide amplitude is one meter with a maximum of 2.0 m$\pm$; the distance from the bay mouth to the mouth of Arakawa River is about 50 km, and within this distance the phase lag is about 28 min.

Beside the two tides per day, there is a characteristic change in the Period, which occurs every half month with irregularity. It can be classified into 2 or 3 types, and can be used to correlate the tidal change with the change in groundwater level. Thus, the cyclic change of groundwater level, associated with the ground subsidence, as observed in wells in the Tokyo area correspond directly and rigidly with the Tide.

DISTRIBUTION OF OBSERVATION STATIONS

Besides the observation of ground subsidence in the Tokyo area by the Tokyo Institute of Civil Engineering, the writer has studied the ground subsidence, groundwater, mechanical soil character, geology of the Tokyo area, in the field and in the laboratory during 1959-1966. The basic data were published in "Result of Levelling, and Records of Landsubsidence and Groundwater Level during 1968-1969" and in other papers. These data are from the observation wells whose locations are shown in figure 1.

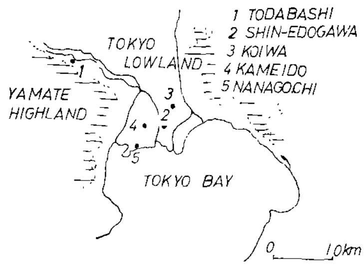

FIGURE 1. *Distribution map of observation well*

Todahashi well Nos. 1-4, Kameido well and Kameido well No. 2, Nanagochi well, Shin-Edogawa wells Nos. 1-3 etc. proved to be suitable as observation wells. It was determined that the aquifers tapped by the Todahashi wells Nos. 1-4 are isolated from each other and have no connection.

CHARACTERISTICS OF THE OBSERVATION WELLS

Todabashi well No. 1, depth -290 m, strainer -258-286 m. Todabashi well No. 1 and Nos. 2 and 3 are not connected according to the test by the Natural Water Gas Standard. It has been shown that (1) the groundwater level in Todabashi well No. 1 is affected by tides; (2) the amplitude of level fluctuation is 10 cm$\pm$; (3) groundwater level in Todabashi well No. 2 showed tidal effects from the time it was drilled to 1961, but thereafter, the situation was disturbed by pumping in the surrounding area and the tidal effects became extinct; (4) changes of groundwater level in Todabashi well No. 3 were measured and the graph is like a saw-blad evidently due to the interference by pumping in the surrounding area; (5) groundwater records in Todabashi well No. 4 indicate that it is mostly dry but at times contains stagnant water.

KAMEIDO WELL AND KAMEIDO WELL No 2

1. Both wells are situated in the same campus, and are about 20 m apart. It is noteworthy that initially the groundwater levels in both wells showed tidal effects. However, subsequently the levels in the Kameido well did not show tidal effects whereas the levels in the Kameido well No. 2 continued to do so.
2. The gound subsidence shows a characteristic decrease. There is no tidal effect in the Kameido well when the groundwater level descends from the upper confining stratum and forms a groundwater table.
3. When the groundwater level again ascends to a more normal position, the curve of tidal change is deformed in consequence.

SHIN-EDOGAWA WELL Nos 1-2

1. The levels of Shin-Edogawa well No. 2 show tidal changes but those of No. 1 indicate no more than occasional disturbances;

2. The depth of No. 1 is 70.5 m, and that of No. 2, 150.5;
3. The amplitude of level fluctuation amounts generally to 15-20 cm in well No. 2;
4. The change of level in well No. 1 is due to the influence of industrial pumpage.

KOIWA WFLL

1. The water levels of the Koiwa well show very uniform tidal changes.

NANAGOCHI WELL NOS. 1-3

1. Groundwater levels were measured only in well No. 2;
2. The groundwater level change was about 20-35 cm and was the largest amplitude obtained from all observation wells, and the magnitude is about twice that of Todabashi well No. 1.

THE PHASE CORRELATIONS BETWEEN TIDAL CHANGES IN GROUNDWATER LEVEL AND TIDES IN TOKYO BAY

1. Tides were measured by the standard Reiganjima Pole (Arakawa Peil Standard) and the values were published every year by the Meteorological Office and incorporated in this study;
2. The interpretation of tide regularity was made by the harmonic analysis method using the observation well on the Tokyo University campus for about 22 min. but according to its mechanism and the geological structure of the area, precise explanations are reserved for future study;
3. In this study, the analysis and interpretation of the correlation of the tides and changes in groundwater level are as follows:

   The tides show a characteristic period in time every day, and has one cycle each half month, and this is continued for two or three months with 2-3 types. It has been useful in correlating the tide and fluctuation of the groundwater level within the distances of 6 km, 10 km and 20 km from the shore well.

The correlation shows that at these distances, wells are affected almost simultaneously by the tide.

ARTERIAL STRUCTURE OF AQUIFER

The strata are composed generally of fine and coarse grained materials, especially the younger and unconsolidated sediments of the Alluvium and Diluvium show such characteristics. The Yurakucho Formation is composed of clay, silt and fine sand and the Tokyo Group contains mainly coarse grained materials with some conglomerates.

It had been thought that the Tokyo Group has no distinct stratification not widely distributed strata, but rather an alternation of beds of different dimensional and areal extent due to the character of the sediments and depositional environment. Therefore, it can be supposed that the group has homogeneity although it consists of unhomogeneous and variable facies and that the intergranular voids of the sediments are connected and easily saturated by water. Thus the Tokyo Group may be considered an aquifer and the direct decrease of intergranular water pressure in the aquifer by heavy pumping occurs wherever the stratum is saturated. This way of thinking necessarily carries the conception of "squeeze" which refers not only to homogeneous and wide spread stratum, but also to intergranular water flow that takes place from high pressure stratum to low pressure and in most cases from the impermeable to the permeable stratum. In short, the way of thinking of the writer is that flow is not limited to the lateral homogeneous and extensive strata, but that groundwater flow also takes place frequently in the arterial network.

As the Tokyo Group sedimentary facies is neither homogeneous nor of distinct layers its geohydrological character cannot be estimated by mathematical procedures. The sediments were originally supplied to this area under the control of gravity, and an arterial course or path of materials of the same porosity cannot be followed, because the beds may intercept one another and also be deposited against the sedimentational plane. If the finer materials are intermixed unhomogeneously with coarser material, the arterial course may have a very complicated structure. These phenomena are encountered in the actual cases. The transmission of intergranular water pressure and flow of the water itself are influenced by the resistance of the materials to the flow, which is controlled by the magnitude and the duration of pressure. Accordingly, partial isolation or a pocket of water within the arterial netwrork occurs frequently under the different environmental conditions.

An irregular mass consisting of fine granualar materials is subject to three dimensional squeezing, depending upon the pressure trend and the intergranular water flow trend. Therefore, the reactions are very variable. Compactions of these mass do not take place uniformly everywhere within the mass body by action of pressure or water squeezing etc.

It may be understood by the explanation above that the structure along the path of coarse material is not affected, as that of sediment mixed with fine material. As the course or path of an arterial structure is chosen, and the writer propose the term "arterial structure of aquifer" to explain the phenomenon, the tidal change of groundwater level occurs directly and far from the shore in some cases as mentioned later.

PROPAGATION AND ITS DISTINCTION

The funnel of the groundwater table induced by pumping is widely distributed in the area. Therefore, the water level was drawn down below the confining bed and the tidal change became extinct, notwithstanding that the tidal change propagated far inland to the well. For instance, in Kameido well and Kameido well No. 2, the writer has quoted "The Paskal's Law" or "Connected Pipe Conception" to explain this phenomenon. Figures 2, 3, show the case, which was reported by N. Miyabe and T. Shiobara *et. al.*, (1965). In this figure, the groundwater level is drawn down beneath the confining bed after some time had elapsed. The rhythmical record is irregular while the sensitive change of groundwater level continues but not rhythmically. The interpretation may be incorrect as a result of neglecting some factors, for instance the pattern of the connected pipe-like paths etc. General observations show that tides affected the groundwater levels in almost all deep wells in the past, and before disturbance by heavy pumping in the Tokyo-lowland and Yamate-highland areas.

ELASTIC DEFORMATION

The sediments of Tokyo Bay are subjected to differential loading by the weight of water according to tides, therefore the stress under which the sediments are subjected is more uniform in the deeper parts of the Bay because of more uniform load; the decreasing ratio according to the water pressure can be evaluated by the Boussinesq concept. Tokyo Bay measures 30 km in diameter and the water load propagates over more than about 90% of the sediments down to 3 km depth from the surface. Those sediments develop a honeycomb structure, open honey-combs structure, and single grained structure. Diagenetic compaction and elastic deformation of the bottom sediments occur by the following process.

The sediments consisting of fine granular materials frequently contain clay (clay minerals) and silt. This kind of admixture of clay particles contributes to the formation of the honey-comb structure. When sufficient fine materials are contained, the adjoining

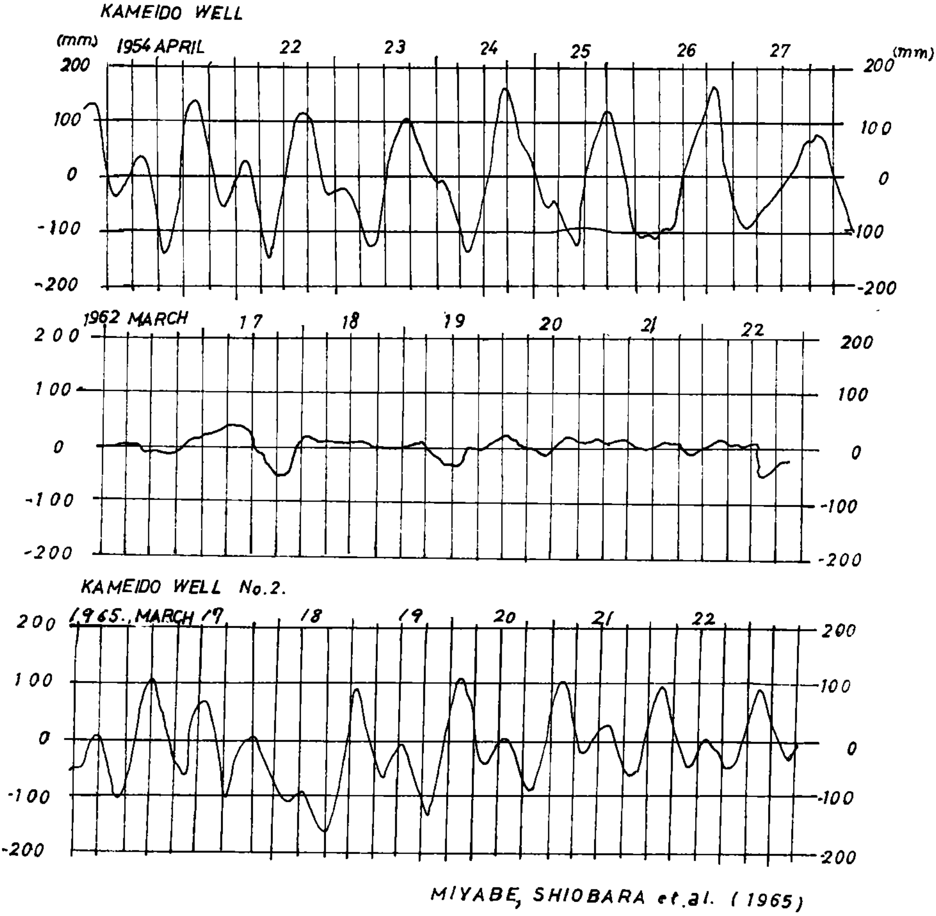

FIGURE 2. *Variation of groundwater level as related to tide change per week*

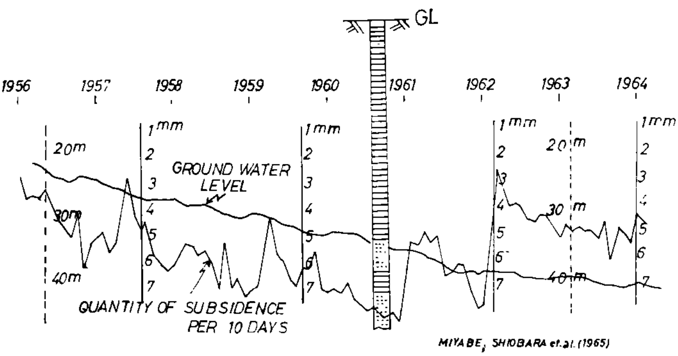

FIGURE 3. *Relation between amount of ground subsidence and lowered quantity of groundwater determined at the Kameido well No. 2*

two or three honey-comb structures may fuse to develop an imperfect septal wall in the mother unit; this is named the "Open honey-comb structure". The structure consists frequently of fine grained materials mixed with coarser ones (silt etc.). In the case of coarse grained material such as gravel, very coarse sand and fine sand, each grain comes into contact directly with one another in general, and the resulting form is called "single grain structure".

DETOUR TRAVELING AND COMPACTION

Groundwater pressure propagation in saturated sediments is controlled like water in a pipe. The ascending of intergranular water by pressure and flow in an arterial network is controlled by the path resistance and its deformation. This effect can be determined by the acting time of the pressure. Thus, the propagated pressure and waterflow can pass more easily along the path of the coarse material distributed along the gradient of waterflow. This phenomenon is named "Detour Traveling".

Heavy pumping causes a drawdown of the groundwater level and decreases the corresponding water pressure. By the duration of such a condition the compaction of fine grain material within the coarse grains goes on gradually. This kind of compaction is controlled by the arterial structure and detour traveling, and the intergranular water is squeezed out by the compaction.

CONCLUSION

The consideration based upon the arterial structure of the aquifer, detour traveling and elastic deformation of the sediments serves well to explain ground subsidence. The groundwater level varies with rhythmical change perhaps due to some large energy in the whole area. The origin of this titanic energy is presumed to be derived from the deformation of the sediments below Tokyo Bay (sea), which suffers change by weight due to the difference of seawater depth by each tide in the whole area of Tokyo Bay. The pulse of the tidal rhythm propagates directly to the Yamate-highland, Tokyo-lowland and surrounding area of the bay. The pressure and the amount of travel of the water are controlled by the arterial structure and detour traveling. In conclusion it can be said that, consideration of the mechanism controlling the compaction of deep sub-surface strata must be based on the proposed conception, especially in and around Tokyo.

ON SUBSIDENCE OF LOESS SOILS OF THE UKRAINE

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About 14% of the territory of the USSR and nearly 65% of the Ukraine are covered with loess. The latter possesses several specific compositional features (predominence of dust fractions, presence of carbonates, poor weathering of the clastogene material) as well as specific properties (fast loosening, rather high filtration rate, anisotropy of the mechanical properties in vertical and horizontal directions, subsidence). Subsidence is among the most adverse features of loess soils. Experience in the construction and operation of different buildings on loess soils furnishes numerous examples of deformation suffered by buildings, industrial installations, and irrigation canals. In several areas of the Ukraine subsidence may be as great as 1.0 to 1.5 meters, with thickness of the loess averaging 20 to 40 meters.

Subsidence results both from soaking of the soils on top (in canals, with emergency in the water supply system) and from rise in the ground-water level following flooding of reservoirs, in the former case subsidence is much more pronounced.

The physical nature of the phenomenon consists in a highly perceptible process of compaction of otherwise in sufficiently compact portions of the loess complex due to desintegration of a greater part of structural bonds in the soil by water (or other liquids), which may be both natural or supplemented by the construction weight. To class a soil with the subsiding type one should employ such values of subsidence relative compression which might reflect sudden commencement and collapse characteristics of the resulting deformation. The lower limit of such a relative compression may be evaluated as 0.02.

Mostly detrimental to a construction (especially at the lower limit of the subsidence rate) is nonuniformity of deformation with time at the construction site or inside the building, rather than the value of the deformation proper.

It is therefore, essential not only to determine the limiting value of the likely subsidence, but to have all the necessary data on the subsidence deformation regime.

The nature of the seemingly simple phenomenon of subsidence is complicated by numerous factors, whose strict consideration is extremely difficult, as yet. A great number of comprehensive determinations of various properties of loess soils, carried out in different areas of the Ukraine, enabled us to get an idea of the role of the main natural and artificial factors in the origine and course of subsidence deformations.

Thus, it has been established that the rate of subsidence is primarily dependent on the value of active free porosity. The term "active" implies prorosity with pores greater than 0.02 mm, involved in the deformation, the term "free" describes free-of-water porosity

Utilisation of the structural-adsorbtion method and investigation of the relation. "porosity-versus relative subsidence" show that the active porosity in loess soils of the Ukraine is over the 30-40% of non-active porosity. Thys makes clear the usually observed relationship: the higher the total porosity of a loess soil, the greater subsidence may result.

Close values of total porosity in similar lithological types (for instance, sandy loams alone or loams alone) are not enough evident to refer them to identically subsiding soils. To cite an example, the asolian loess which is compositionally medium loam with a porosity equal to that of lacustrine medium loam, differs from the latter by marked subsidence properties. Consequently, only soils of the same or identical in origin lithologico-genetic types may be compared in this respect. The difference between subsiding and non-subsiding soils with equal total porosities lies in their structural characteristics. Subsiding soils are characterized by a carcass structure of the skeleton elements with a nonuniform ("cluster" or contact) distribution of a small content of clayey material. The subsidence deformation results from sufficiently fast collapse of such an unstable to soaking carcass structure.

The usual compression strain is a process of slower recomposition of a structure, which may differ in composition but is uniform in structure, from a loess into a more compact state.

With approximately equal values of active porosity a loess soil is found more subsiding if it possesses lower natural humidity, contains a smaller percentage of clayey particles, is characterized by a smaller absorbtion capacity, and has lower indices of hydrophylic properties. All this is abundantly clear and does not require any additional explanation.

The following remains to be discussed: 1) why are clayey minerals of subsiding soils often rich in montmorillonite; 2) why may the subsidence take a stopwise course; 3) why may the subsidence recur with a repeated cycle of soaking.

1. In the mechanism of subsidence the physico-chemical phenomenon of filter water disjoining action is essential. The most subsiding loess soils are formed in steppe and semidesert conditions of arid and semi-arid zones where due to predominance of alkaline conditions of weathering there originate in the clayey portion minerals of the montmorillonite group. Small content of montmorillonite, present in a soil in contact clusters, swells when soaked and still loesens the primarily loose soil structure, thus contributing to a more vigorous manifestation of the water wedge effect.
2. Studies of structural bonds in loess soils have shown that the latter may be represented by clayey materials and various salts. The stopwise character of subsidence results from the nonsimultaneous disintegration of different groups of structural bonds.
3. Recommencement of subsidence following a water-level decline or after a repeated soaking is by no means a result of recovery of the subsidence properties. The phenomenon should rather be explained by the presence in loess soils of a developed system of tubular macropores, owing to which vertical filtering is twice or thrice as great as horizontal, i.e. temporary "closed" hydraulic systems may originate because of an insufficient horizontal discharge of water and suspension of subsidence.

The subsidence properties are studied by means of precise laboratory and field techniques. In the former case the accuracy of the determination of the value of subsidence depends upon the density of sampling and subsequent tests of natural specimens. Calculations of subsidence take account of the anticipated pressure at the base of a construction. The method of odometer testing of two samples-analogues with nearly equal specific weights by two curves is considered very advantageous, as it enables calculations for various loads to be carrid out.

The field techniques include besides direct methods of evaluating subsidence with the help of test pieces, by soaking of exploration areas of canals, groundworks, or parts of experimental constructions, also secondary means of studying density and humidity of the subsiding soils through the use of radioactive isotopes. The techniques of nuclear geophysics together with vibrodrilling installations for driving in the measuring poles make it possible to evaluate density and humidity of soils quickly and without great expenditure.

It is available to conduct such tests between wells with properties accurately measured in laboratory. Besides the analogy method in this case, subsidence may also be qualified by approximate secondary analytical indices.

The above procedure is indispensable for carrying out repeated measurements with a special network of observational vibrodrill wells furnishing data on the regime of variations in soil humidity and density, establishing the character of the soaked soil body, etc. This technique may help in bettering the density of information, especially during explorations of large areas and linear segments.

Other indirect methods of studying density of subsiding soils, such as conventional geophysical or penetration techniques, are less reliable under the conditions. However, the fact should be emphasized that all the above-mentioned seccondary procedures do not ensure any direct answer as to the value of deformation caused by subsidence. Therefore, they should be looked upon as supplementary to conventional techniques, especially when dealing with important engineering tasks.

It should be remembered when studying various deformation properties of soils that the latter are an object of primary geological concern. Soils are generally characterized by a prolonged and rather complicated formation history. Only having thoroughly investigated the geology of a given area, i.e. formation conditions and further trends of development of soil complexes, can one be confident as to the correctness of the selected exploration program, tests, laboratory or field studies, as to the effectiveness of average results furnished by the methods of mathematical statistics, and, finally, as to the reliability of the resulting prognosis.

We shall now consider our experience in studying loess soils of the Ukraine in terms of engineering geology. Despite a substantial body of investigations of loess soils, conducted for various construction sites, the problems of loess top-soil structure, conditions of loess formation, the nature of variations in its composition and properties have not been adequately explained until quite recently.

Thorough analysis of the materials available and of the newly acquired data resulted in the following conclusions. The covering Upper-Pliocene and Quaternary rocks reflect in their composition and appearance the effect of the most significant climatic phase of the period. In particular, the formation of loess top-soils was influenced by an alternating succession of mild interglacial and cold glacial epochs (with Criohygrotic and Crioxerotic subphases each), which is reflected in a distinct rhythmical structure of loess top-soils. Within the accumulative flat geomorphological levels with a minimum degree of different denudation factors, three to four complete loess rhythms are established. Each of the loess rhythms comprises (from the bottom upwards): 1) loessoid loam, which is a comparatively compact, relatively weathered, little subsiding soil formed during humid and cold Criohygrotic subphase; 2) loess-compositionally lighter (in weigth and color), loess, subsiding loam whose formation was connected with the cold and arid Criomerotic subphase of each glacial epoch; 3) fossil soil-loam with traces of the degraded humus, weathered, less subsiding as compared to loess, and formed during a mild interglacial epoch.

In the lower portion of the loess complex compact, non-subsiding loams of the loess-like crust of weathering are common.

Such is the general picture of the conditions favoring the formation of loess top-soils, in which we may observe a regular rhytmical succession of not only lithologico-genetic soil types, but of the composition and properties of the soils in the cross-section, as well.

The above regular picture is violated in a number of areas or individual places: the number of rhythms may decrease, elements of the rhythms may be missing. All this seems to be accounted for by concrete geologic-tectonic conditions (recent active uplifts of the earth's crust), conditions of activation of denudation processes, etc. In terms of the regularities discussed above one may treat the problems of engineering stratigraphy, i.e. the principal basis of engineering geology, which enables us to get a deeper insight into rather, complex and seemingly involved geological features.

With all the necessary geological data for the areas under study at hand we may not only choose the right trend of the investigations for a particular construction site, but may also guarantee the reliability of the engineering geology prognosis and direct the melioration works.

DISCIUSSION

Intervention of Prof. George V. Chilingar (U.S.A.)

Question:

Did you use electrochemical methods to stabilize your compacting soils? By introducing electrolytes in conjunction with application of direct electric current, new minerals (such as hisingerite, allophane, calcite, gypsum, hematite magnetite, etc.) may form which will cement the weak ground together. One can also simply change Na-montmorillonite (highly swelling)to Ca-montmorillonite (swelling to a lesser degree) by base exchange. In our laboratories we have also found that application of direct electric current may destroy clay sheet structure.

Answer of Dr. Kravev

Yes, we have used this technique successfully in some instances.

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TOME II — VOLUME II

SHORT PERIOD VARIATION IN LAND SUBSIDENCE

A. ISIGAKI, Shauzow KOMAKI, Takeshi ENDO and Naomi MIYABE

ABSTRACT

On the records of compaction obtained at several stations in and near Tokyo, we notice fluctuations with nearly diurnal and semidiurnal periods superimposed with secular advancement of subsidence. These short period fluctuations are separated by taking deviations from the 24 hours' moving averages of the hourly values of compaction read on the records, and subjected to detailed study.

Of these, the diurnal change is characterized with particularly remarkable daytime swelling of surface soil layer. The swelling of the soil layer appears to be very nearly parallel to the change in barometric pressures, also to the change in the heights of the ground water level, though the latter is slightly out of phase (with a lag of 2-3 hours). We then studied in some detail the relation between the changes in the barometric pressures and the diurnal variation in land subsidence, taking into consideration the changes in the heights of the ground water level, which may have been influenced by the barometric pressure changes.

The terms of tidal changes are also taken out, of which the M_2 term is most remarkable, though the amplitude is far less than that of the diurnal fluctuations.

RÉSUMÉ

Sur les bandes d'enregistrement de la compacité obtenues dans les stations de Tokyo et des environs, nous observons des variations presque diurnes et semi-diurnes qui viennent se superposer à l'affaissement séculaire. Ces fluctuations de courte durée sont distinguées des valeurs horaires de la compacité lues sur les bandes en prenant les déviations des moyennes mouvantes toutes les vingt-cinq heures, et elles sont soumises à l'étude détaillée.

Parmi elles, le changement diurne est caractérisé par un soulèvement diurne particulièrement remarquable de la surface des couches. Le soulèvement de la couche paraît à peu près parallèle aux variations de la pression barométrique et aussi à celle de la hauteur du niveau des eaux souterraines, bien que la dernière soit un peu hors de phase (avec un retard de 2-3 heures). Ensuite nous avons étudié en détail la relation entre les changements de pression barométrique et la variation diurne de l'affaissement du terrain, compte tenu des changements dans la hauteur du niveau des eaux souterraines, qui auraient pu être influencés par les changements de la pression barométrique.

Les termes des fluctuations de marée sont aussi enregistrés, dont M_2 est le plus remarquable, bien que son amplitude soit moindre que celles de variations diurnes.

1. In the land subsidence area of Tokyo and its vicinity, a number of compaction recorders stations have been installed to obtain continuous records of compaction of the surface soil layer which has considerable thickness. In the records thus obtained, various interesting features may be noted, among which the oscillatory fluctuations with approximate diurnal and semi-diurnal periods are rather conspicuous. These oscillatory fluctuations may be caused by periodic external disturbances, such as direct and indirect effects of tidal force and atmospheric pressure changes.

The land subsidence is undoubtedly due to the lowering of the ground water pressure resulting from excessive withdrawal of ground water. The soft surface soil layer, the compaction of which accounts for the main part of the observed land subsidence, may also undergo oscillatory movements in response to external periodic disturbances. These movements may be characterized by the physical properties of the soil layer which may control the development of continuing land subsidence in response to the lowering of the ground water pressure. An explanation of the mechanism of land subsidence may be developed through a detailed study of the periodic variations observable in the compaction records.

2. The following procedures were used to analyze short period variations in the compaction records.

On the actual graphs from the compaction recorders and those from ground water hydrograph recorders operated simultaneously at a particular station, hourly values were read and their deviations from the twenty-five-hour moving average were calculated. These procedures were followed with regard to the records obtained from the stations at Todabasi, Kameido, and Minami-sunamati, for the years 1956, 1961, and 1966. The locations of these stations are given in figure 1.

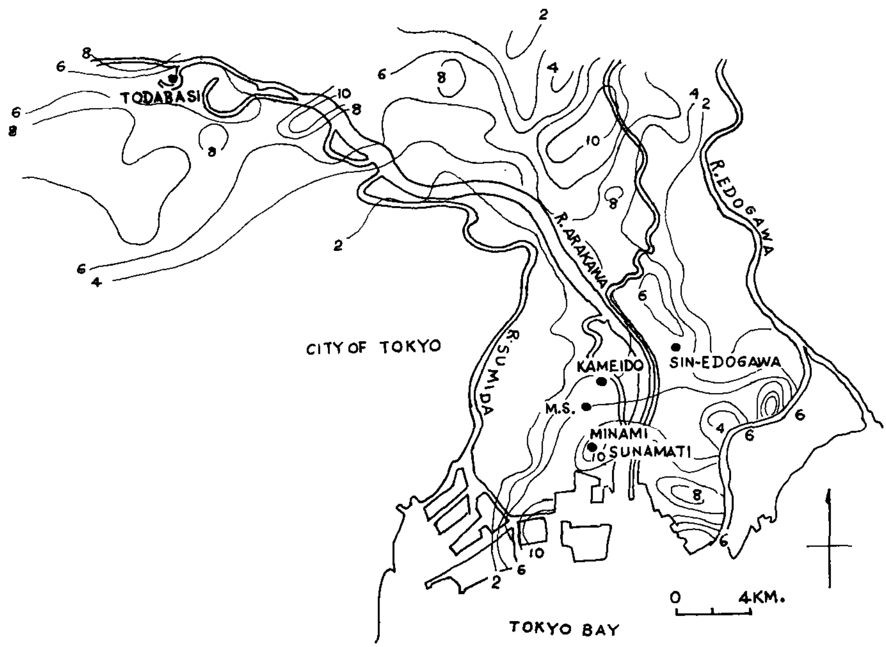

FIGURE 1. *Locations of observation stations in Tokyo*

At Todabasi station are four observations wells, each equiped with compaction recorders and two of them also equipped with hydrographs recorders. For the present analyses, the compaction and hydrograph records that were obtained at No. 1 well were used. The reference tube at No. 1 well is at a depth of 290.0 metres. At the No. 2 well, the reference tube reaches a depth of 113.0 metres. Because the observations at No. 1 well began in the beginning of 1961, and those at No. 2 well began in the end of 1961, it was possible to work with the records obtained during 1961 and 1966 at No. 1 well, but only with those of 1966 from No. 2 well.

At each of the Kameido and Minami-sunamati stations, there are two observation wells, deep and shallow, both equipped with compaction and hydrograph recorders. The reference tubes of the compaction recorders at Kameido station are fixed at the depths of 143.0 metres (deep) and 65.0 metres (shallow), while those at Minami-sunamati station are fixed at depths of 130.0 metres (deep) and 70.0 metres (shallow). Because the operation of the compaction recorders at these stations began in 1954 on the shallow wells and in 1961 on the deep wells, we have available compaction records for 1956, 1961 and 1966 from the shallow wells, and records for 1961 and 1966 from the deep wells, for the present analyses.

3. Attention was first focussed on the component of variation with the exact diurnal period for the short period fluctuations in the land subsidence records because of a conspicuous apparent swell of the surface soil layer which appears late in the morning and decays before evening. Since the hourly deviations of the compactions, computed as mentioned above, may include the components of variations with tidal periods, that is, 12.42 hours (M_2), 12.00 hours (S_2), 25.82 hours (O), 23.93 hours (K_1), and so on, average values of the deviations at each hour of successive days are taken over a period of a year to eliminate the effect of tidal components. In the present report, these average values were taken over 55 days (about two months) for elimination of the most effective component, the M_2 component.

Examples of the average diurnal variations in the land subsidence (strictly, the diurnal variation in the height of the land surface) obtained from the compaction records at the Kameido station are shown in figure 2. Those demonstrate the apparent daytime swell of the surface soil layer.

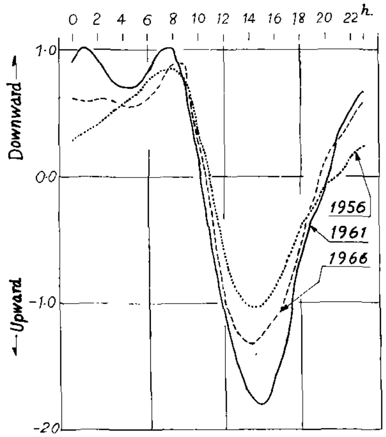

FIGURE 2. *Mean diurnal variation of compaction at Kameido. (Negative values represent swelling)*

The apparent daytime swell of the surface soil layer should be attributed neither to the temperature effect, nor to the mere changes in the height of the ground water level. The atmospheric temperature is relatively high during the period when the swell of the surface soil layer reaches its maximum, and the thermal expansion coefficient of the reference tube of the compaction recorder may not be smaller than that of the soil layer. Hence, the temperature effect on the compaction records, if any, should be apparent contraction of the surface soil layer, in contrast with the swelling observed in the actual records. The change in the height of the ground-water level may not be the only cause of this swelling, because it is quite out of phase with deviations of the compactions. It also was noted that this swelling commences late in the morning and recovers in the evening.

Fortunately, it was found by chance that in the barograph records taken at the compaction recorder station of Sin-edogawa, there was a fluctuation, though with small amplitude, that just was in phase with the fluctuations in the compaction records. Both the barograph records and the compaction records are plotted in figure 3. The similarity of these two fluctuating curves is obvious. Thus it may be concluded that there is a leading component of fluctuation in the compaction records that is caused by the barometric fluctuations.

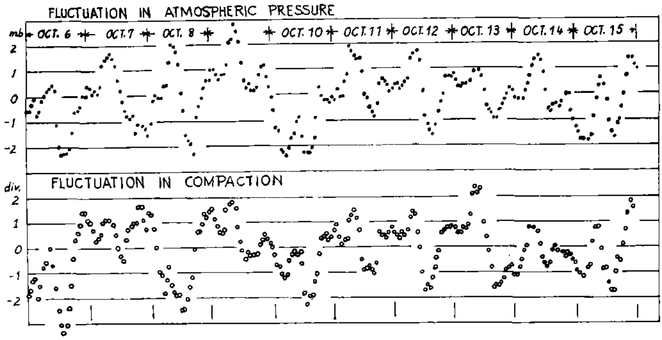

FIGURE 3. *Fluctuations in atmospheric pressure and compaction in records obtained at Sin-edogawa station, during October 1969. (Negative values represent Swelling)*

Another question that then arises is whether the similar relation holds between the smoothed curve of the compaction records and that of the barograph records. The smoothed barograph curves shows quasi-periodic change with an approximate period of several days, and the smoothed compaction records show fluctuations with a similar period. However, the latter fluctuation is more or less out of phase with the former. On the other hand, the fluctuations in the smoothed compaction records seem to be linearly related with those in the smoothed curve showing the gradual change in the positions of the ground water level, observed at the corresponding compaction recorder station.

In order to make this relation clear, the hourly values of compaction, with the short period variations and the secular variation term linearly proportional to time eliminated, are plotted against the corresponding values of the smoothed ground water pressure. The results are shown in figure 4. Since the plot does not cover a sufficiently long period

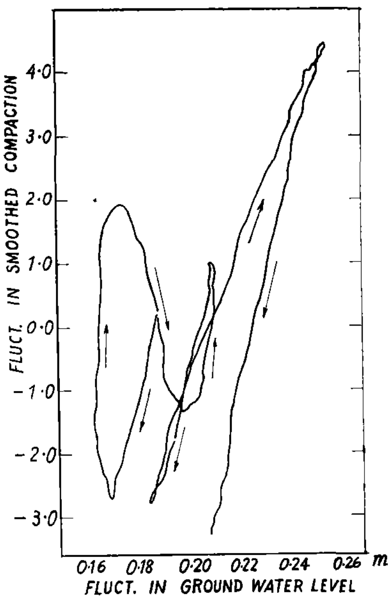

FIGURE 4. *Relation between fluctuations in the smoothed compaction and those in the smoothed heights of ground water level*

the relation revealed in figure 4 may not be descissive. So far as the curve shown in figure 4 is concerned, the dependency on the relation of the compaction and decreasing ground water pressure is different from that of the swelling and increasing ground water pressure. If this relation is found in other cases, it is presumable that there may be retardation of strain yielded in the soil layer under the influence of alternate applications of positive and negative stresses. The soil mass under consideration may then be of the nature of a rheological substance.

4. The analyses were carried out using compaction records from the Kameido, Minami-sunamati, and Todabasi station in Tokyo, for the years 1956, 1961 and 1966, and using compaction records from Tidori-tyo (Kawasaki) in 1968. However, because the operation of the compaction recorders at No. 2 wells of the Kameido and Minami-sunamati stations began in the beginning and fall of 1961, the analyses of compaction records from these stations for the years 1956 and 1961 were impossible.

The average diurnal variations of compaction at these stations thus determined are approximately the same in their phases, but generally different in their amplitudes. The curves of average diurnal variation of compaction at these stations for the year 1961 are shown in figure 5.

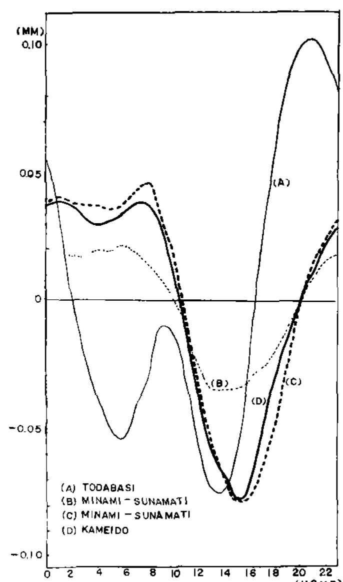

FIGURE 5. *Mean diurnal variations of compaction at Todabasi (A), No. 2 well of Minami-sunamati (B), No. 1 well of Minami-sunamati (C), and No. 1 well of Kameido, for the year 1961. (Negative values represent swelling)*

The amounts of apparent swelling in the diurnal variation of compaction thus obtained for the stations mentioned above are given in table 1, together with the data of annual subsidence (annual amounts of compaction), thicknesses of soil layer, the heights of the ground water level (annual mean values), and annual fall of the ground water level, at respective observation stations.

As for the time variation, if any, of the apparent swelling of the soil layer, refer to the curves in figure 2. In this figure, the average diurnal variations of compaction at Kameido station are given referred to an arbitrary datum, and are converted into actual

TABLE 1. Apparent Swelling and Reference Data

Amount of swelling	Thickness of layer	Annual Compaction	Height of G.W.L. (Annuel Mean)	Fall of G.W.L.
Kameido No. 1 Well				
0.11 mm	64 m	74.4 mm	−37.9 m	3.9 m
Minami-sunamati No. 1 Well				
0.12 mm	70 m	121.5 mm	−32.2 m	2.9 m
Minami-sunamati No. 2 Well				
0.053 mm	130 m	109.0 mm	−33.1 m	1.5 m

amounts of swelling of about 0.06 mm in 1956, 0.11 mm in 1961, and 0.09 mm in 1966. In 1961, the annual compaction at Kameido station reached nearly a peak value and then declined. The amounts of apparent swelling seem to have followed the rise and fall of the amounts of yearly compaction.

5. If the apparent swelling is, caused mainly by the depression in the atmospheric pressure, it also may be influenced by the change in the height of the ground water level which too is affected by the change in the barometric pressure.

An analysis in this direction was made with the compaction records obtained at Tidori-tyo (Kawasaki) and at Sin-edogawa observation stations. On analyzing the compaction records obtained at Tidori-tyo station, for the sake of simplicity, the diurnal variation curve of compaction is worked out by taking average values of each hourly readings over a two month period, from September to October of 1968. This is done by putting the value of the 0-hour reading of each day equal to zero, in order to eliminate the effect of secular variation of compaction. The curve thus obtained is compared with the mean diurnal variation in barometric pressure and with the height of the ground water level observed at the same station. From the results, we notice discrepancies between the minima of these three types of curves. Therefore, the compaction, or strictly speaking, the height of the land surface, u may be expressed, as a function of atmospheric pressure and the height of the ground water level, by

$$u = ah + bp + u_0, \tag{1}$$

where u. h, p are respectively the hourly values of the height of the land surface recorded, those of the ground water level and the atmospheric pressure, and u_0, a,b are the constants to be determined statistically, using the data from the compaction records obtained at the Tidori-tyo station. The values of constants thus determined are $u_0 = 104.89$, $a = 0.81$, and $b = -3.68$, where the negative sign of b designates that the land surface moves upwards in response to the depression of the atmospheric pressure. The estimated compaction curve calculated on the basis of equation (1) is given by filled circles in figure 6. The empty circles in figure 6 show the actual observed diurnal variations in the height of the land surface.

Similar analysis was made of the compaction records obtained at the Sin-edogawa observation station. The compaction records used were obtained during October 1968, and the results are expressed by

$$u = 2.54h - 1.91p, \tag{2}$$

where the fluctuation in the height of the ground water level, h, is measured in cm. and those in the barometric pressure, p, in milli-bars. Although the magnitudes of the coefficients are different from those obtained with the compaction records in Tidori-tyo, Kawasaki, the phenomena of apparent swelling observed at stations in Tokyo and in Kawasaki are thus recognized as of similar nature.

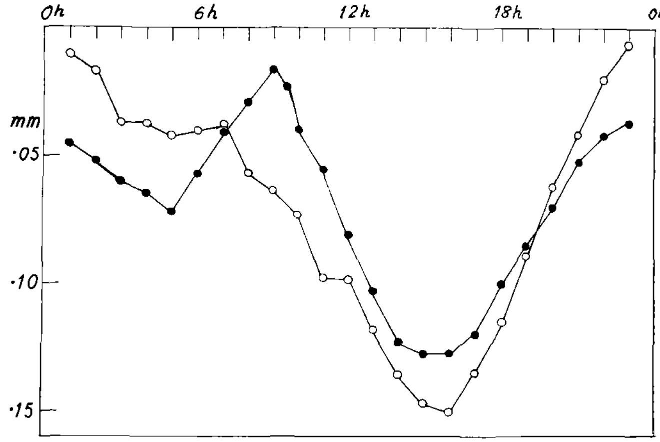

FIGURE 6. *Observed (empty circles) and estimated (filled circles) mean diurnal variations of compaction at Tidori-tyo, Kawasaki, during September to October, 1968. (Negative values represent swelling)*

7. Apparent swelling of the surface soil layer is generally observed at various observation stations in Tokyo and Kawasaki, as previously described and is caused in the main by the fluctuation in the atmospheric pressure. However, several exceptional cases also are notable. One of them is the mean diurnal variation curve obtained from the compaction records from No. 1 well of the Minami-sunamati station for the 55-day period in the beginning of 1961. The curve is shown in figure 7a, in which the curves for different periods also are shown for reference. Another example is given in figure 7b, which shows

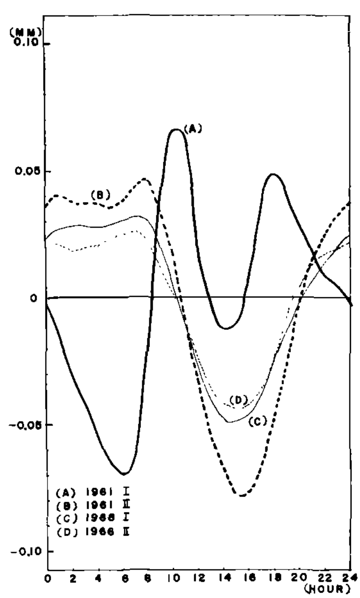

FIGURE 7a. *Anomalous mean diurnal variation of compaction at No. 1 well of Minami-sunamati station, obtained for the first 55 day period of 1961. (Negative values represent swelling)*

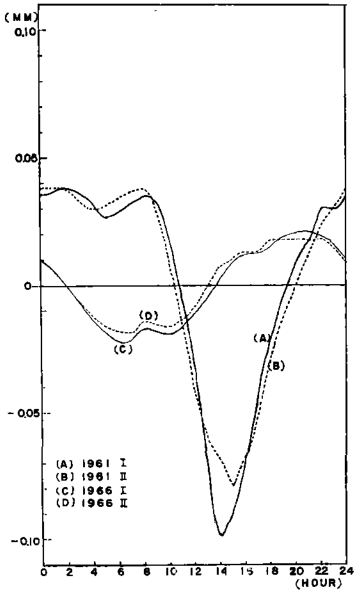

FIGURE 7b. *Anomalous mean diurnal variation of compaction at No. 2 well of Kameido station, obtainet for the first and second 55 day period of 1966. (Negative values represent swelling)*

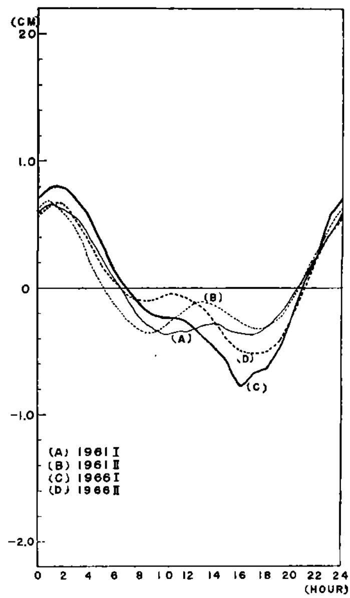

FIGURE 7c. *Mean diurnal variation in the height of ground water level at No. 2 well of Kameido station*

the diurnal variation curves deduced from the compaction records obtained at No. 2 well of the Kameido station for the first and second 55 day period of 1966. For reference, the curves designating the diurnal variation of compaction for similar periods of 1961 also are shown in the figure.

These exceptional diurnal variations seem to be more or less affected by the diurnal change in the heights of the ground water level. But there is a considerable phase difference between these two kinds of curves. The difference may be noted by comparing the curves in figure 7b with those of figure 7c, which show the curves of diurnal changes in the heights of the ground water level at No. 2 well of the Kameido station for the corresponding periods.

The diurnal variation in the compaction of the surface soil layer may then be regarded as having occurred under the influences of change both in batometric pressure and in the heights of the ground water level. This assumption, however, may not be a satisfactory approach to the solution of the problem in that the soil layer occasionally is unresponsive to the change in the barometric preessure. Because this assumption does not imply the source of change in the response characteristics of the soil layer, the problem should therefore be reserved for future investigations.

8. Lastly, additional remarks should be made on the results of tidal analyses in association with the compaction records. Among a number of tidal components, the M_2 comnent only is determined, and the results are given in the table below.

TABLE 2. M_2 Component in Compaction

Station	Time interval	Amplitude	Phase
No. 1 Well, Minami-sunamati	January 1-February 27,1961	0.0024 cm	208
No. 2 Well, Kameido	January 1-February 15,1961	0.0019 cm	271
No. 2 Well, Minami-sunamati	May 23-September 4,1961	0.0010 cm	271

The results show no remarkable change from those worked out with the compaction records obtained at the observation stations in the Yokkaiti land subsidence area [1], and those with the compaction records obtained at stations in Tokyo in 1934 [2]. The amplitudes of the tidal components in the compaction thus obtained are of lower order of magnitude in comparison with those which are recognized as caused by the fluctuations in the barometric pressure.

Tidal analyses on the compaction records obtained at stations in Tokyo, however, are in progress, and the further results will be published when the analyses have been completed.

In conclusion, the authors would like to express their sincere thanks to the members of the Tokyo Institute of Civil Engineering for their kind assistance in the preparation of this paper.

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ON THE INFLUENCE OF OCEANIC TIDES UPON ARTESIAN HEAD AND SURFACE LEVEL OF LAND

Shauzow KOMAKI

ABSTRACT

In our observations of artesian head and subsidence in and near Yokkaichi, diurnal and semi-diurnal period fluctuations are superposed upon secular variations. We investigated those fluctuations caused by oceanic tides, meteorological and other factors.

After eliminating secular variations the hourly values of artesian head and land surface were harmonicaly analysed by the method of least squares.

The same calculation was made concerning the tide curve observed near one of the observation stations.

Regarding M_2 and S_2, the similarities of the different values in phase lag between oceanic tides and artesian head are found in individual wells. This is also the same with K_1 and O_1. Such relations are found between oceanic tides and land surface too, though somewhat less in similarity.

The respective ratios in amplitude of S_2 to M_2 in regard to artesian head and of land surface in each well are distributed around the ratio of oceanic tides, 0.455.

RÉSUMÉ

Dans nos observations de la hauteur piézométrique dans les couches artésiennes ainsi que des affaissements à et autour de Yokkaichi, des fluctuations diurnes et semi-diurnes sont superposées aux variations séculaires. Les auteurs étudient les fluctuations dues aux marées océaniques, aux facteurs météorologiques et autres, après avoir éliminé les fluctuations séculaires.

Les valeurs horaires des hauteurs piézométriques dans les couches artésiennes et celles de la surface du sol sont analysées par la méthode des moindres carrés.

Le même calcul est fait pour les courbes de marée observées près de l'une des stations.

En ce qui concerne M_2 et S_2, la similitude de phase entre les marées océaniques et les hauteurs piézométriques artésiennes est trouvée dans les puits individuels. Il en est de même pour K_1 et O_1. Des relations analogues sont également trouvées entre les marées océaniques et les hauteurs du sol, ces relations indiquent cependant une similitude moindre.

Les rapports entre les amplitudes de S_2 et de M_2 en regard de la pression artésienne et de l'affaissement de surface dans chaque puits sont distribués autour de 0,455.

1. INTRODUCTION

For a number of years the influence of oceanic tides upon ground water and surface layer of land has been known. By comparing harmonic constants of a component of oceanic tides with those of the same component of periodic fluctuations of ground-water level or of land-surface level, some investigators concluded that oceanic tides exerted some influence on both phenomena. However, their discussions are not always sufficient, which may be due to the short period for harmonic analysis and to imperfection in eliminating a zero line, namely a secular variation curve.

As one of the methods for the investigation of subsidence and of the causes of land-surface subsidence, observation wells were installed at three stations in and near Yokkaichi. Artesian head and subsidence have been observed since August 1963. As given in detail in the other article[1], the variations of both artesian head, that is ground-water level, and land-surface level observations are composed of two parts. The first part is a secular variation with long period, due probably to ground-water withdrawal only, and the other is a fluctuation caused by oceanic tides, meteorological and other factors, such as temperature, solar radiation and daily withdrawal, with periods of one day or less. In the previous article[1], the secular variations of both the ground-water level and land-surface level in each observation well were discussed. In the present paper, the fluctuation due to oceanic tides, meteorological and other factors is investigated in detail, after removing the secular variation.

2. ANALYSIS

The methods available for determination of secular variation, successive moving means 24 and 25 hours duration, have been in use for a long time. The moving mean of 25 hours duration is satisfactory for the lunar components, but not for the solar ones. Conversely, the moving mean of 24 hours duration is satisfactory for the solar components, but not for the lunar ones. An excellent method satisfactory to both the components was devised by A.T. Doodson and H.D. Warburg in 1941, as a method for estimation of a mean sea level by eliminating oceanic tides. This is a 30 hours' selected mean and is usually called the Admiralty method. The method, however, requires considerable labour in the calculation procedure. In 1957, B.P. Pertzev succeeded in halving the calculation time by symplifying the Admiralty method without any lowering in accuracy. His efficient method is a 15 hours' mean, as shown in the following formula:

$$A_t = [a_{t-18}+a_{t-13}+a_{t-10}+a_{t-8}+a_{t-5}+a_{t-3}+a_{t-2}+a_t+a_{t+2}+a_{t+3}+a_{t+5}+a_{t+8} +a_{t+10}+a_{t+13}+a_{t+18}]/15,$$

where A and a are respectively the secular and observed values at t hours and a_{t+i} is the observed value at $t \pm i$ hours [4]. The method has been shown by many investigators, to be excellent for estimation of the drift curve of gravity [2, 3]. This method can also be applied for determining hourly secular values from hourly observed values of ground water level and land-surface level.

Observations were made at hourly intervals for about a one-year period from 00 hour 1st January 1964 to 07 hours 4th January 1965. For each observation, therefore 8864 hourly read values were used in calculation, assuming there was no omisson in recording. After eliminating the hourly secular values from the hourly read values, the obtained hourly values of both the ground-water level and the land-surface level were harmonically analysed by the method of least squares into components, the periods of which are the same as those of the main components of oceanic tides, i.e. M_2 (principal lunar semi-diurnal component), S_2 (principal solar semi-diurnal component), K_1 (luna-solar declinational diurnal component) and O_1 (lunar declinational diurnal component), and the meteorological diurnal component S_1, in order to obtain harmonic constants. These hourly values used in each calculation range from 18 hours 1st January 1964 to 13 hours 3rd January 1965, and are 8828 in number, assuming there is no omission in recording. All calculations were carried out by the "IBM-7090". In the same way, calculations were made for the tide curve which was observed at the tidal station, situated about 200 metres from the 2nd observation station.

From the harmonic constants, hourly expected values, hourly residuals—differences between the hourly expected values and the hourly observed values eliminating the zero line—, and a mean square error were calculated in the course of the calculation of the harmonic constants. Both the residuals and the mean square error were very small. It becomes evident that the amplitudes of the components which are not dicussed are too small to show any significant results from the observational data.

3. DISCUSSION

In table 1 are shown the observed amplitude and the phase lag at 00 hour 1st January 1964 of each component obtained by harmonic analysis. The standard error of each harmonic constant computed from the mean square error and coefficients of normal equations in the method of least squares is sufficiently small. This means that thee error of each amplitude is of the order of 10^{-4}, and that of each phase lag is of the order of 0.1° at the most.

With regard to the S_i-component for the ground-water level, a glance at table 1 shows that the amplitudes are fairly small as compared with those of the other components and the phase lags are scattered over the whole range. On the other hand, for the land-surface level, the amplitudes are considerably larger and the phase lags are within the range −155°.5 to −168°.0. It seems supposedly that the *S*-component has a considerable influence on the surface layer but not on ground water level. From considerations in the other article [1], however, the phenomenon is believed to be attributed to bending of the steel beam to which the apparatus for measuring land subsidence is attached, due to expansion caused by changes of air temperature and solar radiation.

TABLE 1. Results obtained from the harmonic analysis after elimination of secular variation by Pertzev's method

	Component	GWL		LS	
		Amplitude	Phase lag	Amplitude	Phase lag
		m	°		
	M_2	0.6296	−160.4		
	S_2	0.2867	−158.9		
Oceanic tides	K_1	0.2410	−175.2		
	O_1	0.1805	−174.1		
	S_1	0.0094	158.5		
				mm	°
	M_1	0.0076	83.0	0.0892	130.3
	S_2	0.0031	91.9	0.0340	161.9
1 - 4	K_1	0.0054	− 8.3	0.0399	35.3
	O_1	0.0024	− 13.3	0.0159	15.1
	S_1	0.0029	64.6	0.3158	−167.7
	M_2	0.0205	15.1	0.0826	88.4
	S_2	0.0085	34.9	0.0322	121.2
1 - 3	K_1	0.0265	− 38.4	0.0248	2.72
	O_1	0.0147	− 45.2	0.0064	− 5.8
	S_1	0.0721	112.5	0.2759	−155.5
	M_2	0.0041	168.8	0.0819	−127.1
	S_2	0.0015	99.4	0.0280	179.6
1 - 2	K_1	0.0021	−120.5	0.0313	− 70.4
	O_1	0.0023	−128.1	0.0093	−94.9
	S_1	0.0028	−158.2	0.3071	−158.0
	M_2	0.0016	158.4		
	S_2	0.0004	89.0		
I - 1	K_1	0.0008	97.7		
	O_1	0.0007	96.5		
	S_1	0.0026	141.4		
	M_2	0.3049	−121.8	0.3006	− 66.2
	S_2	0.1394	−119.9	0.1328	− 70.8
II - 4	K_1	0.1282	−148.0	0.1521	−116.7
	O_1	0.0951	−152.3	0.1189	−127.8
	S_1	0.0221	−114.8	0.1366	−168.0

	Component	GWL		LS	
		Amplitude	Phase lag	Amplitude	Phase lag
II - 3	M_2	0.1820	−133.1	0.2440	− 61.7
	S_2	0.0809	−132.5	0.1063	− 72.1
	K_1	0.0660	−163.8	0.1148	−133.0
	O_1	0.0510	−162.5	0.0970	−135.8
	S_1	0.0116	−177.0	0.1716	−165.9
II - 2	M_2	0.0755	− 45.8	0.1467	19.2
	S_2	0.0329	− 49.6	0.0630	10.3
	K_1	0.0472	− 67.6	0.0984	− 36.9
	O_1	0.0329	− 67.7	0.0554	− 39.6
	S_1	0.0062	− 94.7	0.2186	−160.7
II - 1	M_2	0.1171	− 69.8		
	S_2	0.0475	− 58.6		
	K_1	0.0623	−111.2		
	O_1	0.0469	−114.5		
	S_1	0.0091	−111.7		
III - 3	M_2	–.1645	− 85.0	0.0646	− 27.7
	S_2	0.0856	− 80.0	0.0318	− 5.3
	K_1	0.0772	−124.7	0.0134	− 90.5
	O_1	0.0475	−128.3	0.0207	−103.3
	S_1	0.0818	91.0	0.2275	−160.6
III - 2	M_2	0.1511	− 79.5	0.0433	− 23.2
	S_2	0.0758	− 78.6	0.0212	− 9.4
	K_1	0.0648	−140.3	0.0104	−108.8
	O_1	0.0476	−144.9	0.0028	−122.9
	S_1	0.0682	149.5	0.1584	−163.3
III - 1	M_2	0.0054	− 47.2		
	S_2	0.0026	− 76.0		
	K_1	0.0079	− 71.4		
	O_1	0.0084	− 62.1		
	S_1	0.0018	136.5		

In the following, relations between oceanic tides and the ground-water level or the land-surface are described for the observation wells at the three stations.

1. Phase lag. In table 2 are given values of phase lag between each component of oceanic tides and the ground-water level or the land surface at the respective observation wells. For ground-water level, the phase lags at individual wells are similar for the two semi-diurnal components: M_2-component and S_2-component, and for the two diurnal components: K_1-component and O_1-component. This relation in phase lag is found in the case of the land surface too, though it is somewhat less marked.

Similarly, differences in phase lag between the land surface and the ground-water level for each component besides S_1-component at the respective observation wells, in the form of angle of rotation and time, are shown on the left- and right-hand sides of table 3.

TABLE 2. Phase lag_t—Phase $lagt_{OT}$

		M_2	S_2	K_1	O_1	S_1
		°	°	°	°	°
GWL	I - 4	243.4	250.8	166.9	160.8	266.1
	I - 3	175.5	193.8	136.8	128.9	314.0
	I - 2	329.2	258.3	54.7	46.0	43.3
	I - 1	318.8	247.9	272.9	270.9	342.9
	II - 4	38.6	39.0	27.2	21.8	86.7
	II - 3	27.3	26.4	11.4	11.6	24.5
	II - 2	114.6	109.3	107.6	106.4	106.8
	II - 1	90.6	100.3	64.0	59.6	89.8
	III - 3	75.4	78.9	50.5	45.8	292.5
	III - 2	80.9	80.3	34.9	29.2	351.0
	III - 1	113.2	82.9	103.8	112.0	338.0
LS	I - 4	290.7	320.8	210.5	189.2	33.8
	I - 3	248.8	280.1	197.9	168.3	46.0
	I - 2	33.3	338.5	104.8	79.2	43.5
	II - 4	94.2	88.1	58.5	46.3	33.5
	II - 3	98.7	86.8	42.2	38.3	35.6
	II - 2	179.6	169.2	138.3	134.5	40.8
	III - 3	132.7	153.6	84.7	70.8	40.9
	III - 2	137.2	149.5	66.4	51.2	38.2

TABLE 3. Phase lag_{LS}—Phase lag_{GWL}

	M	S	K	O	M	S	K	O
					h	h	h	h
I - 4	47.3	70.0	43.6	28.4	1.63	2.34	2.90	2.04
I - 3	73.3	86.3	61.1	39.4	2.53	2.88	4.06	2.82
I - 2	64.1	80.2	50.1	33.2	2.22	2.67	3.33	2.38
I - 4	55.6	49.1	31.3	24.5	1.92	1.64	2.08	1.76
I - 3	71.4	60.4	30.8	26.7	2.47	2.01	2.05	1.92
I - 2	65.0	59.9	30.7	28.1	2.24	2.00	2.04	2.02
I - 3	57.3	74.7	34.2	25.0	1.98	2.49	2.27	1.79
I - 2	56.3	69.2	31.5	22.0	1.94	2.31	2.09	1.58

This shows that the values of time shift are similar at the respective observation wells, and also at the one observation station, though the values scatter slightly.

2. Amplitude. With regard to the M_2 and S_2-components, ratios in amplitude of both the ground-water level and the land surface to that of oceanic tides at each observation well are shown in table 4. The influence of oceanic tides on ground water amplitude is

TABLE 4. Amplitude$_i$/Amplitude$_{OT}$

	GWL		LS	
	M2	*S2*	*M2*	*S2*
			× 10Èé	× 10éÈ
I - 4	0.012	0.011	0.141	0.119
I - 3	0.033	0.030	0.131	0.112
I - 2	0.007	0.005	0.130	0.098
I - 1	0.003	0.001		
I - 4	0.484	0.486	0.477	0.464
I - 3	0.289	0.282	0.387	0.371
I - 2	0.120	0.115	0.233	0.220
I - 1	0.186	0.166		
I - 3	0.261	0.298	0.103	0.111
I - 2	0.240	0.264	0.069	0.074
I - 1	0.009	0.009		

30$\sim$50% in the deeper wells near the coast but is very small in the inland wells, and that on surface layer is of the order of 10^{-4}. In table 5 are shown ratios in amplitude of the S_2-component to M_2-component in regard to the oceanic tides, the ground-water level and the land surface at each well. These ratios are distributed around the ratio for oceanic tides, 0.455.

3. Relation between oceanic tides and the ground water in coastal aquifers. As mentioned above, oceanic tides have an influence upon the ground water in coastal aquifers. Consider the uniform semi-infinite sand and gravel of figure 1 and assume that

TABLE 5. Amplitude$_{S2}$/Amplitude$_{M2}$

	GWL	LS
O T	0.455	
I - 4	0.411	0.381
I - 3	0.416	0.390
I - 2	0.366	0.342
I - 1	0.250	
I - 4	0.458	0.443
I - 3	0.445	0.436
I - 2	0.435	0.430
I - 1	0.405	
I - 3	0.518	0.492
I - 2	0.502	0.490
I - 1	0.482	

a sea level in the effectively infinite coast rises and falls uniformly in accordance with

$$h = h_0 + h' \cos \omega t \tag{1}$$

where:

h_0 is a mean height;
h' is the amplitude of a component of oceanic tides;
ω is its angular velocity and
t is time.

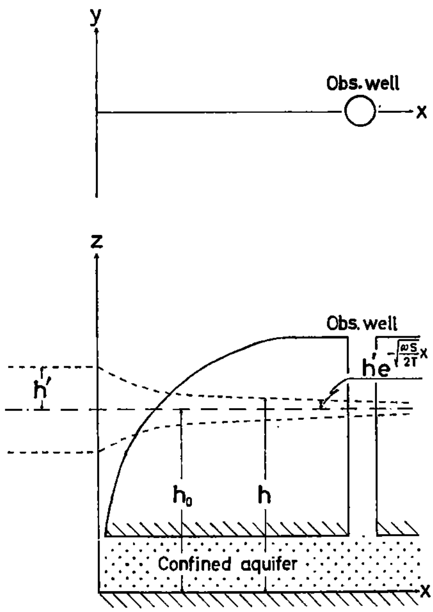

FIGURE 1. *Influence of oceanic tides upon ground water in a confined coastal aquifer*
h : *Height of piezometric surface;*
h_0 : *Mean height of piezometric surface;*
h' : *Amplitude of a component of oceanic tides;*
ω : *Angular velocity of a component of oceanic tides;*
S : *Storage coefficient;*
T : *Coefficient of transmissibility;*
x : *Distance inland from the outcrop.*

In this case, the ground water in a confined coastal aquifer moves as in the following formula

$$h = h_0 + h' \mathrm{e} - \sqrt{\frac{\omega S}{2T}}\, x \cos\left(\omega t - \sqrt{\frac{\omega S}{2T}}\, x\right) \tag{2}$$

where:

S is the storage coefficient of the aquifer;
T is the coefficient of transmissibility and
x is the distance inland from the outcrop.

Then, $e-\sqrt{(\omega S/2T)}\,x$ and $\sqrt{(\omega S/2T)}\,x$ in equation (2) are equal to the ratios in amplitude of the ground-water level to that of oceanic tides in table 4 and the different values φ_{obs} in phase lag between ground-water level and oceanic tides in table 2. Therefore, with regard to the M_2-component and the S_2-component, the different values $\sqrt{(\omega S/2T)}\,x$ in phase lag between the ground-water level and the oceanic tides are calculated using equation (2) and the values of implitude ratio given in table 4. In table 6 are given calculated values φ and observed values φ_{obs}. These values are similar except for those in the shallow wells.

TABLE 6. Observed value (Phase lag$_{GWL}$—Phase lag$_{OT}$): φ_{obs} and calculated value φ from(Amplitude$_{GWL}$/Amplitude$_{OT}$) : $e\sqrt{(ws/2Tx}$

		$e-\sqrt{(\omega S/2T)}x$	$\sqrt{(\omega S/2T)}x$	φ	φ_{obs}
I - 4	M_2	0.012	4.42	253.4	243.4
	S_2	0.011	4.51	258.4	250.8
I - 3	M_2	0.033	3.41	195.5	175.5
	S_2	0.030	3.51	200.9	193.8
I - 2	M_2	0.007	4.96	284.3	329.2
	S_2	0.005	5.30	303.6	258.3
I - 1	M_2	0.003	5.81	332.9	318.8
	S_2	0.001	6.91	35.8	247.9
II - 4	M_2	0.484	0.73	41.6	38.6
	S_2	0.486	0.72	41.3	39.0
II - 3	M_2	0.289	1.24	71.1	27.3
	S_2	0.282	1.27	72.5	26.4
II - 2	M_2	0.120	2.12	121.5	114.6
	S_2	0.115	2.16	123.9	109.3
II - 1	M_2	0.186	1.68	96.4	90.6
	S_2	0.166	1.80	102.9	100.3
III - 3	M_2	0.261	1.34	77.0	75.4
	S_2	0.298	1.21	69.2	78.9
III - 2	M_2	0.240	1.43	81.8	80.9
	S_2	0.264	1.33	76.3	80.3
III - 1	M_2	0.009	4.71	269.9	113.2
	S_2	0.009	4.71	269.9	82.9

From the significant results above-mentioned, it can be concluded that oceanic tides exert a direct influence upon both ground water and the surface layer of land.

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SOIL-WATER DISEQUILIBRIUM AS A CAUSE OF SUBSIDENCE IN NATURAL SOILS AND EARTH EMBANKMENTS

O.G. INGLES and G.D. AITCHISON

ABSTRACT

The widespread occurrence of localised soil subsidences following sub-surface tunnelling, both in natural soils and in earth embankments is explained in terms of a metastable state involving chemical, hydrological and mechanical factors. The sources of excessive voids which precondition the soil for such failures are described, also the conditions under which water flow into these voids adversely affects the safety of the soil structure. The hazards are shown to be most acute when the soil itself is capable of spontaneous dispersion and/or loss of cohesion in contact with water of a high purity or whose chemical composition is not in equilibrium with the soil.

Appropriate procedures for evaluation of the risk of such tunnelling subsidences are described, and available remedial measures noted.

RÉSUMÉ

La production fréquente d'affaissements localisés du sol par écoulement souterrain, tant dans les sols naturels que dans les remblais est expliquée comme résultant d'un équilibre instable dû à des facteurs chimiques, hydrauliques et mécaniques. L'origine des grands vides qui prédisposent les sols à ces affaissements est décrite ainsi que les conditions sous lesquelles le courant d'eau dans ces vides affecte défavorablement la structure du sol. Les chances de production de ces accidents sont augmentées si le sol lui-même est capable d'une dispersion spontanée et/ou d'une perte de cohésion au contact d'une eau de grande pureté ou dont la composition chimique n'est pas en équilibre avec le sol.

On décrit les moyens capables d'évaluer le risque de production de tels vides ainsi que les remèdes qui peuvent être appliqués.

INTRODUCTION

Subsidence, of a highly localised nature, is frequently observed in open field natural profiles (fig. 1). In such regions earthworks—for example, small dams, bridge abutments, etc.—have been observed to suffer a high incidence of tunnelling failure often accompanied by equally localised subsidences in the embankment soil (fig. 2).

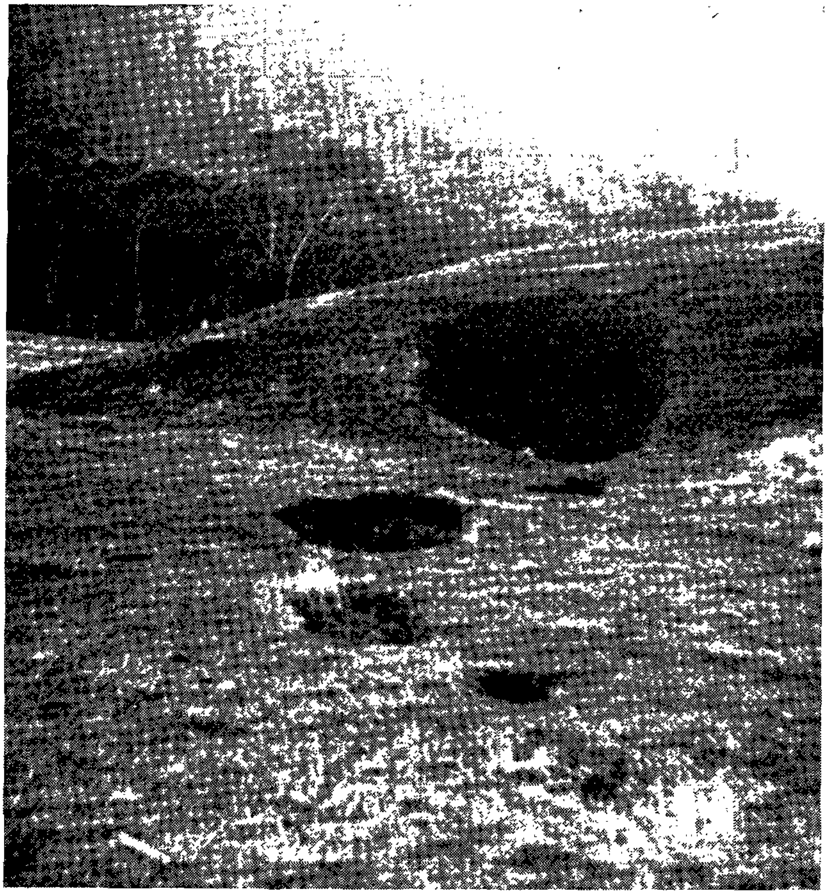

FIGURE 1. *Field tunnelling and subsidences. East Risdon, Tasmania*

The incidence of such failures and subsidences is widespread, being reported from every continent (e.g. Fuller (1922); Parker (1963); Denisov, Bally and Antonescu (1960); Dobrovolny (1962); Downes (1946); King (1951) to quote but a few); and late as 1950 the underlying causes were still considered unclear (Casagrande, 1950). Since that time however, various aspects of the problem have been separately clarified so that it is now possible to present a general statement of the contributory causes and their relative effect, the means available for prior assessment, and remedial measures where necessary. It is shown that these contributory causes appear to be very diverse, encompassing climatic environment, land use, soil type and chemical composition, hydrological factors and constructional procedure; but that they can be interpreted overall as particular expressions of soil-water disequilibrium, the adjustment circumstances of which control the subsidence or safety of the soil.

It is important that these circumstances should be placed in correct inter-relationship, since major structures are still endangered by their unrecognised occurrence, as exemplified by the Baldwin Hills (USA) and Flagstaff Gully (Tasmania) dam failures in recent years (Jessup, 1964; Ingles, Lang, and Richards, 1968).

FIGURE 2. *Tunnelling and subsidence in small earth dam, Stratford, N. S. W.*

THE BASIC MECHANISM OF LOCALISED TUNNELLING AND SUBSIDENCE

Two factors are essential for tunnelling and subsidence to occur. The first, or static factor, which is present *ab initio*, is the existence of substantial porosity (or better, voidage) in the soil. The second, or dynamic factor, which triggers the failure, is the lessening of interparticle bond forces in the soil. The latter, which will be discussed first, can be readily caused by water; and therefore any change in an existing soil-water equilibrium state should be carefully considered both as regards the new equilibrium state to be established and the transient paths by which this may be achieved, whenever reliance is to be placed on the strength and cohesion of a soil mass.

Natural soils are found bonded either with oxides, silicates or carbonates precipitated by seasonal moisture changes at the particle contact zones and which, being insoluble in water, provide an effectively permanent bond (e.g. in lateritic soils, krasnozems, etc.); or else bonded by clays and (within the clays themselves) concentrated salt solutions or partially soluble salts such as gypsum. It is the group of clay-and-salt-bonded soils wherein water advent may seriously lessen the interparticle bonds with dangerous consequence.

Reduction of the interparticle bond forces is usually not sufficient *per se* to lead to a structural collapse of the soil, which commonly requires an additional small displacement force, even quite minute, to disturb the metastable equilibrium which follows the bond force reduction. This condition has been recognised for some years in the case of ‚quick clay' sensitivity (e.g. Bjerrum 1954; Skempton and Northey 1952) and in the phenomenon of collapse in unsaturated sandy clays and loesses (e.g., Jennings and Knight 1957; Larionov 1965) following wetting. What is perhaps less well recognized is that the disturbing force which destroys this metastable equilibrium may equally be a hydrodynamic force, instead of an applied load, and that the movement of the water as well as its presence is an important element (Ingles 1964). Moreover it is apparent that in some circumstances e.g., clayey sands (of arid areas) which have not been previously wetted to the point of saturation of the gross pores, soils may exhibit the collapse phenomenon upon wetting to this maximum extent without the requirement for any additional displacement force. In this situation the perturbation arising from the gross dilution of salts in the electrolyte at points of particle ‚contact' may be sufficient to cause substantial settlements of the land surface.

Soils *in situ* possess a natural salts content which at all points of the profile is in equilibrium with their climatic and hydrological environment. The salts remain in place throughout cycles of wetting and drying and are themselves almost invariably sufficient to maintain the soil in a natural flocculated or coagulated condition. If the environmental conditions are now changed such that either these salts can be eluted from the soil, or diluted far beyond their normal seasonal dilution, then a lessening of the interparticle bonds occurs and a potential subsidence condition is created. On the theoretical side, this behaviour has been particularly well illustrated by the work of Quirk and Schofield (1955) who showed that threshold salts concentrations exist below which the ready dispersion of a clay takes place under quite small hydraulic gradients. It is immaterial to the present discussion whether this lessening of the interparticle bonds arises from an increase in repulsive forces or a decrease in attractive forces, but it is pertinent to note that the latter is certainly the case for sandy soils bonded by clays or salts (cf. Parker, 1963; Cole and Lewis, 1960).

For simplicity, the chemical and hydrological factors may be approximately separated by regarding the former as those producing the metastable state, and the latter as those displacing it to failure; with the previously noted reservation that forces other than hydrodynamic can also occasion such displacements.

(*a*) Chemical origins of lessened bond strength in soil

These are either (*i*) a diminution of salts concentration in the soil by dilution or (*ii*) an elutriation and/or replacement of existing soil salts. The practical circumstances of both have been discussed by Aitchison, Ingles and Wood (1963); Wood, Aitchison and Ingles (1964) and Aitchison and Wood (1965), and have been summarised both theoretically and practically by Ingles (1968). Essentially, those soil cations which provide the weakest bonding are the monovalent cations, and of these particularly the sodium ion. Therefore, saline and sodium enriched soils have been particularly prone to this type of failure.

Experimental evidence suggests that below a threshold value of about 10%Na^+ on the soil exchange complex, the clay bonding forces are sufficiently strong to resist even infinite dilution so that no deflocculation hazard can exist. (Rowell, 1963; Emerson, 1960). This threshold value, however, is not at present defined within a range of perhaps $\pm 5\%$, being probably dependent on the structure (and hence internal forces) of the clays concerned. Soils rich in K^+ are rare, and thus inadequate data exists from which to form a similar statement; such data as exists is in accord with the general weakening effect of monovalent cations. Hydrogen ion occupies a somewhat unique position; and since acidic soils are not at all uncommon, this has some practical importance. It is known that for strongly acid soil conditions, hydrogen ion tends to displace aluminium ion onto the soil exchange complex, (e.g. Jackson (1963)), and that thereby strongly acid soils are also well flocculated soils, not susceptible to ready dispersion. By contrast, weakly acid soils should be, and are observed to be, susceptible to dispersion, so that tunnelling and subsidence have been noted in weakly acid podzols of pH 5-7, as well as with the solodic soil groups whose pH is usually much higher at pH 7-9. Increasing soil acidity lowers the total exchangeable cations on a soil, but *increases* the equilibrium percentage of exchangeable sodium.

Since acidic soil failures occur only in weakly acidic soils, some doubt still remains as to the relative contribution of the acidity and the sodium ion content in such soils. Note that ESP determinations at pH7 may underestimate the risk (cf. figure 5, data of Pratt *et alia*, 1962, for kaolinites).

Though deflocculation by dilution alone is the most commonly observed cause of failure, replacement of cations[1] on the soil exchange complex by percolating waters of a different cation type or ionic ratio has also been observed to affect the soundness of structures. Since most natural surface waters have relatively low monovalent to divalent ionic ratios

$$\left(\frac{[Na^+]+[K^+]}{([Ca^{++}]+[Mg^{++}])^{1/2}} < 0.25, \text{ concentrations in moles}\right)$$

such ionic exchange is usually in the direction of soundness and stability for the final equilibrium position. However, the transient path from initial to final state may well transgress a region of chemical instability (i.e. of lessened interparticle bond forces) and this is best exemplified by the data of Aitchison, Ingles and Wood (1963) and of Ingles, Lang and Richards (1968), as represented in figure 3.

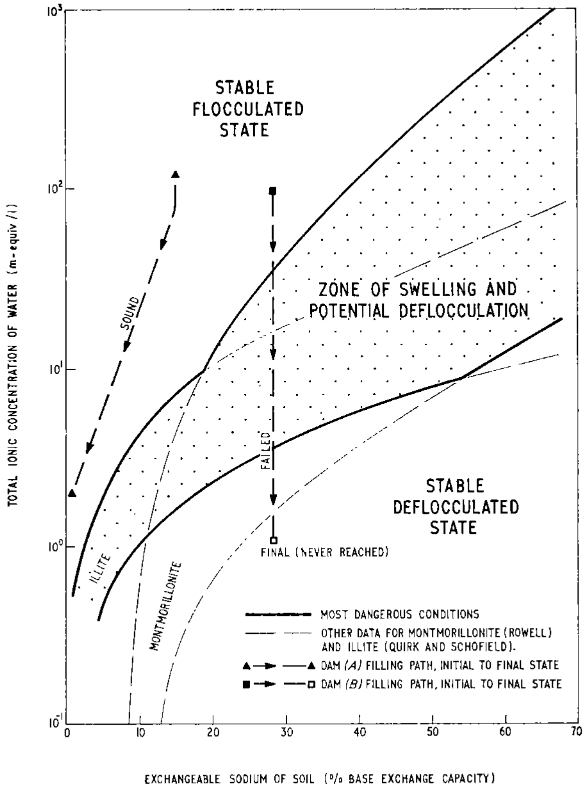

FIGURE 3. *Method for estimation of soil-water instability conditions*

1. Some evidence also exists that bicarbonate anion in the water assists dispersive failures in the field (Ingles, 1964); a situation which might be expected in view of the common use of carbonates as soil dispersants in the laboratory.

Gradual elution of the soluble salts can lead to failures only if the soil is unable to swell sufficiently to seal the flow paths before the salt concentration falls. In principle such failures would be expected most commonly in the nonswelling clays (kaolinites). This is the type of failure observed when small displacements are imposed on the chemically metastable state, as remarked earlier—("quick" clays and "collapsing" soils).

(*b*) Hydrodynamic displacement of metastable soil equilibrium

Once the bonds between soil or clay particles are sufficiently weakened, their displacement under small hydraulic forces becomes possible. For non-swelling clays (e.g. kaolin), silts and sands, the water velocity necessary for particle transport at an exposed surface is a well understood phenomenon (e.g. Sundborg, 1956) and depends on the size of the particles to be transported.

Within the pores of a soil, at least three particle diameters are required to ensure transmission of a particle in suspension, irrespective of the relationship between settling velocity and fluid velocity (based on the assumption of Poiseuille flow). The relationship between permeability and size of soil transported is given in table 1 (Aitchison and Wood, *loc. cit.*). The pore sizes of the quoted permeabilities are just adequate to allow material transport, so that either pore size or sedimentation velocity could be the limiting factor in the size of soil transported, and the actual physical mechanism is thus not relevant.

Table 1.

Value of permeability (cm/sec)	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}
Size of soil transported (microns)	5.2	1.5	0.5	0.15	0.05

The swelling clays (montmorillonites, illites, etc.), have particularly small discrete particle sizes, often as low as .01 micron, and hence can be readily trasported under the smallest hydraulic gradients and in extremely fine channels—i.e. in materials of very low permeability. However, this ready particle dispersion and transport of the swelling clays is opposed by the swelling itself, since these clays when partly swollen still retain sufficient long range interparticle forces to resist small displacement forces. Hence, if the rate of entry of water is such that the rate of swell exceeds the rate of surface attrition, then flow will be reduced below the point at which material transport is possible and a stable non-flow condition will finally result. Conversely, if the rate of attrition exceeds the rate of swell, tunnelling of the soil with the possibility of subsequent subsidence, must ensue. A well defined example of this behaviour in practice has been reported by Ingles, Lang and Richards (*loc. cit.*).

It is important to note that for clay-bonded soils the failure mechanism involves *attrition* not simply *solution*, even though the hydraulic gradients may be extremely low, as was noted by Casagrande (*loc. cit*) .This attrition requirement was diagnosed independently by Parker, Shawn and Ratzlaff (1964) and by Ingles (1964).

The kinetics of the relationship between clay swelling and water infiltration rates for given permeabilities does not appear to have been quantitatively defined to suit the present case. Laboratory experiments by Ingles and Wood (1964) indicated substantial risk of piping (i.e. accelerative hydraulic erosion) for permeabilities $\geqslant 10^{-4}$ cm/sec, and no incidence of piping below 10^{-5} cm/sec. Field observations by Peterson and Iverson (1953) on Canadian dams showed failures at 10^{-3} cms/sec. Thus although in theory montmorillonite particles can be transported in soil permeabilities as low as 10^{-6} (table 1), in practice clay swelling is apparently more rapid than the internal erosion process at least to permeabilities of approximately 10^{-4} cm/sec.

(c) Mechanical and physical preconditions for tunnelling subsidence

As already mentioned, a static factor or precondition for subsidence is the presence of substantial porosity or voidage in the soil. For loessal soils, Larionov (1965) suggested a minimum requirement of 21%. For natural clay soils, this porosity usually arises from cracking, since their internal pore sizes (and hence permeability) are much too low to permit even the low flow rates required for clay particle transport. It is therefore in the swelling clays of an arid environment that tunnelling incidence has been most commonly observed, a fact noted by Parker (1964) who associated the phenomenon with montmorillonites and drylands. This is not wholly so however, since equally clear cut instances have been reported in kaolinites and in areas of ample and well-distributed rainfall such as Tasmania (Wood, Aitchison and Ingles, *loc. cit.*). Even with a cracking soil it is necessary, as discussed in Section (*b*) above, that the advent and passage of water shall be more rapid than the rate of swelling of the soil, if failure is to occur.

Land use is one important way by which the risk of subsidence can be greatly enhanced. Thus for instance the clearing or overgrazing of ,duplex'soils i.e. soils having a shallow silty or sandy top and a heavy clay subsoil, if extensive, permits seasonal drying —intensified by removal of the foliage shade—to crack the clay subsoil; whereupon subsequent heavy rains (which readly penetrate the permeable silt capping) cause subsurface erosion tunnels. When these have enlarged sufficiently, the silt top collapses into the cavity with the formation of surface "sink" holes and deep erosion channels. Similar behaviour in drained and irrigated land has been described by Denisov, Bally and Antonescu (*loc. cit.*). The collapse phenomenon of arid sandy clays and loesses often follows the clearance of natural vegetation which controlled the level of moisture suction in the upper profile.

These phenomena become even more pronounced in earthworks. Any construction condition which gives rise to large pores or fissures in the soil mass will constitute a considerable hazard for soils having clay or salt bonds (note that the strongly bonded soils, e.g. krasnozems, are excellent construction materials for that reason).

One way in which gross pores are built into an embankment is by compaction dry-of-optimum, since voidage and permeability rise sharply below optimum moisture content (Aitchison and Prescott, 1954; Walker and Holtz, 1951). Soils which resist compactive effort, i.e. heavy clays in an unsaturated state, will be particularly hazardous. The moisture content-density-permeability relationships are indicated schematically in figure 4 which represents the essential features found experimentally, though individual values naturally vary considerably according to the particular soil, and must be so determined. Note particularly from figure 4 that steep compaction curves imply large potential settlements for small deviations from the optimum moisture content, but that well-flocculated soils (e.g. Soil $A+2\%$ lime) with flatter compaction curves are much more tolerant of deviation from the optimum moisture content for the same potential settlement.

Burton (1964) has attributed tunnelling and subsidence in earth dams to settlement of the saturated soil below the phreatic line, with consequent unification of the embankment air voids along that line. Thereafter, either total subsidence occurs, or, if the dry upper arch resists subsidence, a later rise of the water level will lead to immediate tunnelling and washout.

Even construction procedures which result in a suitably low permeability of the earthwork at completion will not be sufficient to ensure against future hazard if the earthwork is either:

(*a*) Allowed to dry out and develop shrinkage cracks (often the case with montmorillonite clays, which form a thin highly impermeable layer on the upstream face of a water storage, allowing the core to dry and crack. Subsequent draw down allows the upstream skin to crack also, and washouts follow refilling (Gyamarthy 1962). Or (*b*) Founded on soil horizons already affected by sub-surface cracking (i.e. not below the depth of seasonal moisture movement. A somewhat equivalent case is the placement of embankment layers in conditions which allow any surface drying. Unless a sheepsfoot roller is used, the slight surface drying may prevent bonding between the layers and thus leave potentially troublesome horizontal discontinuities).

Failures of types (*a*) and (*b*) have long been recognised in earth dams and are likely to be equally prevalent in levee and channel banks, bridge abutments, railway embankments, and the like were they always correctly recognised as to their origins.

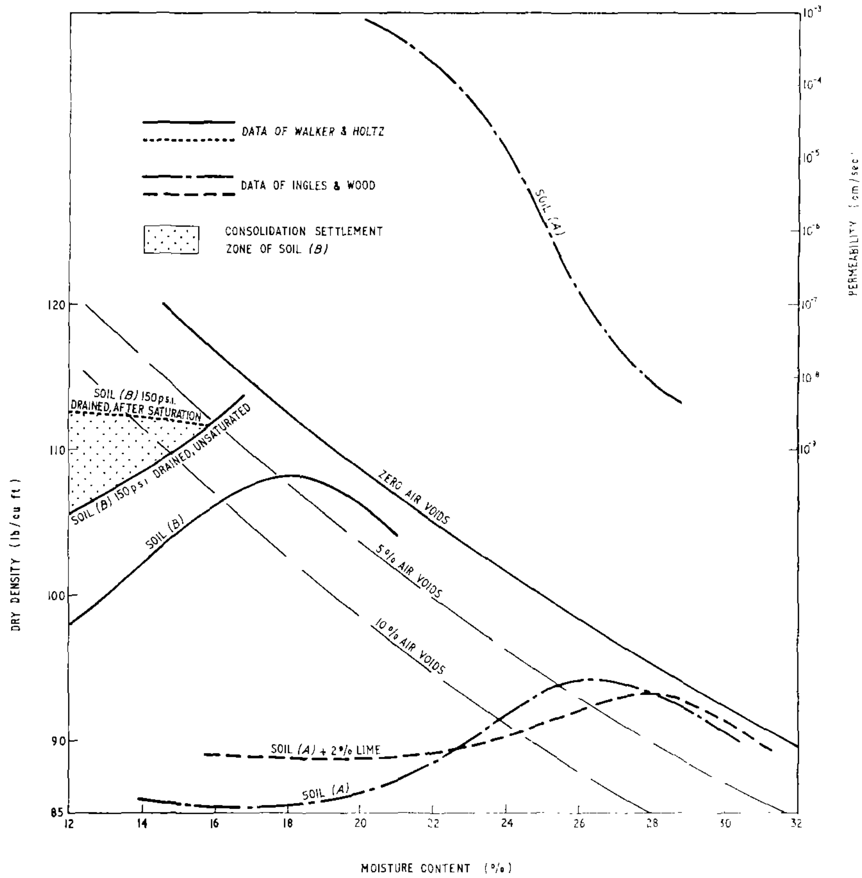

FIGURE 4. *Moisture-density-permeability in relation to settlement*

It should also be noted that the chemical and hydraulic considerations discussed before imply two important considerations when any water-retaining structure is to be filled. These are firstly, that proof filling should be made with a water of no less a dispersive power than that to be finally stored (i.e. sea-water testing is not usually adequate) and secondly that the rate of first filling should be kept as low as possible, particularly if the soil contains expansive clays.

METHODS FOR ESTIMATION OF THE RISK OF TUNNELING AND SUBSIDENCE

(*i*) The chemical hazard. This requires recognition of clay and salt bonded soils. The most general such test is probably the Crumb Dispersion Test (Emerson, 1960) which identifies qualitatively both slaking and dispersive soils. Note particularly that the soil should not be "worked" in any way before this test, to avoid formation of an impervious skin on dispersive clays. Nor does this test apply any displacement force, so that a more severe test might be to swirl the specimen slightly before final observation. Potentially slaking soils should be further examined quantitatively by loading or consolidation tests, a field loading test accompanied by inundation being preferred if sampling is difficult, or, if laboratory testing is possible then a consolidation test in which the dry-placed material is inundated.

Potentially dispersive soils should be examined either by a dispersion index test (Reilly, 1964) or perhaps better by percentage monovalent exchangeable ion determination, accompanied by determination of the ionic ratio of the water against which the structure is to be secure. For present practice, the simplified procedure is adopted of attributing the dispersive character of the soil wholly to sodium ion, and to determine the ESP (exchangeable sodium percentage) of the soil. The latter is quickly determined, with sufficient accuracy for practical purposes, by homogenising an equal weight of soil and distilled water, standing one half hour, then extracting the soil paste under an air pressure of 80 p.s.i. on a Visqueen membrane. The clear extract is conveniently analysed by flame photometer for Na^+ and by versenate titration for $Ca^{++} + Mg^{++}$, and the ESP of the soil deduced from these values via the Gapon equation (Ingles, 1968). ESP values in excess of $7\frac{1}{2}$% should be regarded with caution, those above 15% as posing a serious potential hazard.

The above methods assess only the short-term risk, or *dilution* effect of water in the soil. Long term instability due to changes in the soil-water equilibrium and transient states must be evaluated from the curves of figures 3 and 5, using estimates of the most dangerous path based on conditions of service.

(ii) The hydraulic risk. This is best assessed by reference to the critical permeability for piping and the presence of open fissures or like voids in the soil. As discussed earlier, permeabilities higher than 10^{-5} cms/sec should be regarded as potentially dangerous for dispersive clays with particle sizes below 2 microns. Well structured and flocculated clays tolerate permeabilities as much as 10^{-3} cms/sec, by which point seepage flows become so high as to represent un unacceptable loss of water, so that such clays are sound in practical service conditions. Figure 4, which is typical, shows that even in dispersive soils an acceptable permeability can be achieved for moisture contents not less than 1% dry of optimum.

The presence of open fissuring and voids cannot be assessed by small scale (laboratory) tests and must therefore be evaluated in the field. Infiltration tests are recommended on suitably sited boreholes. The water used should be either treated with aluminium sulphate to prevent clay swell and thus ensure maximum flows, recording a "most dangerous" value; or else a high purity water ($<$ 200 ppm total dissolved salts) can be used provided the results are interpreted with care. Thus, this latter method gives a high initial value of permeability in a tunnelling soil, but when the test is interrupted for some hours, permits the natural swell of the clay to close the fissures and subsequent readings record the lower permeabilities comparable to a "slow-wetting" condition. Note that in all cases the alum infiltration test is a sufficiently severe criterion for assessing the permeability hazard, though conservative when slow wet-up conditions can be maintained in the field.

Both chemical and hydraulic risks can be jointly monitored in the field by the borehole piezometer method described and applied by Ingles, Lang and Richards (*loc. cit.*).

General soil areas in which the risk of such tunnelling and subsidence is unusually high can be conveniently recognised by aerial survey (Ingles and Wood ,1964) relying on the presence of turbid waters, clay outwash fans, and erosion gullies. The latter are also a good guide in ground survey work, since erosion gullies which show undercutting are invariably evidence of a soil horizon whose internal bonds are critically weakened by the natural water of the area.

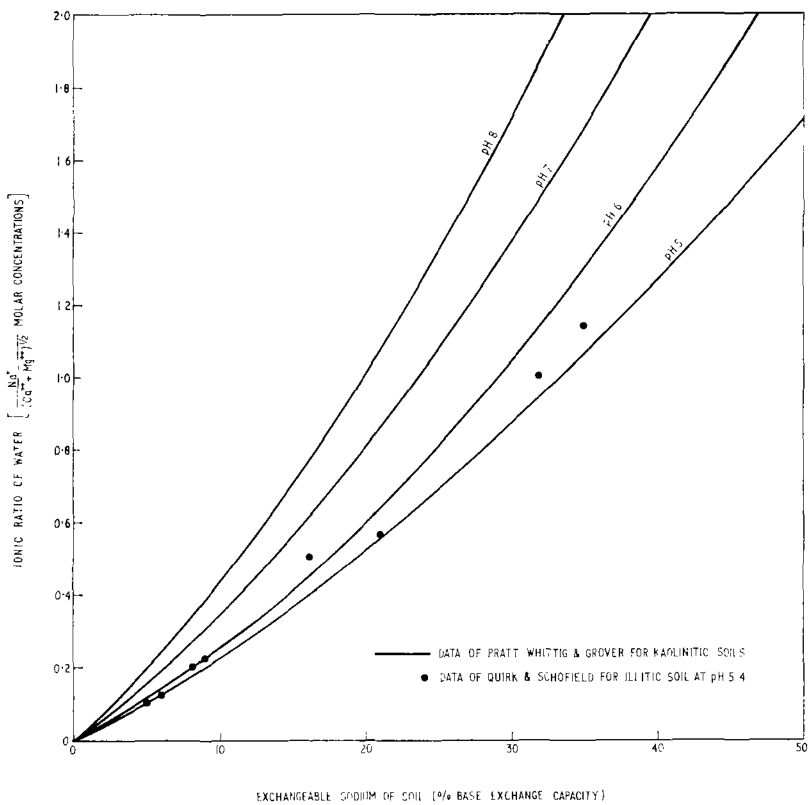

FIGURE 5. *Method for estimation of soil ESP from water ionic ratio*

METHODS FOR TREATMENT OF SUSCEPTIBLE AND AFFECTED SOILS

Since one basic condition for failure is the lessening of interparticle bond strengths by water, remedial measures will include those which either (*a*) increase the soil bond strength or (*b*) diminish the dispersive power of the water. The former is more commonly adopted, and consists usually of treatment of the soil with calcium hydroxide, calcium sulphate, ferrous sulphate, or aluminium sulphate (these being the cheapest commercial chemicals which can flocculate, i.e. bond, clays). In each case good mixing is important since the additive must be thoroughly distributed in the soil. The higher solubility of the latter two salts enables them to be applied as sprayed solutions, whereas the former two must be mechanically mixed. Diminution of the dispersive power of the water is obtained by the addition of any soluble salt; however, this should not be such as to affect the long term stability of the earthwork (*vide* previous section) and moreover the quantities required are usually both uneconomic and undesirable in the water.

As an alternative to the chemical remedy, construction procedures and service procedures must be such as to ensure complete freedom from fissuring or excessive voidage. This can be done only by (*a*) wet compaction using sheepsfoot rollers (*b*) design procedures which ensure the structure against subsequent drying out—e.g. the placement of deep layers (minimum 3 feet) of silty sand on the exposed faces of the earthwork, together with the establishment of natural vegetative cover (grasses) as soon as possible. As previously noted, where a water-retaining structure in such soils is being brought into service, filling should be particularly slow (and when in service, draw down and filling conditions require careful attention). For collapsing soils, heavy compaction is both simple and effective.

It is therefore possible to use soils susceptible to tunnelling and collapse for construction purposes, provided their materials properties are fully recognised, and the appropriate corrective or constructional measures taken.

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DISCUSSION

Intervention of Prof. George V. CHILINGAR (USA):

Question:

I would like to congratulate you on a very important contribution. In as much as your soils contain Na-montmorillonite, I believe that an electrochemical method of stabilization may prove not only effective but also economical. The following processes can be involved:
(1) Destruction of clays on application of direct electric current.
(2) Base exchange—change to less swelling clays (Ca-base or Al-base montmorillonites).
(3) Cementation as a result of mineral neoformations.

Answer of Mr. INGLES:

Electrochemical stabilization methods have been studied in Australia[1]. They are successful in the laboratory but too expensive in the field. I will be very interested to see the comments on electrochemical stabilization in your own paper.

As to the type of clays, the majority of the serious subsidences occur in montmorillonites and illites. Kaolinites do suffer from this but not nearly so frequently as the montmorillonites and illites. Marine clays or clays deposited under saline condition, are notoriously bad. Also wind blown desert sand and loam. Most of the clays in Australia in which subsidence is found would fall into the montmorillonite group.

Intervention of Dr. N. MIYABE (Japan):

Question:

What is the most important factor for occurences of flocculation and deflocculation?

Answer of M. INGLES:

The factors which cause flocculation and deflocculation are the ionic forces between the clay particles. If the clay has a high percentage monovalent cations on the exchange complex, it very readily deflocculates in the presence of excess water. If it has a high percentage of divalent cations, it is much more difficult to deflocculate.

Question of Dr. MIYABE:

In there any difference between the soils?

Answer of Mr. INGLES:

Yes, there are differences. This is quite a complex question to answer, because you can have, for instance, montmorillonites which have substitution in their tetrahedral layer instead of the octahedral layer and which are still mineralogically montmorillonites but which do not disperse. They behave just like sand. On the other hand, of course, if you have extensive substitution in the octahedral layer, instead of the tetrahedral layer, then one observes very ready dispersion as a result of the high surface charge on the clay.

(1) *Reference:* NETTLETON, A.F.S., (1963): *4th. Augst. and N.Z. Conf. Soil Mech. and Found. Eng.*, 93. Also other references cited therin.

STUDIES ON PARTIAL COMPACTION OF SOIL LAYER IN REFERENCE TO LAND SUBSIDENCE IN TOKYO

Shigeru AOKI and Naomi MIYABE

ABSTRACT

Apparent land subsidence recorded by the compaction recorder designates the compaction of the soil layer existing between the ground surface and the level where the reference tube of the compaction recorder is fixed. This part of the soil layer is, however, composed of two or more layers which are of different mechanical properties. It is, therefore thought appropriate to consider that the recorded compaction is the integration of the compaction of each composite layer.

Assuming that the compaction of each soil layer is proportional to the effective thickness of the respective layer, the separation of its bottom from the ground water level, and the change in the height of the ground water level, the integrated compaction obtained by the compaction recorder is analysed into responses of two or three composite layers of different mechanical properties. The results are almost satisfactory within the tolerance limit of errors.

RÉSUMÉ

L'affaissement apparent de la surface de la terre enregistré par l'appareil enregistreur de la compacité indique la compacité de la couche existant entre la surface de la terre et le niveau où le tube de référence de l'appareil est fixé. Cette partie de la couche est pourtant composée d'une ou plusieurs couches qui sont de différentes qualités mécaniques. On croit donc convenable de considérer la compacité enregistrée comme une intégration de la compaction de chaque couche composante.

En supposant que la compacité de chaque couche soit proportionnelle à son épaisseur, à la distance de son fond au niveau des eaux souterraines, et au changement de la hauteur du niveau des eaux souterraines, la compacité intégrée obtenue par l'appareil enregistreur de la compaction est analysée en tenant compte de deux ou trois couches composantes de différentes qualités mécaniques. On juge satisfaisants les résultats dans les limites de la marge de tolérance.

1. As has been discussed by one of the present authors, Miyabe [1], the observed land subsidence is regarded as the integration of compactions of the constituent soil layers that were deposited at various horizons. The mode of compaction of each of the constituent layers may be different according to the differences in their physical properties and their relations to the ground water.

In the previous paper, it was desired that the response of each soil layer would be determined separately as a function of its thickness and the height of the ground water level, combined linearly through constants characterizing the materials of the soil layer concerned.

In the present paper, the writers developed trial studies in the direction mentioned above.

2. For the purpose of the present study, the values of yearly compactions obtained at the observation stations of Kameido and Minami-sunamati, both located in the Koto region of Tokyo, are employed.

At these observation stations, the locations of which are shown in the map of figure 1, geological profiles of the surface soil layer are made using data from borings, and are used in conjuction with data from the compaction recorders and the hydrogarphs. The

(*) This report would be regarded as the „Second Report" of the report published on the same subject by one of the present authors, N. Miyabe.

geological profiles thus obtained at these stations are shown in figure 2a (Kameido) and b (Minami-sunamati).

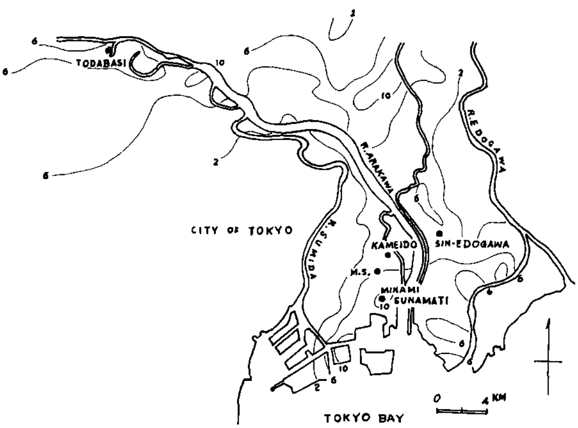

FIGURE 1.

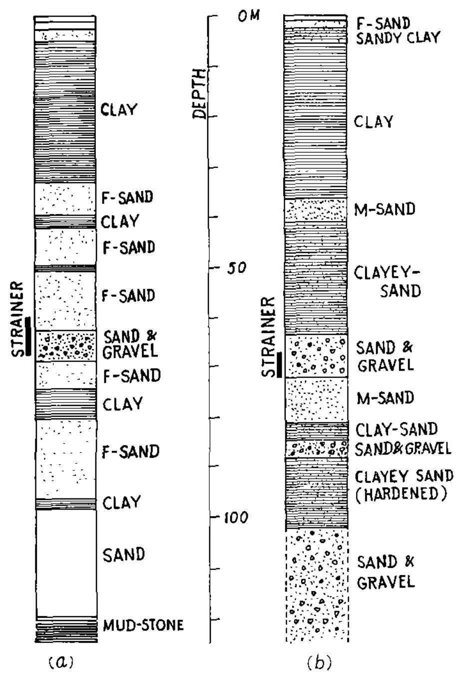

FIGURE 2.

At Kameido, the soil layer present from the surface to the depth of 33.0 metres is composed mainly of a soft clay mass containing shells, and, from that level to the depth of 63.0 metres, soil is composed of rather fine sand with insertions of thin (one or two metres thick) clay sheets. Below 63.0 metres from the surface, there is an aquifer composed of sand and gravel. About 70 metres below this aquifer is another (deeper) aquifer.

Two observation well are installed at the Kameido station, one with its strainer at the depth of 60 to 67 metres, while the second has its strainer at the depth of 135 to 143 metres. At the former well, that is, No. 1 well of the Kameido observation station, the hydrograph records the variation in the height of the ground water level of the shallower aquifer, while, at the latter well, that is, No. 2 well of the Kameido station, the hydrograph shows the changes in the height of the ground water level of the deeper aquifer. These two hydrograph records are similar on the whole, but different in detail. We are, therefore, not sure whether these two aquifers are connected.

In the profile of soil layer, three parts may be distiguished according to the composition and geotechnological characteristics. These are;

1. from the ground surface to the depth of 33.0 metres, the soil layer is composed of loose clay, containing shells;
2. from the depth of 33.0 metres to the depth of 74.0 metres, the soil layer is composed of rather fine sand with insertions of thin clay sheets;
3. deeper than 74.0 metres to the surface of the tertiary bed, which is estimated to be situated at a depth of several hundred metres, the soil layer is composed of alternating beds of fine sand, clay and sand-and-gravel, and this layer is more or less hardened on the whole.

The compaction recorder at No. 1 well of the Kameido station records the sum total of compaction of the clay layer (1) and that of the sand layer (2), caused by decline in the ground-water pressure in the shallower aquifer. On the other hand, the compaction recorder at No. 2 well records the compaction of a part of the soil layer (3), that is, the part from the depth of 74.0 metres to that of 143 metres, in addition to those of layers (1) and (2).

The geologic conditions at the Minami-sunamati compaction recording station are quite similar to those at the Kameido station in that the soil layer may be divided into three parts;

1. from the surface to the depth 36.5 metres, the soil layer is composed of loose clay containing shells;
2. from the depth of 36.5 metres to that of 64.0 metres, the soil layer is composed mainly of clayey sand, underneath which is an aquifer composed of sand and gravel;
3. from the depth of 64.0 metres to the surface of the tertiary bed, the soil layer is composed of alternating layers of clayey sand and sand with gravel, which are hardened to a certain degree.

No. 1 well of the Minami-sunamati station has its strainer in the layer of sand with gravel at the depth of 64-72 metres. The compaction recorder at this well records the sum total of compaction of layer (1) and layer (2), and the hydrograph records the changes in the height of the ground water level of the aquifer existing at the horizon of 62 to 72 metres depth. On the other hand, the compaction recorder at No. 2 well of the Minami-sunamati station, which is inserted to the depth of 130 metres, records the compaction of a part of layer (3), that is, from the depth of 72 metres to the depth of 130 metres, in addition to those of the layers (1) and (2).

The operation of the compaction recorders began in 1952 at No. 1 well, and in 1960 at No. 2 well of the Kameido station, and in 1954 at No. 1 well, and in 1961 at No. 2 well of the Minami-sunamati station.

In this paper, as already indicated, the yearly amounts of compaction are treated for the first step of study. The following table (table 1) shows the yearly amounts of compaction measured at No. 1 and No. 2 observation wells of the Kameido and the Minami-sunamati stations, and yearly mean heights of the ground water level, together with the yearly fall (or rise) of the ground water level at respective observation wells.

TABLE 1. Yearly Compaction and Related Data

Yearly Compaction mm	Mean G.W.L. m	Fall m	Year	Yearly Compaction mm	Mean G.W.L. mm	Fall m
	Kameido No. 1 Well				Minami-sunamati No. 1 Well	
29.6			1953			
31.0	20.8	0.9	1954	22.3	18.0	1.4
37.7	22.0	1.6	1955	32.7	19.2	1.0
50.2	24.1	2.6	1956	37.3	20.5	1.6
59.6	27.4	4.1	1957	53.0	22.7	2.8
73.9	30.4	1.9	1958	82.2	25.1	2.0
75.1	32.4	2.0	1959	68.1	27.3	2.4
92.3	34.8	2.8	1960	71.4	29.6	2.2
74.4	37.9	3.9	1961	121.5	32.2	2.9
53.7	40.1	1.0	1962	91.8	34.2	1.1
61.3	41.3	1.6	1963	77.4	35.4	1.5
66.8	42.5	0.8	1964	65.0	36.6	1.0
27.2	44.2	−0.7	1965	61.1	36.8	−0.6
39.2	42.2	−3.2	1966	54.2	35.4	−2.1
54.9	39.4	−2.4	1967	50.6	33.5	−1.9
	Kameido No. 2 Well				Minami-sunamati No. 2 Well	
70.1	39.9	2.6	1960			
137.2	43.3	4.4	1961	109.0	33.1	1.5
114.2	46.8	2.5	1962	104.9	37.7	1.6
123.0	47.9	−0.3	1963	81.4	35.9	0.7
121.2	49.5	3.6	1964	63.8	36.4	0.2
89.6	50.9	−2.8	1965	61.8	36.3	−0.1
87.1	46.5	−6.7	1966	56.9	35.1	−2.2
80.0	41.7	−3.0	1967	49.1	33.1	−1.8

Mean G.W.L. in the above table designates the mean value of the ground water level, and negative signs prefixed to the numerals in the lines of "Fall" means "Rise" of the ground water level.

3. As have been employed by many authors [2] in their discussion on land subsidence, the compaction of soil layer of thickness 2h is expressed, after Terzaghi's theory of consolidation, in the form

$$\left.\begin{aligned} S &= v \int_0^{2h} \{w(z,0) - w(z,t)\}\, dz \\ \frac{\partial S}{\partial t} &= v \int_0^{2h} \frac{d}{dt} \{w(z,0) - w(z,t)\}\, dz \end{aligned}\right\}, \tag{1}$$

where:

S and $\partial S/\partial t$ are respectively the amount and the rate of compaction;
v the consolidation coefficient;
w the pore water pressure within the soil mass and given as the solution of the equation

$$\frac{\partial w}{\partial t} = \frac{k}{v}\frac{\partial^2 w}{\partial z^2}.$$

The solution of this equation is given, neglecting the terms for stationary state, in the form

$$w(z, t) = \frac{4}{\pi}\sum\frac{1}{n}\left[\exp\int -\frac{k}{v}\left(\frac{n\pi}{2h}\right)^2 t\right]\sin\frac{n\pi}{2h}z. \tag{3}$$

If the change in the ground water pressure, which will result in the compaction of soil layer, is given as a linear function of time, as $q = F(t)$

$$w(z, t) = \frac{4}{\pi}\int_0^t \sum\frac{1}{n}\left[\exp\int -\frac{k}{v}\left(\frac{n\pi}{2h}\right)^2 (t-\tau)\right]\sin\frac{n\pi}{2h}z\cdot\frac{\partial F}{\partial\tau}d\tau, \tag{4}$$

and for $F(t) = at$,

$$w(z, t) = \frac{4}{\pi}\sum\frac{1}{n}\frac{a}{\frac{k}{v}\left(\frac{n\pi}{2h}\right)^2}\left\{1-\left[\exp\int -\frac{k}{v}\left(\frac{n\pi}{2h}\right)^2 t\right]\right\}\sin\frac{n\pi}{2h}z, \tag{5}$$

and the rate of subsidence is given by

$$\frac{\partial S}{\partial t} = v\frac{4}{\pi}\sum\frac{a}{n}\left[\exp\int -\frac{k}{v}\left(\frac{n\pi}{2h}\right)^2 t\right]\int_0^{2k}\sin\frac{n\pi}{2h}z\,dz. \tag{6}$$

It may therefore be rather reasonable to put for a mode of compaction of clayey soil layer as

$$\frac{\partial S}{\partial t} \propto a.h. \tag{7}$$

On the other hand, for the mode of compaction of sandy soil layer when the ground water is withdrawn, the expression

$$\Delta S \propto \Delta p, \tag{8}$$

which has been proposed in determining elastic compression of an artesian aquifer [3], may be applicable.

The actual soil layer, the compaction of which has been recorded, is composed of the clayey soil layer and sandy soil layer with insertion of thin clay sheets, as previously referred to. It is therefore rather appropriate to take into consideration the both modes of compaction mentioned above. Thus for the general expression to be applied to express the annual compaction u, the following form is proposed;

$$u = k_1 a h_1' + k_1' h_1' \Delta p + k_2 a h_2' + k_2' h_2' \Delta p, \tag{9}$$

in which h_1', h_2' are the effective thicknesses of the layers (1) and (2), which participate in the compaction, and k_1, k_1', k_2, k_2' are the contribution factors (so called) of the layers (1) and (2) in the mode represented by equation (7) and that represented by equation (8) (marked by dashes).

4. The values of a in the expression (7), which is necessary in executing the calculation of the amounts of compaction, is available through the curves in figure 3, in which the annual mean heights of the ground water level are plotted against time (year). The values for a obtained for the changes in the heights of the ground water level at Kameido and Minami-sunamati stations are:

at Kameido, during the period from 1955 to 1961 $a = 2.6$ m per year,
and, during the period from 1961 to 1964 $a = 1.3$ m per year,

at Minami-sunamati, during the period from 1956 to 1961 $a = 2.3$ m per year,
and, during the period from 1961 to 1964 $a = 1.1$ m per year,

After 1965, the ground water level has been rising, perhaps as a result of the ground water control pact.

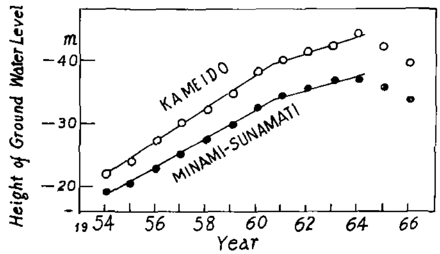

FIGURE 3.

As for the effective thickness of soil layer, which participates in compaction, we considered as follows:

With regard to the layer of soft clay, such as alluvial clay layer, the effective thickness subjected to compaction as a result of its water being squeezed out, will be that from the surface of the layer to the depth (or nearly as) where the ground water level stands. Actually, this effective thickness is given by the positions of the ground water level (depth to ground water from the land surface). It may be remarked that, for this clayey soil layer, the Tersaghi's theory of consolidation would be applicable, and the equation (7) is the facsomile expression for calculation of compaction of this layer. On the other hand, in the sandy layer underneath the clayey soil layer at Kameido, the mode of compaction given by equation (8) will probably predominate. Therefore, we assume, to avoid complication, that the compaction recordeed at No. 1 well of the Kameido station is composed of mode (7) type compaction of layer (1) and mode (8) type compaction of layer (2), until the ground-water level falls to the depth coincident with the bottom level of layer (1). When the ground-water level falls below the bottom of layer (1) the cause of compaction is considered to be both modes of compaction of layer (2) only. The effective thickness of layer (2) in the mode (8) is assumed to be the full thickness of the layer and constant.

Since the ground water level, at Kameido station, reached the bottom of layer (1) at the end of 1959, then layer (2) only is thought effective in compaction during the period from 1960 to 1964. After 1965, the ground water level has been rising, and the same method of analysis will no longer be applicable.

Thus, we put the expression for the compaction at No. 1 well of the Kameido station for the period from 1955 to 1959 as

$$u-\alpha_1 = k_1 a h'_1 + k'_2 h'_2 \Delta p \qquad (10)$$

and for the period from 1960 to 1964

$$u-\alpha_2 = k_2 a h'_2 + k'_2 h'_2 \Delta p \qquad (11)$$

where k_1, k_2, k'_3 are the contribution factors and α_1 and α_2 are constants to be determined statistically.

The numerical values of these constants are determined with regard to the compaction data obtained at No. 1 well of the Kameido station, for the period from 1955 to 1959 as

$$k_1 = 0.49, \quad k'_2 = 0.22, \quad \alpha_1 = -11.4\,(mm),$$

and for the period from 1960 to 1964 as

$$k_2 = 0.17, \quad k_2' = 0.60, \quad \alpha_2 = -8.8\,(mm).$$

With regard to the compaction records obtained at No. 1 well of the Minami-sunamati station, the values of k_1, k_2' and α_1 are deduced as

$$k_1 = 0.0142, \quad k_2' = 0.50, \quad \alpha_3 = 30.0\,(mm),$$

as the results of similar analysis.

5. The results obtained with the compaction records at No. 1 well of the Kameido station seems to show that, during the period from 1955 to 1959, during which the ground water level was higher than the bottom of the upper clayey soil layer, layer (1), the compaction of this clayey layer played the leading part; however, this role was transmitted to the lower sandy soil layer, layer (2), when the ground water level fall below the bottom of the the upper clayey layer.

The compaction conditions observed at No. 1 well of the Minami-sunamati station appear to be somewhat different; that is, although the soil layer that might have played a principal part in the compaction, must be of clayey soil, the principal mode of compaction was the sandy soil type, that is, mode (8). It must also be added that the physical meaning of the constants α obtained above is quite obscure.

The problem is therefore reserved for future investigation to clarify the sources of discrepancies in modes of compaction and the physical meaning of the constant α.

In conclusion, the authors would like to express their heartfelt thanks to their colleagues and the staff members of Tokyo Institute of Civil Engineering for their kind assistances in preparing the present report

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SHORT-TERM MOVEMENT OF THE LAND SURFACE NEAR WATER WELLS

Frank L. PETERSON and Stanley N. DAVIS

ABSTRACT

Stresses related to short-term pumping (or injecting) water from wells result in measurable movements of the land surface. Sensitive transducers coupled to horizontal and vertical extensometers enable relative movements to be measured with a precision of 0.1 micron. The testing periods were limited to about one hour or less owing primarily to strain buildup in the extensometers which produces an instrument drift.

Results from 196 tests on 19 wells indicate the following:

1. Total vertical movement probably ranged between 1 and 100 microns on most tests, and horizontal movement was as great as 15 microns per meter of lateral distance.
2. With only one exception, short-term effects were almost perfectly elastic.
3. Pumping and injecting equal amounts of water produced movements of similar magnitude but in opposite directions.
4. During pumping, the shape at the surface depression resembled the surface deflection produced by a concentrated load on an elastic plate of infinite extent.

RÉSUMÉ

Les forces produites par le pompage (ou l'injection) de l'eau des puits, sur un court intervalle de temps, produisent des mouvements mesurables de la surface de la terre. Des dispositifs sensibles se reliant aux extensomètres horizontal et vertical rendent possible la mesure des mouvements en question avec une précision de 0,1 micron. Les périodes d'essai se limitent à une heure environ ou à moins d'une heure, cette limitation étant due essentiellement à la mise en tension dans les extensomètres produisant une déviation dans les instruments.

Les résultats d'après 196 essais sur 19 puits sont les suivants :

1. Tout le mouvement vertical est probablement de l'ordre de 1 à 100 microns dans la plupart des essais et le mouvement horizontal atteint 15 microns par mètre de distance latérale.
2. A une seule exception près, les effets sur une courte période de temps sont presque parfaitement élastiques.
3. Le pompage et l'injection des mêmes quantités d'eau produisent des mouvements d'importance semblable en sens opposés.
4. Pendant la durée du pompage, la forme de la dépression à la surface ressemble à la déflexion sur la surface produite par un poids concentré sur une étendue élastique d'une extension infinie.

INTRODUCTION

Surface subsidence caused by groundwater removal results primarily from compaction of fine-grained confining beds and fine-grained lenses within aquifers, and to a lesser extent from compression of coarser-grained aquifer materials. Most research has been concerned with subsidence and associated effects that have accumulated during periods of months or years, and which most likely have resulted primarily from compaction of fine-grained confining layers. In contrast, this study was directed almost entirely to minute movements of the ground surface around individual wells during short-term (2 hours or less) pumping and injecting. Because of the rapid and nearly elastic movement of the ground surface, it is probable that the displacements observed during these short interval of fluid flow resulted from compression of the coarse-grained aquifer materials.

There have been few other attempts reported in the literature to measure short-term responses of aquifers to changes in loading. Riley (1960) observed almost instantaneous tilting of the ground surface in response to short pumping tests near San Jose, California. Ito (1961) also measured almost immediate tilting of the ground surface during short intervals of fluid flow to wells near Osaka, Japan.

FIELD MEASUREMENTS AND EQUIPMENT

Vertical and horizontal displacements were measured during approximately 190 pumping and injection tests conducted on 16 different wells located in the Santa Clara Valley, California (Peterson, 1967; Halderman, 1967). The wells tested vary in size from shallow (5-10 meters), low discharge (25 liters per minute) test holes to deep (500-700 meters), high discharge (700 liters per minute) municipal wells. Almost all the wells obtain water from aquifers in Pleistocene alluvium, mainly silty and clayey gravels, and all except one are confined or partially confined.

In addition, an attempt was made to measure horizontal displacement near 3 oil wells at Lost Hills, California, and also to measure horizontal movement above underground brine solution cavities in Hutchinson, Kansas, Detroit, Michigan, and Windsor, Ontario, Canada (Peterson, 1967). Measurable responses were not obtained in these tests.

The apparatus used to measure vertical displacement is similar to many of the vertical compaction-measuring devices in operation today (for example, see Lofgren, 1961). The vertical extensometer measured displacements between the bottom of a test hole and the ground surface (fig. 1). It consisted of a 14-kilogram weight placed at the bottom of a test

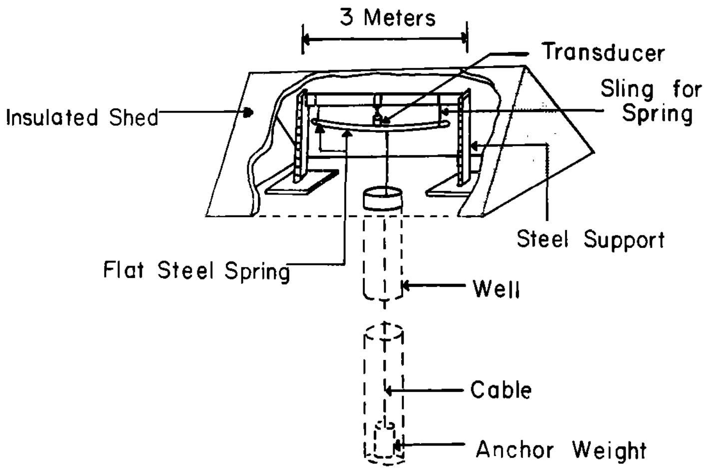

FIGURE 1. *Vertical extensometer used to measure relative motion between land surface and bottom of well. Milivolt recorder and thermistors are not shown*

hole, a 3-milimeter-diameter cable attached to the weight and running to the ground surface, a steel frame resting on the ground surface over the well, a flat steel spring which served to couple the frame to the upper end of the cable, an electrical resistance transducer that measured the relative displacement between the spring and the frame, electronic readout and reconding equipment, and an insulated cover to ptotect the extensometer from the effects of wind and rapid temperature changes.

The horizontal extensometer used in the study measured the change in distance between the tops of two stakes driven into the ground approximately 3 meters apart (fig.2). This provided a measure of the magnitude of horizontal displacement at the ground surface, and in addition, indicated the surface shape of the subsidence depression at the ground surface. When the land surface subsided it assumed a curved shape which was concave upward near the well and convex upward farther away from the well. When the surface was displaced in a concave upward manner the tops of the stakes were tilted toward each other and the distance between them decreased. When the surface was displaced in a convex upward manner, the tops of the stakes moved apart. In addition to the rotational movement of the stakes, some lateral displacement also was observed. It was possible to separate the rotational displacement from the purely lateral displacement so that reliable determinations of the shape of the ground surface were obtained.

The horizontal extensometer consisted of two 1-meter long stakes driven into the ground, a quartz-rich glass rod approximately 3 meters long suspensed between the stakes and firmly attached to one of the stakes, a transducer to measure the displacement between the free end of the glass rod and the adjecent stake, electronic readout and recording equip ment, and an insulated cover to protect the extensometer from wind and rapid temperature changes.

A displacement as small as 0.01 micron could be detected easily with the extensometers Because of the extreme sensitivity of the instruments, they were often affected by external vibrations. People could not walk within several meters of the instruments, and heavy motor traffic within several hundred meters caused troublesome vibrations. Winds of a few kilometers per hour often produced excessive vibrations, particularly if the apparatus was near large trees that swayed in the wind.

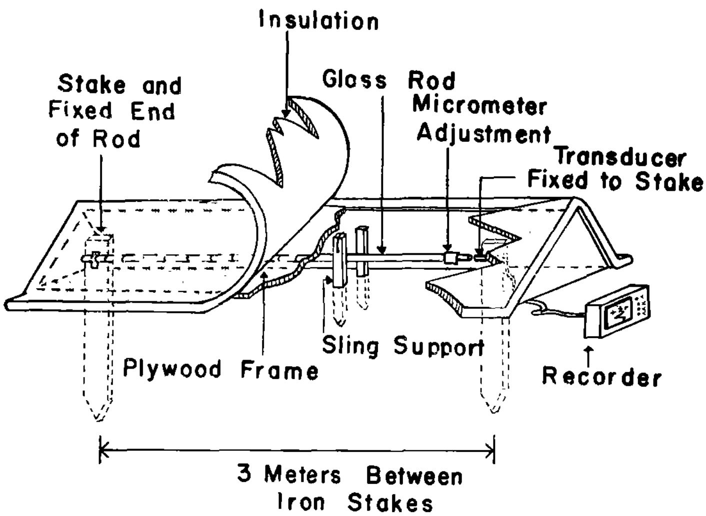

FIGURE 2. *Horizontal extensometer used to measure relative motion between two metal stakes. Thermistors are not shown*

Horizontal displacements were measured at several different distances from each well, and along several different bearings from the well in order to observe horizontal responses over the entire subsidence depression. In addition, horizontal displacements in both tangental and radial orientations with respect to the well were measured. Vertical displacements were measured wherever observation wells were available adjacent to the pumping well.

SUMMARY OF RESULTS OF MEASUREMENTS

Vertical and horizontal displacements of a few tens of microns generally were measured at the ground surface around individual wells during short intervals of fluid flow. Figures 3 and 4 show vertical and horizontal displacements measured near a well at stanford, California. The movement shown in these figures is similar to the response observed during most of the tests conducted in this study. Several characteristics which were common to virtually all the tests are well illustrated: an almost instantaneous respons (within 1 second) of the land surface to change in hydraulic head within the aquifer, an almost perfect

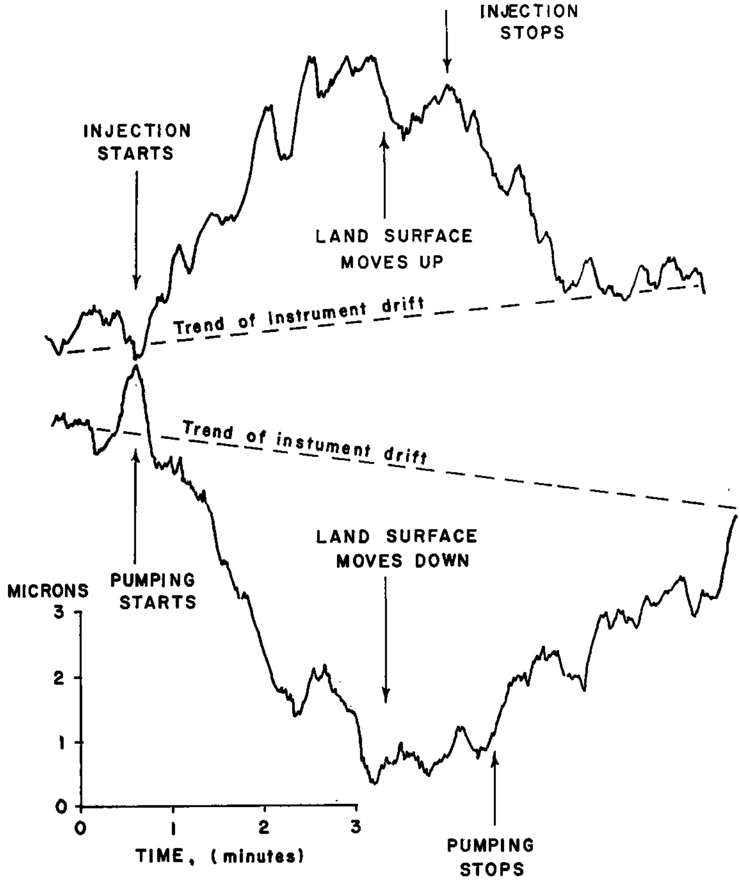

FIGURE 3. *Vertical displacement produced by 100 liters/minute pumping and injection at Lagunita well, Stanford, California*

elasticity of the aquifer and overlying sediments, and displacements produced by pumping and injection which were of similar magnitude but in opposite directions.

Near a pumping well compressional displacements commonly were observed, indicating the curvature of the ground surface was concave upward. At greater distances from a pumping well extensional displacements commonly were observed, indicating the curvature of the ground surface was convex upward. At intermediate distances from the well an inflection zone was observed in which the magnitude of the displacements was somewhat reduced and the direction of displacements was variable, first showing extensional characteristics and then after a few minutes of pumping becoming compressional. This is presumed to represent an outward migration of the inflection zone in response to additional removal of water from the aquifer. Figure 5 shows the probable configuration of the ground surface during short intervals of fluid flow. The distance from the well to the outer edge of the concave upward curved zone varied from a few meters at some wells to approximately 100 meters at others, and extensional horizontal displacements were detected several hundred meters from some wells.

DISCUSSION AND CONCLUSIONS

Deflections of the land surface around pumping (and injected) wells generally produced a circular or elliptical depression (or mound). The depression was only a few microns deep, and formed rapidly during the first few minutes of pumping, and then continued to expand very slowly with additional fluid flow. The depression had a concave upward surface centered on the well, and a convex upward surface at a greater distance from the

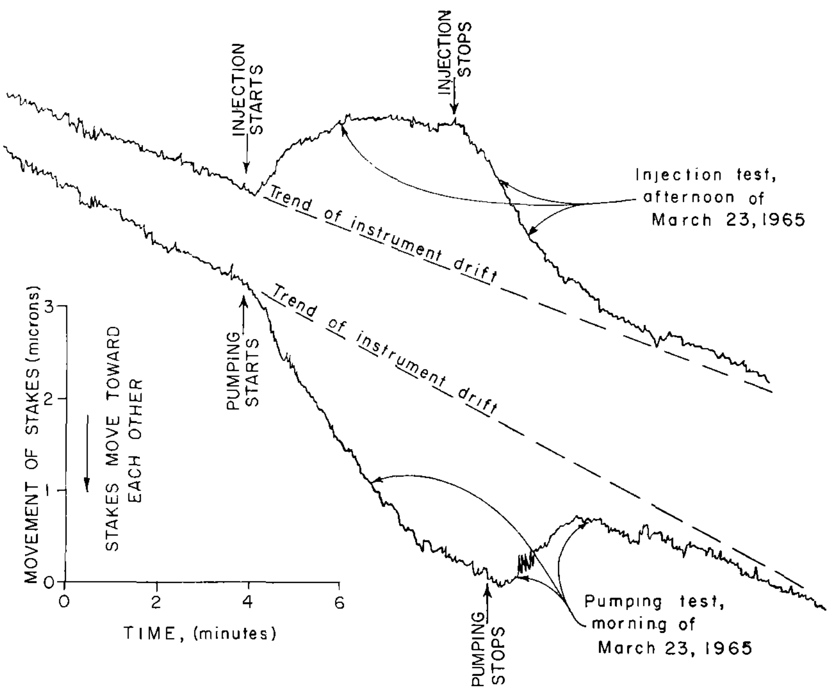

FIGURE 4. *Horizontal displacement of stakes produced by 100 liters per minute pumping and injection at Lagunita well, Stanford, California*

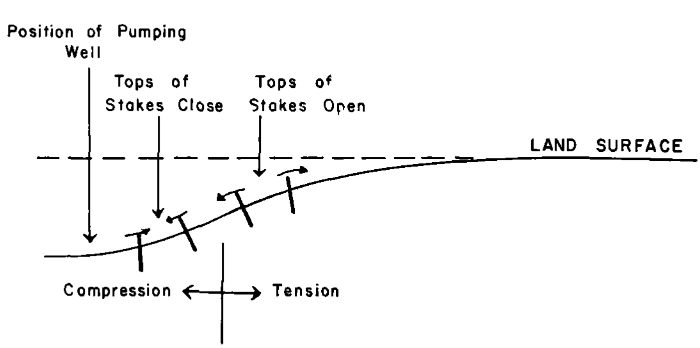

FIGURE 5. *General shape of subsidence depression near a pumping well. Total subsidence in almost all short-term tests was less than 40 microns*

well. The dimensions of the depression varied from well to well and were dependent primarily on the depth of the fluid-yielding zone(s), the rate of fluid flow, the total volume of fluid flow, and elastic properties of the aquifer and overburden layers. Variations of thickness and lithology of the aquifers tested produced depressions that were not always radially symmetrical.

Grant (1954) suggested the subsidence depression of the Wilmington Oil Field, California, was analogous to the theoretical vertical deflections of a point load on a plate of infinite extension resting on an elastic foundation. Deflections of the land surface measured in this study appear to be consitent with this analogy. Peterson (1969) has presented a detailed discussion of thsi apparent analogy.

The results of most interest from this study can be summarized as follows (Davis, Peterson, and Halderman, 1969):

(1) Simple yet precise techniques have been developed to measure displacements of the land surface;

(2) A qualitative description of the depression of the land surface around a pumping well has been provided;

(3) The techniques have been used to distinguish between aquifers subject to rapid compaction and those that are more stable. Better thermal and mechanical stabilities are needed, however, before the horizontal and vertical extensometers can be used to locate all areas of potential land subsidence;

(4) The close coupling of strain and hydraulic conditions in the aquifer is indicated by the almost instataneous response of the land surface to pumping or injection. This suggests, in turn, that strain records may some day be useful in interpreting the transmissivities and storage coefficients of aquifers that are not penetrated by observation wells;

(5) Our measurements show relatively large horizontal and/or rotational displacements at the surface. We assume that some of these non-vertical displacements must also be taking place in the aquifers. If so, equations for transient fluid flow near wells should be modified, because present equations assume only strain in the vertical direction (De Wiest, 1966; Cooper, 1966). However, the effects of nonvertical displacements are probably small, except in unusually thick aquifers.

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DISCUSSION

Intervention by Mr. J.T. CALLAHAN (USA):

Question:

Have you made experiments to test the reaction when pumping water from the weathered zone in crystalline rocks?

Answer by Prof. PETERSON:

We have run no experiments on wells in crystalline rocks, however, an attempt was made to detect horizontal displacements over brine solution cavities in limestone and dolomitic terrains. The purpose was to locate the lateral extent of the cavities by alternately pressurizing and de-pressurizing them in order to induce detectable subsidence of the ground surface. Unfortunately, no measurable surface movement was detected. This probably was owing to the relatively great depths to the cavities and the small pressure changes that could be achieved without hydro-fracturing the rocks.

Intervention by Dr. N. MIYABE (Japan):

Question:

May I understand that there is residual permanent deformation observed during your injection experiments?

Answer by Prof. PETERSEN:

In general, the deformation we observed was essentially elastic. At one of 16 wells tested a small amount of permanent deformation was observed. However, because the deformation is so small, we have been unable to separate the effects of subsidence from other background noises for periods longer 1-2 hours.

Intervention by Prof. K. L. LEE (USA):

Question:

It would appear that studies such as you describe could be used to obtain values of *in situ* soil modulus of deformation, provided a mathematical theory could be developed relating stress, modulus and deformation for the boundary conditions of your problem. Have you considered this, and developed a mathematical method for theoretically predicting the ground surface movements around a pumping well?

Answer by Prof. PETERSON:

Most of our early work has beer simply trying to gather enough basic data to describe the deformation which occurs at the ground surface. We are extremely interested, however, in pursuing the general line of work you suggest, and in particular we would like to develop a model to relate the surface displacement we measure to aquifer properties such as transmissibility and storage coefficient.

We are open to any suggestions.

SAND COMPRESSION AS A FACTOR IN OIL FIELD SUBSIDENCE

James E. ROBERTS
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and Consultand, Gribaldo, Jones and Assoc., Mountain View, Calif. U.S.A.

ABSTRACT

The results of a comprehensive experimental investigation of the compressibility of clastic sediments at high pressures are summarized. Of particular importance was the discovery that, in the pressure range of 1 000 to 20 000 psi, certain sands may be at least as compressible, if not more compressible, than typical clays. This high compressibility, which is due to a shattering of individual grains, indicates that a stratum of oil-bearing sand which is subjected to effective stress changes in the same pressure range may contribute significantly to the surface subsidence.

The relative importance of clay and sand compression as contributory to subsidence and the conditions under which either or both is important are summarized. It is concluded that even at a depth of 3 000 feet compression of oil-bearing sands may contribute significantly to subsidence.

RÉSUMÉ

Les résultats d'une recherche expérimentale de la compressibilité des sédiments clastiques sous de hautes pressions sont présentés. Une découverte de particulière importance est que, sous des pressions de 1 000 à 20 000 psi, certains sables peuvent être aussi compressibles, sinon plus compressibles, que les argiles typiques. Cette haute compressibilité, due à l'éclatement des grains, montre qu'une couche de sable pétrolifère qui serait soumise à de telles pressions peut contribuer de façon significative à l'affaissement de la surface du sol.

L'importance relative de la compression de l'argile et du sable contribuant au tassement et les conditions de leur degré d'importance ensemble ou séparément sont présentées. Il est conclu que même à une profondeur de 3 000 pieds la compression des sables pétrolifères peut contribuer de façon significative à l'affaissement du sol.

INTRODUCTION

The large surface subsidence which has been observed in certain oil fields has been the object of considerable study in recent years.

There has been fairly unanimous agreement among various investigators who have studied the subsidence problem that the primary cause of the subsidence is the compression of sediments due to the reduction of fluid pressures within the producing formation. However, there is still considerable difference of opinion as to whether the compressing sediments are the oil bearing sands, the interbedded shales and/or siltstones, or a combination of both. This disagreement has been due, in part, to a lack of experimental data on the compression behavior of soils at high pressure.

The principal objectives of this paper are:

(*a*) to present a summary of the results of a comprehensive experimental investigation of the compressibility of clastic sediments at high pressure and to show that, within the pressure range of 1,000 to 20,000 psi, sands can have compressibilities equal to or greater than similarly determined compressibilities of some clays; and

(*b*) to demonstrate that, at the high pressures encountered in deep sedimentary deposits, the compression of sands can be of equal, if not greater, importance than the compression of clays.

COMPRESSION BEHAVIOR OF CLASTIC SEDIMENTS

(*a*) General

Factors which must be considered in evaluating the compressibility of deep sediments and more particularly in evaluating the efficacy with which laboratory data obtained from one-dimensional compression tests can be extrapolated to predict the compressions of deposits of natural sediments include:

(1) Sediment composition, particularly its mineralogy, paticle size and distribution, and particle shape;

(2) Pore fluid characteristics at the time of deposition;

(3) Changes in pressure, temperature, and pore fluid subsequent to deposition;

(4) Elapsed time subsequent to deposition;

The results of the tests performed to evaluate the effects of soil composition and pressure are summarized below.

(*b*) Clays and Shales

Experimental work included 30 one-dimensional compression tests on samples of Venezuela Clays which were obtained from core samples taken from depths of from 2 486 to 4 769 feet below ground surface.

The range of results obtained from these laboratory tests on core samples are indicated by the three lines *A*, *B*, and *C* in figure 1. Line *A* represents the test resulting in the largest compression index* and also the highest values of void ratio*. Line *B* represents the test resulting in the smallest compression index. Line *C* represents the test having the smallest void ration throughout the test. To obtain each line, the portion of the compression curve

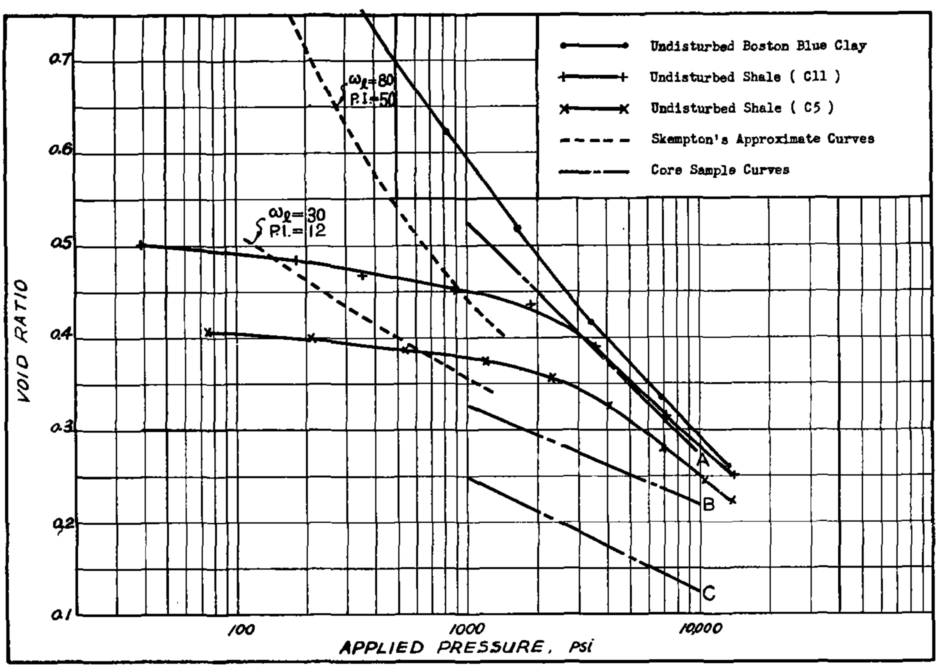

Figure 1. *Compression curves — clay and shale*

above the precompression was approximated by a straight line; this straight line was then extrapolated over the pressure range 1,000 psi to 10,000 psi even though the experimentally determined curve may have been straight over only a portion of the pressure range. Many of the samples showed a precompression of 1,000 psi to 2,000 psi.

The values of virgin compression index thus obtained vary from 0.11 to 0.26.

Compression curves presented by Skempton (1953) for a highly plastic clay with a liquid limit equal to 30 percent are also shown. At a pressure of 100 kg per sq cm (1421 psi) these clays have compression indices equal to 0.35 and about 0.17 respectively.

The results of a high pressure test on an "undisturbed" sample of Boston Blue Clay as representative of an inorganic clay of medium to high plasticity also have been plotted in figure 1.

(*c*) Sands

The results of a number of representative tests on quartz sand are shown in figure 2.

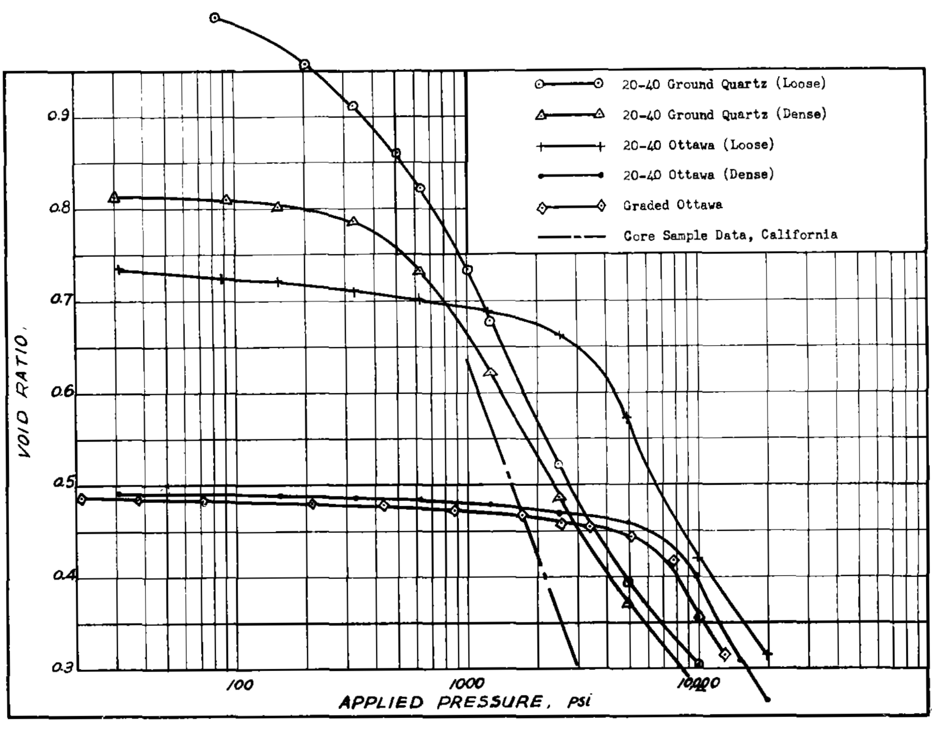

Figure 2. *Typical compression curves—quartz sand*

The effect of mineralogical composition was studied using highly angular sands composed of quartz, feldspar, and dolomite. A natural beach sand of volcanic origin from Hawaii also was tested as were a number of natural beach and river sands. Representative results are shown in figure 3.

In addition, a number of tests were performed on oil sands recovered from depths of about 3 400 feet. Results of representative tests in the "undisturbed" and remolded condition are shown in figure 4.

* See Glossary.

The most important fact brought to light by the experimental investigation is the relatively high compressibility of sands as measured in the one-dimensional laboratory test at high pressures.

At low pressures sands are relatively incompressible and the Compression Index is

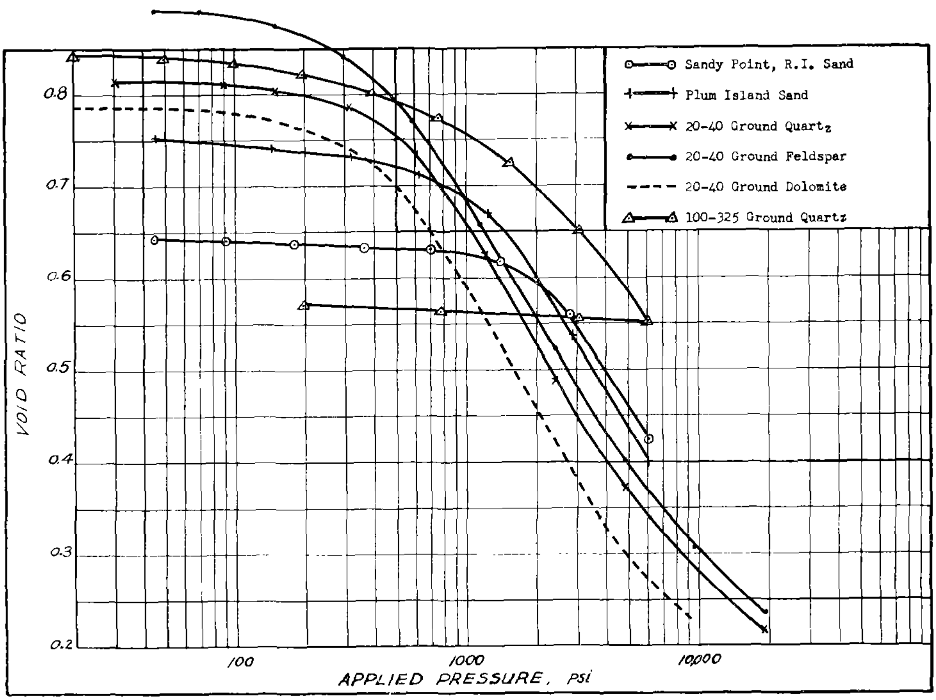

FIGURE 3. *Compression curves — sands*

generally small. The compression at low pressures is due to particle rearrangement. Because of elastic shortening of particles they shift by sliding and rolling. There may be some yielding or crushing at the points of contact, but at pressures as high as 1,000 psi only slight degradation of a uniform quartz sand tested had occurred.

When the pressure on the sample becomes sufficiently high, the load on individual grains becomes large enough to cause shattering or fracturing of the particles. Shattering generally manifests itself as an increased slope of the compression curve shown, for example, by typical curves in figure 2. The pressure at which the shattering becomes evident will be referred to as the "critical pressure" or the break-point. In the case of rounded particles, the shattering can easily be heard as a continual "popping" sound as the grains break. This "popping" only becomes readily detectable at pressures in the vicinity of the "critical pressure" and above.

At pressures high enough to cause shattering, quartz sands have maximum compression indices ranging from 0.35 to 0.70.

Tests on core samples of oil sands from depths of approximately 3 400 feet in one well indicate maximum compression indices ranging from 0.17 to 0.27. A value equal to 0.35 was obtained from one test on a sand core sample recovered from the same depth in a different well. This compares with the range of compression indices for the clay core samples of 0.11 to 0.26.

The pressures at which shattering begins depends on the initial density of the sample, on the angularity of the grains and on the grain size distribution. This pressure varies from about 100 psi for initially very loose, uniform, highly angular sand to about 9,000 psi for initially very dense, well graded, well rounded sand. The higher the initial relative density, the higher the break-point pressure. The more angular the grains, the lower the break-point pressure for any given method of deposition. The smaller the median grain size, the higher the break-point pressure for any given method of deposition.

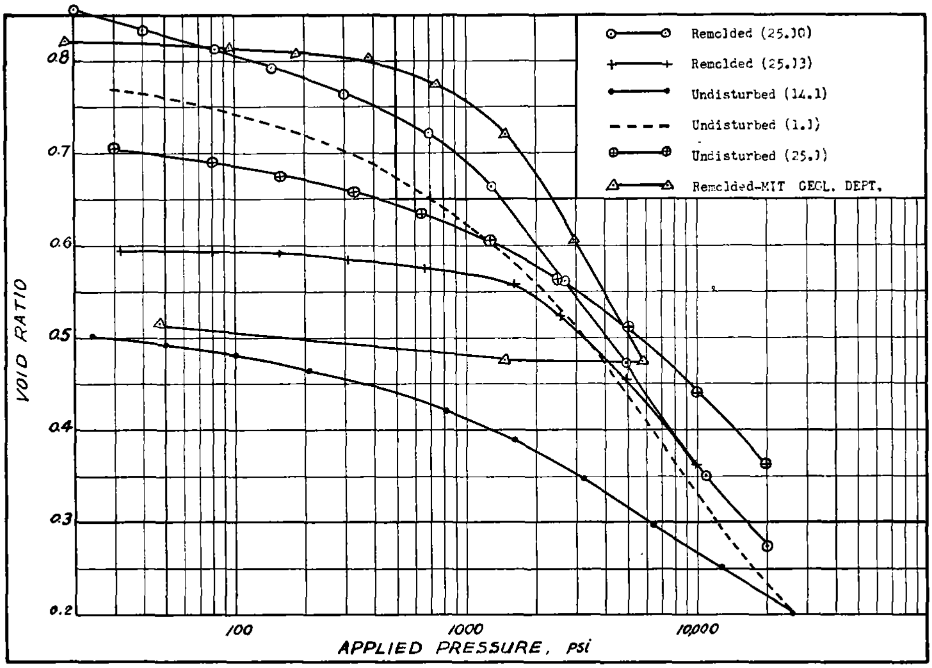

FIGURE 4. *Compression curves—cil sands*

The results of all the laboratory compression tests on uniform quartz sands of different sizes are summarized in figure 5. This plot shows the range of variation of the compression curves for pressures above the break-points.

Thus the limited data presented show that at pressures high enough to cause shattering of individual sand grains, the compressibility of sands as measured by the Compression Index* can be equal to or greater than that of representative clays or oil shales.

Although other investigators: Terzaghi (1925), Terzaghi and Peck (1948), De Beer, (1963) and USSR scientists (Drashevska 1958) observed degradation of sediment as a function of pressure, no comparison of the compressibility thus obtained with the compressibility of clays was made nor was there any apparent consideration of the significance of this high compressibility on the study of settlement due to compression of deep sediments.

* However, an increase in Compression Index cannot always be interpreted as an increase in compressibility.

RELATIVE IMPORTANCE OF CLAY AND SAND COMPRESSION

(*a*) General

In studies of the subsidence associated with the development of oil fields, it is not the total compression which a sediment has undergone since its deposition which is of interest but only the incremental compression which is likely to occur due to a pressure change. Therefore, any comparison or evaluation of the compressibility of sand and clay must be based on a comparison of this incremental compression due to a pressure change after each sediment has been consolidated to a relatively high pressure corresponding to the overburden pressure.

(*b*) Character of Natural Sands

Because the experimental results indicate that the compressibility of sand is quite dependent on initial density and angularity, it is appropriate to consider first the character of natural sands in order to evaluate how indicative the experimental results obtained in this ivestigation might be of the field compressibility of natural sands.

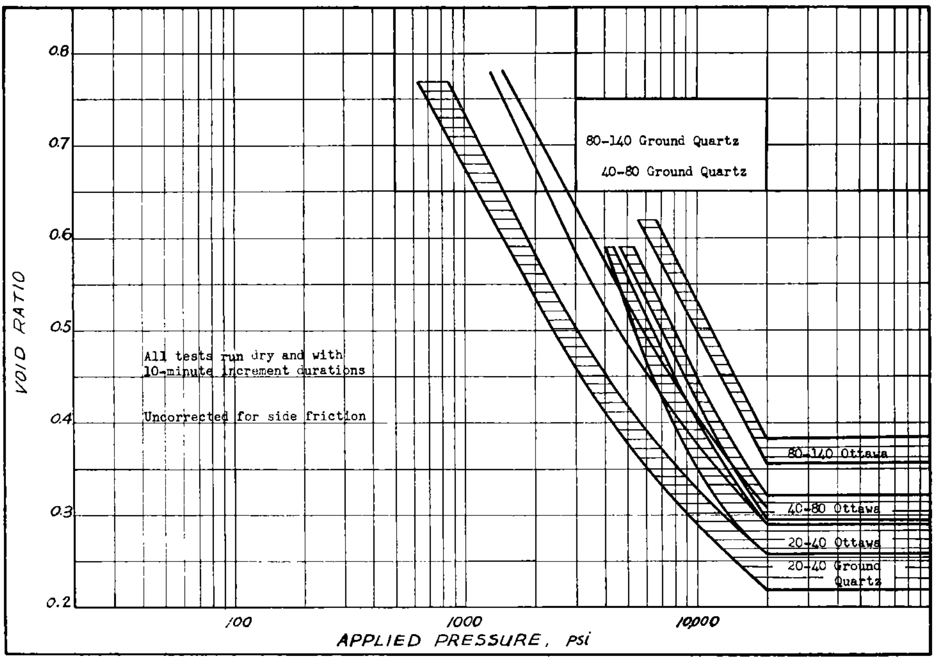

Figure 5. *Generalized compression curves—quartz sands*

The data on the character of natural sands collected by the Waterways Experiment Station (1960) indicate that, in general, one can expect quartz to be the predominate mineral sands. The sands are likely to be fairly uniform and individual particles are likely to be blocky with rounded to sub-rounded edges. Particles finer than about 0.2 to 0.5 mm are not likely to be appreciably rounded if water transported. Data presented by Koelser and Lara (1957) indicate that the initial void ratio of sands with a D_{10} size between 0.125 mm and 2 mm sedimented in water is likely to be of the order of 0.7 to 0.8.

Thus, the compression characteristics of natural sands deposited in water probably will be intermediate between the compressibilities of the well rounded and highly angular quartz sands studied in the present investigation.

(c) Comparison of Clay and Sand Compression

The relative compressibility of the representative sands and clays was evaluated by computing the percent compression, $\Delta H/H_0$, to be expected at various depths for strata composed of various soils if the fluid pressures were reduced from an initial static pressure to atmospheric pressure. This would be the normal maximum pressure change which could be developed due to oil or water production. The results of these computations for depths of 3,000, 5,000 and 8,000 feet are shown in table 1.

At a depth of 3,000 feet the computations show that a stratum of sediment comparable to the Boston Blue Clay would undergo about 6 percent compression. That is, for an initial stratum thickness of 100 feet a total settlement of approximately 6 feet would be

Table 1. Percent compression – Representative soils

Soil	Initial Void Ratio	Percent Compression at Various Depths 3 000 ft	5 000 ft	8 000 ft
Boston Blue Clay		6.0	5.6	4.9
Undisturbed Clay Core				
(≈ 2 500 ft.)		1.75	4.1	4.8
Undisturbed Sand Core				
(2 251 ft.)	0.803	4.3	3.7	5.9
Undisturbed Sand Core	0.54	2.5	2.8	3.3
(≈ 3400 ft)				
Undisturbed Sand Core		2.2	2.9	3.0
(≈ 3400 ft)				
Reformed Oil Sand				
Initially Loose	1.09	5.1	5.6	6.1
Initially Dense	0.60	2.3	3.9	4.8
Ottoawa Sand (20-40)				
Corrected for S. F)				
Initially Loose	0.62	1.35	5.0	10.0
Initially Dense	0.55	0.73	1.7	6.4
Ottawa Sand (40-80)				
Initially Loose	0.75	1.5	2.7	6.9
Initially Dense	0.57	0.7	1.1	1.8
Ottawa Sand (80-140)				
Initially Loose	0.83	1.9	2.9	5.3
Initially Dense	0.65	1.5	2.8	4.8
Ottawa Sand (20-140)	0.66	1.5	2.7	6.9
Ground Quartz (20-40)				
Initially Loose	1.04	8.7	7.6	5.9
Initially Dense	0.83	8.0	7.4	7.0
Ground Quartz				
(100-325)	0.85	6.6	7.0	6.9
Plum Island Sand	0.77	6.6	7.0	6.9
Sandy Point, R.I., Sand	0.65	2.1	7.0	7.4

* For fluid pressure reduction from initial hydrostatic to atmospheric.

expected. The various sands in general show much lower compressibilities; however, even at this depth, sands similar to the ground quartz or Plum Island sand would result in greater settlements than the Boston Blue Clay. Compression of the oil sand which was disturbed and repacked into an initially loose condition would result in a settlement only 15 percent less than that of the blue clay.

At a depth of 5,000 feet, the Blue Clay would undergo about 5-1/2 to 6 percent compression. At this depth the various sands would undergo from about 1 to 7-1/2 percent compression.

At a depth of 8,000 feet, the Blue Clay would undergo about 5 percent compression whereas the various sands would undergo compressions varying from about 2 to 10 percent. At this depth a sand similar to 20-40 Ottoawa would be about twice as compressible as the Blue Clay.

From these comparisons there can be no doubt that even at depths of 3,000 to 5,000 feet compression of the oil bearing sands may contribute quite significantly to the subsidence to be expected if the oil field is developed.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation to the Creole Petroleum Corporation for their support of the experimental work and especially Mr. John Martin. Particular credit is due Mr. John M. De Souza and Paul A. Harremoes, Research Assistants to the author during this investigation; and finally, the author is indebted to Dr. T. William Lambe, Head, Soil Engineering Division, Department of Civil Engineering, Massachusetts Institute of Technology, for continuous encouragement, guidance and patience.

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GLOSSARY

Void ratio, $e = V_s/V_v$

where: V_v = volume of void (pore) space

V_s = volume of solids

$$\text{Compression Index, } C_e = \frac{de}{d\,(\log p)}$$

The compression index is the slope of the compression curve plotted as void ratio vs applied pressure. (logarithmic scale)

DISCUSSION

Question by Mr. Robert F. YERKES (U S A):

Question:

I would like to ask whether you got grain crushing in the pressure range equivalent to oil field pressures?

Answer of Dr. ROBERTS:

Since it is my understanding that producing formations exist at depths of from about 3,000 to 15,000 feet, a pressure range of 3,000 to 15,000 psi after drawdown would be of interest. The pressures for the producing formation we were studying is in the range of 2,000 to 4,000 psi. As indicated by the slides, in the case of well-rounded grains we observed only a very modest amount of crushing at 1,000 psi. We observed progressively increasing amounts of degradation as we increased the pressure above that. On the other hand, with the angular grains, we got crushing at much lower pressures; for the ground quartz sample we observed crushing as low as 100 psi.

Intervention of Prof. Frank L. PETERSON (U S A):

Question:

What portion of the loss in pore space ocurring during compaction of the sands is recoverable?

Answer of Dr. ROBERTS:

That depends on whether you unload partially or completely. For the first increment of unloading, recovery is extremely small. For the first instance of load reduction, the curve is relatively flat and it steepens with progressive increasing reduction of pressures. If the first increment of reduction is about 1/2 of the applied load, then the expansion or rebound is quite small.

Question of Dr. H. OHTA (Japan):

Comment:

I suppose that sands under normal pressure behave mechanically similar to over-consolidated clays, and only under very high pressure sands behave similar to normally consolidated clays. The data obtained by Vesic, or Bishop and Webb for sands under high pressure support my opinion.

I think your data related to the change of the distribution of particle size might give the key for the research on the difference between the skelton of soil structure under normally consolidated state and over-consolidated state.

Answer of Dr. ROBERTS:

Although the appearance of the compression curve for sands even during a first loading cycle is similar to that for a precompressed (or over-consolidated) clay the phenomena are quite different. For sands, the increased compressibility at higher stress is a consequence of significant crushing or fracturing of individual grains. In the case of clay, the compression is undoubtedly primarily a consequence of rearrangement and reorientation of particles with little, or no, fracturing. For both types of soils, the rearrangement is irreversible and both sands and clays would have lower compressibilities on a second loading cycle than on the first. In the case of sands, however, even during a so-called first loading cycle an increased compressibility will be observed when the pressures become large enough to cause significant crushing. This increased compressibility during the first loading cycle has not, to my knowledge, ever been observed in clay.

COMPRESSIBILITIES OF CLAYS AND SOME MEANS OF PREDICTING AND PREVENTING SUBSIDENCE

George V. CHILINGAR, Herman H. RIEKE, III, and Costandi T. SAWABINI*

ABSTRACT

The compressibilities of various dry clays (halloysite, dickite, illite, kaolinite, hectorite, montmorillonite, attapulgite, and soil samples) have been determined experimentally in the laboratory by the writers and compared with those saturated with either fresh water or sea water. Compressibility (c_p) ranges from 8.67×10^{-5} to 2.94×10^{-4} psi^{-1} at 1000 psi and from 1.62×10^{-6} to 7.16×10^{-6} psi^{-1} at 100,000 psi.

Extensive compaction experiments conducted by the writers indicate that salinity of the squeezed-out solutions changes with pressure and that the mineralization of interstitial waters in shales should be lower than that in associated sandstones, if water moves from shale into sand. Thus, the salinity of interstitial solutions may give a clue as to the degree of subsequent compaction. In addition, decreasing salinity of produced waters with time may indicate water influx into sands from the associated undercompacted shales.

A new technique of eletrochemical stabilization of undercompacted formations is offered by the writers as a possible solution to the subsidence problem. This is based on the extensive experimental work in the laboratory and in the field by the writers, which is presented here. Electrochemical stabilization involves application of direct electric current in conjunction with introducing various electrolytes, such as calcium chloride, aluminum acetate and aluminum sulfate.

RÉSUMÉ

Les compressibilités de diverses argiles sèches (halloysite, dickite, illite, kaolinite, hectorite, montmorillonite, attapulgite et des échantillons de sol) ont été déterminées expérimentalement dans le laboratoire par les auteurs et comparées avec celles d'échantillons saturés à l'eau douce ou à l'eau de mer. La compressibilité (c_p) varie de $8{,}67 \times 10^{-5}$ à 2.94×10^{-4} psi^{-1} a 1000 psi et de $1{,}62 \times 10^{-6}$ à $7{,}16 \times 10^{-6}$ psi^{-1} à 100.000 psi.

De larges expériences de compaction conduites par les auteurs indiquent que la salinité des solutions libérées par la pression change avec celle-ci et que la minéralisation des eaux interstitielles des ardoises serait inférieure à celle des grès associés, si l'eau passe de l'ardoise dans le grès. Il en résulte que la salinité des solutions interstitielles peut donner des indications sur le degré de compaction. De plus, une salinité décroissante avec le temps des eaux produites peut indiquer une entrée d'eau dans les sables venant des ardoises sous-compactées.

Une nouvelle technique de stabilisation électrochimique de formations sous-compactées est présentée par les auteurs comme une solution au problème des affaissements. Elle est basée sur un travail intensif en laboratoire et sur le terrain par les auteurs. La stabilisation électrochimique implique l'application directe du courant conjuguée avec l'introduction de divers életrolytes, comme le chlorure de calcium, l'acétate d'aluminium et le sulfate d'aluminium.

INTRODUCTION

The production of fluids from buried sediments causes a reduction in pore pressure and subsequent compaction. As pointed out by van der Knaap and van der Vlis (1967), however, although subsidence is due to compaction of layers at depth, a certain amount of compaction does not give rise to the same amount of subsidence at the corresponding point at the surface. The variables involved include depth of the compacting layers, the area of

* Petroleum Engineering Department, University of Southern California, Los Angeles, Calif.

compacting formations, and mechanical properties of the reservoir sediments and the overburden. The amount of subsidence usually may be calculated from the compaction of producing layers; therefore, the knowledge of clay and sand compressibilities is of utmost importance. The first part of this paper is devoted to the subject of rock compressibilities.

The second part of the paper deals with relationship between the chemistry of interstitial solutions and the overburden pressure. The experimental results of the writers, which show that the chemistry of solutions squeezed out of muds changes with the magnitude of imposed overburden pressure, are presented here. Consequently, the chemistry of solutions in shales and associated sandstones may prove to be of value in predicting degree of subsequent compaction, especially in the case of undercompacted shales.

In the third part of the present paper the writers discuss the possibility of electrochemical stabilization of weak formations in order to prevent subsidence; this involves application of direct electric current in conjunction with introduction of various electrolytes.

I. COMPRESSIBILITY DATA ON VARIOUS CLAYS AND ARGILLACEOUS SEDIMENTS

Any changes in the physical properties of clays under high overburden pressures is a function of the change in the voids present in the clays. The magnitudes of these changes are related to the pore compressibility of the clays, whereas at very high pressures (at "zero" porosity) it is related to the compressibility of the mineral grains themselves. The studies on compressibilities of reservoir rocks (sandstones and limestones) are not numerous, but include some interesting papers by Carpenter and Spencer (1940), Fatt (1953, 1958a and b), Hall (1953), Harville (1967), Harville and Hawkins (1967), McLatchie *et al.* (1958), van der Knaap (1959), Dobrynin (1962), Knutson and Bohor (1963), and van der Knaap and van der Vlis (1967).

There are even fewer published studies on the compressibilities of shales and clays (Macey, 1940; Skempton, 1953; Paulding *et al.*, 1965; and van der Knaap and van der Vlis 1967). Consequently, the writers are attempting to fill this gap in our knowledge on the compressibility characteristics of clays (dry and hydrated) at various overburden pressures (40 psi to 200,000 psi). The bulk compressibility of various clays in this pressure range have been calculated by the writers from void ratio-versus-pressure curves and include the following API clay standards: dickite, halloysite, hectorite (containing 50% $CaCO_3$), illite, kaolinite and montmorillonite. Compressibilities of P-95 dry lake clay and a soil originating on a limestone terrain were also determined in the same manner. Some of these data were presented by Rieke (1969) and Rieke *et al.* (1969); the bulk compressibilities of clays saturated with water ranged from about 10^{-6} to 10^{-3} psi^{-1}. The bulk compressibility of dry clay powders were experimentally determined in the laboratory at pressures ranging from 1000 psi to 100,000 psi. Compressibilities of attapulgite and a lake clay (recently drained lake near Louisville, Kentucky, the sediments of which were derived from the Chattanooga Shale) were also determined.

The knowledge of the degree of compressibility and reduction in the bulk and pore volumes resulting from the overburden compression of sedimentary rocks is important in the studies of settlement and subsidence and in the investigation of abnormally high-pressure zones associated with undercompacted shales. It has been suggested by Hall (1953) and Geertsma (1957) that there is a possibility that neglecting the variation in pore volume due to compaction may introduce errors into material balance calculations for undersaturated petroleum reservoirs. An undersaturated reservoir is one in which the pressure is higher than the bubble point pressure, and, therefore, there is no free gas present.

In order to better understand the pressure-volume relationship of shales and agrillaceous sediments, the theoretical aspects of their bulk and pore volume compressibility variations should be known. Compressibility of a material can be defined as the change in volume of the reference component per unit change in pressure. The removal of fluid from reservoirs gives rise to a change in volume of (*a*) the reservoir rock, (*b*) its associated fluids, and (*c*) the interstratified or surrounding argillaceous rocks. This volume variation results in a decrease of the reservoir's total volume. There are four kinds of compressibilities that should be considered: (1) bulk compressibility,

$$c_b = -\frac{1}{e+1}\left(\frac{\mathrm{d}e}{\mathrm{d}p}\right) = -\frac{1}{h}\left(\frac{\mathrm{d}h}{\mathrm{d}p}\right);$$

(2) pore compressibility,

$$c_p = \frac{c_b}{\phi};$$

(3) grain or matrix compressibility,

$$c_s = -\frac{1}{V_s}\left(\frac{\mathrm{d}V_s}{\mathrm{d}p}\right);$$

and (4) fluid compressibility,

$$c_f = -\frac{1}{V_f}\left(\frac{\mathrm{d}V_f}{\mathrm{d}p}\right)_T.$$

e void ratio = volume of voids/volume solids;
h thickness of sample;
ϕ fractional porosity;
V_s volume of solids;
V_f volume of fluids;
p effective pressure; and
T temperature at which c_f is determined

The common assumption is that if porous rock or clay is completely saturated with fluids, all additional applied pressure is completely borne by the pore fluids and that the initial grain to grain contact stress remains constant. At high pressures this may not be valid because the relative compressibilities of the pore fiuids and grains must be taken into account. Any volume decrease of the material at high pressures ($>15{,}000$ psi) will be the result of continued elastic deformation and fracturing of the mineral grains and the compressibility of the pore fluids. The proper total compressibility expression for a clay, shale, or reservoir rock may then contain the compressibility terms for oil, gas, water and pores. Such an expression for the total compressibility is generally written in terms of separate phase compressibilities by volumetric phase saturation weighting (Craft and Hawkins, 1959, p. 273):

$$c_t = S_o c_o + S_g c_g + S_w c_w + c_p$$

where

c_t is "total system" compressibility;
c_o is oil compressibility;
c_g is gas compressibility;
c_p is pore compressibility; and

S_o, S_g and S_w are the oil, gas and water saturations of the formation, respectively. Ramey (1964, p. 447) presented generalized plots which speed the process of obtaining estimates of fluid and total system compressibilities.

The "effective" shale compressibility can be defined as the sum of the shale pore and water compressibilities. This is especially valid for the Tertiary basins where there is a large influx of water into the producing overpressured reservoirs from the surrounding argillaceous sediments. It should be noted that certain shales possess a large hydrocarbon content and this should be taken into consideration through the addition of the proper variables as shown in the above equation.

Although many consolidation (compressibility) studies on clays and shales have been carried out in soil mechanics laboratories for over 40 years, these tests have been mainly limited to a very small pressure range ($\leqslant 1000$ psi). During this time, high-pressure confining tests on reservoir rocks have exceeded 15,000 psi. It is of interest that most investigators, except for Knutson and Bohor (1962) and van der Knaap and van der Vlis (1967), used mainly well-consolidated, low-porosity sandstones or limestones in their laboratory experiments. Knutson and Bohor (1962) worked with reservoir rocks typical of the Texas-Louisiana Gulf Coast region (orthoquartzites to calcareous subgraywackes), whereas van der Knaap and van der Vlis (1967) worked with unconsolidated clays and sands from the Bolivar Coast in Venezuela.

Carpenter and Spencer (1940) measured the "pseudo" bulk compressibility $[-(1/V_b)(\mathrm{d}V_v/\mathrm{d}p)]$ (V_b = bulk volume, V_v = void volume) of various consolidated sandstones in an attempt to investigate whether or not fluid withdrawal from Gulf Coast reservoirs and the resulting volume reduction could account for subsidence. Compressibilities calculated from Carpenter and Spencer data on the Woodbine Sandstone are presented in figure 1.

Hall (1953, p. 310) utilized the reduction in pore space, at a constant overburden pressure, to obtain the "effective" compressibility values $[-(1/V_v)(\mathrm{d}V_v/\mathrm{d}p_i)_p]$ of sandstones and limestones. In his equation, p_i is the internal fluid pressure in the voids of the rock and p is the constant overburden pressure. He introduced another compressibility term, "formation compaction", $[-(1/V_v)(\mathrm{d}V_v/\mathrm{d}p)\ p_i]$, which is part of the total rock compressibility. Hall's values for "effective" reservoir rock compressibilities ranged from 1×10^{-7} to 3.4×10^{-6} psi^{-1} at p = 3000 psi. Hall showed that as porosity increases the "effective" rock compressibility decreases.

Fatt (1958a and b) and McLatchie *et al.* (1958) attempted to relate compressibility to rock composition. Both reported that unconsolidated sediments which are poorly sorted and contain clay have higher compressibilities than the consolidated and well-sorted sands. Fatt (1958b, p. 1924) found that his measured compressibilities in sandstones are a function of composition, grain-shape and sorting. Fatt's (1958b) and Hall's (1953) procedure was similar to that of Carpenter and Spencer (1940), but in the former case the produced fluid volume measured in the experiment resulted from a pore pressure reduction rather than an increase in the external stress, thereby, duplicating closely reservoir conditions. Fatt (1958b) reported measurements of bulk compressibilities on a variety of petroliferous sandstones (0 to 15,000 psi net confining pressure range) (fig. 1). His "pseudo" bulk compressibility (of a Boise Sandstone sample) is not the same property measured by Carpenter and Spencer (1940, p. 17), and was defined by Fatt as

$$[-(1/V_b)(\mathrm{d}V_b/\mathrm{d}p_i)p],$$

where p_i is the internal fluid pressure and p is the constant external pressure. This volume change is essentially the volume change that a reservoir undergoes as the fluid pressure is depleted by production. The net confining pressure is defined as the difference between the applied stress and the pore pressure. Fatt's work also included the development of several analytical analog models constructed out of rubber and glass spheres and cubes with which he attempted to rationalize the mechanical behavior of porous rocks.

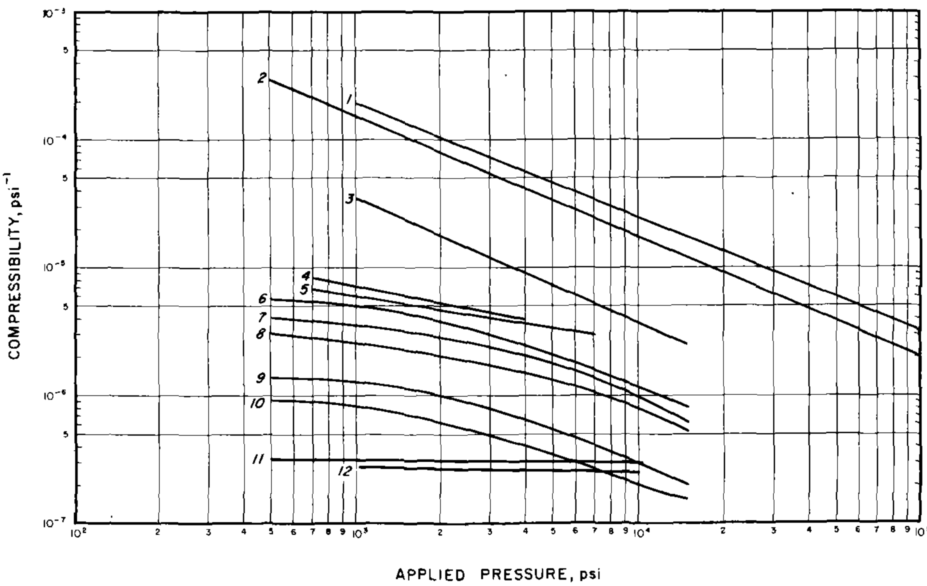

FIGURE 1. *Relationship between compressibility (psi^{-1}) and applied pressure (psi) for an illite clay, a limestone, sandstones and a shale.*

No.	Investigator	Rock type	Type of applied pressure	Compressibility
1	Authors	Illite clay (API No. 35)	Axial	Bulk
2	Authors	Illite clay (API No. 35) (wet)*	Axial	Bulk
3	Knutson and Bohor	Repetto Fm. (Grubb Zone) (wet) †	Net confining	Pore
4	Knutson and Bohor	Lansing-Kansas City Limestone (wet) †	Net confining	Pore
5	Carpenter and Spencer	Woodbine Sandstone (wet)	Net confining	Pseudo
6	Fatt	Feldspathic Graywacke (No. 10) (wet) ‡	Net confining	Bulk
7	Fatt	Graywacke (No. 7) (wet) ‡	Net confining	Bulk
8	Fatt	Feldspathic Graywacke (No. 11) (wet) ‡	Net confining	Bulk
9	Fatt	Lithic Graywacke (No. 12) (wet) ‡	Net confining	Bulk
10	Fatt	Feldspathic Quartzite (No. 20) (wet) ‡	Net confining	Bulk
11	Podio *et al.*	Green River Shale	Net confining	Bulk
12	Podio *et al.*	Green River Shale (wet)*	Net confining	Bulk

* Saturated with distilled water.
† Saturated with formation water.
‡ Saturated with kerosene.

Van der Knaap (1959) found that pore compressibility increases with decreasing porosity.

Dobrynin (1962, p. 361) suggested that between certain minimum and maximum pressures, the relation between pore compressibility and logarithm of pressure can be approximated by a straight line.

Knutson and Bohor (1963) presented pore compressibility data for many types of rocks, except shales.

Paulding *et al.* (1965, p. 70) studied the compressibility of a kaolinite clay sample (95% saturated with water) up to 32,000 psi. In this experiment no water egression from the sample was allowed. A bulk compressibility value of 8.8×10^{-7} psi^{-1} was found at 32,000 psi.

Thomas (1966, p. 4) studied the effects of overburden pressure on shale during retorting. He found that there was an increase, initially, in the bulk volume under low overburden pressures. Above 1000 psig the bulk volume decreased.

Van der Knaap and van der Vlis (1967) found a straight-line relationship between the log of the bulk compressibility and the log of the "effective" pressure, which in this case was equal to the direct applied axial load. The bulk compressibilities of the unconsolidated clays and sands decreased with increasing overburden pressure. The results obtained by Rieke *et al.* (1969) for various clays were of comparable magnitude.

Podio *et al.* (1968) computed the dynamic elastic properties of dry and water-saturated Green River Shale samples from compressional–and shear–wave velocity measurements. Bulk compressibility values of both wet and dry samples and their relation to confining pressure are given in figure 1. Values of compressibility for the wet samples are lower with respect to the values for dry shale. In all cases, the bulk compressibility decreases with increasing confining pressure.

The equipment used by the writers was described by Rieke (1969) and Rieke *et al.* (1969). Experimental data on compressibility of dry and wet clays are presented in figures 2 and 3.

Compressibilities of dry clays were calculated from height versus pressure curves; equations relating bulk compressibility to pressure are presented in table 1. It appears that kaolinite group is more compressible than montmorillonite. The nonlinearity of

TABLE 1. Bulk compressibility equations for various dry and saturated clays as shown in figures 1, 2 and 3.

Clay Type	Condition	Compressibility Equation
Attapulgite	dry	$c_b = 2.028 \times 10^{-2}\, p^{-0.6990}$
Dickite	dry	$c_b = 9.1308 \times 10^{-2}\, p^{-0.8925}$
Halloysite	dry	$c_b = 5.225 \times 10^{-2}\, p^{-0.7330}$
Hectorite	dry	$c_b = 9.563 \times 10^{-2}\, p^{-0.9825}$
Illite	dry	$c_b = 7.834 \times 10^{-2}\, p^{-0.9820}$
Kaolinite	dry	$c_b = 8.251 \times 10^{-2}\, p^{-0.9091}$
Lake Clay	dry	$c_b = 1.350 \times 10^{-2}\, p^{-0.7293}$
Montmorillonite	dry	$c_b = 7.577 \times 10^{-2}\, p^{-0.9348}$
Soil from Ls. terrain	dry	$c_b = 1.350 \times 10^{-2}\, p^{-0.7293}$
Attapulgite	wet (distilled water)	$c_b = 2.669 \times 10^{-3}\, p^{-0.3364}$
Illite	wet (distilled water)	$c_b = 1.02 \times 10^{-1}\, p^{-0.965}$
Montmorillonite	wet (distilled water)	$c_b = 1.55 \times 10^{-1}\, p^{-0.950}$ (1000-200,000 psi)
Montmorillonite	wet (sea water)	$c_b = 9.095 \times 10^{-2}\, p^{-0.9016}$

attapulgite curve is possibly due to the needle shape of the particles which rearrange and close-pack differently than do plate-like clay particles and, possibly, to crushing of these grains when rearrangement cannot effectively take place.

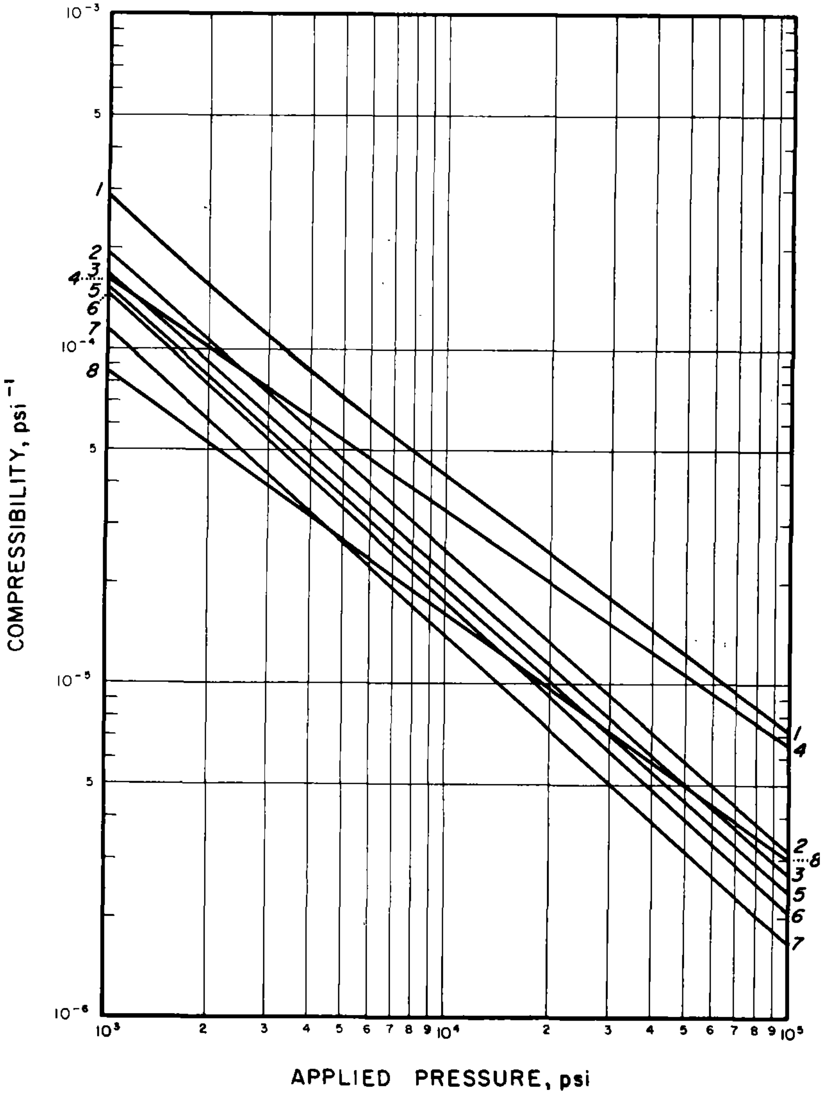

FIGURE 2. *Relationship between bulk compressibility in psi^{-1} and effctive pressure in psi for dry clay powders.* 1. *Halloysite (API No. 12)*; 2. *Dickite (API No. 15)*; 3. *Illite (API No. 35)*; 4. *Attapulgite (API No. 46)*; 5. *Kaolinite (API No. 4)*; 6. *Hectorite containing 50% $CaCO_3$ (API No. 34)*; 7. *Montmorillonite (API No. 25)*; and 8. *Lake clay derived from the Chattanooga Shale and soil derived from limestone terrain (Louisville, Kentucky) (both materials are presented by curve no. 8).*

Compressibility curve of attapulgite clay saturated with distilled water lies above montmorillonite saturated with fresh water curve and the curve is steeper (fig. 3); this is possibly due to the needle shape of attapulgite particles. The compressibility of montmorillonite saturated with sea water is less than that saturated with distilled water (fig.3), although flocculated clays close-pack better than do deflocculated ones.

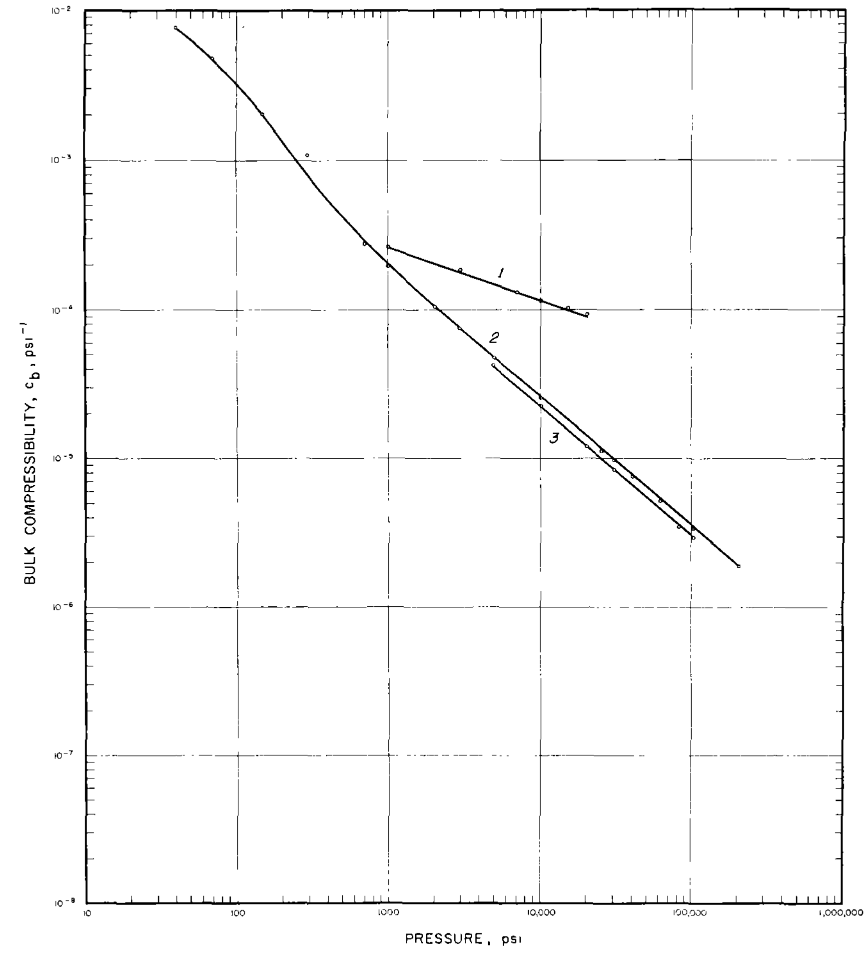

FIGURE 3. *Relationship between bulk compressibility in psi^{-1} and overburden pressure for (1) attapulgite clay saturated with distilled water, (2) montmorillonite saturated with distilled water, and (3) montmorillonite clay saturated with sea water.*

The compressibility of dry illite powder between the pressures of 1000 and 100,000 psi is linear on a log-log plot of compressibility versus effective pressure (figs. 1 and 2), and the curve for dry illite powder lies above that for illite initially saturated with distilled water. The same relationship (higher compressibility values of dry clay powder compared to its hydrated form) holds true for dickite, halloysite, hectorite, kaolinite and the soil from weathered limestone terrain. Apparently, dry clays closepack more than do hydrated, enlarged clay particles. In the latter case, possibly low permeability hinders expulsion of water. This does not, however, hold true in the case of montmorillonite (fig. 3); montmorillonite saturated with distilled water has higher compressibility values than that in a dry state. This can be attributed to the very high porosity of montmorillonite saturated in distilled water (Chilingar and Knight, 1960) as compared to other clays, possibly due to a "card house" structure in which the clay platelets are aligned edge to face forming a honeycombed-like structure. As shown by Chilingar and Knight (1960), the moisture (% dry weight) versus log of pressure curve below 1000 psi for montmorillonite clay hydrated in distilled water swings upward and is not linear. Possibly it is easier to squeeze non-oriented free water out in this lower pressure range.

Like sandstone, clay and shale compressibility is dependent on many variables. In the case of the hydrated clays the compressibility of the pores is influenced by the following variables: (1) the compacting pressure, (2) the fluid pressure in the pores, (3) permeability, and (4) degree of fluid saturation. In most competent rocks the bonding material has a high specific surface which is affected by the nature and amount of fluid in contact with it. Van der Knaap (1959, p. 183) found that wetting causes a certain amount of softening of the bonding material between the rock grains; this could explain why wet competent rocks have higher compressibilities than do dry rocks. This is not true in the case of most clays and shales as shown by the writers' experimental data and that of Podio *et al.* (1968) as presented in figure 1.

Terzaghi (1925) noticed that there was a relationship between the bulk compressibility of the clay, the grain-shape factor, and the forces of molecular attraction or repulsion within the adsorbed water layer. No doubt, the time factor and geothermal gradient also affect the bulk compressibility of clays and shales *in-situ*. Geertsma (1957, p. 331) theoretically showed that compressibility is independent of the pore shape; he assumed a continuous homogeneous matrix that is isotropic in nature. Gondouin and Scala (1958, p. 179) remarked that shale pore compressibilities, as calculated from laboratory data, were at least one order of magnitude larger than the pore compressibilities of sandstones. Geertsma (1957) stated that the laboratory measured compressibilities, by the "net confining" method, may be greater than those in the reservoir by a factor of two. In the "net confining" method, the sandstone and limestone cores are placed under a simulated hydrostatic overburden pressure, whereas the stresses in a reservoir may not be hydrostatic.

II. EFFECT OF COMPACTION ON CHEMISTRY OF SOLUTIONS SQUEEZED OUT OF CLAYS AND MUDS

Large amounts of water are squeezed out of the continental and marine sediments during compaction and lithification. The overburden pressures on these sediments may reach magnitudes of 14,000-36,000 psi in geosynclinal basins. Most of the salts present in the waters, which are trapped during sedimentation, are squeezed out during the initial stages of compaction. The laboratory results obtained by Buneeva *et al.* (1947), Kryukov and Komarova (1956), Kruykov and Zhuchkova (1963), and Rieke *et al.* (1964) showed that mineralization of squeezed-out solutions progressively decreases with increasing overburden pressure.

In order to illustrate the changes interstitial waters undergo during compaction, experimental data by the writers are presented in figures 4 and 5. It can be seen that the mineralization of solutions squeezed out during the different stages of compaction is definitely a function of overburden pressure. The degree of reduction in mineralization and the content of various ions in solutions squeezed out at higher overburden pressures from montmorillonite clay (API No. 25) saturated with sea water was presented by Degens and Chilingar (1967). All these results support the finding of Kryukov *et al.* (1962, p. 1365) that the mineralization of squeezed-out solutions changes with pressure.

In some experiments, the concentrations of the principal cations and anions decreased at about the same rate under pressure. This suggested that (1) the ions being removed represent interstitial electrolyte solution and do not include the adsorbed cations, and (2) the analysis for a single ion in the effluent (for example, Cl^-) might reveal as much as the analysis for all of the ions.

The salinity of squeezed-out solutions progressively decreases with increasing overburden pressure. Thus, the mineralization of interstitial solutions in shales is possibly less than that of waters in associated sandstones. The mineralization of solutions moving upward through a thick shale sequence as a result of compaction probably will progressively increase in salinity. It should be remembered, however, that if water from a sandstone bed moves through a shale layer into another sandstone bed, the water in the latter bed may be less mineralized because of filtration through a charged-net membrane.

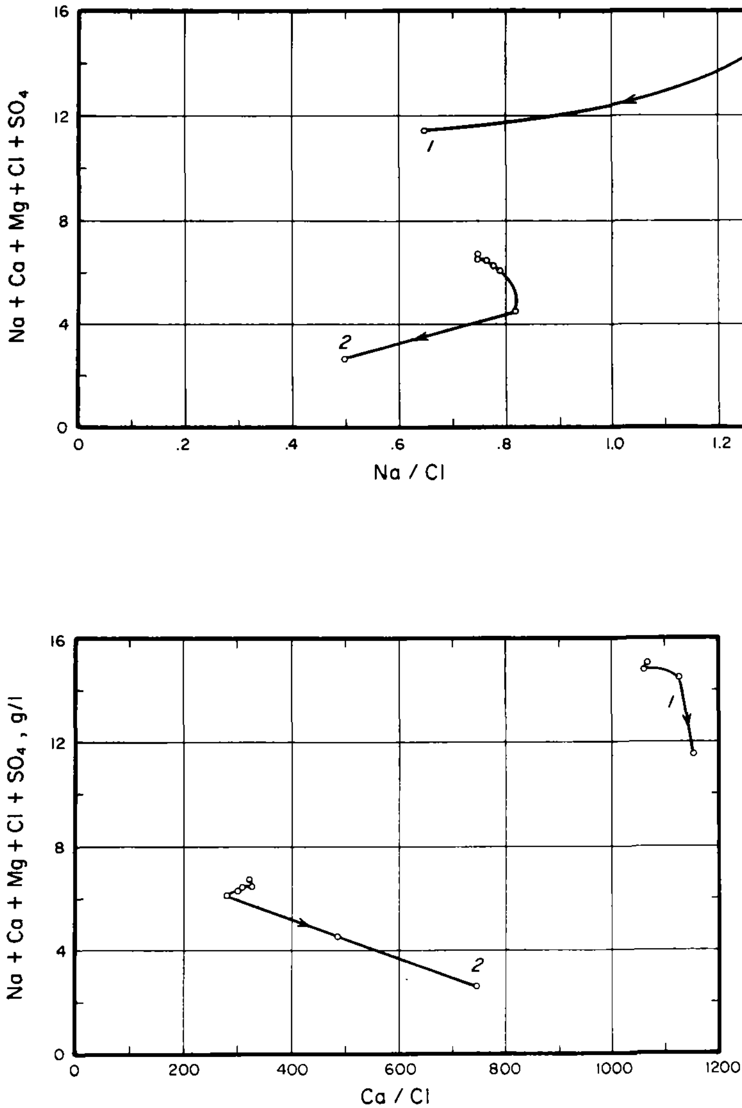

FIGURE 4. *Variation in Na/Cl and Ca/Cl ratios with total solids and pressure.* 1. *Kaolinite clay and* 2. *Montmorillonite clay. Arrows point in direction of increasing pressures.*

The reasons for the gradual decrease in the mineralization of squeezed-out solutions can be best understood on visualizing a capillary. When the cappillary is filled with water, the density of water next to the capillary walls is maximum, whereas along the center-line of the capillary the density is lowest an approaches one if the radius of the capillary is large enough. According to Martin (1960, p. 32), the density of adsorbed water on sodium montmorillonite varies with the amount of water present. The highest densities of bound water occur when the H_2O/clay weight ratio is less than one; these densities range from 0.980 to 1.41. The dissolving capacity of water is inversely proportional to density. As a result of compaction, the radius of capillary is decreased and the water is squeezed out, with the least bound water close to the center of the capillary being squeezed out first. The remaining adsorbed waters poor in electrolytes are squeezed out at the end.

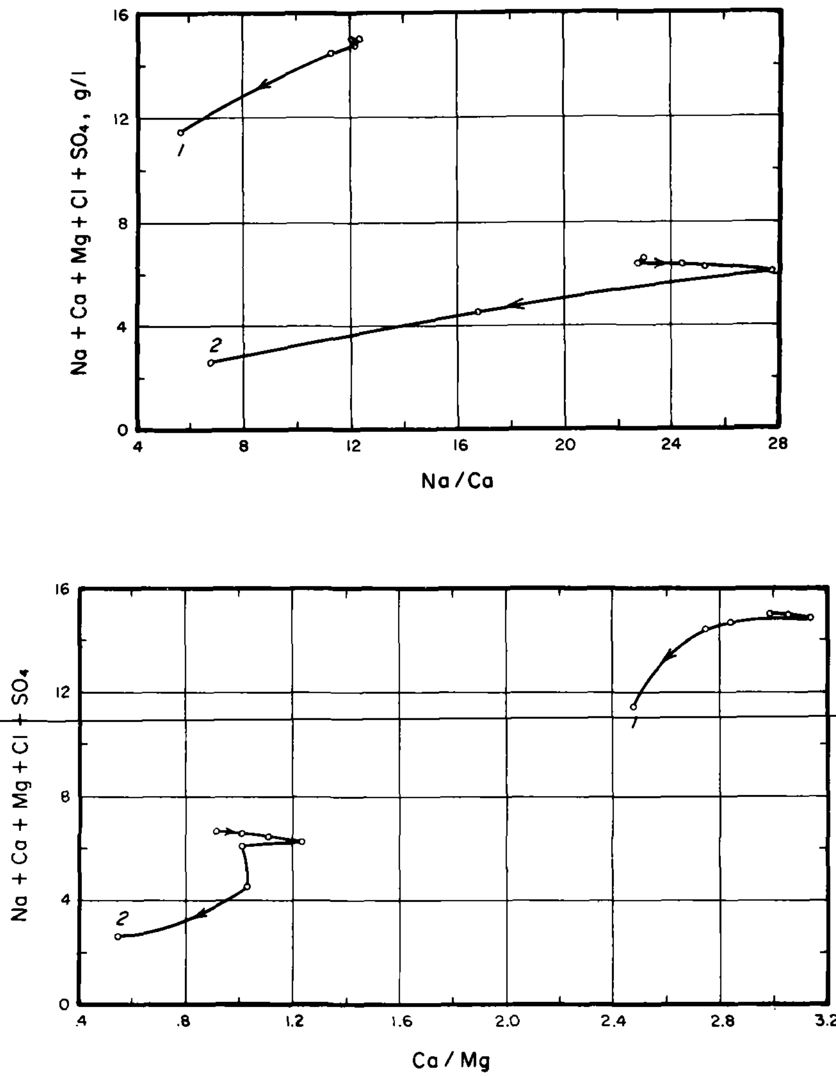

FIGURE 5. Variation of the Na/Ca and Ca/Mg ratios with total solids and pressure. 1. *Kaolinite clay and* 2. *Montmorillonite clay. Arrows point in direction of increasing pressures*

The percentage increase in the resistivity of squeezed-out solutions with increasing overburden pressure was shown by Rieke *et al.* (1964). The mud was obtained from the Santa Cruz Basin, off the coast of southern California.

It has been shown by many investigators that there is a tendency for the calcium ions to increase as salinity increases. According to White (1965, p. 361), there is also a tendency foı the calcium ions to increase relative to the sodium ions as salinity increases, because the Na^+ ions are more mobile than the Ca^{++} ions. The Ca/Cl ratio, with a few exceptions also increases with salinity (total solids). As pointed out by White (1965, p. 350), fine-grained sediments are not equally permeable to all ions, and it seems reasonable to expect less mobility for calcium (radius of hydrated ion is 9.6 Å) than for sodium (radius of hydrated ion is 5.6 Å) According to White (1965, p. 351) there is strong evidence that molecular water is considerably more mobile than chloride ions.

Experiments in the laboratory show that the Na/Cl ratio first increases and then decreases (for montmorillonite) with decreasing total solids and increasing pressure (fig. 4); the latter predominates at higheı pressures. In the case of kaolinite, the Na/Cl ratio decreases with decreasing total solids and increasing pressure. The Ca/Cl ratio first decreases and then increases with decreasing total solids and increasing pressure, with the latter predominating at higher pressures in the case of montmorillonite. For kaolinite clay the Ca/Cl ratio increases with decreasing salinity and increasing pressure (fig. 4). This is also true for the Na/Ca ratio, which in the case of montmorillonite, first increase before decreasing with decreasing total solids and increasing pressure. The Na/Ca ratio, in the case of kaolinite, decreases with decreasing salinity and increasing pressure (fig. 5). The Ca/Mg ratio first increases and then decreases with decreasing total solids and increasing pressure for both kaolinite and montmorillonite clays (fig. 5).

To further investigate this problem, 10 ml of sea water were added to 3.1958 g of clay and allowed to hydrate for 4 days. At the end of this period the supernatant fluid was decanted and analyzed. The remaining mud was centrifuged at 1500 rpm for 15 min

TABLE 2. Variation in the composition of the supernatant liquid and interstitial solution centrifuged out of montmorillonite clay (API No. 25). The chlorinity ratios (Ca/Cl, K/Cl and Na/Cl) are presented along with the Na/Ca and Ca/Mg ratios.

Ions	Composition in pppm			
	Sea water (S_t) (V_t = 10 ml)	Supernatant liquid (S_1) (V_1 = 2.95 ml)	Centrifuged liquid (S_2) (V_2 = 2.9 ml)	Remaining liquid (S_3) (V_3 = 4.15 ml)
Ca^{++}	480	444	462	518.2
Mg^{++}	1283	765	794	1992.9
B^{+++}	*	*	24	—
K^+	427	260	274	652.6
Na^+	10,554	13,949	14,813	5164.5
$SO_4^=$	2172	4380	4471	— a
Cl^-	19,574	20,355	21,823	16,202.4
Total solids	34,490	40,153	42,661	24,530.6
Na/Cl	0.539	0.685	0.678	0.319
Ca/Cl	0.0245	0.0218	0.0212	0.0320
K/Cl	0.0218	0.0128	0.0126	0.0403
Na/Ca	21.9	31.4	32.	9.966
Ca/Mg	0.374	0.580	0.581	0.260

* Tested for, but undetermined because the fluid sample volume was too small.
α The results are not reported because the clay tested has high $SO_4^=$ content.

(p = 6 psi) and the expelled fluid was also analyzed. The results are presented in table 2. The last column in this table shows the composition of remaining fluid in the mud, calculated using a material balance equation ($S_t V_t = S_1 V_1 + S_2 V_2 + S_3 V_3$, where S is the salt content in ppm and V is the volume in ml; table 2). These results indicate that total salinity of initially squeezed-out solutions, in this case, first increases and the remaining interstitial portion has lower salinity.

In 1947, DeSitter reported that the salinity of formation waters in sandstones varied from that of fresh water to ten times the salinity of sea water. The distribution of salinity of interstitial waters present in young geosynclinal sediments along the Gulf Coast has been well documented by Timm and Maricelli (1953), Myers (1962), and Fowler (1968).

Timm and Maricelli (1953, p. 394) stated that high salinities up to $4\frac{1}{2}$ times that of normal sea water characterize the interstitial solutions in Miocene-Pliocene sediments, where the relative quantity of undercompacted shale is small. In Eocene sediments, where the relative quantity of shale is large and its degree of compaction is high, interstitial solutions have salinities as low as $\frac{1}{2}$ that of normal sea water. Figure 6 illustrates their concept that the formation waters in down-dip, interfingering marine sandstone members, which have proportionately less volume than the associated massive shales, have lower salinities than that of sea water. More massive sands updip have salinities greater than that of sea water. Salinity was determined on using the following techniques: electrical resistivity, complete chemical analysis, and titration (see Gullikson *et al.*, 1961).

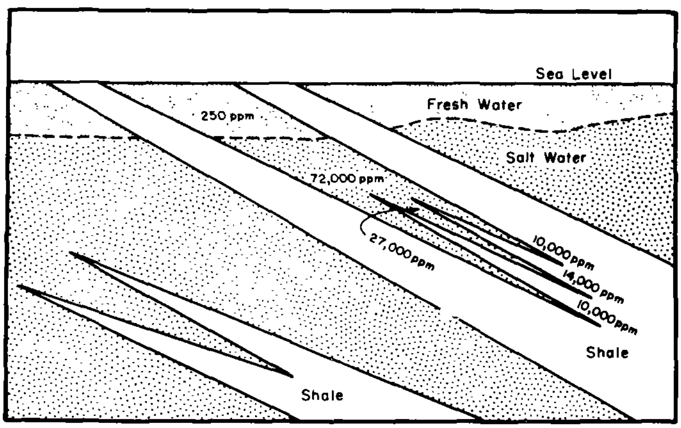

FIGURE 6. *Idealized typical cross-section of some sands in southwest Louisiana showing generalized salinity relationships. (After Timm and Maricelli, 1953, p. 396, 397 and 408).*

Myers (1962) studied the chemical properties of formation waters, down to a depth of 12,400 feet, in four producing oil wells in Matagorda County, Texas. The salinities of interstitial waters ranging from 5000 ppm to 12,500 ppm were found below 10,200 ft in each of the four wells. He commented that in this deeper section, the proportion of massive shale is large and the sands are near their down-dip limits (become thinner). These results were in close accord with those of Timm and Maricelli (1953).

Many investigators (i.e., Hottman and Johnson, 1965) observed that the sands with abnormally high pore-water pressures are associated with undercompacted shales having very high porosity. In an excellent paper, Dickey *et al.* (1968) pointed out that faults, which cut oil reservoirs, form pressure discontinuities and act as seals for zones of very high fluid pressure for very long times. The high porosity of shales in such zones is reflected by the high values of conductivity. The depth marking the beginning of the abnormally high fluid pressures in the sandstones coincides with abnormal increase in conductivity of associated undercompacted shales (Williams *et al.*, 1965; Wallace, 1965). Yet, calculations by the writers indicate that possibly high prosity of shales alone could not account for this abnormal increase in conductivity; the salinity of interstitial waters also seems to play an important role.

The findings of Fowler (1968) for the Chocolate Bayou Field, Brazoria County, Texas, seem to confirm that the salinity of water in under-compacted shales are higher than in well-compacted ones. He found a definite correlation between the high salinity of interstitial fluids and abnormally high pressures. In addition, he studied variation in salinity of produced water with time. The typical pattern is one of decreasing salinity with time, and the freshest water is found in sands receiving most of this water from associated shales. This is in line with experimental results of the writers which show that salinity of waters in shales is less than that in associated sands. Thus, the following factors can be considered in predicting possible subsidence:

1. Presence of undercompacted shales, which may compact upon withdrawal of fluids from sands and subsequent movement of water from shales into sands.
2. Undercompacted shales are associated with abnormally high pressures and exhibit very high conductivity due to the high porosity and possibly higher salinity of interstitial fluids as compared to those in well-compacted shales.
3. The interstitial fluids in shales are fresher than those in associated sandstones. Thus, decreasing salinity of produced waters with time may indicate their influx into sands from the associated undercompacted shales; this may eventually cause subsidence.

III. ELECTROCHEMICAL STABILIZATION OF WEAK FORMATIONS

During the past eleven years extensive research has been conducted at the University of Southern California Petroleum and Civil Engineering laboratories on electrochemical stabilization of weak grounds (Adamson *et al.*, 1966a, 1966b, 1967; Rieke, *et al.*, 1966; Harton *et al.*, 1967).

Many processes occur as a result of application of direct electric current such as ion diffusion, ion exchange, dessication, buildup of pH and osmotic gradients, decomposition of some minerals, precipitation of secondary minerals, electrolysis, hydrolysis, oxidation, reduction, physical and chemical adsorption, and reorientation of clay particles. The above-mentioned authors concluded that electrochemical stabilization is largely due to cementation. Various minerals form during the electrochemical treatment using iron and aluminum electrodes along with introduction of various electrolytes such as calcium chloride, aluminum sulphate, aluminum acetate and sodium silicate. These minerals include gibbsite, aluminite, hisingerite, allophane, opal, calcite, magnetite, hematite, hydrogoethite, limonite, nontronite, and gypsum. Titkov *et al.* (1964) also found these neoformations after electrical treatment. Chilingar *et al.* (1968a, 1968b) discovered that electrical treatment may also result in destruction of clays and formation of complex non-swelling silicates.

In some recent tests, upon electrochemical treatment of clayey silt, the cohesion at the anode increased from 300 to 900 lb/sq ft, whereas at the cathode to 510 lb/sq ft. The angle of internal friction was raised from 15° to 21°. About 20-40 kW-hr are necessary for stabilization of 1 m^3 of ground. It seems, however, that in the case of massive subsidence only a portion of settling mass needs to be stabilized to prevent further subsidence. This may be analogous to diagenetic dolomitization: in the case of presence of solid framework (crinoid stems, etc.) porosity created by dolomitization remains intact, whereas in the case of early diagenesis of soft muds created porosity is destroyed by subsequent compaction (Chilingar *et al.*, 1967, pp. 287-297).

CONCLUSIONS

The findings of the present study can be summarized as follows:

1. The compressibility of various dry and wet clays studied (attapulgite, dickite, halloysite hectorite (containing 50% $CaCO_3$), illite, kaolinite, montmorillonite, P-95 dry lake clay, and soil from limestone terrain) range from about 10^{-6} to 10^{-2} psi^{-1}. Compressibility (c_b) of dry clays ranges from 8.67×10^{-5} to 2.94×10^{-4} psi^{-1} at 1000 psi and from 1.62×10^{-6} to 7.16×10^{-6} psi^{-1} at 100,000 psi.
2. The compressibilities of clays saturated with fresh water lie below those tested in a dry state.
3. Compressibilities of illite saturated with fresh water ($c_b = 3 \times 10^{-4}$ to 2×10^{-6} at 500 to 100,000 psi) lie below the values for montmorillonite saturated with fresh water ($c_b = 7.765 \times 10^{-3}$ to 1.883×10^{-6} at 40 to 200,000 psi).
4. Compressibilities of montmorillonite clay saturated with sea water lie below those for montmorillonite clay saturated with fresh water.
5. The chemistry of interstitial solutions squeezed out of clays in the laboratory gradually changes with overburden pressure. The less mineralized solutions extrude at higher pressures. Thus, it appears that chemistry of solutions in shales should be fresher than those in associated sandstones and may serve as an indicator of degree of prior compaction. Some field data support these findings. Consequently, the chemistry of interstitial solutions may indicate degree of undercompaction of shales and amount of subsequent compaction upon efflux of fluids into associated producing sandstones.
6. Electrochemical treatment should be tried in stabilization of subsiding grounds.

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FINITE ELEMENT ANALYSIS OF LAND SUBSIDENCE

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ABSTRACT

Application of the finite element method to the problem of land subsidence is presented. The settlement of a land mass is viewed as an immediate or time dependent surface deformation caused by direct application of surface loads or by the loss of support associated with mining or withdrawal of pore fluids. Practically all cases involving static or quasi-static subsidence can be treated. The method permits consideration of complex geometrical configurations and arbitrary boundary conditions. Non-homogeneity, anisotropy, viscoelasticity and creep, temperature effects, residual stresses, plastic behavior can be allowed for. The method is applicable to two or three dimensional deformation and thus takes into account horizontal as well as vertical movements.

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RÉSUMÉ

Les auteurs présentent une application de la méthode des "éléments finis" au problème des affaissements. Le tassement d'une masse de sol est considéré comme une déformation immédiate ou dépendant des temps causés par l'application directe de charges en surface ou par la perte de support provoquée par l'exploitation minière ou l'extraction du fluide des pores. Pratiquement tous les problèmes d'affaissements statiques ou quasi statiques peuvent être traités. La méthode permet la prise en considération de configurations géométriques complexes et de conditions aux limites arbitraires. On peut tenir compte de la non-homogénéité, de l'anisotropie, de la visco-élasticité et du fluage, des effets de température, des contraintes résiduelles, du comportement plastique. La méthode est applicable à la déformation à deux ou à trois dimensions et tient compte par conséquent tant des mouvements horizontaux que verticaux.

INTRODUCTION

Land subsidence, natural or artifically induced, is of considerable importance in engineering. However, analytical solutions are available only for the cases of simples loadings, idealized homogeneous linearly elastic materials and simple geometry. Problems involving non-homogeneous materials and complex geometry are intractable by the classical approach. For these numerical procedures have to be used to generate approximate solutions.

Settlement of a land mass may result from a natural process or may be caused by man. Creation of large reservoirs, loading of soft soil to consolidate it faster to improve its supporting qualities, construction of structures that apply large loads to the surface are some examples of external loads on the land mass. Removal of material by mining or deep excavation in a stressed medium or a change in the pressure-flow regime of the pore fluids results in land-mass deformations due to ,loss of support'. The actual deformations evidently will depend upon the manner of loading and also upon the mechanical properties of the constituent materials. Presence of internal or external boundaries and the condition of saturation profoundly influence the magnitude of subsidence.

For a satisfactory analysis of land subsidence, therefore, we need a method that will allow consideration of external loads, residual stresses, incremental excavation or mining, arbitrary geometrical configuration, presence and influence of pore fluids, lack of homogeneity, and actual mechanical behavior of the materials involved. Approximate solution techniques using the finite element idealization hold the best promise.The theory of the finite element method is well documented in literature [3, 17, 20]. It has been used to solve boundary value problems in two and three-dimensional elasticity [3, 5, 17, 20] and seepage [7, 13, 16, 19]. Creep and viscoelasticity [8, 15], incremental construction [4, 8, 15], non linear elasticity [5, 18], effect of joints [6] have been analyzed by this method. The method has been applied to predict horizontal as well as vertical settlement [9], and to determine the time dependent deformations caused by excavating a large cavity in a material having large initial stresses and nonlinear stress-strain-time law [10]. Recently the method has been extended to consolidation, natural or antifial, of an initially stressed saturated land mass [13, 14]. Elasto-plastic material bahavior has also been considered [1].

BASIC EQUATIONS OF DEFORMATION AND FLOW

We shall consider the stresses and deformation of a fluid-saturated medium. The case of fluid free land mass will be included as a particular application of the general formulation.

Neglecting chemical reaction and inertia affects, the equilibrium equations are [13]:

$$\sigma_{ki,k}+\delta_{ik}\pi_{,k}+\rho F_i=0 \tag{1}$$

where:

σ_{ki} are the components of the symmetric stress tensor for the solid phase;
π the hydrostatic stress in the pore fluid;
ρ the total mass density of the saturated material;
F_i are the components of the body force and
δ_{ik} the Kronecker delta.

Subscripts after a comma denote spatial differentiation in the indicial notation and repeated indices indicate summation. In cases where there is no pore fluid or we have a free draining soil mass, $\pi = 0$ and the equilibrium equations reduce to the usual form

$$\sigma_{ki,k} + \rho F_i = 0 \tag{2}$$

The equation of flow is a generalization of Darcy's law.

$$v_i = K_{ij}(\pi_{,j} + \rho_2 F_j) \tag{3}$$

where:

v_i are the components of the relative velocity vector;
K_{ij} are the components of the permeability tensor and
ρ_2 the mass density of the fluid, assumed to be incompressible.

Taking divergence of equation 3, we note that

$$v_{i,i} = -\dot{u}_{i,i} = -\dot{e}_{ii} \tag{4}$$

where:

u_i are the components of the displacement vector;
e_{ii} are the components of the strain tensor, for the solid phase,

and a superposed dot denotes differentiation with respect to time. Hence

$$\dot{u}_{i,i} + [K_{ij}(\pi_{,j} + \rho_2 F_j)]_{,i} = 0 \tag{5}$$

Equation 5 is the condition of saturation. In integral form it can be written, assuming an initially undeformed system:

$$u_{i,i} + \int_0^t [K_{ij}(\pi_{,j} + \rho_2 F_j)]_{,i} \mathrm{d}t = 0 \tag{6}$$

Assuming a strain displacement and a stress-strain relationship, equations 1 can be written in terms of the three components of displacement. Then these equations along with equation 6 are sufficient for determination of deformations and fluid pressures. Evidently for a free-draining soil mass or no pore fluid, equations 2 written in terms of displacements are sufficient to govern the problem.

For the case of infinitesimal deformations, the strain-displacement law is:

$$e_{ij} = \tfrac{1}{2}(u_{i,j} + u_{j,i}) \tag{7}$$

For linear elasticity, the general stress strain law is:

$$\sigma_{ij} = C_{ijkl}\, e_{kl} \tag{8}$$

where C_{ijkl} are the components of the anisotropic elasticity tensor such that

$$C_{ijkl} = C_{klij} = C_{jikl} = C_{ijlk} \tag{9}$$

It is to be noted that K_{ij} and C_{ijkl} are completely unrelated i.e. hydraulic anisotropy is independent of mechanical anisotropy.

THE FINITE ELEMENT METHOD

The finite element method involves the replacement of the actual continuum by a finite number of discrete subregions called elements. The geometry of the elements is completely defined by a set of points in space called the nodal points of the system. The values of the unknown field variables within each element are expressed in terms of the nodal point values by means of suitable interpolation functions. Thus the entire solution can be expressed in terms of the nodal point values which are evaluated through solution of governing equations written in the matrix form. Variational methods provide a convenient procedure for setting up the matrix equations and have been widely used in finite element analysis [7, 11, 13, 14, 18]. However, in what follows, we shall adopt the direct approach [16, 20].

Interpolation scheme for displacements within an element m can be expressed as

$$u_i^m(\underset{\sim}{x}, t) = \{\phi_u^m\}^T \{u_i(t)\} \tag{10}$$

where:

$u_i^m(\underset{\sim}{x}, t)$ is the ith component of the displacement vector at (x, t) in the element;
$\{\phi_u^m\}$ is the set of displacement interpolation functions and
$\{u_i(t)\}$ is the set of ith components of nodal point displacements fo the entire system.

The symbols x and t in the parentheses indicate the space and time dependence of the quantities. u_i^m are functions of space and time and u_i are functions of time only.

The strain-displacement relationship, equation 7, can then be written as:

$$\{e^m(\underset{\sim}{x}, t)\} = [\phi_e^m]^T \{\underset{\sim}{u}(t)\} \tag{11}$$

where $\{e^m(x, t)\}$ is the reduced strain tensor

$$\{e^m(\underset{\sim}{x}, t)\} = \begin{Bmatrix} e_x \\ e_y \\ \gamma_{xy} \end{Bmatrix} \tag{12}$$

and $[\phi_e^m]^T$ is the transformation matrix derived from the displacement interpolation functions by suitable differentiation and re-arrangement of terms. Application of equation 8 gives the reduced stress tensor for the element as:

$$\{\sigma^m(\underset{\sim}{x}, t)\} = [H^m]\,[\phi_e^m]\,\{\underset{\sim}{u}(t)\} \tag{13}$$

where $[H^m]$ is the reduced elasticity tensor for the element.

If residual stresses are present in the soil mass, e.g. horizontal stresses caused by geologic factors, gravity load stresses, etc., these can be allowed for by simply adding these to the stresses associated with the deformations. Thus equation 13 will be modified to

$$\{\sigma^m(\underset{\sim}{x}, t)\} = [H^m]\,[\phi_e^m]\,\{u(t)\} + \{\sigma_0^m(\underset{\sim}{x}, t)\} \tag{14}$$

where $\{\sigma_o^m(\underset{\sim}{x}, t)\}$ are the residual stresses for element m.

It can be shown [17] that the transformation matrix relating nodal point forces to element stresses is transpose of the matrix relating element strains to nodal point displacements. Thus,

$$\begin{aligned}\{P_1^m\} &= [\phi_e^m]^T \{\sigma^m(\underset{\sim}{x}, t)\} \\ &= [\phi_e^m]^T [H^m]\,[\phi_e^m]\,\{\underset{\sim}{u}(t)\} + [\phi_e^m]^T \{\sigma_0^m(\underset{\sim}{x}, t)\} \\ &= [A^m]\,\{\underset{\sim}{u}(t)\} + \{M_1^m\}\end{aligned} \tag{15}$$

where:

$\{P_1^m\}$ is the vector of nodal point loads;
$[A^m]$ is the element stiffness and
$\{M_1^m\}$ is the vector of loads corresponding to release of residual stresses.

Similarly, we use an interpolation scheme for pore-fluid pressures and carry out appropriate differentiations to establish a relationship between nodal point fluxes and the fluid pressures. Summing up the relations for the entire system and including effect of pore fluid pressure and body forces, we obtain:

$$[A]\,\{u(t)\}+[C]\,\{\pi(t)\} = -\,\{M_1\}+\{M_2\}+\{P_1(t)\} \tag{16}$$

$$[C]^T \int_{t_0}^{t_1} \{\dot{u}(t)\}\,dt-[B]\int_{t_0}^{t_1} \{\pi(t)\}\,dt = \int_{t_0}^{t_1} \{M_3(t)\}\,dt - \int_{t_0}^{t_1} \{P_2(t)\}\,dt \tag{17}$$

where:

[A] is the stiffness matrix for the solid phase;
[C] is the coupling matrix;
[B] is the flow matrix;
$-\{M_1\}$ is the load vector due to residual stresses;
$\{M_2\}$ is the load vector due to the body forces, and
$\{P_1\}$ is due to the boundary pressures.

Locally applied loads can be included in $\{P_1\}$. In the second equation $\{M_3\}$ is the vector of nodal point flows due to body forces (generally gravity) and $\{P_2\}$ is the vector of specified boundary flow and local drainages (sinks or sources).

Assuming linear interpolation over a short time interval t_0 to t_1, and writing $\Delta t = t_1 - t_0$, equation 17 becomes:

$$\begin{aligned}[C]^T\,\{u(t_1)\} - \frac{\Delta t}{2}[B]\,\{\pi(t_1)\} = {} & [C]^T\,\{u(t_0)\} + \frac{\Delta t}{2}[B]\,\{\pi(t_0)\} \\ & + \frac{\Delta t}{2}\{M_3(t_0)+M_3(t_1)\} \\ & - \frac{\Delta t}{2}\{P_2(t_0)+P_2(t_1)\}\end{aligned} \tag{18}$$

Equations 16 and 18 express the values of the unknown nodal displacements and fluid pressures at any time in terms of the prescribed data and the values at the previous time step. It is noted that the boundary data may be varied with each time step. These equations can be solved by standard procedures. Prescribed boundary values for displacements and pressures can be allowed for using techniques explained by Wilson [17].

The above derivation applies to the linear elastic case. Other material laws can be incorporated in the analysis as desired following standard procedures (1, 5, 6, 8, 10, 12, 15, 20).

In the case of free draining soil or no pore fluids, equation 16 is the only one to solve. The terms containing $\pi\,(t)$ are dropped. Thus:

$$[A]\,\{u(t)\} = -\{M_1\}+\{M_2\}+\{P_1(t)\} \tag{19}$$

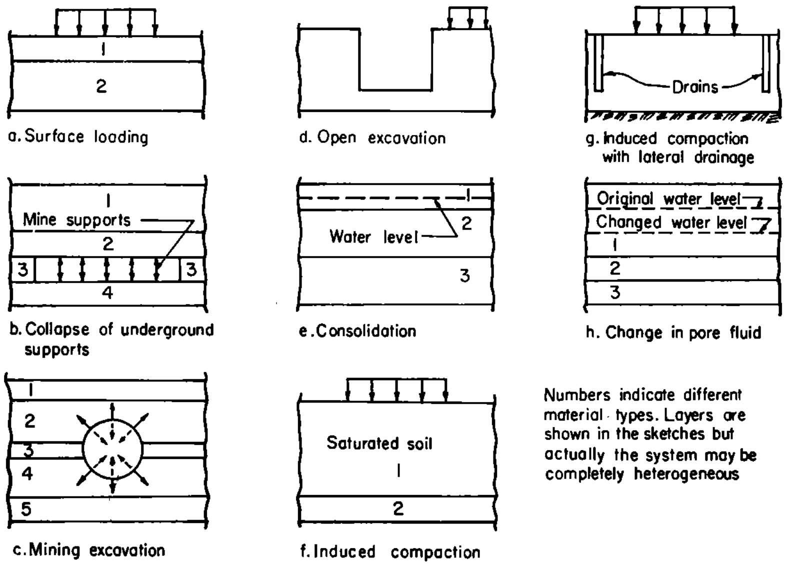


FIGURE 1. *Typical land subsidence*

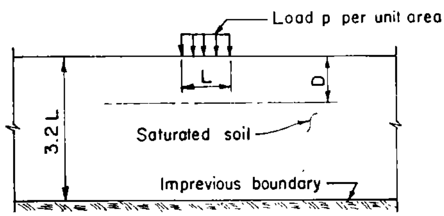


FIGURE 2. *System analyzed for settlement*

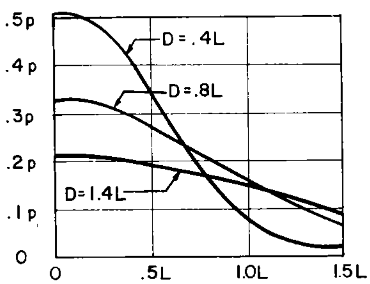


FIGURE 3. *Distribution of excess pore water pressures immediately after loading*

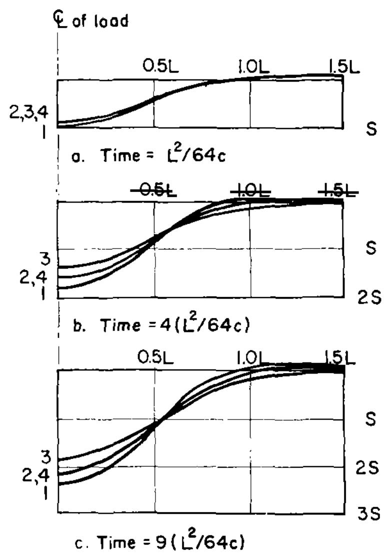
This equation represents the equilibrium equation under the effect of residual stresses, body forces and surface loads. In general, gravity is the only body force. If the vertical residual stresses are also due to gravity alone, these effects will cancel the body force effect. However, when mining is involved, the difference will represent the negative load due to removal of support. Similarly, mining will involve negative loads in the horizontal direction over surface of the cavity to represent relief of stress, with associated displacements.

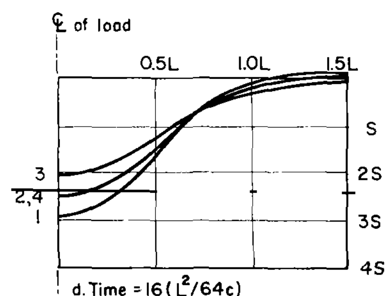
DISCUSSION

The finite element method furnishes a powerful tool for the analysis of land subsidence in a variety of situations and due to a wide range of causes. Some of these situations are illustrated in figure 1. The method permits complex geometry, arbitrary time varying boundary conditions, non-homogeniety as well as anisotropy to be considered. Nonlinear and time-dependent material behavior can be allowed for.

The procedure outlined will predict land subsidence and ground movement associated with mining, surface loading, fluid withdrawal, and changes in fluid pressures, in the presence of residual stresses. Any or all of these factors can be simultaneously allowed for. Overconsolidated or consolidating soil masses can be considered. The method can be extended to situations where the degree of saturation changes during application of the cause of subsidence. The analysis presented treats the saturated soil mass as a mixture and is free from the semi-empirical assumptions often made in such analyses to allow for the pore fluid pressures.

As an illustrative example, the finite element method was used to investigate the influence of elastic properties on the consolidation settlement of a thick clay layer under a rectangular surface lead. Figure 2 shows the system analyzed. It was found that over a range of values of the elastic modulus and Poisson's ratio, the distribution of pore





Material	E	μ	c	l/a
1	62500	0.0	.625	62500
2	56250	0.2	.625	62500
3	29170	0.4	.625	62500
4	62500	0.2	.694	69440

S = Consolidation settlement in time = $L^2/64c$ if load were $(-\infty, \infty) = apL/4\sqrt{\pi}$

FIGURE 4. *Influence of elastic properties on consolidation settlement*

pressures immediately after application of load was as shown in figure 3. A history of consolidation settlements is shown in figure 4. It is seen that the settlements vary with Poisson's ratio even for the same value of the coefficient of consolidation.

In the example the soil is assumed to be linearly elastic, isotropic and homogeneous. However, as explained earlier, the approach is quite general and complex material laws can be considered.

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OBSERVATION OF COMPACTION OF FORMATION IN THE LAND SUBSIDENCE OF NIIGATA CITY

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ABSTRACT

A method for measuring vertical distribution of compaction of a formation was developed by utilizing radioisotopes. Radiation sources made of cobalt 60 were shot into formations by a gun-perforator in a well at various depths and vertical distances of the sources were measured by a specially designed gamma-ray logger at intervals of about a year and accordingly, compaction during the period between two successive measurements was derived from difference of the distances. The two radioisotope observation wells were sunk at Niigata which has suffered remarkable subsidence since around 1956, and the observation at these wells was carried out since 1961. Although the accuracy of measurement was not always enough mainly owing to weak radioactivity of the sources, the observational results could-be compared with drainage of ground water and injection to aquifers.

RÉSUMÉ

Une méthode de mesure de la compaction verticale de formations a été développée en utilisant des radioisotopes. La source des radiations était constituée par du cobalt 60 injecté dans la formation par un injecteur dans un puits à différentes profondeurs et la distance verticale des sources radioactives était mesurée par un appareil à rayons gamma, imaginé spécialement, à des intervalles d'environ un an ; la compaction au cours de cet intervalle était déduite des distances trouvées. Les deux puits d'observation des radioisotopes furent forés dans la ville de Niigata où se sont produits des affaissements notables vers 1956 et les observations sur les puits commencèrent en 1961. Bien que la mesure ne soit pas toujours précise à cause surtout de la faiblesse de la radioactivité, les résultats des observations ont pu être discutés par comparaison avec le drainage de l'eau souterraine.

1. INTRODUCTION

Niigata City, located on the coast of the Sea of Japan, has suffered from severe land subsidence, the maximum rate reaching about 1.4 millimetre per day during 1958 and 1959. The Quaternary sediments on which the city lies contain six aquifers with natural gas dissolved in water, the total thickness of the sediments being several hundred to a thousand ~~metres. Since a large quantity of the water has been pumped out in order to separate the~~ natural gas, it was considered that the land subsidence was caused by shrinkage of the thick, soft beds owing to drainage of this ground water.

In order to measure the vertical distribution of compaction or the compaction of the sediments, the writer developed a method utilizing radioisotopes, using observations in two observation wells constructed for this purpose during the periods from 1961 to 1966 [1].

2. METHOD OF OBSERVATION

In this method for measuring the compaction of a formation, radioactive sources which radiate gamma-rays are shot into the stratum around the observation well by a gun-perforator through the iron well casing. These sources act as depth markers and the vertical displacement of the markers is determined by repeatedly measuring the depth of the radioactive sources using a radioactivity well logging system.

Because precise measurement of absolute depth is difficult, a dual radiation detector system was introduced, in order to measure the relative displacement. The dual detector system consists of two radiation detectors with a spacing of approximately 20 metres. Then, if the radioactive sources are shot into the stratum with a vertical interval of approximately 20 metres and the vertical variation of the gamma-ray emission of the sources is recorded simultaneously by the two detectors, the relative distance of two adjacent sources to the spacing of detector system can be determined by scanning a small vertical distance of ten centimetres to a few metres. Thus, the relative displacement of the two adjacent sources can be obtained by repeating the measurement. Then, the compaction is given as the ratio of relative displacement to the vertical interval of the two sources.

For the observations carried out at Niigata City, compaction of the iron casing of the observation wells was measured using radioactive sources attached to the casing at intervals of 20 metres. The compaction of the formation was derived by combining the measurements of the sources attached to the casing and those shot into stratum with the interval of 40 metres of more, i.e. in multiples of 20 metres.

The structure of the present observation well and typical records are presented in figure 1.

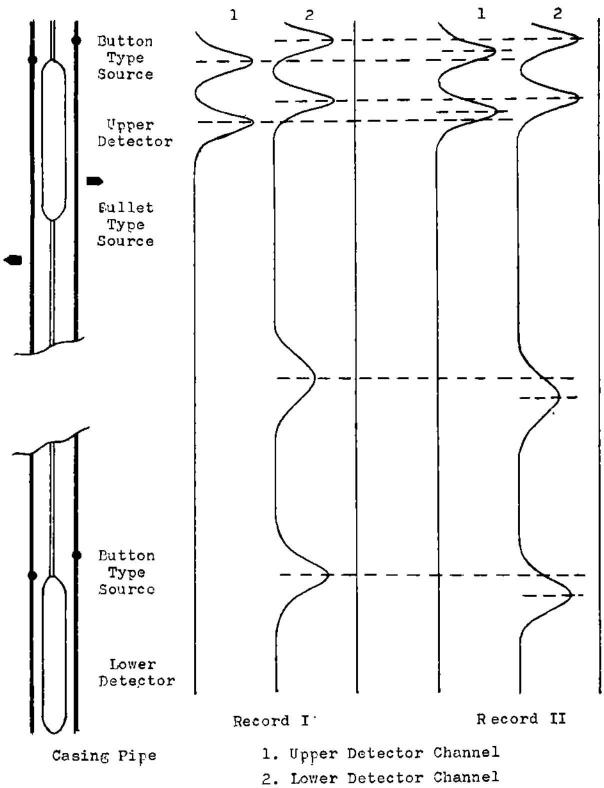

FIGURE 1. *Principle of observation*

The radioactive sources attached to the iron casing and those shot into stratum are called button type sources and bullet type sources respectively due to the shape of their steel capsules, which prevent radioactive contamination of the stratum and the ground water. The radioisotope used as the source is cobalt-60 in metallic form, which radiates high energy gamma-rays and is chemically stable. In the present wells, a pellet of 3 to 6 micro curie (Ci) is used for a button type source and a small wire of 1 milli curie for a bullet type source, but these strengths are rather low when the results of the observations are considered.

A scintilation counter with a NaI(Tl) phosphor 6 millimetres thick and 1/2 inch in diameter is used as a detector. A shield made of tungsten alloy with a slit located just around the phosphor covers the detector, so that it gives a sharp response to gamma-ray source.

Because the slower scanning speed of the detector yields a more accurate result in the measurement of radioactive intensity, a winch was specially designed for this particular logging system in order to prevent slip of the cable and to make the winding speed continuously variable from 1 centimetre per minute to 1 metre per minute. The chart driving device of the two channel radioactive intensity recorder was directly coupled to the cable so that the scale of the chart was exactly the same.

Observations from 20 minutes to 2 hours to a complete record of a measuring station with an interval of 20 metres in the observation well, and thus from 5 to 7 days are needed for the overall measurement of a well.

3. ERRORS OF MEASUREMENT

The position of a source is represented by the peak intensity in the record. If the centre of the peak is determined as the mid point of the width at 3/4 of the maximum intensity and the upper part of curve showing vertical change of radioactive intensity is approximated by a normal distribution curve, the standard deviation E of the centre due to statistical fluctuations of the radiation is

$$E = 0.17d/\sqrt{n_0 t}, \tag{1}$$

where n_0, d and t are respectively, the maximum intensity of the peak, the width at 3/4 of the maximum intensity and the time constant of a ratemeter used for measurement of gamma-ray intensity [2].

The standard deviation is decreasing with an increasing time constant, but the time constant should be small to avoid significant deformation of the recorded peak. Since the maximum intensity n_0 is usually about 10^3 to 10^2 counts per minute in the existing wells, a time constant of 5 to 10 seconds and a scanning speed of 1 to 2 centimetres per minute are employed. For example, a sharp peak from the button type source is characterized by $n_0 = 1{,}900$ counts per minute and $d = 0.8$ centimetres; then $E = 0.6$ millimetres when $t = 5$ seconds and $v = 2$ centimetres per minute. A broad peak from a bullet type source is characterized by $n_0 = 420$ counts per minute and $d = 8.8$ centimetres; then $E = 7.5$ millimetres when $t = 5$ seconds and $v = 2$ centimetres per minute. If it is desirable that $E = 2$ millimetres for the same t and v, a 28 milli curie source giving $n_0 = 1.2 \times 10^4$ counts per minute should be used.

When the positional error of the two sources are E_1 and E_2, the error E_{12} of distance between the two sources is given by

$$E_{12} = \sqrt{E_1^2 + E_2^2}, \tag{2}$$

assuming that the normal distribution law is applicable. The error of relative displacement *Ed* of two adjacent button type sources, *Ed* is expressed as

$$E_d = \sqrt{E_{12}^2 + E_{12}'^2} \tag{3}$$

where $E_{12}'^{2}$ is the error of the second measurement of distance while the error of relative displacement of two adjacent bullet type sources can be derived in a similar way.

The error of compaction is obtained from the error of displacement and for the present observations, the error of compaction of the iron casing is of the order of 10^{-4} and the error of compaction of formation is about 3×10^{-4} on an average, ranging from 10^{-4} to 10^{-3}.

Another error arises from the change in inclination of the casing of the observation well, which amounts to several tens of minutes per annum at a maximum. Such an inclination of the casing is probably from buckling due to the shrinkage of the formation while the relative position of the bullet type source to casing is unchanged. If the inclination of the casing increase by $d\theta$ from the original inclination θ_0 and the original inclination of the line connecting the two sources is ϕ, as shown in figure 2, the distance L between the two sources will decrease by dx by the measurement through the inclined casing and if θ_0, $d\theta$ and ϕ are small,

$$\begin{aligned} dx &= L \sec(\theta_0 - \phi) \cos(\phi + d\theta) - \cos\phi \\ &= L \cos(d\theta - 1). \end{aligned} \tag{4}$$

The effect of inclination can, therefore, be corrected [3].

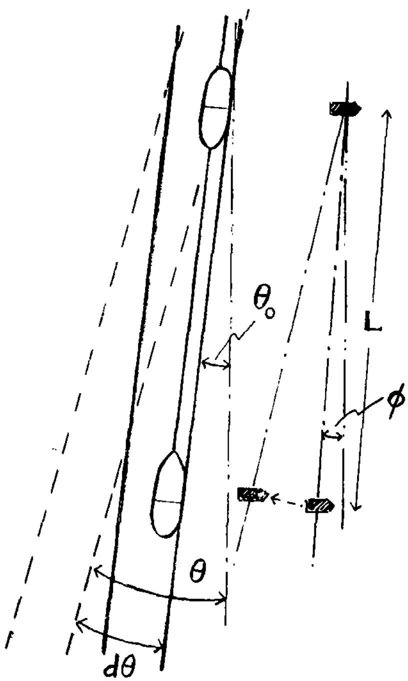

FIGURE 2. *Inclination of the casing pipe*

4. RESULTS OF OBSERVATIONS

4.1 YAMANOSHITA (RINKO) OBSERVATION WELL

This observation well was completed in March 1960 at Momoyama-cho, Niigata City, where the land subsidence was the most remarkable in the city during from 1956 to about

1962. This well, whose depth is 650 metres, was sunk to the base of the six aquifers in the Quaternary sediments. These aquifers are named G_1, G_2, G_3, G_4, G_4', and G_5 successively from the surface. Measurements in the observation well had been made every year. The Niigata Earthquake occurred in June 1964 and caused great damages. The upper part of the casing became inclined, but breakage of the well was not noticed [4]. However, the measurements became difficult to make especially in the upper part of the well of large diameter, because signals from the sources were weakened due to the attenuation of the radioactivity.

A four stage casing was adopted for this observation well, each stage being of smaller circumference than the preceding one. The base, or toe, of each stage was grouted to the surrounding stratum and overlapped the head of the stage below it, as shown in figure 3. It was intended that the increase in the overlapped length of each stage should represent the shrinkage between the strata to which the casing pipe was grouted. This system was, however, unsuccessful since the casing pipe contracted with the shrinkage of the formation.

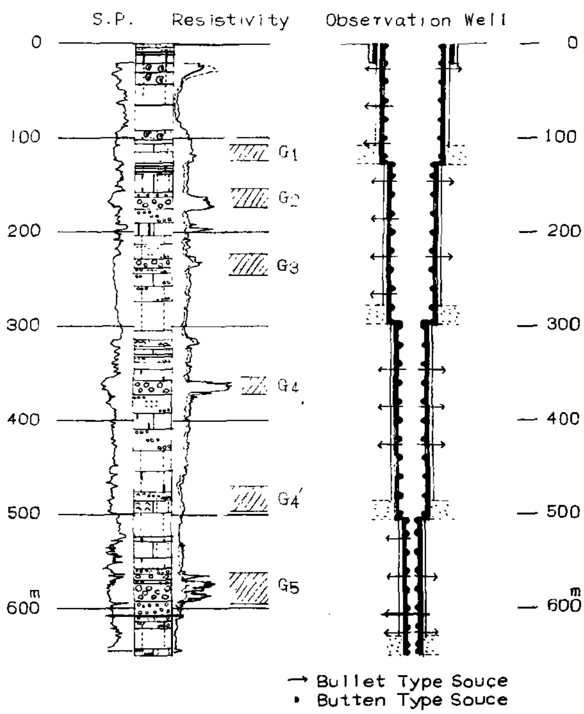

FIGURE 3(a). *Structure and electrical logs of Yamanoshita (Rinkō) observation wells*

The total compactions of the formation and the iron casing since July 1961 are shown in figure 4 and 5 respectively.

Shrinkage of the formation has in general gradually decreased, during 5 years from 1961 to 1966. Compaction of the formation from the ground surface to the G_1 and G_2 aquifers increased in 1962 and then decreased. Compaction of the formation between the G_3 and G_4 aquifers decreased in 1963 and 1964 and increased thereafter. Compaction of the formations between the G_4 and G_4' aquifers has decreased on average, while the formations stopped compactiv of even expanded in 1962. This was when the experimental injection of water was undertaken mainly in the G_4' and G_5 aquifers in the Rinko area for about a year. Compaction of the formation between the G_4' and G_5 aquifers was small even in the early stages of the observations and in 1962 compaction was interrupted.

4.2 Uchino Observation Well

This observation well was finished in March 1961 at Shin'ei-cho, Niigata City, where subsidence only became significant in 1962. This well, whose depth is 950 metres, was sunk to the base of the six aquifers contained in the Quaternary sediments. Measurements in the observation wells have been carried out every year and no influence of the Niigata Earthquake was observed. In 1966, however, the well was collapsed and measurements under the G_5 aquifer became impossible.

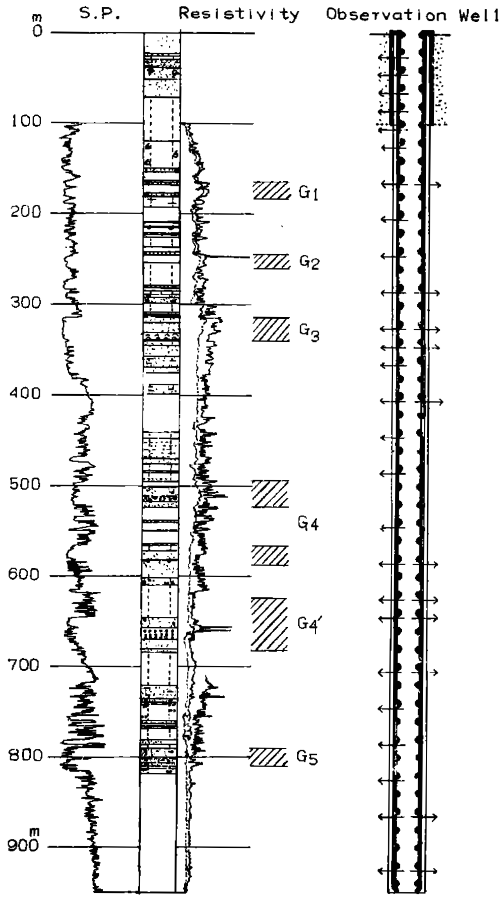

Figure 3(b). *Structure and electrical logs of Uchino observation wells*

The structure of this observation well is shown in figure 3 and the total compaction of the formation and the casing are shown in figure 6 and 7 respectively.

In this observation well, the compaction of the casing was similar to that for the formation. However, the variation of the compaction of the formation against depth was irregular and even some expansion was noticed.

In general shrinkage of the formation has not progressed so remarkably as that around the Yamanoshita Observation Well, during the five years from 1961 to 1966.

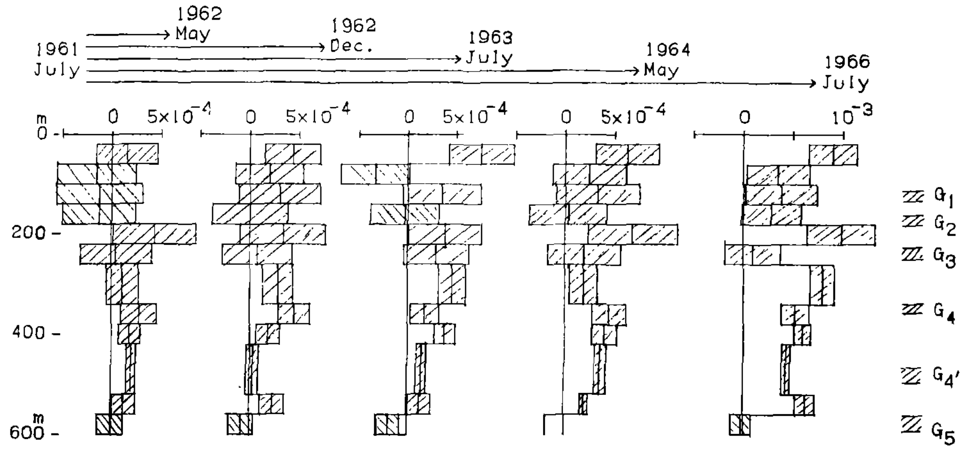

FIGURE 4(a). *Total compaction of the formation in the Yamanoshita (Rinkō) observation well*

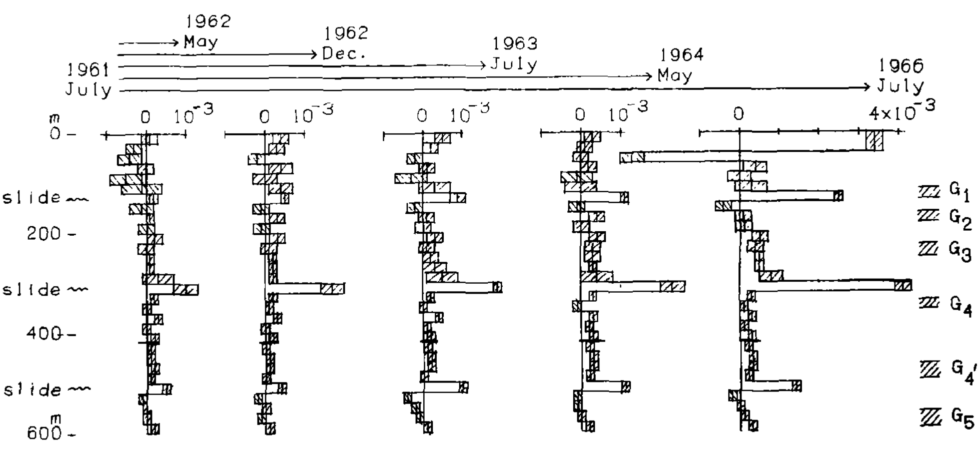

FIGURE 4(b). *Total compaction of the casing pipe in the Yamanoshita (Rinkō) observation well*

Compaction of the formation from the ground to the G_1 aquifer increased from 1962 to about 1964 and then decreased. Compaction of the formation between the G_1 and G_2 aquifers was seldom observed. Compaction of the formation between the G_2 and G_3 aquifers was irregular. On average, the compaction was small from 1962 to 1964 and increased thereafter. Compaction of the formation between the G_3 and G_4 aquifers was similar to that in the overlying formations. Compaction of the formation between the G_4 and G_4' aquifers has gradually decreased, in general, at through the compaction

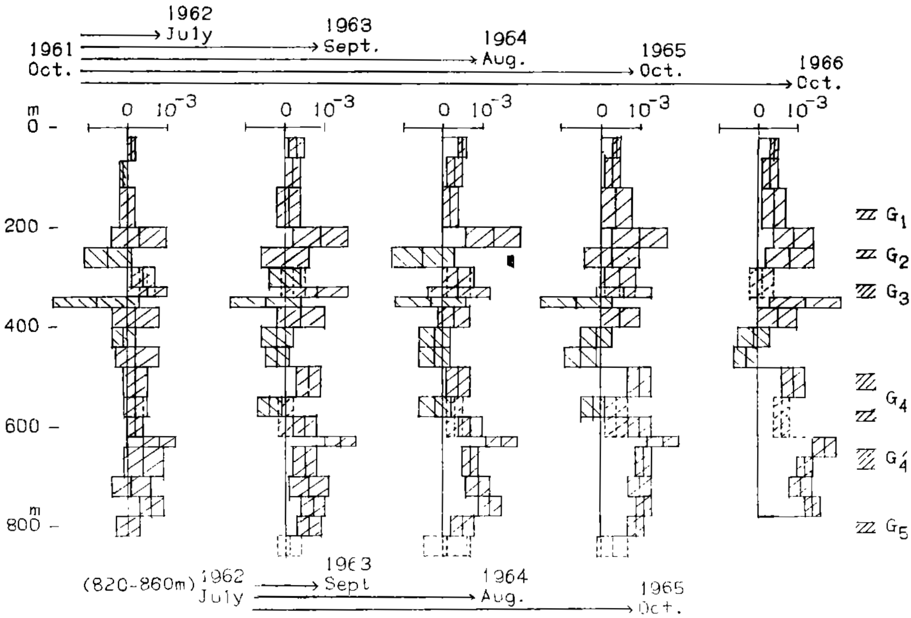

FIGURE 5(a). *Total compaction of the formation in the Uchino observation well*

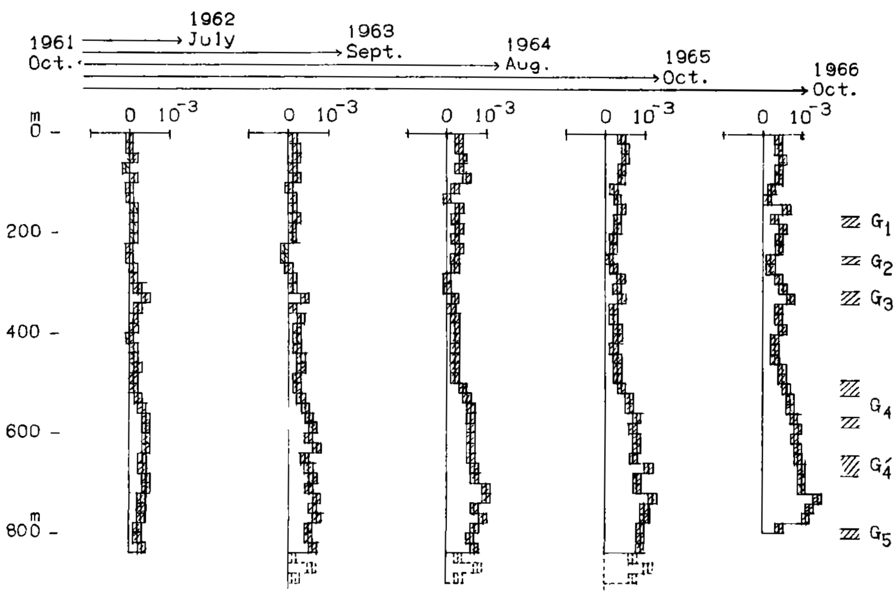

FIGURE 5(b). *Total compaction of the casing pipe in the Uchino observation well* increased somewhat from 1963 to 1964. Compaction of the formation between the G_4' and G_5 aquifers gradually decreased.

4.3 Summary of Results

Pumping of the ground water for producing the water-soluble-type of natural gas was suspended in November 1961, in a semi-circular area of about 5 kilometres' radius around Yamanoshita, several months after the first observations in the radioisotope observation well. In the surrounding area of width about 5 kilometres, discharge from the aquifers above G_6 has been compensated by recharge of the same amount to each aquifer. Therefore, the apparent drainage of the ground water has only been continued outside these areas.

The drainage from each aquifer was reduced to about half to one third against the quantity just before regulation in November 1961. Approximate drainage rates in the Niigata District have been 700 cubic metres per day from the G_1 aquifer, 10,000 cubic metres per day from the G_2 aquifer, zero from the G_3 aquifer, 27,000 cubic metres per day from the G_4 aquifer, 15,000 cubic metres per day from the G_4' aquifer, 45,000 cubic metres per day from the G_5 aquifer and 75,000 cubic metres per day from the G_6 and other deeper aquifers. These rates have been kept nearly constant except for a few months after the Niigata Earthquake in June 1964.

In the observation wells, compaction of the deeper formations was generally decreasing on average and this may correspond to the decreasing drainage from the aquifers due to the regulation of the production of natural gas that came into operation in 1961. During the period from 1962 to 1964, compaction of the shallower formations has been increasing An increase in the discharge of the ground water was reported for these years, but exact statistics were not available because drainage for the production of the gas was carried on by the individuals and small factories without mining rights.

The decrease in the compaction of the deeper formations was observed in the Yamanoshita (Rinko) Observation Well during the period from 1962 to 1963. This decrease may be correlated with the injection of water in Rinko area, with a mean rate of 1,500 cubic metres per day to the G_4' aquifer and 4,000 cubic metres per day to the G_5 aquifer during 1962.

As for the recent increase in compaction of the formation between the G_3 and G_4 aquifers, a corresponding change of drainage has not been discovered.

The bullet type sources were so coarsely distributed and the accuracy of the observations were not so good that difference of compaction between aquifer and aquiclude could be clearly observed. However, it seems that compaction of the aquifer was generally smaller than that of the aquiclude.

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THE MECHANICS OF COMPACTION AND REBOUND, WILMINGTON OIL FIELD, LONG BEACH, CALIFORNIA, U.S.A.

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ABSTRACT

The ground surface above the center of oil production subsided about 9 meters over 27 years. Most observers related subsidence to compaction in the oil reservoirs caused by fluid production and pore pressure reduction.

Axial loading tests on the reservoir sands and shales (siltstones) show the sands to be either as or more compactable than the shales. Reservoir calculations, oil well casing measurements, and laboratory tests, indicated a reduction as great as three to four percent in the bulk sand volumes (about 10-11% of the pore volume). It is concluded that most of the compaction prior to 1952 occurred in the reservoir sands. Compaction percentage calculations place 32.4 percent of the cumulative compaction in the shales and 67.6 percent in the sands.

Water injection programs arrested subsidence and caused elastic rebound, which in conjunction with pump pressures has uplifted the surface as much as 33.5 cm over 8 years. Rebound is believed to be confined principally to the sand intervals. The coefficient of rebound is estimated to be .005/unit of vertical sand thickness; of which only about 50 percent is seen at the surface due to unrelieved stresses in the overburden.

RÉSUMÉ

Le sol au-dessus du centre de la zone productrice s'est affaissé d'environ 9 mètres en 27 ans. La plupart des spécialistes ont attribué cette subsidence à la compaction du réservoir due à la réduction de pression dans les pores résultant de l'enlèvement du fluide.

Des essais de compression triaxiale sur les sables et les argiles de la formation productrice ont montré que les sables étaient autant, ou plus, compressibles que les argiles. Les calculs de réservoirs, les mesures sur les tubages, et les essais de laboratoire, ont indiqué une réduction de 3 à 4 % du volume total des sables (environ 10-11 % du volume des pores). Il en a été conclu que la majeure partie de la compaction antérieure à 1952 s'est produite dans les sables du réservoir. D'après les calculs effectués, 32,4 % de la compaction cumulée sont attribuables aux argiles, et 67,6 % aux sables.

Les programmes d'injection d'eau ont arrêté la subsidence et provoqué une expansion élastique qui, en conjonction avec la remontée de la pression, a rehaussé la surface de 33,5 cm en huit ans. On pense que l'expansion est limitée principalement aux intervalles sableux. Le coefficient d'expansion est estimé à 5 ‰ (5 pour mille) de l'épaisseur verticale des sables ; mais la moitié seulement de cet effet est visible en surface, en raison de la pression maintenue par la couverture.

INTRODUCTION

Wilmington Oil Field is located in the southern part of California, U.S.A. in the physiographic area known as the Los Angeles sedimentary basin (fig. 1). Approximately 35 fields produce from sandstone reservoirs of Upper Miocene and Pliocene ages. Surface subsidence has occurred in minor amounts (1 meter or less) over several of these fields. The extreme case and most publicized is that of Wilmington Oil Field located in and near the City of Long Beach. World-wide attention has been attracted to his area because of it's location in the center of a highly industrialized port and naval shipyard (fig. 2).

Cumulative subsidence had reached 9 meters in 1968, at the center of an elliptical shaped surface depression, before compaction was controlled in the oil reservoirs by water injection. Differential horizontal movements as great as 3 meters accompanied

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the vertical subsidence, causing extensive damage to wharves. pipe lines, buildings, streets and bridges, necessitating costly repairs and surface filling (fig. 3 and 4).

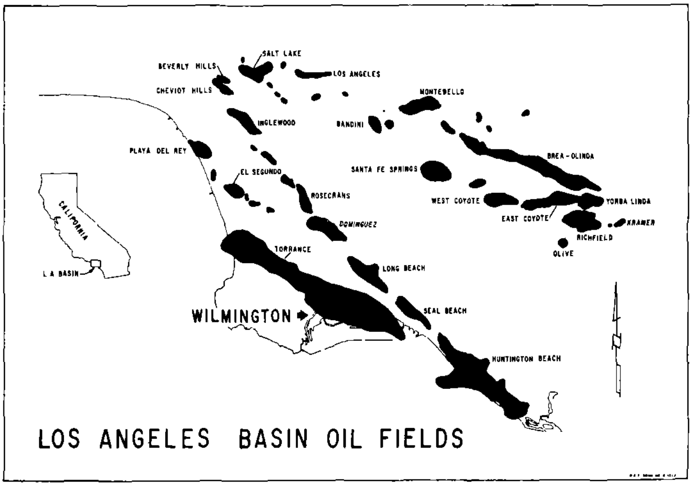


FIGURE 1.

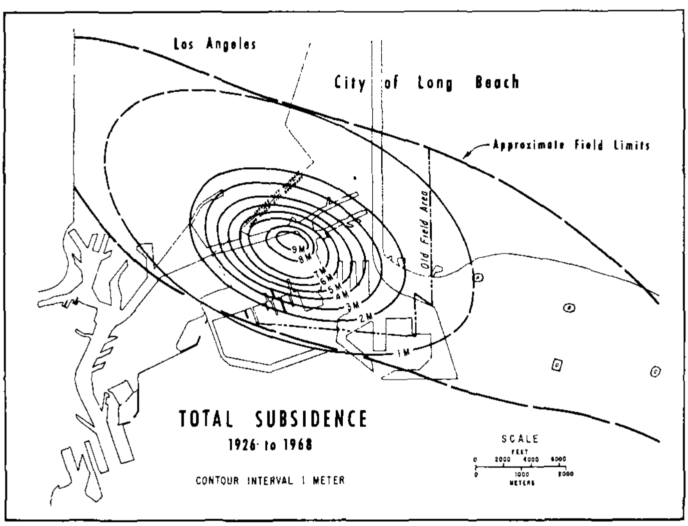


FIGURE 2.

CAUSATION AND LOCATION OF COMPACTING STRATA

During the period from about 1948 to 1958, when the surface was sinking at rates from 30 to 65 cm/per year, many investigative reports were written as to causation and many predictions made as to the ultimate subsidence.

The first two comprehensive studies were made by Gilluly, Johnson and Grant [6] and by Harris [8] in 1945. Both of these reports surveyed possible causes, including

FIGURE 3. *Bucked Railroad Track*

ground water withdrawals and tectonic movements, related subsidence to compaction in the oil zones, theorized as to the mechanics of compaction, and predicted a maximum subsidence of about 3 meters. Gilluly *et al.* concluded that the compaction was occuring within the fluid producing sands, while Harris concluded that it was the shale members within the oil zones which were compacting. This controversy as to the mechanism of compaction has not been resolved and still has proponents for both viewpoints.

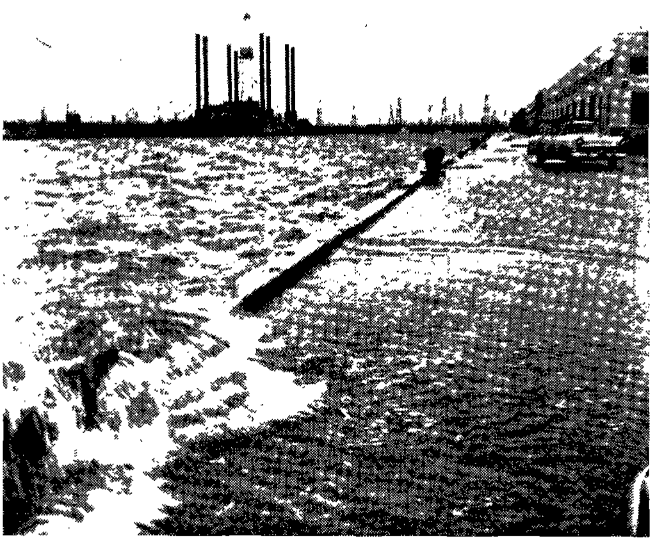


FIGURE 4. *Subsided Pier*

Location of Compacting Strata

During 1948-1949 while there still existed some doubt regarding the depth of the compacting strata as well as the mechanics of compaction, a method was developed for locating and estimating compaction by measuring the deformation in oil well casing [11]. This procedure, called collar counting, is as follows:

1. Locate the recessed area between coupled casing joints in oil wells with a magnetic locator connected to a vernier cable odometer at the surface;
2. Determine casing joint lenghts by the difference between collar locations;
3. Compare these joint lenghts with accurate surface measurements or a prior collar survey.

Casing joints of about 14 meters in length were found to be vertically shortened as much as 36 cm (fig. 5). Based on the casing compression data from collar counting, most observers since 1949 have conceded that compaction was localized in the four uppermost oil producing zones.

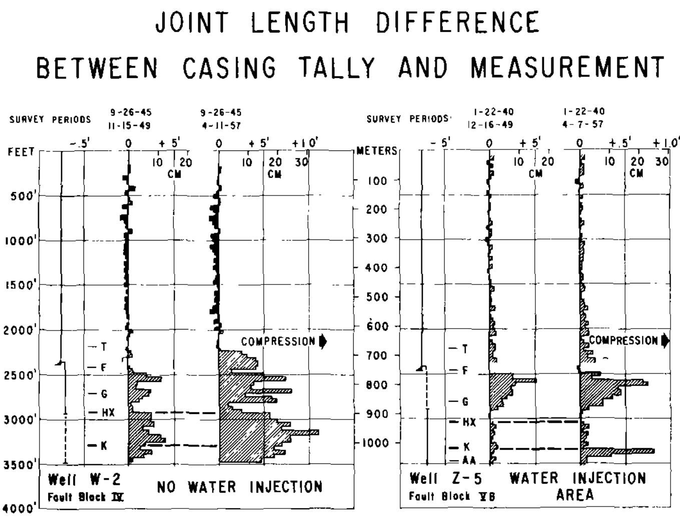


Figure 5.

Prerequisites for Subsidence

The consensus of most authorities from many disciplines is that the extraction of reservoir fluids and the consequent lowering of pressures caused reservoir compaction and surface subsidence regardless of whether the compacting strata were primarily sand or shale.

Accepting the above consensus as factual, three statements of condition can be made which are axiomatic for Wilmington type subsidence. These are:

1. Reservoir fluid pressures must be lowered;
2. The reservoir rocks must be compactable (uncemented) and unable to effectively resist deformation upon a transfer of load from the fluid phase to the grain to grain contacts;
3. The overburden must lack internal self support and be of such a nature as to easily downward and supply a constant load to the underlying formations.

PREDICTIONS

Along with the reports on the causes of subsidence, there were many predictions made as to the maximum amount which might be expected. These estimates ranged from about 3 meters to over 23 meters, dependent upon the date when the prediction was made. In general it may be said that accurate extrapolations based on known conditions and cause and effect relationships were not made until 65 or 75 percent of the probable total subsidence had occurred. Two accurate extrapolations are shown on figure 6, both of which were heavily weighted to an historical base [3 and 13]. The Law extrapolation assumed no injection influence while the Branson extrapolation did.

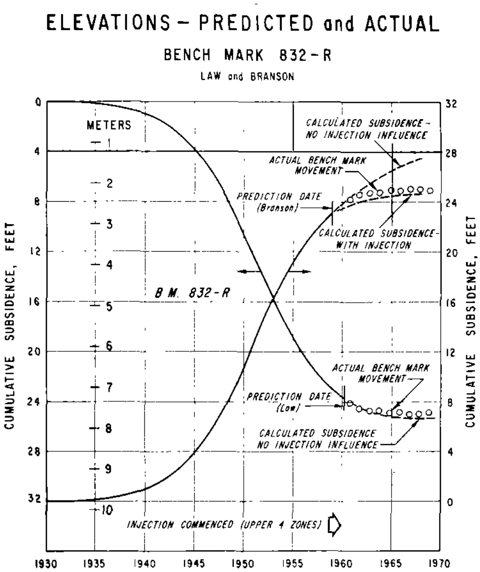


FIGURE 6.

MECHANICS OF COMPACTION

Many of the basic mechanics of compaction are related to formation stability which involves geological structure, sand sizing, composition, cementation and shale induration and competency. Following is a brief summary of these characteristics in the Wilmington field:

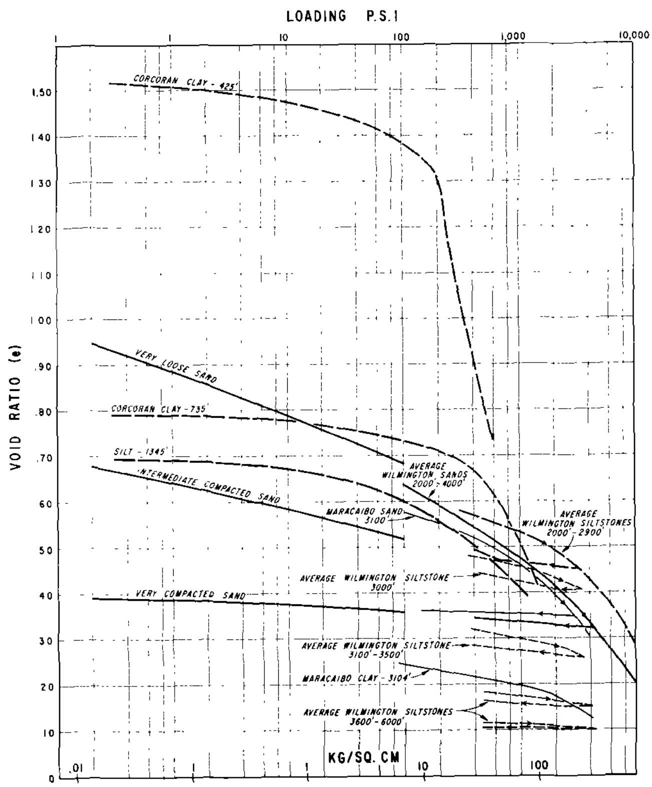
The structure is a tension faulted anticline about 17 kilometers long with a span of about 4.7 kilometers. The beds are nearly flat or dip gently for about 1.6 kilometers across the crest. The sands are composed of about 35 to 70 percent quartz, 12 to 40 percent feldspars and 8 to 12 percent silt and clay minerals. Porosities range from about 25 percent at 1500 meters to 35-40 percent at 700 meters. Above 1200 meters the sands are uncemented and loose and grade in grain size from fine to coarse. Below 1200 meters there is an increase in cementation and the degree of induration.

Wilmington shales are usually termed siltstones by soils engineers. Below 1200 meters they are indurated and appear to have good structural strength. Toward the surface the shales become progressively softer and grade to clay. The siltstones and sand layers are intercalated and often do not have a sharply discernible boundary.

The vertical section is divided into seven oil producing zones or intervals for convenience of production based on reservoir characteristics and oil-water interfaces [15].

Laboratory Tests

Axial loading tests were made on sand and siltstone cores by various engineering firms including both petroleum and soils specialists. The averaged results of these tests are compared with data from textbook examples [9], Lake Maracaibo [18] and deep water wells in California [10] on figure 7. Wilmington sands above 4000 feet (about 1219 meters) fall in a trend with a silt from a California water test well at 1345 feet (410 meters), a Lake Maracaibo sample from 3100 feet (945 meters) and a text book example of a very loose sand. The siltstones show an increasing preconsolidation with depth.



COMPARISON OF

VOID RATIO vs. APPLIED PRESSURE

SANDS, SHALES & SILTS - VARIOUS DEPTHS & AREAS

Figure 7.

The deeper samples agree generally with the Lake Maracaibo "clay sample". The preconsolidation break [9] in the void ratio versus pressure curves is seen fairly well in the siltstones but not in the sands due to their uncemented nature. All of the tests show that the unconsolidated sands compact easily by grain rearrangement, possible crushing of grain points and plastic flow of softer materials when subjected to loading. The recovery cycle of these tests (unloading) shows a small portion of the compaction to be elastically recoverable. The siltstones in the shallow zones compacted readily when subjected to pressures higher than preconsolidation. Siltstones from below 3100 feet

(942 meters) compacted a les er amount of which a large percentage was elastically recoverable, indicating a strong internal structure. Siltstone permeabilities are 1/10,000 or less than that of the sands, which, in conjunction with a stronger internal structure, makes a rapid consolidation unlikely.

Field Measurements

Collar logging results from Well W-2 are summed by years and by oil zone on figure 8. (For zone depths refer to fig. 9.) These data show a rapid compaction in the Upper Terminal zone to about 1952 with a lesser but similar pattern in the Ranger zone. It is here assumed that a rapid compaction rate is indicative of primarily sand compaction.

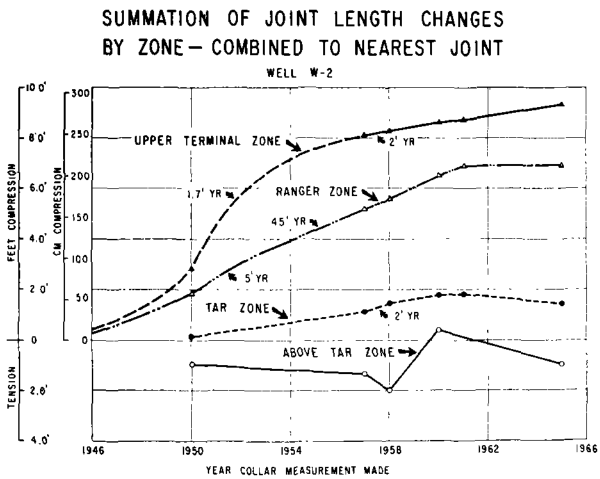


FIGURE 8.

This can be verified by observing the data on figure 9. Casing compression data from two nearby wells are compared with their electric logs wich enable the primarily shale and sand intervals to be observed. From 1945 to 1965 the data from Well W-2 show that most of the compaction occurred in the sand intervals. Well W-279 data from 1959 to 1965 show that most of the compaction occurred in the shale intervals. (Note scale difference).

Calculations

Theoretical sand compaction can be calculated from the average sand curve on figure 7, using the following expression: $\Delta H = \Delta e/1+e \times H$ where: ΔH = change in thickness, Δe = change in void ratio between varying loads, e = original void ratio, H = original interval thickness. Applying this formula to a total compactable sand thickness of about 242 meters in the upper four zones and using an average pressure reduction of 93 kg. sq. cm (at a depth of 913 meters), than ΔH = about 10.2 meters. Using this same equation and appropriate pressure and depth ranges for the shales, a theoretical shale compaction of 6 meters is calculated, giving a total of 16.2 meters of theoretical compaction in the upper four zones. Subsidence rate decline curves indicated a maximum subsidence might be about 10.7 meters. The zone compaction data shown in figure 8 were proportionally adjusted to reflect the actual surface subsidence of about

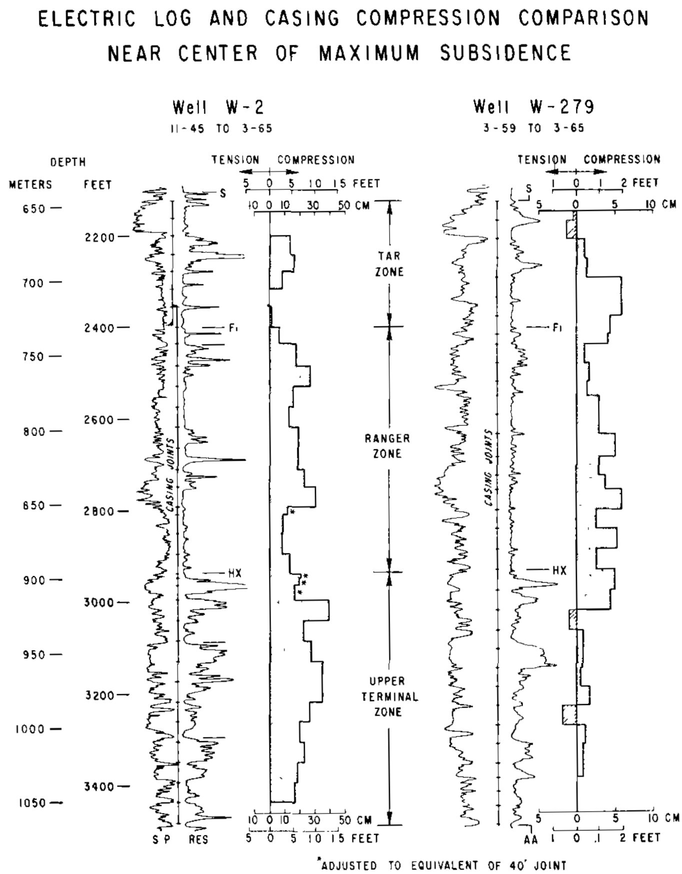


FIGURE 9.

9 meters and are listed in the table below. By using the changes in the rate of compaction in figure 8 as a guide to the relative contribution of the sands and shale to compaction, the following percentages were calculated:

	Shale	+	*Sand*	(*Sand*%)		*Zone Compaction*
Tar	27 cm	+	63 cm	(70.0%)	=	90 cm
Ranger	162 cm	+	153 cm	(48.6)%	=	315 cm
U.T.	72 cm	+	333 cm	(82.2%)	=	405 cm
L.T.	30 cm	+	60 cm	(66.7%)	=	90 cm
	291 cm	+	609 cm		=	900 cm
						(Surface subsidence)

Compaction factors (unit of compaction/unit of sand or shale)

Tar	.007	+	.015	=	.022
Ranger	.013	+	.039	=	.052
U.T.	.014	+	.029	=	.043
L.T.	.014	+	.013	=	.027

Total compaction percentage calculations place 32.4% of the compaction in the shales (siltstones) and 67.7% in the sands. The above table of compaction factors show the bulk volume of the sands to be reduced in a range of about 1 to 4 percent, all of which is at the expense of reservoir porosity.

MECHANICS OF REBOUND

The concept that a partial restoration of depleted reservoir pressures will lift a thousand meters of overburden can be difficult to grasp. Supporting data connecting surface rebound with repressuring is found in both laboratory loading tests and practical oil field measurements.

Laboratory Tests

All samples tested (fig. 7) exhibited a small rebound (expansion or swelling) during the unloading cycle. The swelling portion of these curves is normal for earth materials tested in this manner and is generally considered to be due to the elastic properties of the samples.

Field Measurements

Surface response to water injection was measured in 1961, shortly after large scale injection was commenced into the oil reservoirs (fig. 10). The bench marks shown are located in various areas and show differing times of injection response. Elevation is

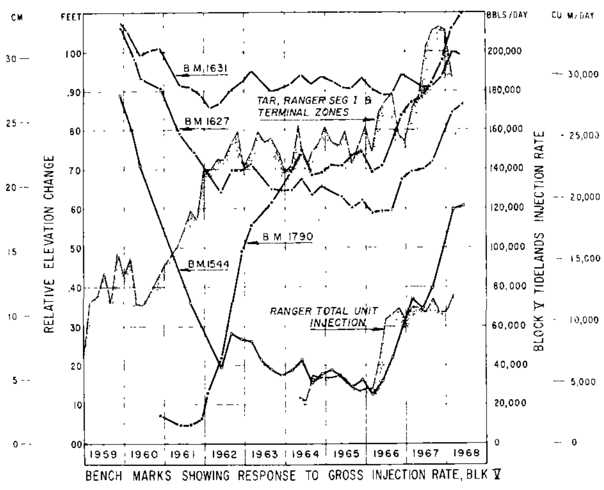


Figure 10.

gained and lost by the bench marks in step with the water injection rate. Figure 11 illutrates movements of a bench mark relative to changes in reservoir pressure. Rebound is directly related to changes in reservoir pressure and only indirectly to water injection rate; however, pressure data usually are not as readily available. Total rebound over the entire area is shown in figure 12. The maximum rebound recorded through 1968 was 33.5 cm.

The collar count technique was also used to measure zone expansion (fig. 13) in areas of high water injection rate. Some casing joints indicate an elongation more than that theoretically possible without parting in the threaded ends.

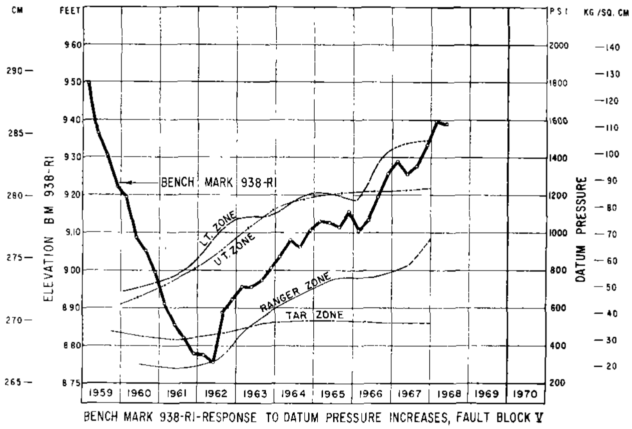


FIGURE 11.

Because of the rapid surface response to injection rate, rebound is qualitatively judged to be confined to the sand intervals. Supporting this premise is the obvious difficulty which would be experienced moving water into the low permeability shales with their relatively high pore pressures versus the ease with which water enter the sands because of their higher permeabilities and lower pore pressures. Some rebound might be generated in the shales given a large number of years but the amount is considered to be insignificant.

The area adjacent to an injection well bore is at a considerably higher pressure than the average pressure in the reservoir. This pressure, which is much greater than hydrostatic, hydraulically lifts and supports the surface in excess of normal elastic rebound.

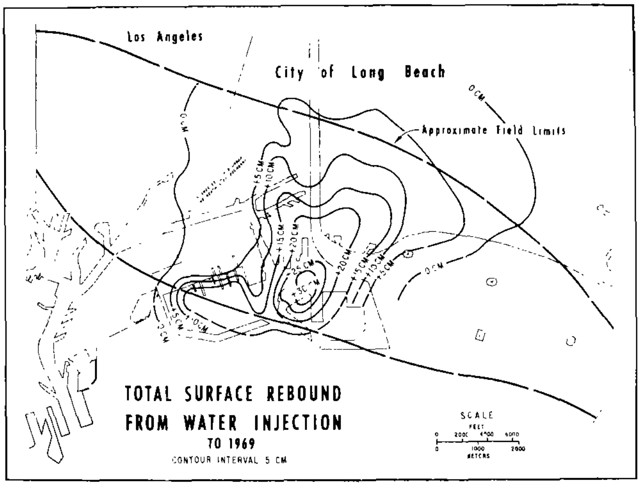


FIGURE 12.

About 6 cm of rebound are attributed to this mechanism, all of which is considered to be highly unstable and subject to rapid loss.

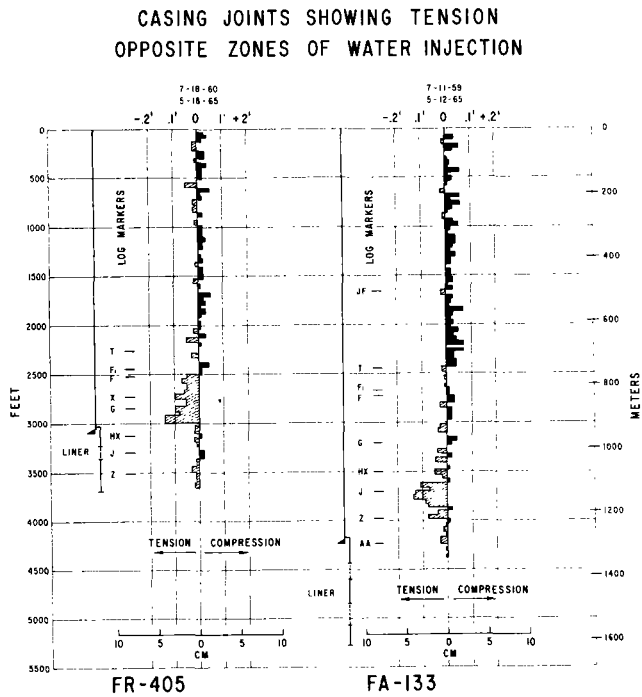


FIGURE 13.

ESTIMATES OF ULTIMATE REBOUND

Estimates of ultimate rebound are highly conjectural because of many complicating factors such as the multiplicity of reservoirs, varying production practices and overburden restraint. A rebound factor (unit of expansion per unit of sand being repressured) of .005 was calculated from the unloading cycle of the date on figure 7 using pressure ranges representative of reservoir conditions.

An expansion factor of .004 to .006 per sand unit also was calculated using the expansion of the zones being repressured as indicated on figure 13. At the time these well measurements were made only 40 to 50 percent of the apparent elongation had appeared as surface rebound.

At bench marks 1790 (fig. 10) about 251 vertical meters of reservoir sand are being repressured. Using the expansion factor of .005 and a 50 percent surface response as indicated by collar counting, a surface rebound of 62.7 cm is calculated. This bench mark has currently risen 33.5 cm and is still rising.

At some point in the rebound cycle, surface response may be closer to unity than to 50 percent. This point is believed to be related to the unknown amount of unrelieved stress in the overburden.

Figure 14 is a composite of the subsidence at the center of the subsided area and the maximum rebound which has occurred in the other areas where the zones are being repressured. Ultimate rebound is extrapolated to be about 76 cm, dependent upon actual repressuring and the percentage of zone rebound which appears at the surface. The rebounded elevations are shown as being unstable and subject to rapid loss if the pressures are reduced.

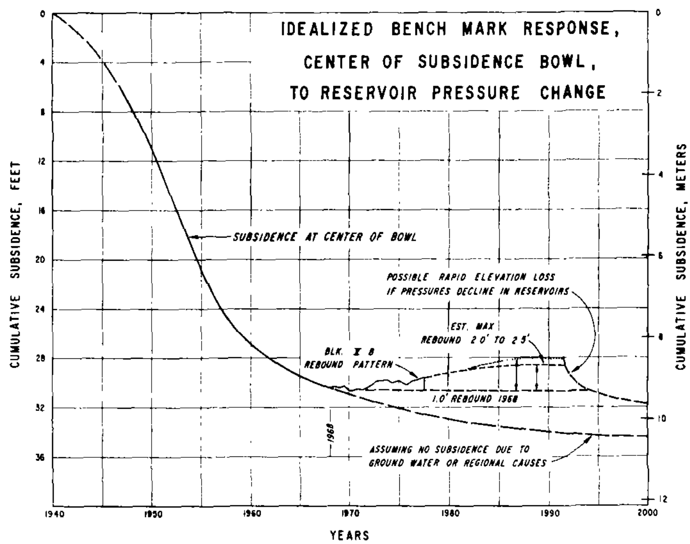


FIGURE 14.

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DISCUSSION

Intervention of Mr. Joseph F. POLAND (USA):

Question:

I believe you have concluded that the compaction has occurred chiefly in the sands. Do you have any evidence as to whether the compaction is primarily by grain rearrangement or by crushing?

Answer of Mr. ALLEN:

We have only our judgement for this, we really do not know. Based on the work, some of which we saw this morning, it appears that you might have some laboratory data now, which we do not have for proposing the crushing of sand grains. We always felt that this was quite obvious because in the arkosic sands which we have in Wilmington, meaning that they are composed at least in part of feldspar, that they are soft and in general they are irregularly shaped. So we feel that when the reservoir pressures are depleted there are irregular transfers of loading, some of which on grain to grain points is quite higher than you might think from the average pressure withdrawn in the reservoir, and that after the primary rearrangement of sand grains and plastic flow takes place, we do feel that we do have shattering, but we have no proof of it.

Intervention of Dr. James E. ROBERTS (USA):

Question:

I have heard that during subsidence the upper non producing formation expands about 0.3 ft for each foot of subsidence. Is it true? If it is, was this phenomena considered in your evaluations and computations relating formation compaction with observed surface subsidence?

Answer of Mr. ALLEN:

That may be correct. I am not sure. The collar count data indicated that there was some expansions in the upper zone but when you get into areas of very small rebound, of tenths of a foot, such as you are speaking of, you are not sure of your data. So, assumming that this happens, the amount was small and we did not take it into account in subsidence. We are speaking only of measured subsidence of the surface related to the oil zones. So it is possible that the compaction of oil zones is greater that what we have actually shown here. These would be minimum figures.

Intervention of Prof. Kenneth E. LEE (USA):

Question:

You mentioned that if the sand has any cementation in it, it will not compact.This does not seem to follow logically. We know that all materials compress under changing load. And I wondered if you had any data from other fields where the material is essentially a cemented material that show definitely that this does not happen?

Answer of Mr. ALLEN:

I think we are speaking of the difference between qualitative and quantitative here, and what will compact and what any compaction means. There is no question that any material with pores can be compacted when enough load is put on it. There are many studies which have been made on cemented sandstones. I have a bibliography of them, and I believe I used some of them as references in this paper, in which cemented sandstones had been stressed and the deformations measured. Now, when the sandstones are cemented, deformations are found, usually within the elastic range. You find that within the range of pressures, that might be suffered in producing oil field reservoirs, the strains which are imposed normally will not crush a cemented sandstone. They will deform slightly elastically. There is a very slight reduction of pore volume, the amount is perhaps a few percent of what we see by rearrangement of sand grains.

ANALYSIS OF BOREHOLE EXTENSOMETER DATA FROM CENTRAL CALIFORNIA

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U.S. Geological Survey, Sacramento, California

ABSTRACT

In subsiding areas of central California, highly sensitive borehole extensometers provide data that define the compression characteristics of the compacting artesian aquifer systems. The extensometer records are combined with hydrographs for the confined and unconfined aquifers to form stress-strain diagrams on which the annual cycles of head decline and recovery generate a series of open loops. Gross compaction, elastic expansion, and net permanent compaction are clearly defined. The average level of preconsolidation stress, the elastic compressibility at lesser stresses, and the much larger plastic compressibility at higher stresses can often be determined from such diagrams. At a given location, these compressibilities may differ by as much as two orders of magnitude. Under favorable circumstances, estimates of the average vertical permeability and average compressibility of the fine-grained strata can be derived.

RÉSUMÉ

Dans les régions d'affaissement en Californie centrale, des extensomètres ultra-sensibles installés dans les trous de forage tubés, fournissent des renseignements qui indiquent les caractéristiques de compression des nappes aquifères artésiennes. Les enregistrements de l'extensomètre sont combinés avec les relevés du débit, en un temps donné, des nappes aquifères artésiennes ou libres, afin de former des diagrammes de pression-tension sur lesquels les cycles annuels de diminution ou de regain de pression artésienne produisent une série de boucles ouvertes. La compaction brute, l'expansion élastique et la compaction permanente nette sont clairement indiquées. Le niveau moyen de la pression de préconsolidation, la compressibilité élastique à des pressions moindres et la compressibilité plastique à des pressions plus grandes peuvent souvent être déterminées d'après ces diagrammes. A certains endroits, ces compressibilités peuvent varier au point d'avoir deux ordres de grandeur. Dans des circonstances favorables, les évaluations de la moyenne de perméabilité verticale et de la moyenne de compressibilité des couches à grain fin peuvent être obtenues.

INTRODUCTION

Recording borehole extensometers of the taut-cable type described by Lofgren (1961) have been used by the US Geological Survey since 1955 in investigations of subsidence due to ground-water withdrawal in central California. As experience was gained with the instruments, it became apparent that the relationship between the recorded compaction of the confined aquifer system and the decline of artesian head was both complex and fundamental to an understanding of the mechanics of subsidence. The purposes of the present paper are to summarize certain new techniques for analyzing this relationship and to present some preliminary results of their application.

STRESS AND COMPACTION AT PIXLEY, CALIF.

The multiple-extensometer installation near the town of Pixley, Calif., is described briefly in Lofgren's contribution to the present symposium. His figure 11 presents the basic head-change and compaction data used in the following analysis. Figure 1 of my paper shows, in the uppermost graph, the history of compaction between depths of 355 and 760 feet (108 to 232 meters), an interval that corresponds approximately to the zone of principal pumpage from the confined aquifer system. The second graph is a somewhat generalized history of the changes in the stress applied to all strata within this depth interval, as a result of changes in head in the confined and overlying unconfined aquifer systems. (See Lofgren, 1968, for the method of calculating stresses.) For convenience, stresses in this illustration and throughout this paper are expressed in equivalent units of water head (1 ft of water head equals 0.433 lb in^{-2} or 0.030 kg cm^{-2}; 1 m of head equals 0.1 kg cm^{-2}). The stress-change graph is plotted with stress increasing downward to emphasize its close correlation with declining artesian head.

From 1958 through 1968, 3.4 ft (1.04 m) of compaction has been recorded in the 355- to 760-ft interval. Despite the fact that the stress fluctuated through about the same range year after year, each major episode of stress increase was accompanied by additional permanent compaction. As stresses diminished during the fall and winter seasons of head recovery, compaction ceased, and in most years a slight expansion of the aquifer system was recorded. However, no simple and quantitatively consistent relationship between stress change and compaction is evident in the data.

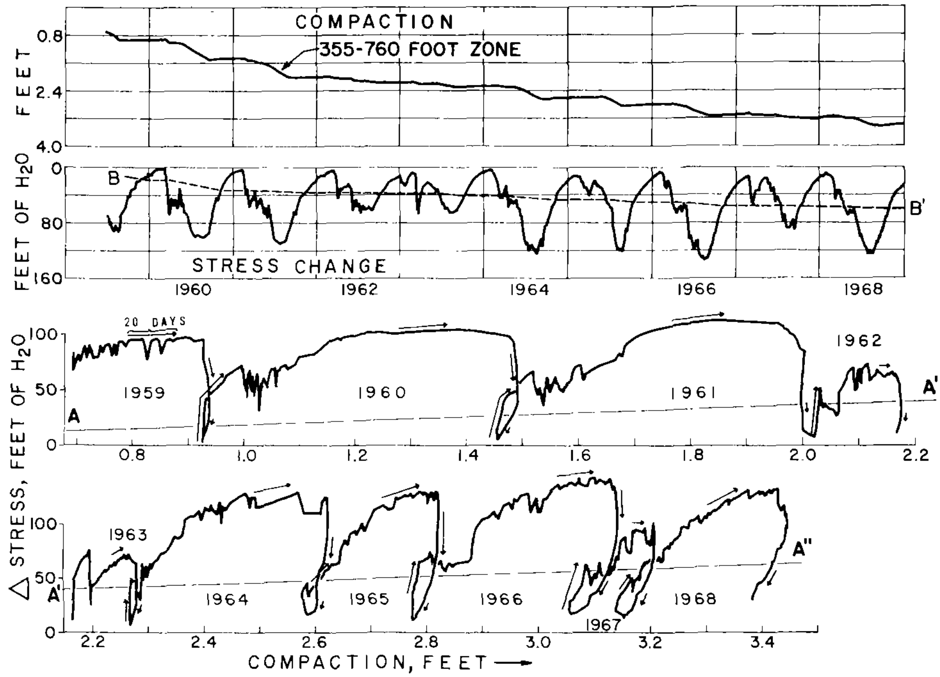

FIGURE 1. *History of compaction and stress change, and the relationship between stress change and compaction near Pixley, Tulare County, California*

THEORY OF AQUIFER-SYSTEM COMPACTION

The well known hydrodynamic (Terzaghi) theory of soil consolidation can provide a semi-quantitative explanation for the phenomenon of repeated permanent compaction during successive cycles of loading and unloading through about the same stress range. In the context of this problem a central tenet of consolidation theory states that an increase in stress applied to a "clay" stratum (aquitard) becomes effective as a compressive grain-to-grain load only as rapidly as the heads (pore pressures) in the aquitard can decay toward equilibrium with the head in the adjacent aquifer(s). Because of the low permeability and relatively high compressibility of the interbedded aquitards, the consolidation (compaction) of a multi-layered aquifer system in response to increased applied stress is a strongly time-dependent process, and complete or "ultimate" consolidation is not attained until a steady-state vertical distribution of head exists throughout the aquifer system. Transient heads in the aquitards higher than those in the adjacent aquifers (termed residual excess pore pressures) are a direct measure of the remaining primary consolidation that will ultimately occur under the existing stress. When pore-pressure equilibrium is attained throughout the aquitard, it is said to be 100 percent consolidated for the prevailing stress and no further permanent compaction will occur if the same stress is repeatedly removed and reapplied. (The possible role of secondary, or nonhydrodynamic, consolidation in aquifer-system compaction is not well known, but is assumed in this discussion to be minor.)

For a single homogeneous aquitard, bounded above and below by aquifers in which the head is instantaneously and equally lowered, the time, t, required to attain any specified dissipation of average excess pore pressure is a direct function of: (1) the volume of water that must be squeezed out of the aquitard in order to establish the denser structure required to withstand the increased stress, and (2) the impedance to the escape of this water. The product of these two parameters constitutes the aquitard time constant. For a specified stress increase, the volume of water is determined by the volume compressibility m_v, of the aquitard, the compressibility, β_w, of the water, and the thickness, b', of the aquitard. The impedance is determined by the vertical permeability, K', and thickness of the aquitard. Thus, the required time, t, is a function of the time constant, τ, where

$$\tau = \frac{S_s'(b'/2)^2}{K'} \tag{1}$$

and where S_s' is the specific storage of the aquitard, defined as

$$S_s' = S_{sk}' + S_{sw} \tag{2}$$

in which

$$S_{sk}' = m_v \gamma_w = \frac{\Delta b'}{b' \Delta h_a} \tag{2a}$$

and

$$S_{sw} = n \beta_w \gamma_w \tag{2b}$$

S_{sk}' is the component of specific storage due to compressibility of the aquitard, S_{sw}, is the component due to the compressibility of water, h_a is the average head in the aquitard, n is the porosity, and γ_w is the unit weight of water. For consolidating aquitards $S_{sk}' >>> S_{sw}$.

For convenience, it is customary to define a dimensionless time factor, T, such that

$$T = \frac{t}{\tau} \tag{3}$$

when T equals unity, t equals the time constant. The degree of consolidation, $U\%$, at any time, t, is then expressed as a function of T, the form of the functional relation being determined by the initial conditions of the problem. For the commonly used time-consolidation functions, $U\%$ is somewhat more than 90 percent when T is unity. Detailed development of the time-consolidation theory summarized above may be found in Scott (1963, pp. 162-197).

STRESS-STRAIN ANALYSIS

From the foregoing, it seems evident that reliable techniques for determining, preferably *in situ*, the specific storage (or compressibility), vertical permeability, and the thickness of every stratum in the aquifer system would provide the data required for accurately analyzing and predicting the time-dependent compaction of the aquifer system as a whole. Such an undertaking would be technically formidable and in most cases prohibitively expensive.

Under favorable circumstances, however, the bulk compressibility and average time-response characteristics of the confined-aquifer system as a whole can be approximated from the stress-strain relationship defined by the recorded head changes in the aquifers and the recorded compaction of the total system. To illustrate this relationship, the basic data presented in the two upper graphs in figure 1 have been combined in the lower part of the illustration to form a stress-strain diagram. Following the usual convention, stress is plotted increasing upward and strain (in this case cumulative compaction, not unit strain) increasing to the right. In subsidence studies, it is convenient to deal with the modulus of compressibility, which is here designated α when it applies to the response of the entire aquifer system in the elastic range. Accordingly, "slopes" of the stress-strain curve will be characterized numerically by strain per unit stress, the reciprocal of the conventional ratio, and the values will increase as the slopes become flatter.

The yearly cycles of stress increase and decrease, resulting from seasonal demand for irrigation water, produce a series of annual stress-strain loops, each of which is identified in figure 1 by irrigation year. Although time is not displayed as a variable in this type of illustration, it is obviously a strong influence on the shape of the stress-strain loops, because of the time-dependent character of consolidation. In order to convey some idea of the different rates of change of stress and strain at various times during the annual cycle, I have added arrows, each of which indicates by its length and direction the change in stress and strain during a 20-day period represented by the portion of the stress-strain curve adjacent to the arrow.

The descending segments of the annual loop are of particular interest since they represent the resultant of two opposing tendencies – one toward continuing compaction and one toward elastic expansion in response to decreasing applied stress. Expansion of the more permeable strata of the aquifer system must be essentially concurrent with the observed rise in head in wells. However, the first reduction of stress may produce only a slight reduction in compaction rate. Evidently, initial expansion of the aquifers is concealed by continuing compaction of the interbedded aquitards as water continues to be expelled under the influence of higher pore pressures remaining within the medial regions of the beds.

Consolidation theory requires that the maximum excess pore pressure, which is in the middle of a doubly-draining aquitard, be related to the same parameters (embodied in the time factor, T) that control the time-consolidation function. It is, therefore, inevitable that there be, at the end of a relatively short pumping season, a large range of maximum excess pore pressures in a sequence of aquitards of widely varying thicknesses and physical properties. Thus, as head in the aquifers rises and stress declines, the thinnest and (or) most permeable aquitards, containing the least excess pore pressure, will quickly assume an elastic response; but the thickest and (or) least permeable beds may continue to compact at diminishing rates through most or perhaps all of the period of head recovery and stress relief.

Evidence for this type of behavior is contained in the continuously curving stress-strain line characteristic of much of the descending portions of the annual loops (fig. 1). If the lower part of the descending curve approximates a straight line with a positive slope, as it clearly does, for instance, in 1967 and 1968, we probably are justified in assuming that essentially all excess pore pressures have been exceeded by the rising heads and that the entire aquifer system is expanding in accordance with its elastic modulus.

Because stress is expressed in units of water head, h, the compaction-stress-ratio under steady-state conditions equals S_k, the component of the aquifer-system storage coefficient, S, attributable to deformation of its skeleton (Riley, 1968). ($S = S_s \cdot m$, where m is the thickness of the aquifer system.) Therefore, for the confined aquifer system at Pixley we calculate, from the reciprocal slope of the straight-line approximation:

$$S_{ke} = \frac{\Delta m}{\Delta h} = \frac{-0.10}{-87} = 1.1 \times 10^{-3}$$

and

$$S_{ske} = \frac{S_{ke}}{m} = \frac{1.1 \times 10^{-3}}{405} = 2.8 \times 10^{-6}\ \mathrm{ft}^{-1} = 9.3 \times 10^{-6}\ \mathrm{m}^{-1}$$

and finally the compressibility of the aquifer system in the elastic range is computed as

$$\alpha = \frac{S_{ske}}{\gamma_w} = (2.8\times10^{-6})/0.433 = 6.6\times10^{-6}\,\text{in}^2\text{lb}^{-1} = 9.3\times10^{-5}\,\text{cm}^2\text{kg}^{-1}$$

where the subscript, $_e$, identifies the parameter as being in the elastic range. This value may be compared with β_w, the compressibility of water, which is $3.6\times10^{-6}\text{in}^2\text{lb}^{-1}$ ($5.1\times10^{-5}\text{cm}^2\text{kg}^{-1}$). The elastic compressibility slope is best defined in the 1967 and 1968 loops, but is approximated by the lowest segments of the loops in most years, except 1961 and 1964, when the expansion data were degraded by instrumental problems.

By extending construction lines with a slope equal to S_{ke} from the annual point of minimum stress to an intercept with the zero stress-change line, the net annual permanent compaction can be measured directly as the difference between the intercepts. For example in 1967, a maximum compaction of 0.15 ft (4.6 cm) was recorded, but the net permanent compaction was only 0.07 ft (2.1 cm).

The elastic hysteresis loops resulting from the steep ascent and subsequent flattening of the stress-strain curves in the first weeks of each pumping season are attributable in part to the frictional "dead-band" of the extensometer (0.01 to 0.03 ft) and in part to the hydrodynamic lag associated (even in the elastic range) with rapid stress increase. The effects of hydrodynamic lag, though relatively minor during the interval of slow elastic expansion, are sufficient, during the brief period of most rapid stress increase, to delay the appearance of a substantial percentage of the potential elastic compaction. Accordingly, the descending segments of the loops are used for calculating the elastic compressibility parameters.

By judiciously appraising the stress level at which the descending expansion curve approaches tangency with the S_{ke} line and the level at which the following elastic compaction curve crosses over the expansion curve, it is possible to select each year a stress level below which no appreciable excess pore pressures remain within the aquifer system. Although a distinctly subjective element enters into the selection of the stress level representing zero excess pore pressure, it was found that the straight line A-A′-A″ will closely fit the selected values. The intercepts of line A-A′-A″ on the annual expansion curves have been used to control the dashed line B-B′ on the time-stress graph. Line B-B′ defines the seasonal increments and long-term trend of decline in maximum excess pore pressure. Thus, it separates an overlying stress range, within which the sediments are fully consolidated, from an underlying range of higher stresses in which at least some of the aquitards are much less than 100 percent consolidated; therefore, the line B-B′ may be called a "preconsolidation line." Stress changes above the preconsolidation line cause only minor elastic deformations of the aquifer system, but each year, as stresses increase below the line, permanent "plastic" compaction resumes.

The slope of straight line A-A′-A″ approximately represents the ratio of the annual decrement of maximum (midplane) excess pore pressure to the annual increment of total compaction. By correcting for the annually increasing amounts of elastic expansion we can determine a corrected slope equal to the ratio of midplane pore-pressure decrement to permanent compaction increment. Since permanent compaction is a function of the decrement of average pore pressure, the corrected slope may be interpreted as characterizing the ratio of midplane excess pore-pressure decline to average excess pore-pressure decline. The apparent linear relation between these variables is reasonable if an initially sinusoidal distribution of excess pore pressure within the aquitards is assumed. During the first cycle of stress increase into a new range, a linear distribution (with depth) of initial excess pore pressure is more likely, but in all subsequent cycles in the same range a roughly sinusoidal initial distribution should prevail.

It is evident that if the stress repeatedly cycles through the same range – each year extracting an approximately equal percentage of the remaining excess pore pressure – the annual compaction increment will become progressively smaller and 100 percent consolidation will ultimately be approached; further stress cycles will then produce essentially congruent hysteresis loops of purely elastic compression and expansion. Under these conditions, the line A-A′-A″ will terminate at the top of the loops – a point representing full dissipation of midplane pore pressures and maximum compaction (plastic-plus-elastic) for the existing maximum stress. At this point, it is evident that the reciprocal slope of the line A-A′-A″ represents the compaction-stress ratio, which is S_k, the component of the "long-term" (steady-state) system storage coefficient attributable to plastic (-plus-elastic) skeletal compression. Furthermore,

$$\frac{S_k}{m} = S_{sk} = M_v \gamma_w ; \; M_v \gg \alpha$$

where M_v is the overall compressibility of the entire aquifer system for the stress range involved.

Determination of S_k from the slope of line A-A′-A″ on figure 1 is somewhat complicated by the fact that the stresses did not cycle through exactly the same range each year. If the applied stress is abruptly increased beyond the maximum previous level, there will be superimposed on the existing time-consolidation pattern a new response equivalent to the time-consolidation curve for the new stress increment under initially uniform pore-pressure distribution (Scott, 1963, pp. 214-218). Episodes of this type took place near the end of the pumping seasons in 1960, 1961, 1964, and 1966.

The principal immediate effect is a moderate increase in compaction of perhaps 2 to 5×10^{-3} ft per ft of increase over previous maximum stress. A probable secondary result is a reduction in the increment of compaction during the next cycle of equal or lesser stress. In any case, the reasonably good fit of the straight line A-A′-A″, while admittedly based on field data too imprecise for rigorous analysis, suggests that, at Pixley, occasional moderate and short-lived excursions of stress to new maxima have little net effect on either the linearity or average slope of the zero-excess-pore-pressure line (A-A′-A″). Accordingly, the reciprocal slope of this line is taken to be representative of the gross compressibility of the aquifer system, and the system compressibility parameters are estimated as follows:

$$S_k = 5.7 \times 10^{-2}$$

$$S_{sk} = \frac{S_k}{m} = 1.4 \times 10^{-4} \text{ft}^{-1} = 4.6 \times 10^{-4} \text{m}^{-1}$$

$$M_v = \frac{S_{sk}}{\gamma_w} = 3.2 \times 10^{-4} \text{in}^2 \text{lb}^{-1} = 4.6 \times 10^{-3} \text{cm}^2 \text{kg}^{-1}$$

By fitting the steepest and flattest reasonable lines to the data, we can further estimate tha the minimum probable value of S_{sk} is $1.1 \times 10^{-4} \text{ft}^{-1}$ and the maximum is $1.8 \times 10^{-4} \text{ft}^{-1}$ Since M_v is about 2 orders of magnitude larger than β_w, the compressibility of water, for practical purposes S_{sk} is equal to S_s, the steady-state specific storage for the entire aquifer system during compaction. The derived estimate of $S_{sk} = S_s$, although useful for comparing one aquifer system with another, is not directly applicable to determining the average compressibility for the aquitards, since it is based on the total thickness, *m*, of the aquifer system. To calculate the average aquitard compressibility, the plastic storage coefficient, S_k, should be divided by the aggregate thickness of compacting aquitards to obtain the average specific storage for the aquitards, designated $\bar{S}_s$, from which the average plastic compressibility, $\bar{m}_v$, can be obtained.

Thus

$$\bar{S}_s = S_k/\Sigma b' = \frac{5.7\times10^{-2}}{2.46\times10^{2}} = 2.3\times10^{-4}\,\text{ft}^{-1} = 7.5\times10^{-4}\,\text{m}^{-1}$$

$$\overline{m}_v = \frac{\bar{S}_s}{\gamma_w} = 5.3\times10^{-4}\,\text{in}^2\,\text{lb}^{-1} = 7.5\times10^{-3}\,\text{cm}^2\,\text{kg}^{-1}$$

After an estimate of the gross aquifer-system storage coefficient has been obtained, the percent of ultimate consolidation, $U\%$, can be calculated for each annual episode of compaction. Thus

$$\frac{U\%}{100} = \frac{\Delta m}{\Delta h \cdot S_k} \tag{4}$$

where Δm is the annual increment of compaction beyond the elastic range, as measured on the stress-strain graph, and Δh is the increase in stress beyond the preconsolidation level. The computed percent consolidation ranges from a minimum of 4.6 percent in 1967 to a maximum of 8.3 percent in 1961.

The determination of $U\%$ raises the obvious possibility of calculating the system time constant by equation 3, provided the appropriate functional relation, $T = F(U\%)$, can be defined. The character of this function depends upon the nature of the time-loading function (instantaneous or time-dependent) and the distribution of initial excess pore pressures. None of the available analytical functions closely approximates the highly irregular and arbitrary loading functions defined by the increasing-stress segments of the time-stress graph (fig. 1). However, the long time constant of the system, indicated by the small annual percent consolidation, suggests that short-term variations in the rates of stress increase may not be of great importance. Accordingly, an attempt has been made to estimate the time constant, τ, using an approximate procedure developed by Terzaghi and described by Leonards (1962, pp. 169, 170). The procedure is based on the nearly correct assumption that at the end of a period in which stress increases uniformly with time, the consolidation is the same as if the entire load had been applied instantaneously half way through the loading period.

As applied to the Pixley data, the procedure requires generalizing the steeply declining part of the seasonal stress-increase curve as a straight line from time, t_0, when the time-stress graph crosses the preconsolidation line, B-B′, to t_1 the time of maximum stress. If there are more than one stress maxima of about the same magnitude, the earliest is used. For those years in which there was an appreciable reduction of stress in late spring. following an initial stress increase beyond the preconsolidation level, the last peak (minimum stress) before the major stress increase was used as t_0. (If this last peak rises above line B-B′, the next intercept of the descending time-stress curve on B-B′ is used as t_0.) The stress increase, Δh, between t_0 and t_1, and the total compaction, Δm, during the same period are entered in equation 4 calculate percent consolidation. The time factor, T, is then read from the $T = F(U\%)$ curve for instantaneous loading and sinusoidal distribution of initial excess pore pressure. (The function is tabulated in Leonards, 1962, p. 164, 165.) In accordance with the Terzaghi approximation, the time constant for aquifer-system compaction may then be estimated by

$$\tau = \frac{0.5(t_1 - t_0)}{T}$$

The calculated values for τ for each year are tabulated below.

Year	Time constant (years)	Year	Time constant (years)	Year	Time constant (years)
1960	4.1	1963	4.7	1966	5.1
1961	4.9	1964	4.6	1967	3.8
1962	4.9	1965	4.7	1968	5.0
				Average	4.6

Considering the crudeness of the analytical procedure, the values of the time constant are surprisingly consistent, particularly in view of the heterogeneity of the aquifer system, the large differences from year to year in the time-stress graphs, and the even greater variation (by a factor of 6) in the increments of compaction.

The average vertical permeability of the aquitards may be estimated by equation 1 as 3.0×10^{-3}ft yr^{-1} (2.9×10^{-9} cm sec^{-1}).

The compressibilities and time constants developed in this analysis certainly cannot be regarded as representative for all or most of the aquitards in the very heterogeneous confined aquifer system at Pixley. However, they may provide, to a first approximation, the numerical values required to construct a highly idealized model whose time-consolidation behavior is reasonably similar to that of the real aquifer system. Testing of the predictive capabilities of this model by numerical methods and electrical analogy is planned for the near future.

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MODEL EXPERIMENTS ON LAND SUBSIDENCE

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ABSTRACT

In order to investigate the characteristic relations between land subsidence and withdrawal of ground water in the aquifer, some model experiments on a large scale were performed.

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Model soil layers were set in a water filled tank. The tanks used were a reinforced concrete tank (Tank-RC) of a rectangular form of 5 m × 2.7 m and 2.3 in height and 2 steel cylindrical tanks (Tank -A and -B) of 2 m in diameter and 1.5 m high each.

One series of tests was performed on the soil layers which consist of a clay layer placed on a sand aquifer. In the other series of tests, 2 clay layers and 2 sand layers were alternately filled in the steel tank. On the top surface of the soil layers free surface water was filled.

The following experiments were performed on these models:

1. The land movement due to lowering and recovering of the artesian head in the aquifer.
2. The land subsidence due to the repetitious change in the artesian head in the aquifer
3. The effects of change in the level of the surface water on the land subsidence and on the artesian head.

RÉSUMÉ

Afin de rechercher les rapports caractéristiques entre l'affaissement du sol et la baisse de la nappe aquifère, l'auteur a fait quelques expériences à l'aide d'un modèle à grande échelle.

Des couches de sol ont été placées dans un grand réservoir rempli par de l'eau.

Une série d'essais a été faite sur des couches de sol qui consistaient en une seule couche d'argile placée sur un sable aquifère. Deux couches d'argile et deux couches de sable aquifère ont été essayées dans une autre série d'essais. Sur la surface supérieure de la couche de sol, on a rempli de l'eau à surface libre.

Les expériences ci-après ont été faites sur ces modèles :

1. Le mouvement de terrain dû à l'abaissement et au rétablissement de la pression artésienne.
2. L'affaissement du sol dû au changement répété de la pression artésienne.
3. Les effets du changement de niveau de l'eau sur l'affaissement du sol.

1. INTRODUCTION

Land subsidence results from various causes. Among these causes, the subsidence due to the consolidation of clay strata resulting from pumping of much underground water becomes an important public problems. Vsually in such areas of land subsidence, various field observations of the phenomenon have been made. However, the geological, physical, and mechanical properties of soil layers and the hydraulic conditions at the actual sites are so complicated that the data obtained by these observations are almost beyond the scope of rigorus theoretical treatment. In order to avoid the confusion of the complex factors on the land subsidence, the experimental tests with large-scale models under simplified hydraulic and soil mechanical conditions were performed in the writer's laboratory. These tests included investigation of the effects of change in the artesian head as well as in the head of free surface water on the land subsidence 1, 2, 3.

2. MODEL SOIL LAYERS IN THE EXPERIMENTS

Model soil layers were set in a water-filled tank. The first series of tests used a reinforced concrete tank (Tank-RC) of a rectangular form, 5 m × 2.7 m and 2.3 m in height. In this tank, a single clay layer 1.2 m thick was filled on a sand layer 0.4 m thick, as shown in figure 1. These soils were always placed in the tank in water to expel air bubbles contained in the soils. The free water surface in the clay layer and the artesian water in the sand aquifer were connected with separate water supply systems so each water head could be independently controlled. To eliminate the disturbance effect in the clay layer and in the pore-water pressure distribution, the soil layers were let stand in the model for a few months before testing while both water heads equalized and were kept at 1.5 m high from the top of the sand aquifer.

The properties of soils filled in the tank are as follows: specific density of sand grain = = 2.56, dry density of sand filled = 1.55 gr per cc; specific density of clay particle = 2.65,

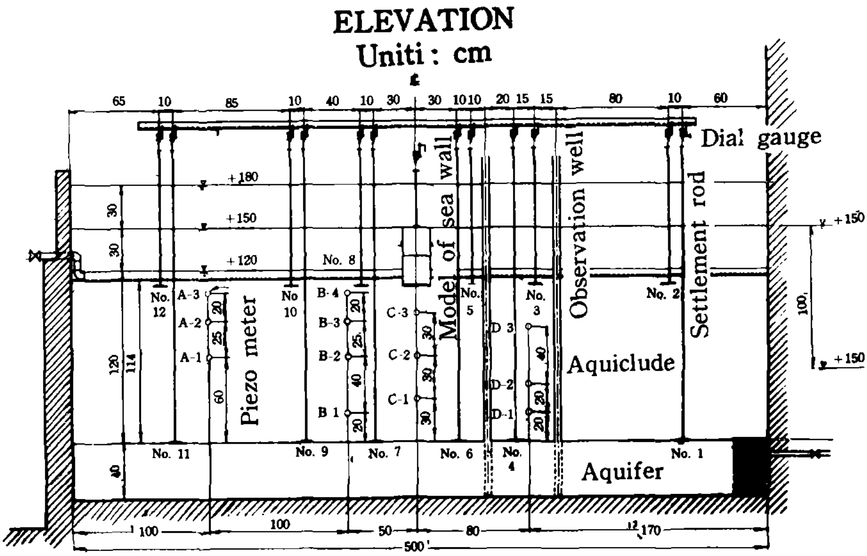

FIGURE 1. *Reinforced concrete tank (Tank-RC)*

unit weight of clay fill = 1.61 gr per cc, its water content = 64.4 percent, its degree of saturation = 100 percent. The grain size distribution curves are shown in figure 2. The

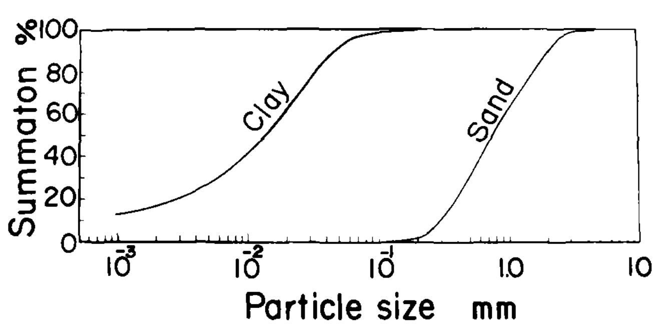

FIGURE 2. *Grain size distribution curves*

consolidation characteristics of the clay sample taken at 20 cm depth from the top surface of clay layer are shown in figures 3 and 4.

In the soil layers, 12 settlement rods and 13 piezometers were installed, as shown in figure 1. The settlement rod is a rod which stands vertically on a small plate embedded horizontally in a soil layer and can move freely in a soil-protecting pipe. Movement of the rod indicates the vertical movement of the soil layer where the plate is embedded.

The second series of tests was performed in a cylindrical steel tank (Tank-A or B) of the same type, of 2 m in diameter and 1.5 m in height as shown in figure 5. To the clay in the water in the tank, suction pressure can be applied through the bottom opening of the tank or compressed air through the air tight top cover of the tank to accelerate the consolidation of the clay. As model soil layers in Tank-A, a clay layer of 0.8 m thick was filled on a sand layer of 0.25 m, while in Tank-B, 2 clay layers and 2 sand layers were filled alternately as shown in figure 6. In these soil layers the settlement rods and the piezometers are also installed. The level of the surface water and the artesian head in the sand aquifer can be controlled independently as in the first series of tests.

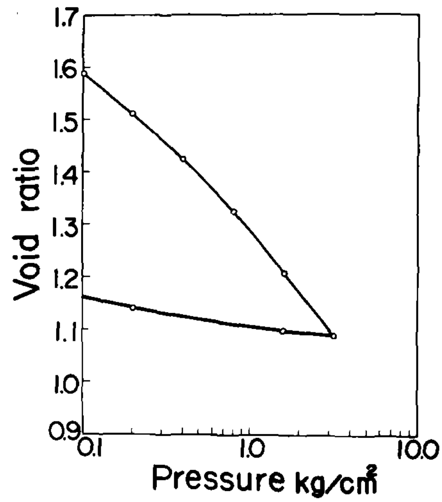

FIGURE 3. *Void ratio vs. logarithm of pressure curve of clay*

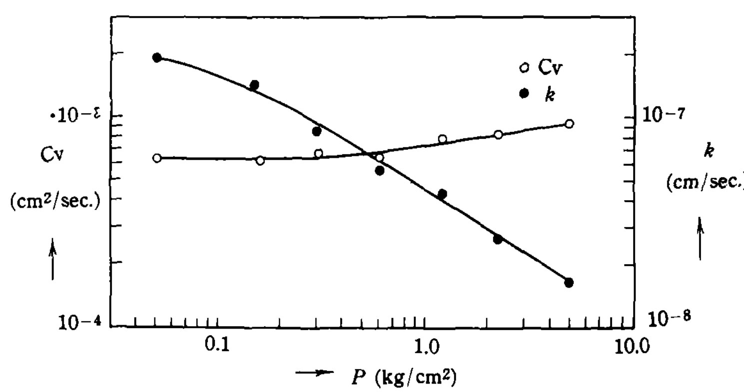

FIGURE 4. *Coefficient of consolidation and coefficient of permeability vs. log. of pressure curves.*

The properties of soils filled in the tank are as follows: dry density of sand = 1.40 − 1.45 gr per cc, specific density of clay particle = 2.62, unit weight of clay fill = = 1.77 − 1.76, water content of clay = 43.0 percent (Tank-A), 42.8 percent (Tank-B), their degree of saturation = 100 percent. The grain size distribution curves and the consolidation properties of the clay are shown in figures 7, 8 and 9.

3. TESTS ON THE GROUND MOVEMENT DUE TO CONSTANT LOWERING OR RECOVERING OF THE ARTESIAN HEAD

(*a*) IN THE CASE OF A SINGLE CLAY LAYER

As stated above, before the experiment in Tank-RC, the heads of the free water surface and the artesian water in the sand aquifer were equalized and kept at $h = 1.5$ m above the top surface of the sand aquifer. Under such hydraulic equilibrium conditions, after the pore water pressure in the clay layer (its thickness = $2H$) reached the hydrostatic pressure (see fig. 10), only then was the artesian head in the sand aquifer suddenly drawn

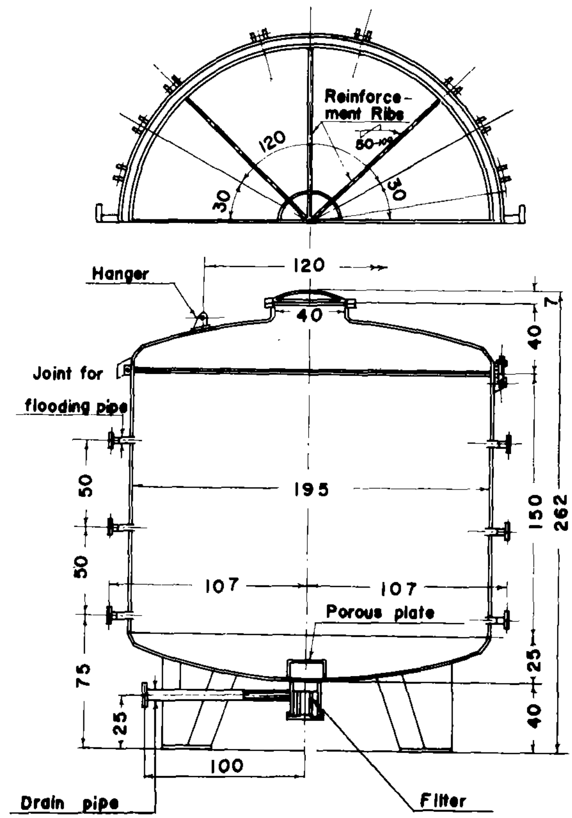

FIGURE 5. *Steel tank (Tank-A or-B)*

down by Δh = 1 m from the initial head and kept at this state. Due to such lowered head, the distribution of pore water pressure in the clay layer moved with time as shown in figure 10 and in consequence subsidence of the clay layer proceeded because of the decrease in pore water pressure.

After continuance of such lowered head for 220 days, the artesian head was let recover to the initial head. Then the distribution of pore water pressure moved backward toward the initial hydrostatic line as shown in figure 11. As shown in this figure, the rate of recovery of the pore water pressure is quicker than that under consolidation.

In the case of consolidation, the degree of consolidation U is given by the ratio of area ACBDA and area of ΔABC in the pore water pressure distribution diagram as shown in figure 12. Such change in the excess pore water pressure, u, in the clay layer is usually determined by the following basic equation:

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \qquad (1)$$

where:

C_v is the coefficient of consolidation;
t is time and
z is depth from the top of the clay layer.

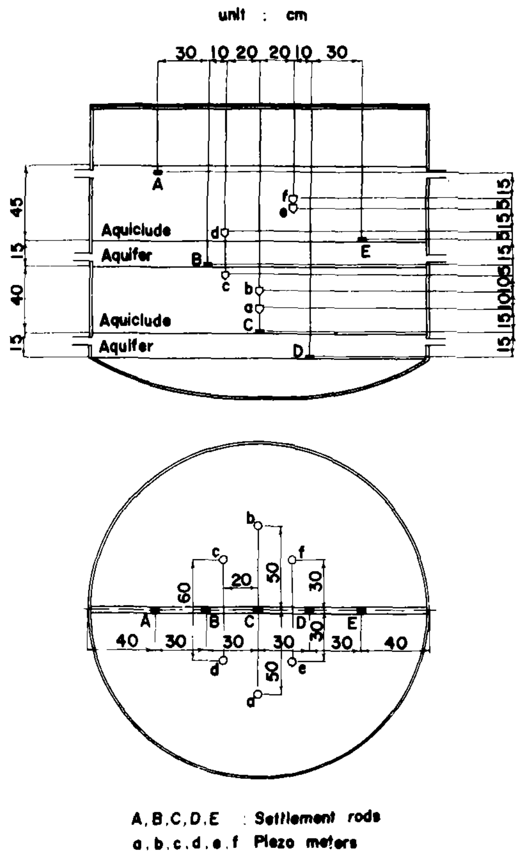

FIGURE 6. *Soil Profile and position of measuring rods in Tank-B*

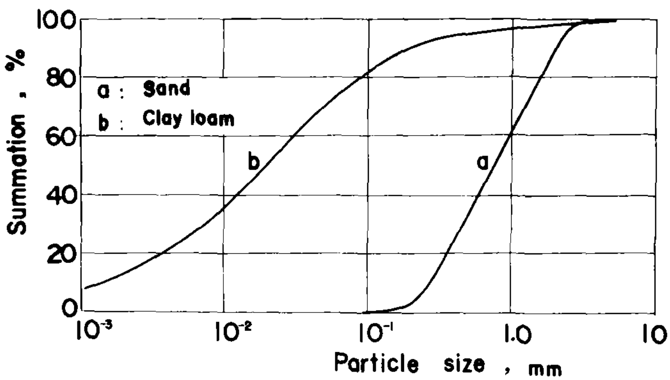

FIGURE 7. *Grain size distribution curves*

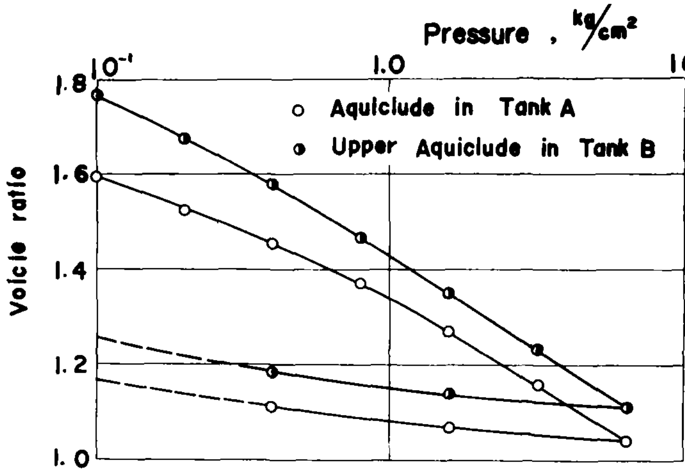

FIGURE 8. *Void ratio vs. logarithm of pressure curve of clay*

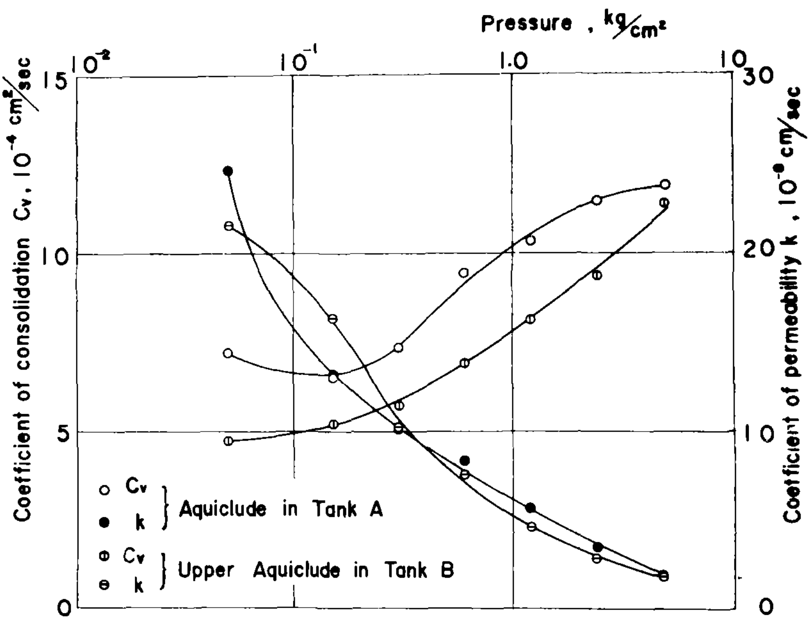

FIGURE 9. *Coefficient of consolidation and coefficient of permeability vs. log. of pressure curves.*

In the case shown in figure 12, the initial and boundary conditions are as follows:

$$\left.\begin{aligned} &u(z,0) = (z/2H)\cdot \Delta h \cdot \gamma_w, \\ &u(0,t) = 0, \quad u(2H,t) = 0. \end{aligned}\right\} \tag{2}$$

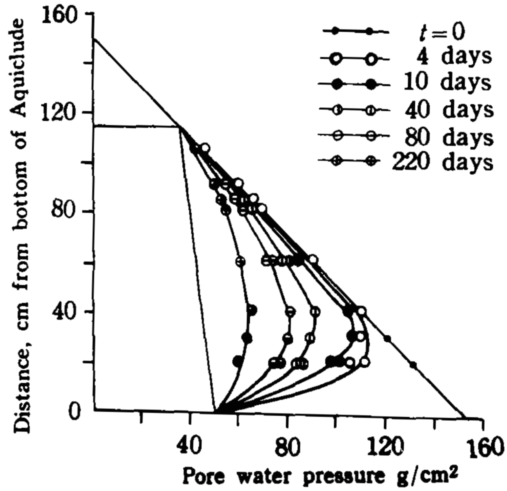

FIGURE 10. *Measured pore water pressure at initial state and after the artesian head in the aquifer was lowered*

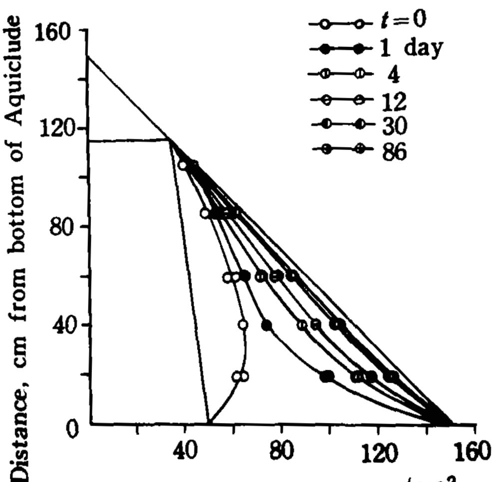

FIGURE 11. *Measured pore water pressure after the artesian head was recovered*

If C_v is constant, equation (1) has been solved by Terzaghi and Frölich under the above conditions. The result is as follows:

$$u(z, t) = \frac{2}{\pi} \cdot \Delta h \cdot \gamma_w \sum_{m=1}^{\infty} \frac{(-1)^{m+1}}{m} \cdot \exp\left(-m^2 \cdot \frac{\pi^2}{4} \cdot T\right) \cdot \sin \frac{m\pi z}{2H} \tag{3}$$

and

$$\begin{aligned} U &= 1 - \frac{1}{\Delta h \cdot \gamma_w \cdot H} \int_0^{2H} u(z, t)\, dz \\ &= 1 - \frac{8}{\pi^2} \sum_{m=1,3,5,\dots}^{\infty} \frac{1}{m^2} \exp\left(-m^2 \cdot \frac{\pi^2}{4} \cdot T\right). \end{aligned} \tag{4}$$

where:

γ_w is the unit weight of water, and
$T = C_v.t/H^2$.

However, the $U-t$ curve computed by equation (4) does not coincide with that obtained experimentally from figure 12. This discrepancy may be considered to be caused by the dependancy of C_v on the effective pressure or time during consolidation. If C_v could be assumed to vary stepwise with time (i.e. C_v is constant for $t_{i-1} \leqslant t \leqslant t_i$), the general theory for variable C_v has been developed by Schiffman [4] as follows:

For the ith step, the excess pore water pressure u_i is given by

$$\frac{\partial u_i}{\partial t} = C_{vi}\frac{\partial^2 u_i}{\partial z^2}, \tag{5}$$

$$\left.\begin{aligned} &u_i(Z, 0) = u_{i-1}(Z, t_{i-1}), && \text{for} \quad 0 \leqslant Z \leqslant 2H, \\ &u_i(0, \bar{t}) = 0, && \text{for} \quad 0 \leqslant \bar{t} \leqslant \infty, \\ &u_i(2H, \bar{t}) = 0, && \text{for} \quad 0 \leqslant \bar{t} < \infty, \end{aligned}\right\} \tag{6}$$

where:

$$\bar{t} = t - t_i.$$

Under the above conditions, the solution of equation (5) is given by

$$\left.\begin{aligned} &u(z, T') = \frac{2}{\pi}\cdot \Delta h \cdot \gamma_w \sum_{m=1}^{\infty} \frac{(-1)^{m+1}}{m}\cdot \exp\left(-m^2\cdot\frac{\pi^2}{4}\cdot T'\right)\cdot \sin\frac{m\pi z}{2H}, \\ &U = 1 - \frac{8}{\pi^2}\sum_{m=1,3,5,\ldots}^{\infty}\frac{1}{m^2}\cdot \exp\left(-m^2\cdot\frac{\pi^2}{4}\cdot T'\right) \\ &\text{where} \\ &T' = \frac{1}{H^2}\left[C_{v1}\cdot t_1 + C_{v2}(t_2 - t_1) + \ldots + C_{vn}(t - t_n)\right]. \end{aligned}\right\} \tag{7}$$

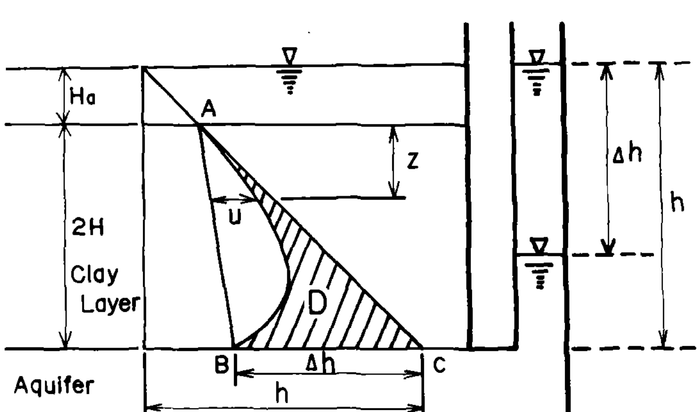

FIGURE 12. *Boundary conditions of soil layers and a distribution curve of pore water pressure*

If the time interval of each step is assumed to be equal, T' becomes

$$\left.\begin{aligned} T' &= \frac{t'}{H^2}(C_{v1}+C_{v2}+C_{v3}+\ldots+C_{vn}), \\ \text{where} \quad t' &= t_1 = t_2-t_1 = t_3-t_2 = \ldots = (t_n-t_{n-1}). \end{aligned}\right\} \qquad (8)$$

Applying this theory to the experimentally obtained U, the relationship between C_v and time is obtained as shown in figure 13, where the time interval t' is taken as 10 days. The shaded part in figure 13 represents the range of C_v-values obtained by the oedometer tests. Applying C_v thus obtained to equation (7), $u(z, t)$ can be calculated as shown by the dotted lines in figure 14. As shown in this figure, each dotted line coincides well with the experimental line, illustrated as a solid line.

In the case of the recovery of the artesian head, a treatment similar to the above can be applied to the swelling of clay by introducing the coefficient of swelling C_s, instead of C_v. Figure 15 shows the relation between the coefficient C_s thus obtained and the time elapsed after the recovery of the artesian head. Comparing figure 15 to figure 14, it seems that the coefficient of swelling is about five times as large as that of consolidation.

The land settlement or rebound due to lowering or recovery of the artesian head were measured by the settlement rods whose detecting plates were embedded on the top surface of the sand aquifer (No. 1, 4, 6, 7, 9, 11 in figure 1) or at the point 5 cm from the top

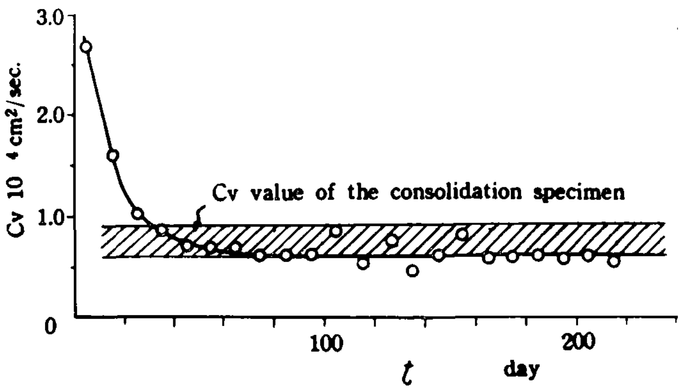

FIGURE 13. *Variation of coefficient of consolidation with time*

surface of the clay layer (No. 2, 3, 5, 8, 10, 12). Therefore, the initial vertical distance between the upper plate and the lower one was 115 cm. The measured settlement of the clay layer due to lowering head and the degree of consolidation calculated from the distribution curves of pore water pressure (fig. 10) are shown in figure 16. The relation between the clay layer's rebound and the degree of swelling are in figure 17.

In figure 16 the settlement-time curve is not similar to U-t curve. The unconformity between these curves may be considered to be due to the secondary consolidation.

Besides the above stated experiments, similar tests were performed in Tank-A. The pore water pressure obtained under lowering artesian head is shown in figure 18 and that under recovering head in figure 19.

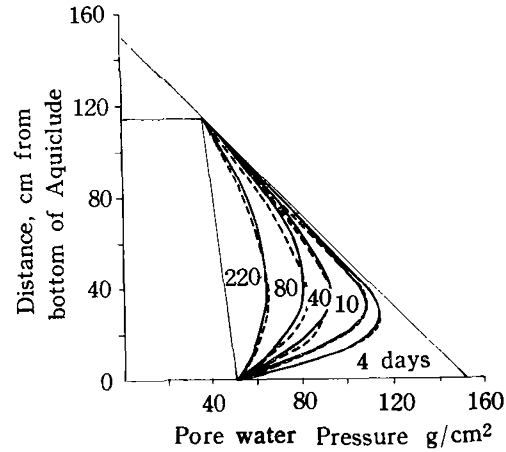

FIGURE 14. *Comparison of the distributions of pore water pressure actually-measured (dotted lines) and those computed from equation (7) (solid lines)*

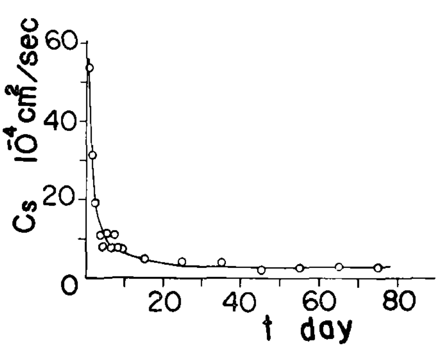

FIGURE 15. *Variation of coefficient of swelling with time*

(*b*) IN THE CASE OF ALTERNATIVE LAYERS OF CLAY AND SAND

Usually, land subsidence is analysed for the case of a single clay layer placed on a sandy aquifer. But the actual ground constitution often consists of alternating sandy and clayey layers. In such a case, if the artesian head in the sand aquifer is drawn, consolidation occurs in both upper and lower clay layers. Therefore, if both clay layers have the same physical properties and thickness, the amount of land subsidence becomes twice as much as that of a single clay layer.

In order to demonstrate such subsidence, two alternative sand and clay layers were placed in the Tank-B as shown in figure 6. Free water in the upper clay layer, artesian water in the upper sand aquifer and that in the lower one were connected individually to each head-control tank. At first each water head was kept at the same level until the pore water pressure in each clay layer reached the hydrostatic pressure. Then the following experiments were performed. (1) Only the artesian head in the upper sand aquifer was lowered by 0.8 m for 14 days from its initial head. As one example of the results, the distribution of the pore water pressure is shwon in figure 20. (2) Subsequently the lowered

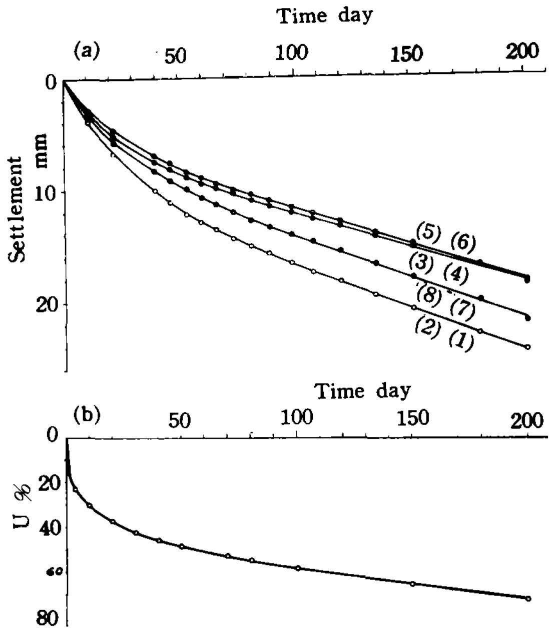

FIGURE 16. (a) *Settlement measured vs. time curves* (b) *Degree of consolidation constructed from figure 10 vs. time curve*

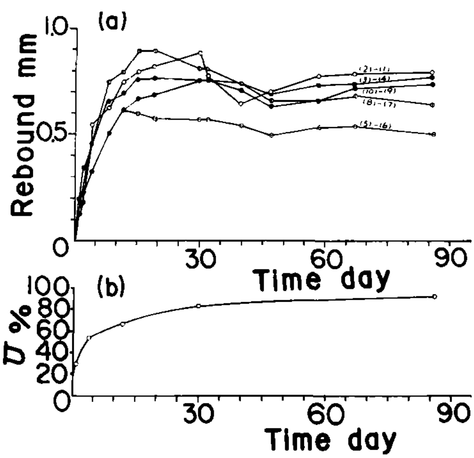

FIGURE 17. (a) *Rebound measured vs. time curves* (b) *Degree of swelling constructed from figure 11 vs. time curve*

artesian head was recovered to the initial head. The increasing pore water pressure in both clay layers are shown in figure 21. (3) After recovery of the pore water pressure in the clay layers, the artesian heads in both upper and lower sand aquifers were lowered simultaneously from the initial head. The results are shown in figure 22.

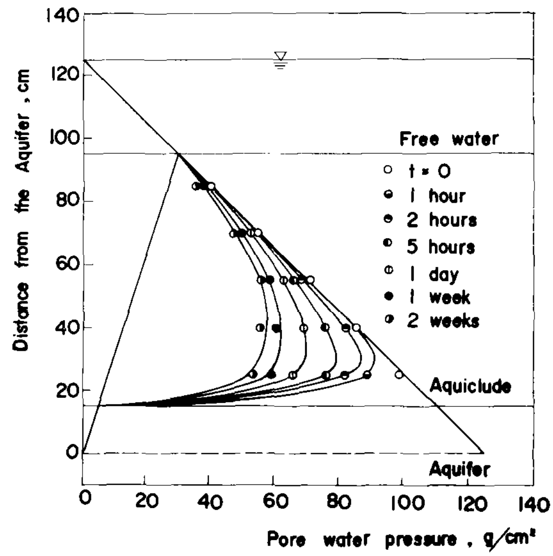

FIGURE 18. *Measured pore water pressure after the artesian head was lowered in Tank-A*

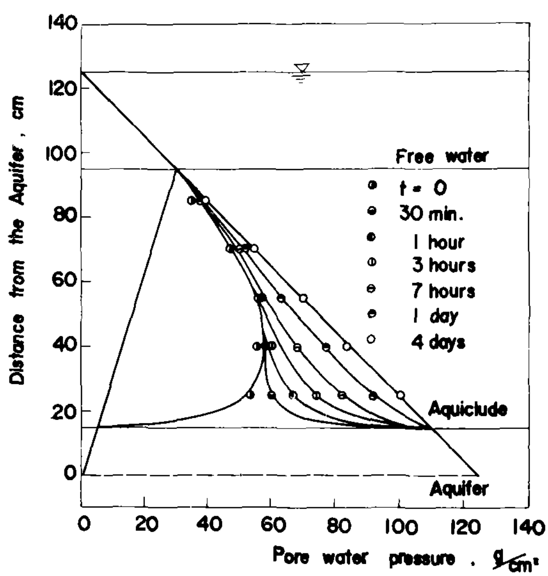

FIGURE 19. *Measured pore water pressure after the artesian head was recovered in Tank-A*

4. LAND SUBSIDENCE DUE TO THE REPETITIOUS CHANGE IN THE ARTESIAN HEAD

The artesian head in a natural sand aquifer usually shows daily and seasonal variations due to unequal withdrawal of the underground water. In order to investigate the effect of such a repetitious change of the artesian head on the land subsidence, the artesian head

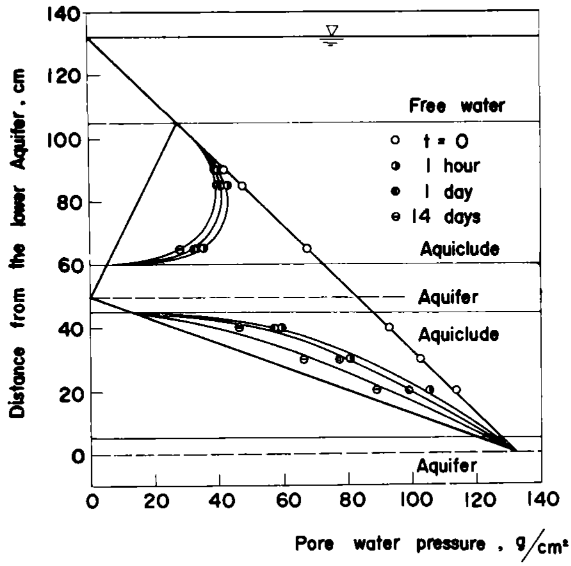

FIGURE 20. *Measured pore water pressure after the artesian head in the upper aquifer was lowered*

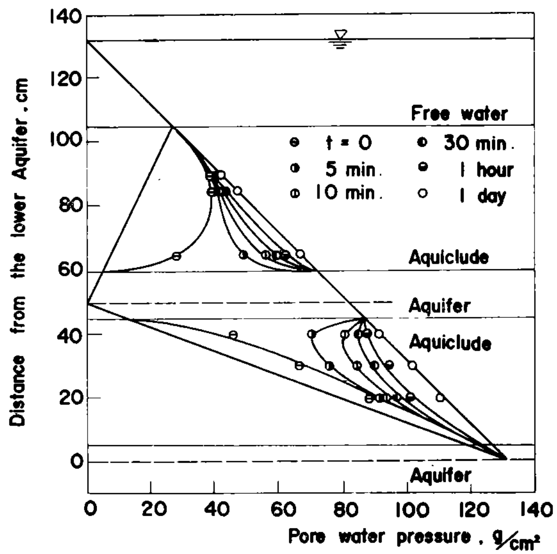

FIGURE 21. *Measured pore water pressure after the artesian head in the upper aquifer was recovered*

in the sand aquifer in Tank-RC, which had been kept at the level of the free water surface, was repetitiously lowered and recovered by 1.0 m at various cycle times i.e. 20, 40, 60, 120 min and 48 hr. As an example, figure 23 presents the subsidence and rebound of the sand aquifer and the clay layer during one cycle of the repetitious change of artesian head of 48 hr-cycle, viz. 24 hr-lowering and 24 hr-recovering. Figure 24 summarizes the relation between time and the observed subsidence and rebound caused in the clay layer under various repetition-cycles of the artesian head. In this figure, the subsidence due to static lowering of the artesian head (i.e. no repetitious change) is also illustrated by a thick line.

From these figures, it can be said that the longer the cycle of the repetition, the greater the rate of land subsidence becomes.

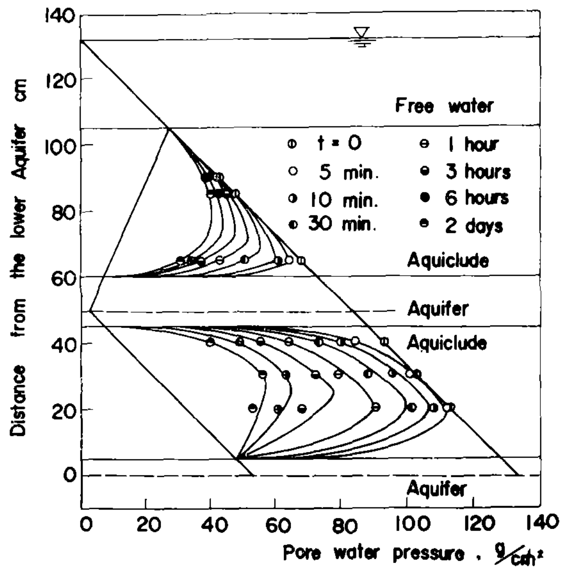

FIGURE 22. *Measured pore water pressure after the artesian heads in both upper and lower aquifers were lowered*

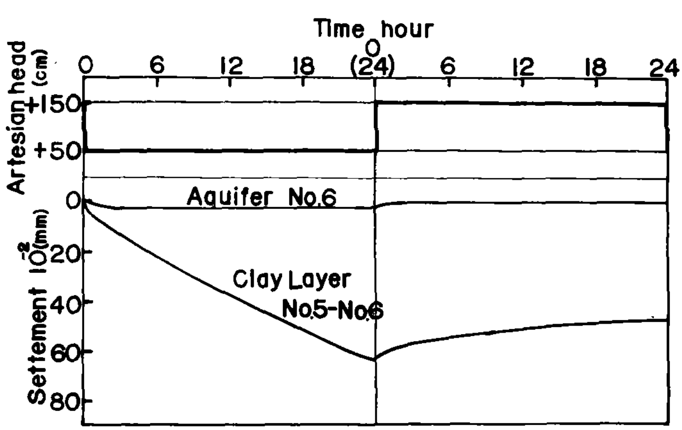

FIGURE 23. *Settlement and rebound of soil layer caused by one cycle of the repetitious change in the artesian head (in Tank-RC)*

Under such repetitous artesian head, the sand aquifer also showed the subsidence due to the compacting mechanism. Therefore, the amount of subsidence caused in the sand layer depends on its initial density and the number of repetitions of the artesian head, provided that the amount of lowering in the head is constant. The movement in the sand layer synchronizes with the variation of the artesian head but the movement in the clay layer shows some time lag. Moreover, as already stated, the rebound in the clay layer is quicker than its consolidation.

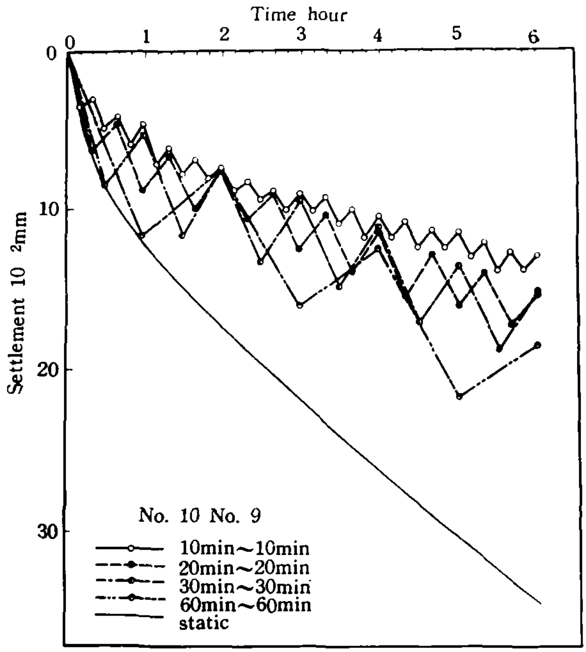

FIGURE 24. *Settlement of clay layer vs. time curve under various periods of repetitious change in the artesian head (in Tank-RC)*

5. EFFECT OF CHANGE IN THE LEVEL OF THE WATER SURFACE ON THE LAND SUBSIDENCE

A change in the pore water pressure in the clay layer results when the level of the free water surface which has been equalized with the artesian head, is changed. In such a case,

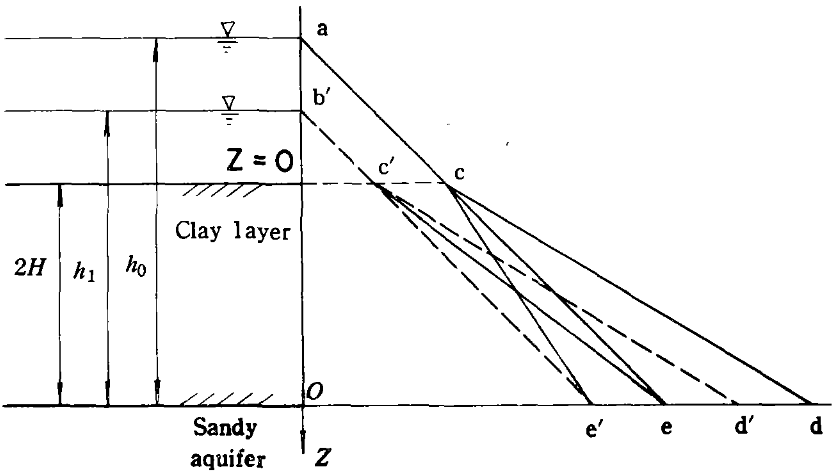

FIGURE 25. *Illustration representing the relation between the change of surface water level and the pore water pressure in the clay layer and the sand aquifer*

if the artesian head is kept at its initial head, the lowering of the free water level causes negative pore water pressure or suction (fig. 25) which leads to the swelling of clay, while raising the water level results in consolidation of the clay.

One of the noticiable phenomena caused by the change in the level of the water surface is the change of the pore water pressure in the sand aquifer. These can be explained by the following paragraphs.

In figure 25, the line *acd* represents the total stress distribution, where *ac* is the line of static water pressure and *cd* is $\int \gamma . dz$ (γ is the unit weight of clay, z is the depth from the clay surface). Since the extended straight line of *ac* or line *ace* shows the static water pressure, Δced represents the area of the effective stress in the clay layer. When the level of the water surface is lowered suddenly from h_0 to h_1, the total stress line is given by the line $b'c'd'$, where $b'c'$ is parallel to *ac* and $c'd'$ is parallel to *cd*. Since the void ratio of the clay can not be altered instantaneously by the external stress, the effective stress in the clay immediately after the lowering of the surface water remains as it has been, and it can be expressed by $\Delta c'e'd'$ which is identical with Δced. Therefore, the straight line $c'e'$ becomes the initial pore water pressure line produced immediately after the lowering of the water surface, from which a change in the pore water pressure will start.

However, the final pore water pressure line, to which the pore water pressure approaches, can not be determined unless the condition of water supply to the sand aquifer is determined.

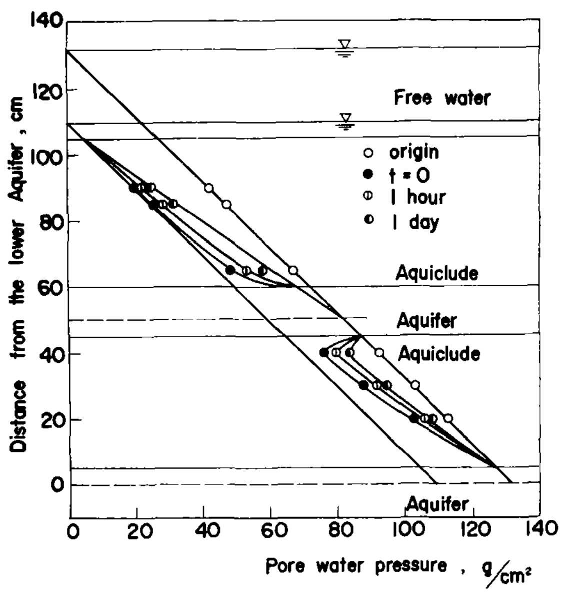

FIGURE 26. *Measured pore water pressure after the level of free surface water was lowered*

If water is freely supplied to the aquifer and its artesian head is always kept at the initial head of $\gamma_w . h_0$ (represented by the lenght *oe*), the final pore water pressure is given by the line $c'e$. Therefore, $\Delta\ c'\ e'\ d'$ represents the initial effective stress and shrinks with time to $\Delta c'ed'$ of the final state. Such decrease in the effective stress promotes the swelling of the clay layer.

As for the effective stress in the sand aquifer, its initial and final values can be expressed by the lengths *ed* and *ed'*, respectively. The decrease of the effective stress due to lowering of the water surface (expressed by the length *dd'* which equals *ed-ed'*)causes the swelling of the sand aquifer. Thus the lowering of the water surface results in the swelling of soil layer and consequently the land heaving.

On the contrary, when the level of the water surface is raised from its initially ballanced level at the artesian head level, the consolidation of the clay layer, as well as the compaction of the sand layer, occur and the land subsidence proceeds with time.

If the water surface is lowered and no additional water is supplied to the sand aquifer, the sand is restrained from swelling freely from its initial void ratio. Such an undrained condition the causes decrease in the artesian head in the sand aquifer. On the contrary, raise the of the water level under the same water supply condition as above results in an increase of the artesian head. Therefore, if the level of the water surface changes repetitiously, as with a tidal change, the artesian head in the aquifer, which has no additional water supply, oscillates in harmony with the water surface oscillation.

In order to investigate the phenomena stated above, an experiment was performed in Tank-RC. The level of the water surface was changed repetitiously after the pore water pressure had reached equilibrium. Figure 27 shows the observed relation between the repetitious change of the water surface level and its effect on the artesian head of the aquifer, into which no additional water from the outer supply system was fed. The amplitude of the artesian head was 2.5 cm and was less that that of the water surface level of 30 cm. This relation may be due to a certain amount of water being fed to the aquifer from the clay layer and from the water-filled measuring piezometers.

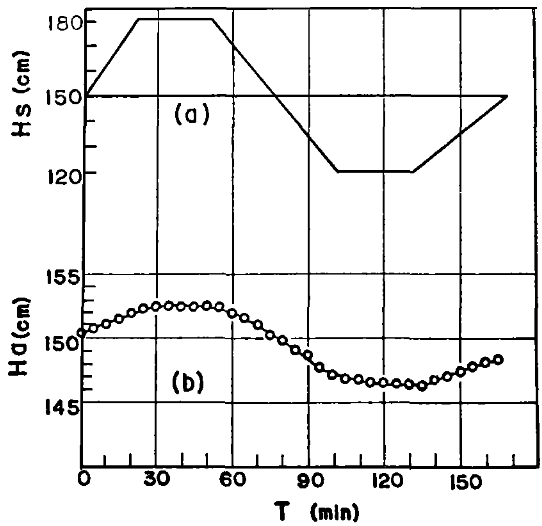

FIGURE 27. *One cyclic change in the level of surface water* (H_s) *and its response in the artesian head* (H_a) *when additional water supply was closed*

As the responsive amplitude of the artesian head decreases with an increase in the feeding capacity of artesian water, the ratio of the amplitude of artesian head and that of the free water surface may be adopted as an indicater of the supplying capacity of underground water to the aquifer. In the actual region of the land subsidence near the sea-coast, it can be usually observed that the artesian water level changes repetitiously in the similar phase with the tidal change. One of the examples is shown in figure 28. From the considereration stated above, such a phenomena seems to indicate the lack of underground water being supplied to the sandy aquifer as a result of pumping of much underground water. The writer whishes to spend more precise investigation on these relations.

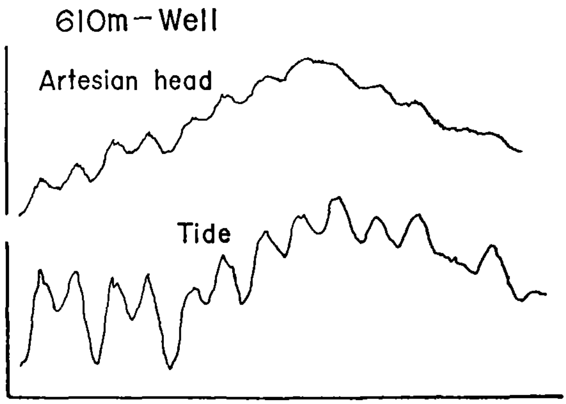

FIGURE 28. *An observed record of tidal level and artesian head in land-subsiding area in Niigata*

ACKNOWLEDGEMENT

The model experiments in Tank-RC were mainly performed by Messrs. Junichi Yamamoto, Yoshikazu Shinmi and Kosuke Takano and those in Tank-A, B were mainly by Mr. Minoru Matsuo. The author is also indebted to Dr. Toru Shibata for his effective co-operation through the experimental research. Again, the author expresses his sincere appreciation for all their fruitful co-operations.

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MODEL STUDIES OF DIFFERENTIAL COMPACTION
MODÈLES POUR ÉTUDE DE COMPACTION DIFFÉRENTIELLE

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ABSTRACT

Phenomena associated with differential compaction of sediments can be reproduced in models employing liquid-solid suspensions subjected to differential vacuum. Suspensions of clay-sized particles which approximate the porosity of young sediments are easily prepared and have viscosities and shear strengths much lower than other, commonly used, fracturable materials.

Both field observation and experiment indicate that response to differential compaction is, in many cases, differential subsidence coupled with fracture. Fracture effects vary in scale of displacement from thousands of meters, in regional contemporaneous faults that parallel the coast of Texas, U.S.A. to decimeters in fractures caused by recent withdrawal of ground-water and petroleum in Texas and Arizona. Model studies indicate that a differentially compacting sediment mass fails in shear in a manner analogous to the failure of a simply supported beam. Therefore, fracture of the sediment mass should be predictable, in time and space, if sufficient information about physical properties of the sediment mass is available.

RÉSUMÉ*

Les phénomènes associés à la compaction différentielle des sédiments peuvent être reproduits au moyen de modèles où l'on emploie des suspensions liquides-solides soumises à un vide différentiel. Des suspensions d'argile avec la porosité de sédiments récents sont faciles à préparer ; leur viscosité et résistance au cisaillement sont de beaucoup inférieures à celles de matériaux cassants généralement employés.

Les observations de terrain et les essais de laboratoire indiquent que la compaction différentielle est souvent une subsidence différentielle accompagnée de cassures. Les déplacements observés au voisinage des fractures varient de milliers de mètres dans le cas des failles régionales contemporaines qui sont parallèles à la côte du Texas, Etats Unis, jusqu'à quelques décimètres observés dans les fractures causées par le pompage de l'eau souterraine et du pétrole dans la région du Texas et de l'Arizona.

Des études à l'aide de modèles indiquent qu'une masse de sédiments en compaction différentielle se fracture par cisaillement d'une façon analogue à la cassure d'une poutre en support simple. On devrait donc pouvoir prédire, dans le temps et l'espace, les fractures de la masse sédimentaire si on a suffisamment de données sur ses propriétés physiques.

INTRODUCTION

The fact that removal of fluids from permeable rocks may cause compaction and consequent subsidence of the overlying land surface probably was first recognized by Minor (1925) and Pratt and Johnson (1926) in relation to subsidence at the Goose Creek Oil Field, near Houston, Texas, USA. About the same time Meinzer and Hard (1925) deduced that loss of pressure in the Dakota Sandstone artesian aquifer was accompanied by compression of the aquifer. This observation led to Meinzer's statement of the compressibility and elasticity of aquifers (1928) and to subsequent development of theories of elastic and inelastic storage coefficients by Theis (1935), Jacob (1940, 1950), Lohman (1961), Domenico and Mifflin (1965), De Wiest (1966) and Lofgren (1968). Tolman (1937) appears to have been the first to recognize the connection between compaction (inelastic compression) and land subsidence.

* For which I am indebted to Dr. Gilles O. Allard, Department of Geology, University of Georgia, Athens, Georgia, U.S.A.

Both Minor (1925) and Pratt and Johnson (1926) reported surface fractures associated with subsidence at the Goose Creek Field, but reports of fracture in connection with land subsidence between 1926 and 1960 appear to be rare. Weaver and Sheets led a field trip to locations of recent fracture in the vicinity of Houston, Texas in 1962. Weaver and Sheets apparently thought that all of the subsidence and movement on fractures was due to withdrawal of fluids, both petroleum fluids and fresh water, but that some of the fractures were rejunevated tectonic features. Beginning in 1961, at least five reports of fracturing related to land subsidence and fluid withdrawal in various places in Arozina appeared in rapid succession (Pashley, 1961; Robinson and Peterson, 1962; Peterson, 1965; Kam, 1965). Carver (1968) proposed that regional contemporaneous faults of the Gulf of Mexico Coastal Plain were produced by differential compaction of thick, rapidly accumulating sediment wedges with pronounced facies change from coarse to fine in the seaward direction. Yerkes and Castle (1968) reported that faulting has accompanied oil-field development in the United States in at least twelve cases.

Some very recent developments emphasize the importance of differential compaction mechanics in land subsidence due to fluid withdrawal. Peterson (1968) reconfirmed that storage capacities of some aquifers are irreversibly reduced by withdrawal of fluids and

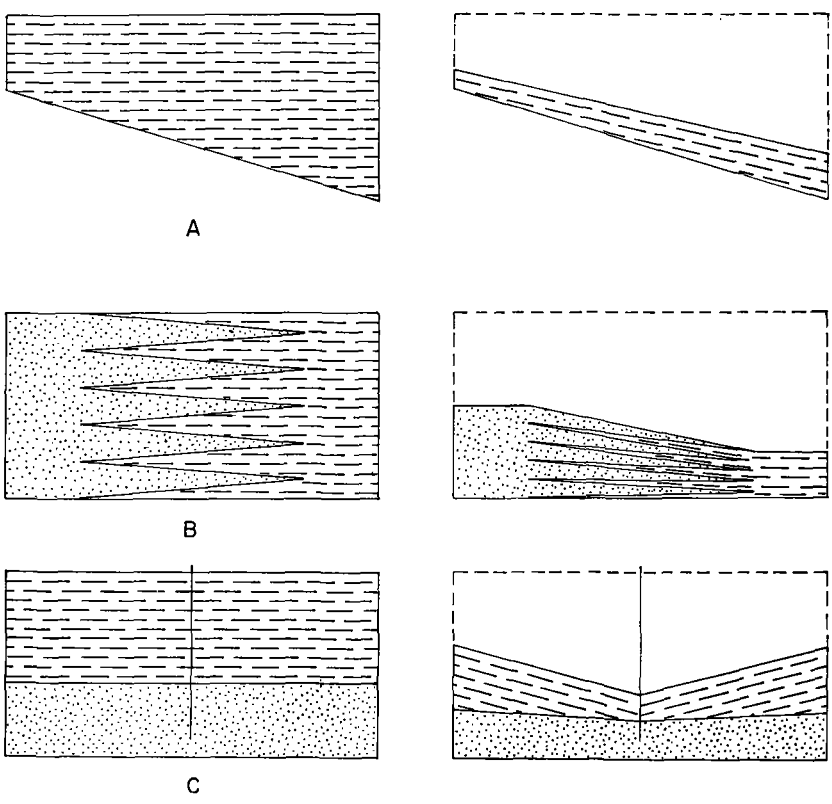

FIGURE 1. *Mechanisms of differential compaction. A, difference in original thickness. B, difference in compactibility of mixed lithologies. C, differential compaction rates, in this case produced by a well, or well field located at the center of the illustration. Original configuration is at left in each illustration, compacted configuration at right. Compaction is to scale, with sand compacting from original porosity of 40 percent to a final 20 percent; clay from 80 percent to 20 percent. C was assumed to be precompacted, with initial sand porosity of 25 percent, initial clay porosity of 50 percent and final porosities as above*

noted a direct correlation between loss of storage capacity and land subsidence, as expected; but, surprisingly, Peterson also observed that horizontal surface movements, to half the value of the vertical subsidence, occurred. The horizontal component of subsidence in this case is strongly suggestive of diagonal shear into the area of maximum differential compaction near the well.

The objectives of this report are to show that shear fracture is a common, if not universal, consequence of withdrawal of fluids from unconsolidated sediments, to outline mechanism of fracture failure of unconsolidated sediment masses, and to suggest methods of estimating and minimizing engineering hazards associated with differential compaction fracture.

MECHANISMS OF DIFFERENTIAL COMPACTION

Differential compaction of unconsolidated sediments eventually occurs wherever unequal thickness of sediments are deposited (Hedberg, 1926). Nevin and Sherrill (1929) studied models of differential compaction over symmetrical and asymmetrical hills and developed the concept of supratenuous folding related to differential compaction. Nevin and Sherrill did not mention that differential compaction also caused faulting, but minor fractures are clearly visible in their photographs of the completed asymmetrical model.

Differential compaction results from interfingering of lithologies with different initial to final volume ratios. Clays (shales) may have initial porosities as high as 80 percent and compacted porosities of 20 percent, as opposed to 40 percent initial and 20 percent final porosities for sands. Interfingering of sands and shales may therefore cause large differences in the vertical compaction of different parts of a sediment mass, as illustrated in figure 1B.

The reduction of aquifer pressures around a well, or across a well field, produces non-uniform stresses which may cause differential compaction. Prevailling opinion is that nearly all such compaction, or inelastic compression, occurs in fine-grained components of the aquifer system (Theis, 1935; Domenico and Mifflin, 1965) and consideration of the large differences between compactibility of clays and sands would tend to support this concept. Coarse sediments probably compact to near-ultimate density much more quickly than fine sediments under the same stress conditions, but if the fine-grained sediments are far from ultimate compaction, the coarse sediments probably are not fully compacted. In this case some compaction of coarse sediment should be expected to accompany compaction of fine sediment in an aquifer system. In figure 1C precompaction of sediments is assumed. Compaction of coarse sediment was 75 percent complete and compaction of fine sediment 50 percent complete at initiation of pumping.

Differential compaction may also occur where fluids escape from one part of the sediment mass more easily than from other parts. For example, clays between the thin sand stringers on the left side of figure 1B would be expected to compact more quickly than clays on the right side of the figure because contained fluids have a free path of escape through the sands. With time, the locus of maximum compaction would shift from left to right across the diagram until all sediments involved reached ultimate compaction. This process is essentially the natural analog of differential compaction produced by withdrawal of fluids, as discussed above.

A final cause of differential compaction is differential loading, either natural, as in the case of an alluvial fan building out over lacustrine sediments of uniform lateral composition and thickness, or artificial, as in the case of construction of one or more heavy structures on a uniform sediment.

The major causes, or mechanisms, of differential compaction can be summarized as: compaction of sediments of non-uniform thickness; compaction of sediments of non-uniform lithology or compressibility; and compaction of uniform sediments at unequal rates, either through loss of fluids at unequal rates or loading at unequal rates. Loss of fluids and loading may be either natural or induced processes, and there are probably very few cases of differential compaction which do ot involve all three factors simultaneously.

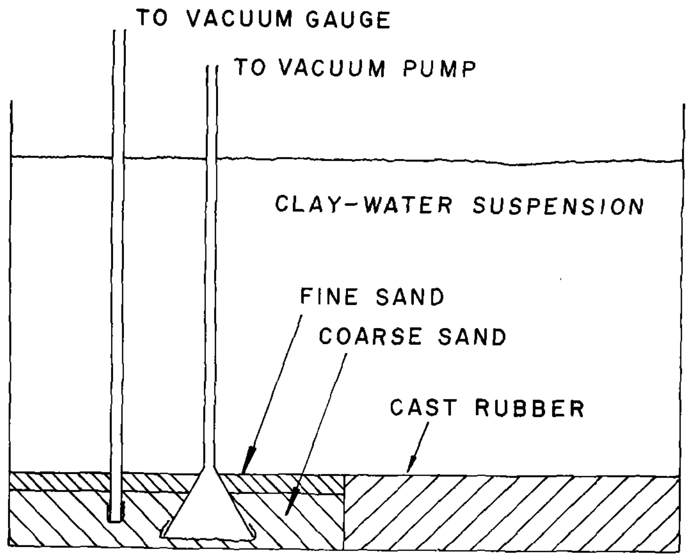

FIGURE 2. *Construction of differential compaction model*

DIFFERENTIAL COMPACTION MODEL STUDIES

Model studies of differential compaction were originally undertaken to determine whether differential compaction could account for very large contemporaneous-with-deposition faults of the Gulf Coast province of the United States. Model studies, and analysis of known features of the faults, appeared to support the hypothesis that differential compaction could be the single mechanism involved in development of growth-fault systems with vertical displacements of 1,500 m or more (Carver, 1968). On review of the Weaver and Sheets (1962) field guide, it became apparent that subsidence, or differential compaction, on the order of the observed 2 m would, by analogy with the large faults and models, produce the fractures, on the scale of 0.5 m, observed by Weaver and Sheets in the Houston, Texas area. If differential compaction is responsible for both the 1,500 m and 0.5 m fractures observed in the Gulf Coastel Plain, it is a geological process of fundamental importance with a range of scale as great as that of tectonic faulting.

The model designed to test the hypothesis that differential compaction would produce fracture rather than subsidence alone, differed from traditional clay-cake models in that the deformed medium consisted of a clay-water slurry and that stress was applied through the deformed medium by differential hydraulic pressure, rather than being applied to one surface of the medium.

An aquarium was fitted with permeable and impermeable bottom sections composed of graded sand and castable rubber (fig. 2). Prior to placing the sand, a funnel and a gauge tube were supported in the aquarium. The openings of both the funnel and gauge tube were covered with cloth to keep sand out. The inverted funnel was connected to an aspirator pump and the gauge tube to an inexpensive vacuum gauge.

Several mixtures of clay, water and water conditioners were used. Both ground Conasauga Shale (Cambrian, Georgia, USA) and commercial paper-grade koalin (Upper Cretaceous to Lower Paleogene, Georgia, USA) were used in the proportion 5 kg of ground clay to 7.5 liters of deionized water. The initial porosity of the clay-water mixture was, therefore, approximately 80 percent. In different experiments, plain deionized water, water with 35 parts per thousand sodium chloride as a flocculent, and water with 0.72 gm/liter sodium hexametaphosphate as a dispersant . Results of the model experiments were independent of the clay mineral or water solution used.

Water was added to the aquarium until the sand in the permeable bottom section was saturated and the funnel mouth and gauge tube filled to the level of the top of the permeable block. Clay-water suspension was introduced to the aquarium and allowed to stand for several hours. Some settling, or passive compaction, occurred during this period. If the funnel and gauge tubes were not filled with water and clamped off, prior to filling the aquarium, differential compaction developed as the tubes filled with water derived from compaction of sediment over the permeable block. The result of early, passive compaction in the model was minor subsidence over the entire area of the model, development of mud vents in the area of greatest compaction, and development of mud flows down the slope between the least-compacted and most-compacted areas (fig. 3).

Differential compaction of the clay-water suspension was produced by applying a vacuum to the permeable block. Flow through the aspirator pump connected to the inverted funnel was started and increased gradually until vacuum approximating 0.9 atm was indi-

FIGURE 3. *Surface of model after passive compaction. Note mud vents and mud flow. Mud flow moved from top to bottom of photograph, along the slope produced by the earliest differential compaction*

cated on the vacuum gauge. The 0.9 atm differential pressure (vacuum) developed within 5 minutes. Fractures appeared in the model surface within 3 minutes of first application of vacuum and continued to develop for at least 15 minutes (fig. 4), after which fracture scarps began to collapse and the experiment was concluded. Maximum differential compaction achieved was about 25 percent of the original sediment thickness.

In all trials of the experiment fracture patterns were similar. The fractures are essentially normal faults, downthrown on the most compacted side, which occur in zones of multiple and complex fractures. The model studies were specifically designed to test the hypothesis that fracture normally accompanies subsidence in differential compaction. The success of the model studies is strong presumtive evidence that the hypothesis is correct.

MECHANISMS OF DIFFERENTIAL COMPACTION FRACTURE

If we consider an aquifer system of uniform properties, with a well field in the center of the area producing differential compaction and subsidence, it would be possible to think of any horizontal slice of the aquifer system, above the locus of fluid withdrawal, as a beam being stressed by loading from above. If the slice were very thin, the effect of any differential compaction occurring within the slice itself could be ignored, and as sediments below the slice subsided, it would behave as a simply-supported, uniformly-loaded beam.

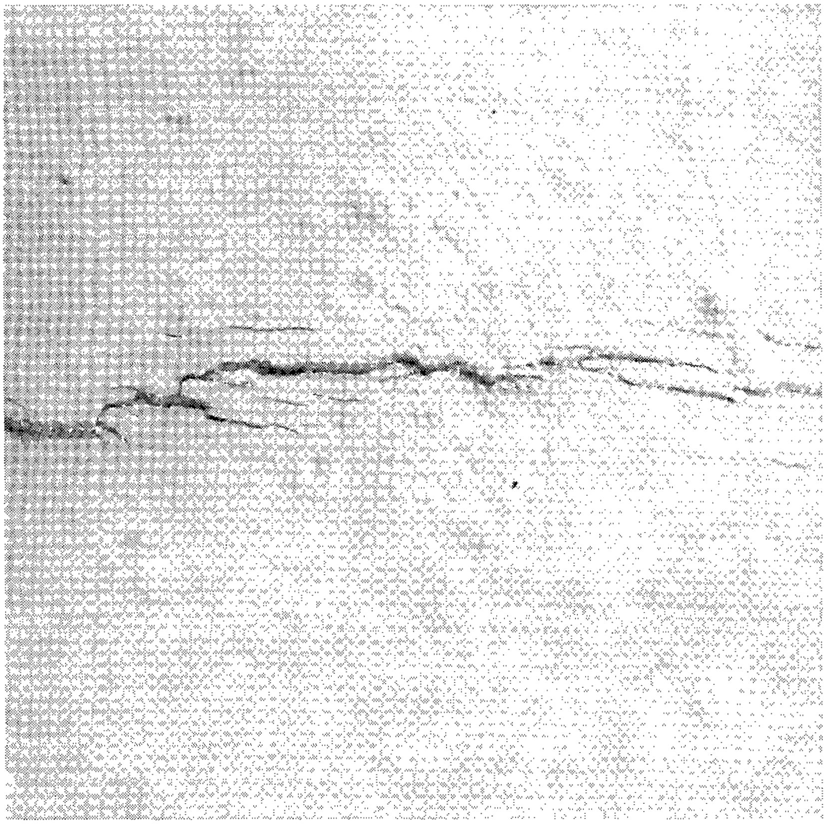

FIGURE 4. *Surface of model. Same area as fig. 3) after active compaction. Fractures shown developed 3 minutes after vacuum was turned on, photograph was taken 10 minutes after vacuum was turned on. Most-compacted side is toward bottom of photograph.*

Stress diagrams for a simply-supported and uniformly-loaded beam are shown in figure 5. Maximum shear stress occurs at the ends of the loaded section of the beam, regardless of how far from the ends of the loaded section the reaction points are. Location of the maximum moment, or fiber stress, is at the center of the loaded section, and always occurs at that point if the reaction points are equidistant from the ends of the loaded section

Failure of the beam, in tension or shear, will occur anywhere that the tensile or shear strength of the beam is exceeded. Because bending (fiber) stress is greatest near the center of the loaded section, first failures in tension will occur near the well, or center of the well field, causing differential compaction. Shear stress is greatest near the ends of the loaded section and first shear faillures will occur near the ends of the stressed section of the beam, or near the edges of the area subject to differential compaction.

Very similar arguments can be developed for several beam-loading and beam support analogies which might be applicable to differential compaction problems, and all lead to essentially the same result, maximum fiber stress occurs near the center of the stressed area and maximum shear near the perimeter of the stressed area.

It is commonly assumed that mechanical failure in an unconsolidated sediment would be plastic failure, but model experiments and observations previously cited indicate that

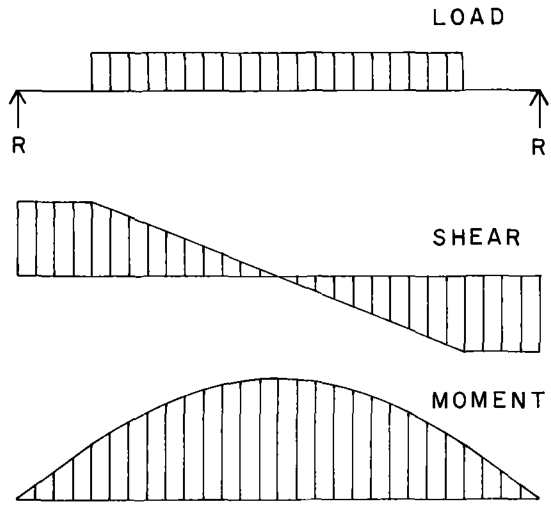

FIGURE 5. *Stress diagram. Reaction points are shown at positions away from the ends of the loaded section to illustrate the independence of maximum shear and location of the reaction points, so the beam illustrated is a simply-supported beam with a partial uniform load*

brittle failure of unconsolidated sediments is not only possible, but common. Whereas plastic failure may be the rule for sedimentary rocks subjected to high confining pressure and low strain rates, brittle failure appears to be normal in weak rocks subjected to low stress at reasonably high rates. If major fractures are present in the sediment prior to application of stress failure will occur along the pre-existing fractures, but this probably is an uncommon occurrence in ground water basins because of their relatively short geologic histories. Minor fractures, or physical discontinuities, and zones of lower than average strength have their counterpart in even the most carefully manufactured beam and, as in civil engineering, there will always be some degree of unpredictability as to the precise point of failure. However, given sufficient information on the physical configuration of an aquifer system, distribution of rock types within the system, and physical properties of the rock types (compaction characteristics, shear and fiber strengths), it should be possible to make reasonable approximations of the future subsidence and fracture of an area of ground-water development.

CONCLUSION

Carver (1968) has demonstrated that differential compaction fracturing occurs naturally in some geological conditions and, in the coastal plain of the Gulf of Mexico, is responsible for faults with displacements on the order of 1,500 m.

Fracturing and subsidence of sediments as a result of withdrawal of petroleum fluids from the Goose Creek Oil Field near Houston, Texas, has been reported by Minor (1925) and Pratt and Johnson (1926), and by Weaver and Sheets (1962) in connection with both oil fields and water-well fields in the same area. Yerkes and Castle (1968) note that faulting has occurred in connection with petroleum production in at least twelve United States oil fields. In addition, there have been several reports of fracturing as a result of groundwater withdrawal in the state of Arizona (see previous citations, Introduction).

Perhaps the most surprising aspect of fracturing as related to differential subsidence is that there are so many areas of major induced subsidence in which fracturing has not been reported. It is suggested that, in heavily populated, or extensively cultivated areas, evidence of fracturing associated with differential compaction is quickly obliterated by agricultural activity or repair of damaged construction; or is attributed to drought, local foundation problems, or regional tectonics. Strain rates may also be a major factor in the plastic or brittle behaviour of subsiding aquifer systems and future studies should consider this aspect of the subsidence-fracture phenomenon.

Until the processes of subsidence and fracture of well systems are better understood, precautionary measures to limit differential compaction should be exercised. These include spacing wells within a given area as widely as possible and limiting withdrawal as much as possible under given economic circumstances.

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ON THE COMPRESSION SUBSIDENCE OF PEAT AND HUMIC LAYERS IN THE KAMI-SHINBASHI AREA, KURATE-MACHI, KURATE-GUN, FUKUOKA PREFECTURE

T. NOGUCHI, R. TAKAHASHI and Y. TOKUMITSU

ABSTRACT

In the alluvial plain along the Onga River in the Chikuho Coal Field, there is an extensive distribution of the so-called SORA-SO (Peat layer), which has hitherto led to serious cases of mining damage as it invites subsidence due to the consolidation through dehydration from digging coal. The authors, who investigated the mining damage near and around Kamishinbashi in the central part of the coal field, reached the following conclusions: In this district there are distributed widely SORA-SO of some ten meter thickness and soft clay bed of various thickness. As it requires quite a long time for those layers to be consolidated completely the surface that had once been restored subsided again by the load of banked soil, and at the part where the thickness of soft clay bed changes rapidly, there has occurred irregular subsidence accompanied by inclination of the ground surface.

RÉSUMÉ

Dans la plaine alluviale, le long de la rivière Onga, dans la région minière de Chikuho, on trouve une distribution étendue de couches de tourbe (SORA-SO), qui ont provoqué jusqu'à présent des dommages miniers sérieux, du fait des affaissements produits par la consolidation due à la déshydratation de ces couches par l'exploitation minière. Les auteurs qui ont étudié les dégâts miniers près et autour de Kamishinbashi dans la partie centrale du bassin minier, sont arrivés aux conclusions suivantes :

Dans ce district, il existe des couches de tourbe de quelque dix mètres d'épaisseur ainsi que des lits d'argile fluente de diverses épaisseurs. Comme la consolidation complète de ces couches exige un temps très long, les affaissements reprennent sous l'action de charges de surface et là où l'épaisseur d'argile fluente change rapidement, il se produit des affaissements irréguliers accompagnés d'inclinaison de la surface du sol.

I. INTRODUCTION

The area of this report is situated in the north-western part of the Nogata district of the Chikuho Coal Field. The greater part of the area is a low flat land embraced between the Onga river and its branch, the Nishi river. To the south and south-west the land becomes hilly, with hills as high as 30-40 m. The flat land reaches northward to a seashore, forming the so-called Onga flat land, with scattered small hills of the Diluvium. Since early days, the area as a whole had been the scene of extensive coal mining. Indeed, since 1918 the coal seams of the Sanzyaku-Goshaku formation of the Paleogene Nogata Group had been worked by the Mitsubishi Coal Mining Co. as Pit No. 6 of the Shinnyu Colliery, but in 1946 the mining work was terminated. Damages due to the mining operation during that period, i.e. inclination of houses and the ground surface or exhaustion of water, occurred directly after mining work. In 1954 and 1961 these were throughly restored to their original states. About 1960, however, the inclination of floors and pillars of houses and cracking of walls began to occur, simultaneously land subsidences recurred again. The causes of the new damages are thus investigated from the viewpoints of mining conditions, the geology of the area, and soil mechanics.

II. OUTLINE OF GEOLOGY

The Cretaceous Wakino Subgroup and the Hirao Granodiorite constitute the basement rocks of the present area. The latter, which is also of Mesozoic Age, intruded into the former and more or less thermally metamorphosed surrounding rock. The Paleogene strata are divided into two groups, the Nogata and Otsuji in ascending order, and these are further subdivided into 4 and 2 formations, respectively.

The lowermost formation of the Nogata Group, the Oyake lies disconformably on the basement rocks, and is distributed in a narrow belt from Mt. Tsurugi to Michinaka. Although it is an important coal bearing formation in the Chikuho Coal Field, it is not well developed in the present area and thins out in the northwestern part. The Sanzyaku-Goshaku formation, which conformably overlies the Oyake, has the most important coal bearing beds, actually coal seams, which attain 12 m in total thickness and are concentrated in the strata 20-30 m thick. It is exposed in the northeastern side of the Oyake formation, but as in the case of the Oyake it thins towards the north and finally pinches out.

The Takeya formation, lying conformably over the Sanzyaku-Goshaku, comprises two sedimentary units, the lowermost is a thick pebble-bearing sandstone, and the uppermost is a coal seam called "Takeya". This formation also decreases in thickness 50 to 100 m toward the north, as do the underlying two formations. Its rock facies changes from coarse—to fine—grained, and finally the sediments become rich in tuffaceous matter. A greater part of the Uwaishi formation, the uppermost of the Nogata group, is covered with Alluvium, except in the southern and south-western part of the area. It is therefore impossible to know the detailed succession and lithologic characteristics of this formation from surface investigation. According to the data obtained from borings, however, the Uwaishi formation conformably covers the Takeya formation and is distributed widely on the northeastern side. It attains a thickness of more than 250 m. The lower part of the formation composed mainly of conglomeratic coarse-grained sandstone, and the upper, in which a few coal seams are intercalated, is composed of grey or brown shale and fine—to medium—grained sandstone. With a slight disconformity the Otsuji Group lies on the Nogata Group, and is divided into the Ideyama and Onga formations in ascending order. Because the distribution of this group is limited only to the tract of this area, comments on the group will be omitted here (fig. 1).

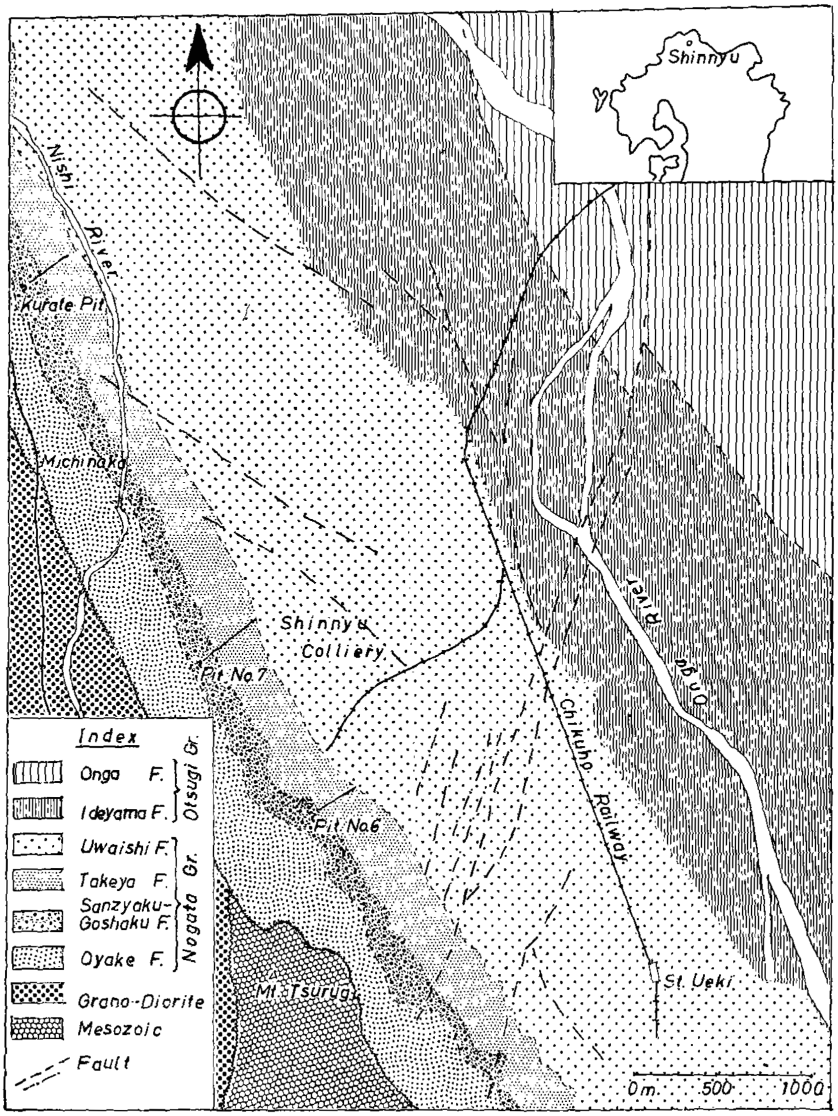

FIGURE 1. *Geological Map faced to Quaternary Base*

As mentioned before, the overburden covering the Paleogene is the Diluvium and the Alluvium. No comments will be given on the former here, since its distribution is extremely limited and has little relation to the problems discussed in this paper. The latter, on the other hand, is noteworthy because it is a fluviatile deposit of an unusual rock facies containing peat and humic clay.

The alluvial deposit constitutes a superficial layer covering the alluvial plane, and covers nearly the half of the present area. As evidenced from its extensive distribution along the Onga and the Nishi river, it is clearly a fluviatile deposit of these rivers. Peat and humic clay beds intercalated in the Alluvium are better developed in the area close to a hilly part than in the present river ways. This same tendency of distribution is recognized also in the whole flat area of the Chikuho Coal Field. The peat and humic clay, called "so-ra-so" and muddy "so-ra-so" respectively, are distributed in a considerably irregular pattern, which may be the results of the meandering of the Onga river. Figure 2 shows boring localities of the area, and representative columnar sections taken from the boring cores. The sections demonstrate that the "so-ra-so" and humic clay beds are distributed quite irregularly, and rapidly merge both horizontally and vertically into sand and gravel beds. These beds vary greatly in thickness, being 20-30 m thick in some places and only a few centimeters thik in other. Frequently a humic layer diverges horizontally into 2-3 sheets, with intercalation of sand layers between them.

III. COAL MINING WORK AND SUBSIDENCE

As mentioned before, coal seams of the Sanzyaku-Goshaku formation were worked throughout this area. A representative profile of the coal seams is shown in figure 3, of which the "Kan-kan", "Goshaku" and "Sanzyaku" were mined at pits No. 6 and 7, as well as at the Kurate pit of the Shinnyu colliery.

Pit No. 6 was opened in June 1915, and through this pit the above mentioned coal seams lying directly under the area of this report were mined during 1925-1928, 1935-1936 and 1945-1946. Coal of the deeper levels, however, was dug until 1918, and then the pit was closed. Pit No. 7 was opened in 1918, and coal dug from the shallow levels between the years 1931 and 1937, but digging of the deeper levels was carried out by August 1960. By the way, the Goshaku coal seam was mined in 1963 in opencut at the north side of pit No. 7. The Kurate pit located at the northside of pit No. 7. was opened in November 1934 and closed the work in April 1926 after digging coals of the northern part. Each pit was operated mostly by the long wall method but in some places by the pilar method. The depth of the working face in the area was more than 35 m below the ground surface. The shallower the working face is, the earlier the damage due to coal mining works appeared. Damages were clearly recognized in this area since about 1930. The extent, amount, and period of the land subsidence due to the underground mining work, of course, depend upon the thickness of the coal seam to be dug, the depth of digging, the working method, and the geological conditions such as rock facies etc. It is expected, from findings gathered, that the land surface in this area was stabilized within 2 years of the stopping of mining. The restitution works that were undertaken after 1949 might therefore be said as timely. But since the Alluvium is composed generally of saturated unconsolidated sand and clay, subsidence caused by dewatering and compaction of such soft sediments did not come to the end, even after the land subsidence of the Tertiary hard rocks, resulting directly from the mining work, had finished.

During the period from 1949 to 1955, the mining in the deeper working faces in the eastern side of the area continued; there still remains, therefore, a possibility of further subsidence caused by lifting of water from the mines. As the rate of compaction of soft sediments depends on the natures of soil, the land subsidence will be discussed from the viewpoint of soil mechanics.

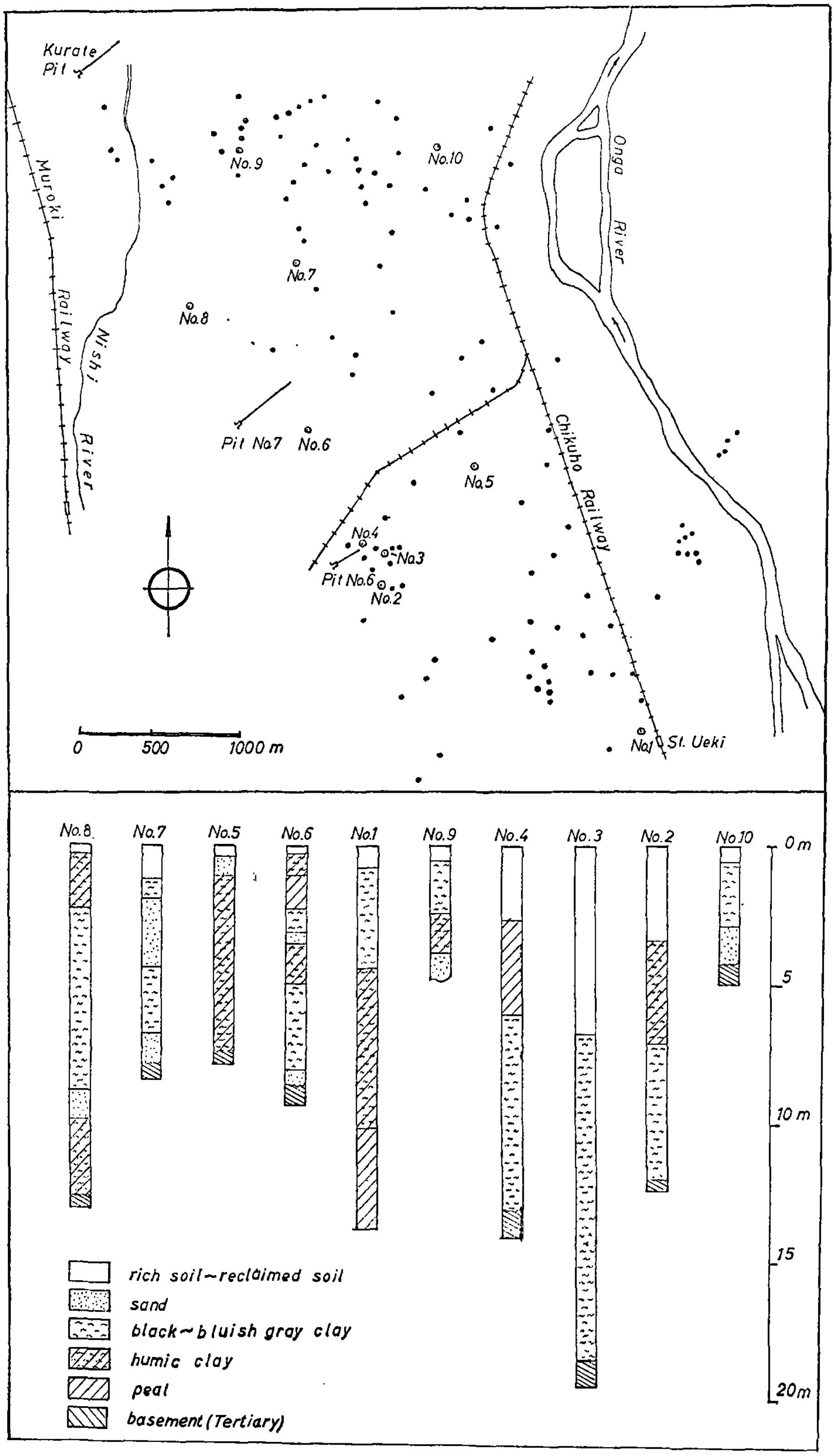

FIGURE 2. *Localities of Borings and Columnar Sections.*

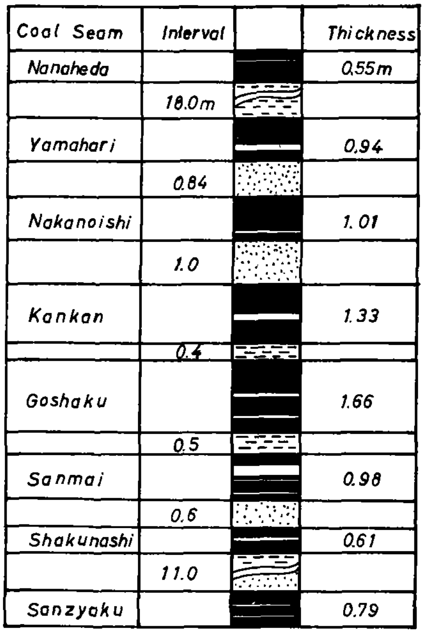

FIGURE 3. *Standard columnar section in the Shinnyu Colliery*

IV. RECURRENCE OF SURFACE SUBSIDENCE AND SOIL CHARACTERISTICS

The Alluvium forming the surface layer of this area is composed of peat ("So-ra-so") clay and sand, all of which are irregularly distributed, both vertically or horizontally. Sand layers are mostly thin, whereas peat or humic clay layers are rather thick; these comprising a great proportion in the Alluvium.

The fundamental properties of the undisturbed samples obtained from several borings are given in table 1.

As clearly shown in table 1 and figure 4, the "so-ra-so" and humic clay can be judged as being liable to cause subsidence by compaction. Especially, "so-ra-so" which has a void ratio as large as 4, although its compression index is small. Hence it is considered that if the thickness of the peat and humic clay layer to be compressed is large, the subsidence will continue for a long time.

As to the time required for consolidation, the following equation holds:

$$t = T\frac{H_1}{C_v}$$

where:

t time required for consolidation sec;
C_v consolidation factor ... 25×10^{-4}cm/sec;
H_1 thickness of peat or humic clay bed ... 1000 cm;
T time factor.

Substituting these values we get

$$t = 1.27 \times 10\ T\,(\text{year})$$

TABLE 1. Fundamental Properties of Peat* *and* Humic Clay

		GANDA I	GANDA II	NAMAZUTA*	NOGATA	KAMISHINBASHI I*	KAMISHINBASHI II
Depth (m)		0.75 ~ 1.35	1.60 ~ 2.00	3.30 ~ 4.30	2.07	3.00 ~ 4.00	5.00 ~ 6.00
Sp. Gravity		1.64	2.08	2.02	2.33	2.24	2.15
Gransize (mm)	2 - 0.074	71	77	54			3
	0.074 - 0.005	19	14	31			66
	0.005 -	10	9	15			31
Water content ratio (%)		909	381	233	268	187	168
Void ratio		16.8	7.98	4.63	5.95	4.63	4.03
Apparent density		0.93	1.12	1.16		1.14	1.15
Pre-Consolidation load		0.08	0.083	0.22		0.33	
Compression index		15.2	7.42	1.96		1.95	

The relation between time factor T and consolidation ratio U is as follows:

U(%)	20	40	60	80	90	95
T	0.03	0.13	0.29	0.57	0.85	1.15
t (year)	3.8	16.5	37	72	108	146

It is seen from the calculation (it takes more than 100 years for 90% consolidation) that "so-ra-so" and humic clay require quite a long time for their complete consolidation, even if we use 25×10^{-4} cm/sec for the consolidation factor, which is an average value for clay.

Since the Alluvium contains very few beds of sand or gravel, which are considered to be permeable strata, nothing is known of the extent the "so-ra-so" and humic clay has dewatered. But it is reasonable to assume that the condition for dewatering once, occurred, because the basement rock below the Alluvium probably cracked due to the exploitation of coal directly below the present area. Table 2 shows the comparison of the pre-consolidation values obtained experimentally from boring samples with those calculated theoretically. From this table, about 2 m lowering of the underground water level is estimated. This value, however, is the amount of lowering under the current conditions. Undoubtedly the value was greater when there was no banked soil. Consequently dewatering of "so-ra-so" and humic clay layer must be caused by the drainage of the coal mines. From the relation between the degree of consolidation and the required time, mentioned above, we estimate that about 40% of the surface subsidence due to consolidation occurred within about 20 years from digging to restitution work, and that there was yet the possibility that 50% of the consolidation subsidence would occur after restoration work. There are refuse heaps piled over the Alluvium in the northern part of Kamishinbashi-machi. For restitution of the ground surface, these refuse heaps were excavated and banked on the subsided surface. Recurrence of surface subsidence has taken place on the

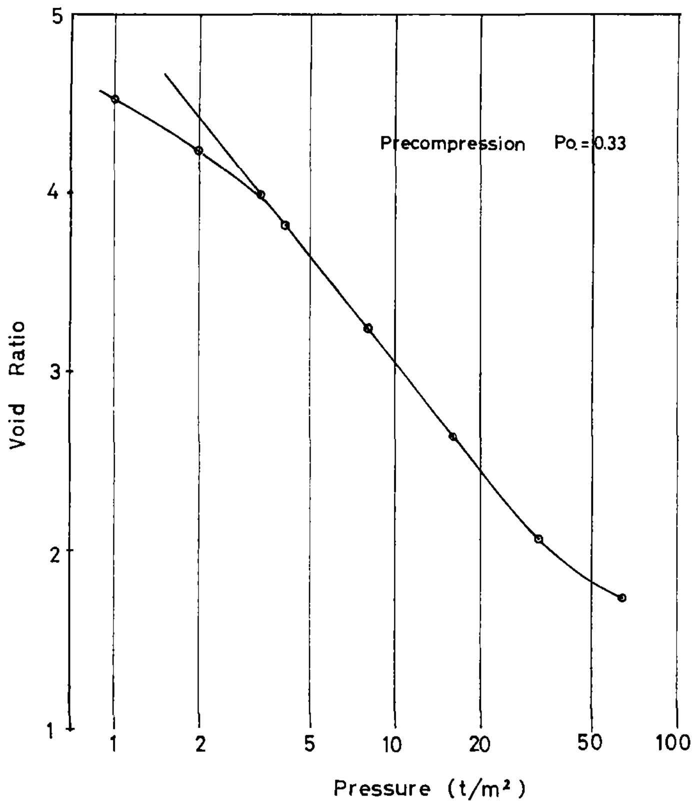

FIGURE 4. *Compression diagram for Peat-Humic clay, Kamishinbashi*

Table 2 Comparison of theoretical— with experimental preconsolidation load

Boring No.	Depth	theoretical	experimental
No. 3	9 - 10m	3.37 t/m²	5.0 t/m²
	11 - 12	3.97	6.2
	13 - 14	4.57	7.0
	15.5-16.5	5.32	7.4
No. 4	3 - 4	1.12	3.6
	5 - 6	1.87	—
	7 - 8	2.31	4.4
	9 10	3.11	6.8

ground surface of residential lots which were releveled, but not on the places where refuse heaps once stood. This fact indicates that consolidation subsidence was caused not only by dewatering of the peat layer, but also by the load of banking, and in this case the thickness of banking amounted to 2-3 m in average. Recurrence of subsidence after releveling was therefore caused by dewatering and banking.

Though we cannot decide immediately which process, banking or dewatering, caused the greater surface subsidence. The former may be said to represent a much greater factor than the latter, because the higher the banking of house lots, the greater the damages of houses, and because those house lots constructed after closing of mining are subjected to a considerable amount of subsidence.

V. CONCLUSIONS

In this area, surface subsidence once occurred due to coal exploitation, and after the ground was stabilized restitution work was carried out several times. Thickness of banking is estimated to be 2-3 m in average. From the viewpoint of soil engineering, the "so-ra-so" and blueish-gray humic clay are so loose that they are liable to cause consolidation subsidence or flow phenomena. For that reason, even after the stabilization of the ground base, the surface is subject to the reoccurrence of subsidence resulting from the dewatering by drainage at deeper working face, and also from the consolidation due to the load of banked soil for restitution work. The authors consider that surface subsidence is affected more by the load of banked soil than by dewatering of the alluvial deposits, because the same amount of surface subsidence is observed in the house lots built after the stopping of drainage, and also, the amount of subsidence is greater where the thickness of banking is larger. It is therefore impossible to carry out a perfect restitution work where such loose deposits as the "so-ra-so" are thickly distributed and where subsidence will be accelerated by banking. The "so-ra-so" and humic clay, which are here widely distributed along the Onga River and its branches, are also present in almost all parts of the low-land of the Chikuho Coal Field. The restitution of ground surface in the Chikuho Coal Field, therefore, will have to be repeated, because the surface subsidence will continue for some time, until the Alluvium is completely consolidated.

ACKNOWLEDGMENT

We are indebted to the members of the Fukuoka Branch of the National Commerce and Industry Bureau for their kind assistances to our field investigation.

Thanks are also extended to the Mitsubishi Coal Mining Co. for providing considerable boring data carried out in the present area to determine the characteristics of the superficial layers.

SMALL SINKING HOLES IN LIMESTONE AREA WITH SPECIAL REFERENCE TO DRAINAGE OF COAL MINES

T. NOGUSHI, R. TAKAHASHI and Y. TOKUMITSU

ABSTRACT

The area investigated is situated close to the south-eastern end of the Tagawa section of the Chikuho coal field. The strata forming this neighborhood involve the Quaternary, the Tertiary and the Paleozoic and the land subsidence of this area is traced in numerous occurrences of small sinking holes of about 4-10 m in diameter. The site where sinking holes have occurred is composed of limestone which is directly covered with the Quaternary of 2-5 m thickness. The Tertiary which is distributed in the south-eastern part of the area bears coal seams and was once worked by several collieries. According to the survey of groundwater level, it was found that sinking holes are distributed mainly in the concave part of groundwater surface. It is known that numerous cracks, dolina and caves lie concealed in surrounding limestone, and this area is no exception to that. Sinking holes are considered to have been caused by the change of water level and velocity of groundwater which is flowing through caves and cracks in limestone. When groundwater spouts in the Quaternary eddy currents occur, and surface soil is collapsed. Phenomena similar to that were confirmed by model experiment. The cause of the change of groundwater can be traced back to the pumping of water at collieries in the south-eastern area, but the change still goes on even after those collieries were closed owing to the pumping of gob-water from the wells drilled for irrigation purpose, and subsidence is continued as ever.

RÉSUMÉ

La surface étudiée est située près de l'extrémité Sud-Est de la section de Tagawa du bassin charbonnier de Chikuho. Les couches constituantes sont du Quaternaire, du Tertiaire et du Paléozoïque et l'affaissement du sol dans cette zone est constitué en de nombreux endroits par de petits puits d'environ 4 à 10 m de diamètre. Le terrain où se produisent ces puits est constitué par un calcaire qui est directement recouvert par une couche de Quaternaire de 2 à 5 m d'épaisseur. Le Tertiaire qui existe dans la partie Sud-Est de la région porte de faibles couches de charbon qui ont été autrefois exploitées par plusieurs charbonnages. L'étude des niveaux d'eau souterraine montre que les puits d'affaissement sont surtout distribués dans la partie concave de la surface de l'eau souterraine. On sait que des fissures, des dolines et des cavernes existent dans le calcaire et la région en question ne fait pas exception à cette règle. Les puits d'affaissement sont considérés comme ayant été causés par les changements du niveau de l'eau souterraine et des vitesses de l'eau qui coule par les crevasses et les cavernes du calcaire. Quand l'eau souterraine pénètre dans le Quaternaire, des courants très turbulents se produisent et le sol s'effondre. Des expériences sur modèles ont confirmé ces considérations. La cause des changements de niveaux de l'eau souterraine peut être recherchée dans les pompages de l'eau des houillères dans la région Sud-Est, mais les changements se poursuivirent après la fin de l'activité des houillères du fait des pompages par puits pour irrigation et les affaissements continuèrent.

I. INTRODUCTION

In Itoda-machi, which is located in the south-eastern part of the Chikuho coal field, small sink holes began to break out on the surface in 1952—accompanied by damage such as inclination of houses and collapse of walls. Approximately 90 holes have subsided in the 17 years since 1952. Sometimes sinking occurred gradually and other times quite suddenly without any warning. Although the caved-in sites have been filled in, some of them have started sinking again. Though there is no regularity in the occurrence of sinking, it is said that more sites have caved during the dry season than during the wet season. Many of the sink holes are small and almost round, being 2-4 m in diameter and 1-3 m in depth. The largest one has an oval shape, 12 m in its long span and 4 m in its short span, and 4 m in depth.

This area is situated in a limestone zone. There were about 200 springs earlier, but at present the number has decreased considerably due to lowering of the underground water level. The representative spring in this area is Tagiri spring, which formerly supplied sufficient to irrigate a paddyfield of 2×10^6sq m but now is almost dry. The occurrence of sink holes is restricted to the inside of the spring area.

Wells drilled into the soil layer hold water in them but wells drilled into the bed rock often dry up ; once a cavity was found when a well was drilled deeper.

II. OUTLINE OF GEOLOGY

The basement forming this area is composed of paleozoic limestone stretching from Mt. Kawara to Mt. Funao. Usually alluvium directly covers the basement, but in a few cases diluvium or the Tertiary are found on the Paleozoic. The soil layer, including the alluvium and the diluvium, is as thick as 3-7 m and is composed of clay, gravel contained sandy clay, silt and sand-gravel layer.

The base part of the Tertiary is distributed only at the concaved place of limestone, and its thickness increases gradually as it turns away to the east, and where it contains coal seams. There are no coal seams in the area where sink holes occurred so exploitation of coal was not carried out there. In the Tertiary in this area there are found cylindrical faults (1). In the coal pits, a circular crushed zone scores of meters in diameter is often encountered. The existence of calcite is found at the fault plane, and probably is a secondary deposit originated from the limestone. The crushed zone is considered to have been formed when the hangingwall of the limestone cavity fell or eroded and the Tertiary over the hangingwall collapsed. A crushed zone of this kind is found only in the Tertiary and has not reached to the surface. The phenomena of the sinking of the soil layer considered in this paper is smaller in scale than the crushed zone, and its mechanism of formation is quite different from the latter.

III. FACTORS INFLUENCING UNDERGROUND WATER

1. Relation between ground-water and surrounding coal mines

Relative positions of the damaged area and the surrounding coal mines are shown in figure 1, and working and drainage conditions of each coal mine are given in table 1. As seen in the table, the surrounding coal mines encountered the protruced part of limestone during mining operations, causing flooding accidents. In 1939, when Kigyokomatsu coal mine flooded (23 cu m per min), the water at Tagiri spring dried up. This spring recovered after the pit was abandoned. A similar phenomenon occurred in the case of flooding (10 cu m per min) at Hokoku coal mine.

It has been confirmed by underground tunneling or boring that limestone in this area is distributed extensively from Mt. Kawara to Mt. Funao. Since limestone contains many cavities and cracks, it is reasonable to assume that ground-water in the damaged area flows into the underground mining through these channels. Furthermore, we cannot say there is no relation between daily drainage and lowering of ground-water level, though the drained water is rather small in quantity.

2. Ground-water level in the damaged area

A long-term observation was carried out on the water-level of 4 newly-drilled boreholes and a large number of domestic wells. As seen in figure 1, there is a strip of ground-water valley running from south-west to north-east in this area. The line of underground water-level generally presents the shape of a mild contour of the terrain, but in this area the line has a shape quite different from that of terrain or basement. The water-level of each

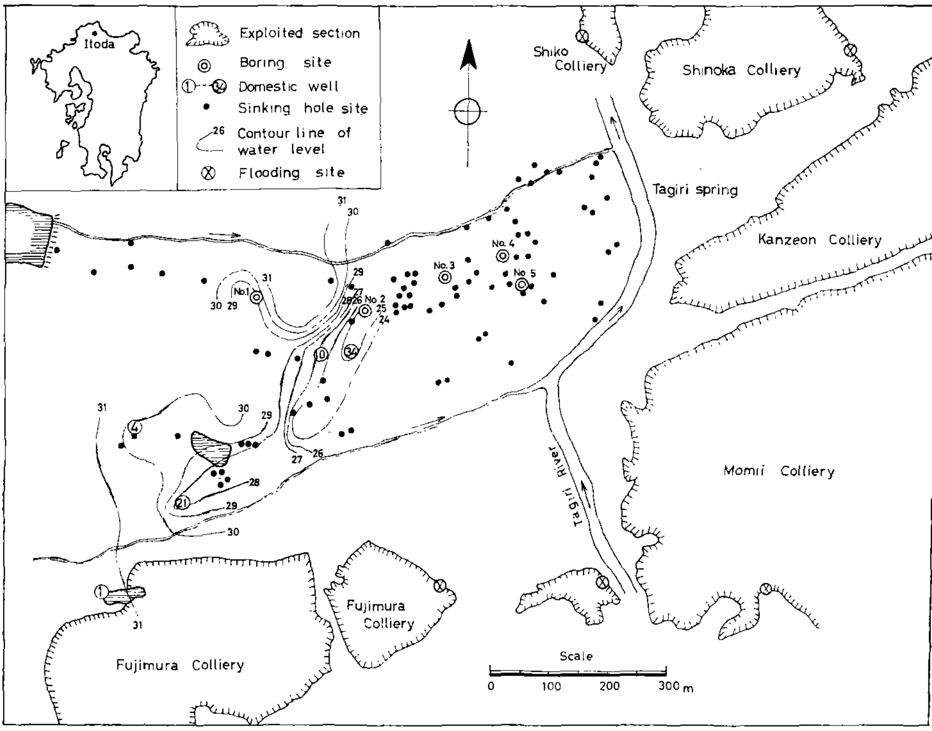

FIGURE 1. *Damaged area and surrounding collieries*

domestic well has suffered little effect from rain, as seen in figure 2, but well number 33 shows sudden rise of water-level after a rain. This seems to be due to the existence of a crack in limestone near the well. It may be understood that the water-level of the mountainous district behind rose after a rain, affecting well No. 33 directly. Well No. 33 is situated in the underground water valley, and the valley is closely related with the crack in limestone.

3. Relation between ground-water level and pumping-up of water for irrigation

Four deep wells provide water for irrigation to the gob of Momii coal mine. Figure 3 shows the change in ground-water level when water is drawn from these wells at the rate of A(10 l per sec); B(5 l per sec); C(7 l per sec); D(6 l per sec). Borehole number for observing water-level is located in the damaged area separated from the place where coal was once dug, as shown in figure 1. The ground-water level shows a tendency to rise after a rain and fall when the pump is operated, which tells us that lifting water from the place where coal was dug causes the descent of underground water-level down in this area.

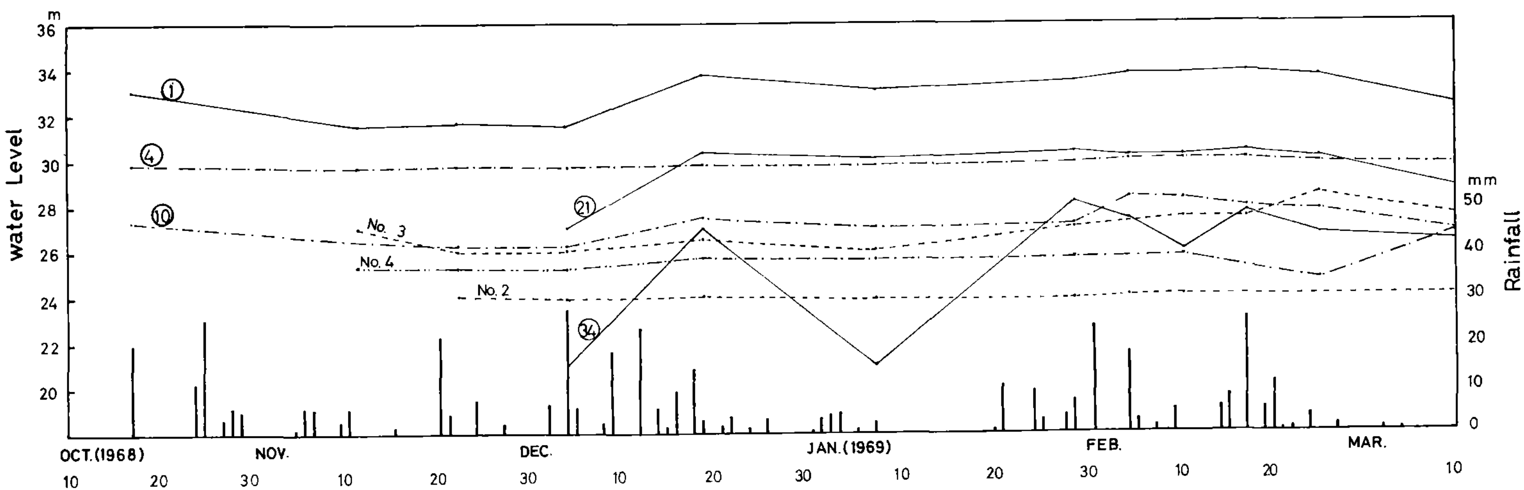

FIGURE 2. *Relation between rainfall and the water level of wells*

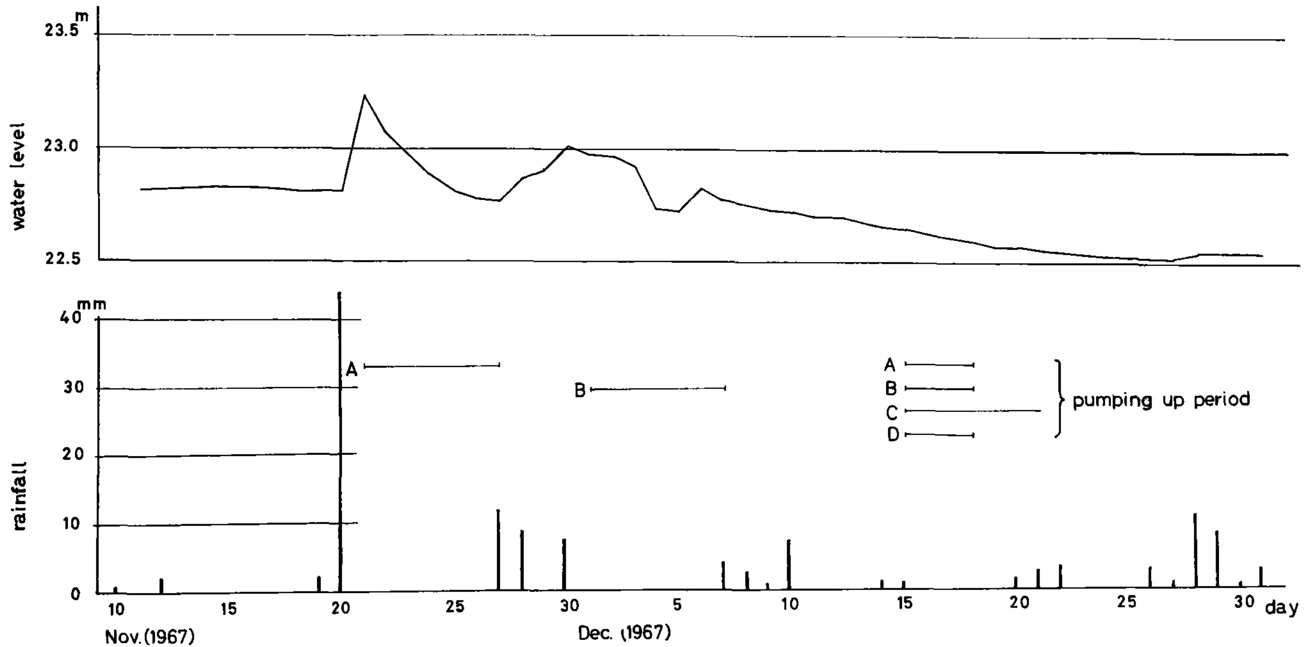

FIGURE 3. *Influence of rainfall and gob water drainage to water level of observation well*

TABLE 1. Exploiting State of Surrounding coal mines

Coal mine	working period	amount of drainage m^3/min	amount of drainage at flooding accident m^3/min	exploited coal seam
Momii	1948 ~ 1962	2.70		Sunazakai Shingoshaku Mumei
Kanzeon	1949 ~ 1959	4.05	2.61	Sunazakai Shingoshaku Yoshinoya
Fujimura	1944 ~ 1958	0.08	2.43	Sunazakai Shingoshaku Yoshinoya
Shinoka	1948 ~ 1958	1.22	1 35 ~ 1.62	Sunazakai Shingoshaku
Hokoku	1938 ~ 1951	8.10	10 80	Sunazakai Shingoshaku Yoshinoya
Kigyokomatsu	1931 ~ 1938		27 00	Yoshinoya

IV. MODEL EXPERIMENT

The experimental apparatus is as shown in figure 4 (2). The experimental tank A is as large as 80 cm × 40 cm × 30 cm, and its front is covered with glass so that the condition inside the tank may be observed. Sand and mine waste were used for samples, and the results of physical tests are given in table 2. The amount of flow was controlled with a value after water was pumped up from the water tank. Figure 5 is the result of a test where sand was used for the sample and water pressure was controlled by a 5 cm mercury column. Inside the solid line near the water discharge pipe the sample is flowing with water, and the broken line drawn around it shows the range of water penetration. The water flowing out of the discharge pipe has penetrated into the surrounding part of the outlet, whereas in the penetrated part, the upper part has become more loose than the lower part, and the

TABLE 2. Grain size of the Sample of model experiment

		Coal mine waste	sand
Specific gravity		2 60	2.65
Apparent density (t/m^3)		1 70	1.58
Water content ratio (w_t %)		35 00	38.40
Grain size %	~ 2.0 mm	39 00	6 40
	2.0 ~ 0.25 mm	15.20	78.60
	0 25 ~ 0.05 mm	18 80	13 00
	0 05 ~ 0 005 mm	16.00	1.30
	0.005 ~ 0 001 mm	8.00	0.50
	0 001 mm ~	3 00	0 20

water form the pipe flows upward where resistance is small. The sample in the upper part of the water flow is gradually washed out and left in the downstream. Since the volume of settled sample is smaller than its original state, a cavity grows in the neighborhood of the upper stream. The cavity gradually moves upward, ultimately letting the surface subside.

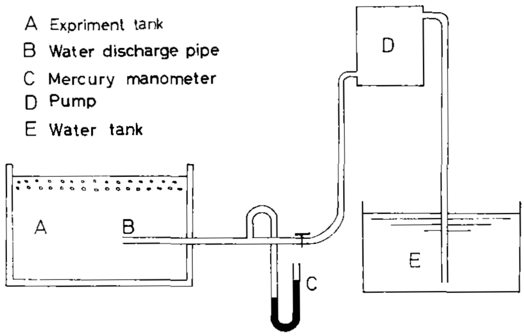

FIGURE 4. *Model experiment apparatus*

Similar phenomena are seen when mine waste was used, but the ascent of the cavity is slow as it requires larger water pressure than in the case of sand. When a clay pipe is buried in the sample, its bent portion is broken, and the ascending water current due to the water flowing out of clay pipe gives rise to circular subsidence, as in the case of metal pipe.

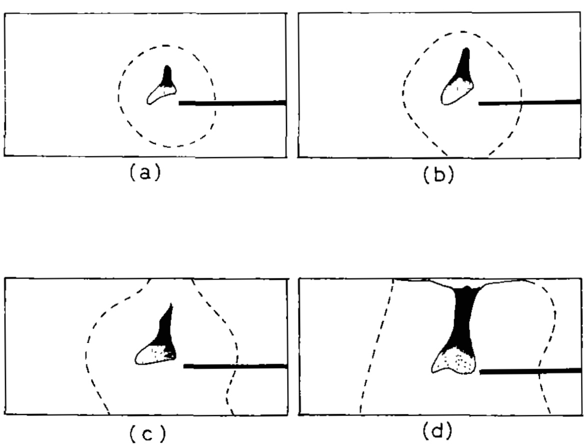

FIGURE 5. *Showing the growth of cavity in sand*

V. DISCUSSION ON THE CREATION OF SINK HOLE

1. NATURAL PHENOMENA

Since there are many cavities and cracks contained in limestone and Mt. Funao is higher than the damaged area by 200 m, the underground water in this area is subject to considerable water pressure. In view of the rise of water level at Mt. Funao after a rain, water pressure naturally will be increased. When there is a crack in the surface of the limestone underground water flows out through the crack into the soil layer.

When the soil is loosely packed and its resistance to flow is small, it is presumed that there occurres a phenomenon similar to the model experiment and sink holes will be generated. On the contrary, when soil is compact and its resistance is large, piping phenomena will occur, along the loose part of soil layer. Existence of such piping in the soil layer was observed when the banks of the Tagiri River collapsed.

In the soil layer of this area, a compact part and a loose part lie one upon another alternately. Piping provides a favorable condition for ground-water to pass through the compact part of soil. Formerly there were about 200 springs in this area, and these are the sites depressed as a natural phenomena in the course of many years.

2. Influence of mine water drainage

Since the damaged area is located outside the critical angle, we cannot think of any direct effect due to coal exploitation. However, it is natural to consider that generation of sink holes was influenced by mine drainage, because the working period of coal mines and the time of occurrence of sink holes are in perfect coincidence and water has dried up in some wells. Daily mine drainage and flooding accidents from the basement consequently have served to increase the flow rate of underground water running through the soil layer and encouraged the generation of sink holes.

As a natural phenomena, only about 200 springs have been formed for a long period of time, whereas during the working period of the coal mines, about 90 sink holes have been brought forth in 17 years, which is due to the above-mentioned reason.

VI. CONCLUSION

This area belongs to the zone of limestone and was formerly the noted site of springs Recently, underground water level has dropped an numerous sink holes have been formed in the area. This is due to the lifting of water at the surrounding coal mines. Limestone, in this area has many cavities and cracks and lots of piping is formed in the soil layer. These all serve as channels for underground water. The drainage of mine water or the pumping-up of artificial gob water has increased the flow rate of underground water and encouraged the formation of sink holes.

ACKNOWLEDGMENTS

We thank the Fukuoka Commerce and Industry Bureau for supplying many valuable. data.

We also thank Mr. M. Takino, Ehime University, for assistance on model experiment.

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CONSOLIDATION PHENOMENON CAUSED BY MINE DRAINAGE IN THE AREA OUTLYING FROM WORKING PLACE

T. NOGUCHI, R. TAKAHASHI and Y. TOKUMITSU

ABSTRACT

This area is situated at the west end of the Saga alluvial plain, and its northern hills are composed of Tertiary. In the Kishima collieries they are working coal at the underground of the northern part of the plain, the surface has subsided in places so detached from the working spot that it is impossible to explain by ordinary draw angle. Here the Omachi fault (throw 300 m, inclining southward) runs from east to west, and surface subsidence stretches nearly along this fault line.

The alluvium is composed of the alternation of clay and gravel beds, and its thickness is 40-50 m on the north of the fault and 350-400 m on the south side. The alluvium is formed in order from the surface, the first clay bed, the first gravel bed, the second clay bed, the second gravel bed, the third clay bed and the third gravel bed.

It has been made known from the results of soil testing that the main cause of subsidence on the north side of the Omachi fault is in the consolidation of the first clay bed, but that on the south side is in the consolidation of the third clay bed. In this area there are several faults running from north to south, through which the Omachi fault is connected directly with the underground working place of the Kishima coal mine. This fact has been proved by the chloric ion test of underground water near and around the Omachi fault and mine water.

RÉSUMÉ

La région en question est située à l'ouest de la plaine de Saga et ses collines du nord sont constituées par du Tertiaire. Dans les collines de Kishima, on exploite le charbon dans la partie nord de la plaine. Le sol s'est affaissé en des endroits si éloignés des endroits d'exploitation qu'il est impossible d'expliquer ces affaissements en utilisant l'angle habituel. Dans cette région, la faille d'Omachi (rejet 300 m, s'inclinant vers le sud) s'étend de l'est à l'ouest et l'affaissement superficiel s'étend à peu près le long de cette ligne de faille.

L'alluvium est constitué de produits d'altération de l'argile et de gravier dont l'épaisseur est de 40-50 m au nord de la faille et de 350-400 m au sud de celle-ci. L'alluvium est constitué en partant de la surface d'une première couche d'argile, suivie d'une première couche de gravier, puis une seconde couche d'argile et après une seconde couche de gravier, une troisième couche d'argile et une troisième couche de gravier.

Les résultats de l'étude du sol ont montré que l'affaissement au nord de la faille est dû principalement à la consolidation de la première couche d'argile, alors qu'au sud de la faille, il est dû surtout à la consolidation de la troisième couche d'argile. Il existe dans cette région de multiples failles courant du nord au sud, permettant à la faille d'Omachi d'être directement reliée avec les travaux des mines de Kishima. Ce fait a été établi par l'essai à l'ion de chlore dans l'eau souterraine près et autour de la faille d'Omachi et dans l'eau de la mine.

1. INTRODUCTION

Kohoku-machi is a town in the western part of Saga Prefecture, centering around Hizenyamaguchi Railroad Station, which constitutes a junction of Nagasaki Main Line and Sasebo Line. There is a mountainous zone of the Tertiary in the north of the town and in the south there is extensive and well-developed plains of alluvial deposits stretching as far as the Ariake Bay. The alluvial soil along the Ariake Bay is known to be rather thick clay. The Ushizu River runs to the east of Kohoku-machi and the Rokkaku River to the west, both of which have brought forth fertile granary zones. The Kishima Coal Mine in Omachi-machi is situated west of Kohoku-machi, and the coal mining has gradually progressed from west to east, that is toward Kohoku-machi, since 1920.

According to the record, when a fault is encountered in the working faces of the mine through which a great deal of water floods, the surface of the flat plain subsides.

The alluvial deposits along the Ariake Lay seem to have been consolidated naturally and hence the surface has subsided. The amount of sinking in the neighborhood of Kohoku-machi is, however, exorbitantly larger than that by natural consolidation. Indeed, about 40 to 80 cm of subsidence is recorded during 10 years between the year 1947-1957. Such a large amount of subsidence is observed over all the flat area, but especially in the zone around Omachi fault where surface sinking has occurred since 1926 and is spreading slowly from west to east. In the area around Kohoku-machi, the water level in wells was nearly as high as the land surface, but as the wells gradually dried up, people must utilize only deep wells.

2. BRIEF NOTE ON GEOLOGY AND COAL EXPLOITATION IN THE KISHIMA COLLIERY

The Paleogene Kishima Group, and andesite which intrude into the Paleogene, are widely distributed in the northern mountainous part of the present area. On the other hand, diluvium and alluvium are widely found in the southern, flat part of the area. The Paleogene distributed in the Saga Coal Field is divided into two groups—the Kiuragi and the Kishima in ascending order. Only the Kishima, as mentioned above, crops out in this area. The

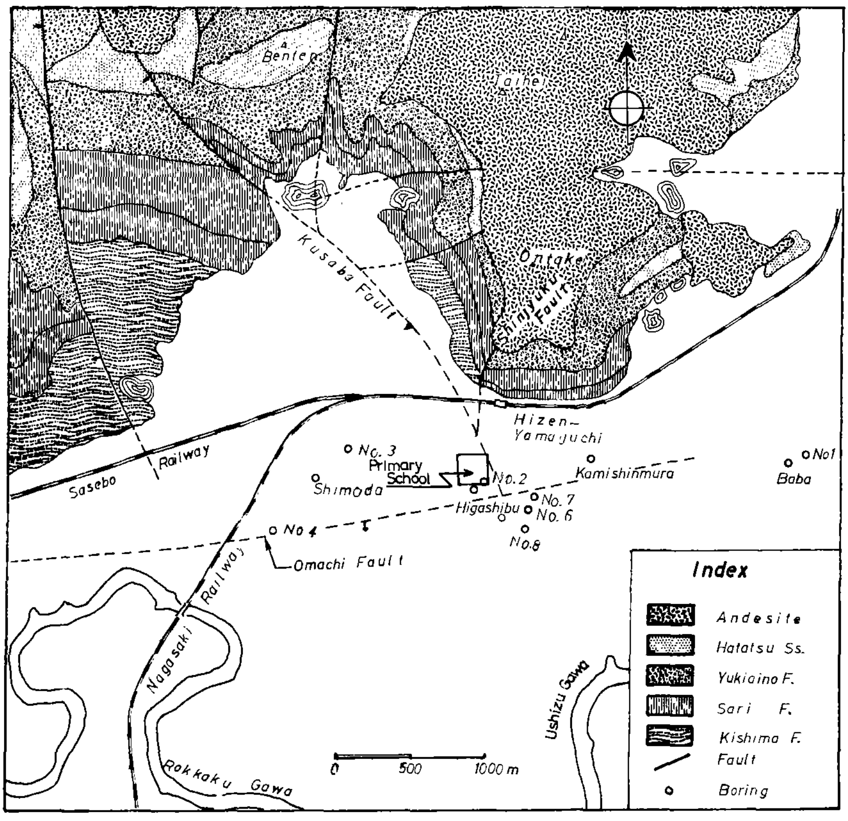

FIGURE 1. *Geological map surrounding Kohoku-machi*

Kiuragi Group is further subdivided into the Kiuragi and the Yoshinotani Formation; and the Kishima Group into the Kishima, the Sari, the Yukiaino, the Hatatsu Sandstone and the Hatatsu Shale Formation in ascending order respectively. The distribution of each formation is shown in figure 1.

A lot of coal seams are intercalated in the Yoshinotani Formation and hence this formation is very important in the coal field. The formation is composed mainly of white coarse-grained sandstone and subordinately of shale or siltstone. Of the coal seams intercalated in the formation, only the Sanjaku and Goshaku seams are worked in the Kishima Colliery.

The diluvial deposit is composed chiefly of sand and gravel and is called the "Sarushi Gravel Bed". Although it is important for geological studies, it has little relation to the problems to be discussed here, so any detailed reference to it will be omitted. The alluvium is distributed extensively in the plain and consists of very loose clay, sand, and their alternation. While on the north side of the Omachi fault (head = 250 to 300 m), the alluvial deposit is just 40-50 m in thickness, on the southernside of the fault it gets as thick as 350 to 400 m. The reason for the abrupt change of thickness of the alluvium bordering the Omachi Fault is not yet clear, but there is a possibility that the Omachi fault has moved during the time the alluvium was being deposited. Besides the Omachi fault there are several faults in the field, of which the Kusaba and the Shinjyuku faults must be taken into consideration for the problem discussed.

At Kishima Coal Mine, Pit numbers 3, 4 and 5 have been newly opened one after another, from west to east, and the Kishima Goshaku and Kishima Sanjaku coal seams, which are intercalated in Yochinotani formations, are being worked. The former is 1.2 m thick and the latter is 0.9 m; hence, the former constitutes the principal workable coal seam. All three pits are developed by inclined shafts and worked under the long wall system, with a monthly yield of about 50,000 tons. The depth of current mining is between 100 to 450 m under the surface, but the amount of drainage is quite high, being 20 cu.m. per min. Flooding is reported mostly where there is fault, but sometimes underground water flows from the worked out part when the long wall face has advanced as far as 30 to 40 m after the work was started. Again, the underground water, which contains carbon monoxide gas, sometimes springs out from the foot rock. In any case, the amount of drainage is proportional to the mining area, and there is a good relationship between the progress of the surface subsidence area and the progress of development in the pit, or increase and decrease of underground water.

3. DISTRIBUTION OF CHLORINE ION IN THE UNDERGROUND WATER

With a view to investigating the flow path of underground water, the chloride content was measured. The method of measurement was an ordinary one in which caustic silver is used for titration.

There are a great number of creeks in this area, and the creek water contains chloride of 15-50 p.p.m. The chloride content is larger in the northern part near the mountainous area than in the south.

A boring was drilled some meters deep in the first clay bed to investigate the chloride content of the water permeating through the strata. The content was 15 to 50 p.p.m. on the north side of the Omachi fault and 150-300 p.p.m on the south side, a distinct difference between the two sides separated by the fault. From electrical exploration, there was also a clear difference in the resistivity value bordering the fault. This is apparently due to the difference of chloride content in underground water. The chloride content of the first permeable layer that lies below the first clay bed amounts to several hundred p.p.m, though the amount varies considerably from place to place. In this way, the alluvial deposit in this area contains salty underground water, a part of which may be supplied from the Ariake Sea.

The chloride content of mine water of various coal mines at Kitagata, Kogayama, Meijisaga, and Nishiki is 7 to 15 p.p.m. Notwithstanding, the Kishima coal mine is working the same coal seam as those coal mines, with the chloride content of the mine water showing the large value of 70 p.p.m. This may be due to salty underground water in the alluvium flowing into the pit of Kishima coal mine through the Omachi fault, and Shinjuki fault that crosses the former. The chloride content of the Kishima mine water is smaller than that of the first permeable layer; probably resulting from the mixing of groundwater with surface-water of less salinity.

4. CONSIDERATIONS ON SUBSIDENCE OF GROUND SURFACE

The phenomena of surface subsidence have occurred largely around the Omachi fault, especially in the neighborhood of the primary school where the Kusaba Fault and the Shinjuku Fault join with each other. If there were some relation between the phenomena of subsidence and the drainage of mine water of Kishima colliery, there should be some similarity between the water of the permeable layer of the alluvial plain and the mine water. Such an assumption was verified by tests of chloride content. As shown in figure 2, there also is the possibility that all of the 1st, 2nd, and 3rd permeable layers of the alluvium in this area are connected to the Omachi Fault, and that the underground water in the permeable layers flows out in the pits passing through the Omachi Fault and others. This presumption may be justifiable considering that the underground water level has lowered toward the fault line from the seaside to the mountain side, as observed in the water level of the permeable layers in borings number 6, 7 and 8.

From the relation between the lowering of water level in the underground permeable

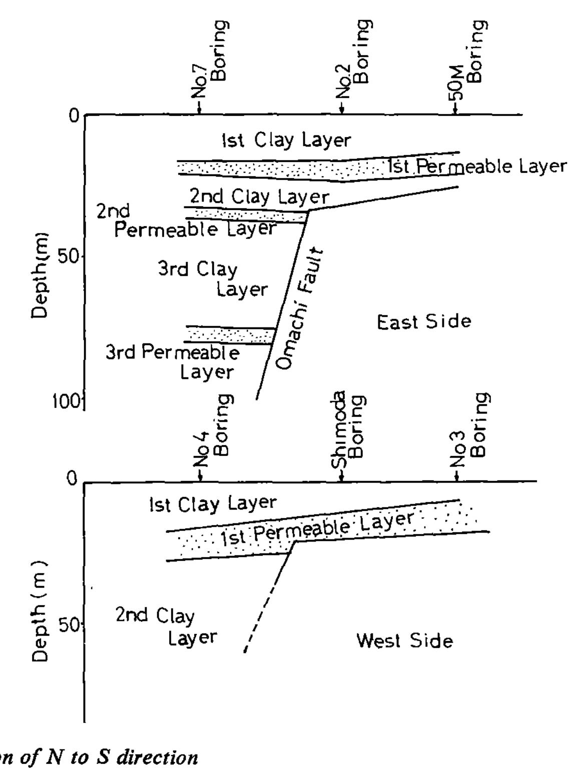

FIGURE 2. *Section of N to S direction*

layer and the subsidence of the surface, is evident about saturated clay that stress, p, acting from the outside is equivalent to the sum of stress, σ, imposed on the skeleton of clay particle and the pore pressure, u, of internal water, or

$$p = \sigma + u$$

If the underground water pressure stays unchanged, u is proportional to the water level, and σ is proportional to depth. Let us consider a simple case in which there is a permeable layer below a clay bed, as shown in figure 3, and the water pressure of the permeable layer and the impermeable layer have changed the same amount. Of course, such a state never occurs since water is supplied from the surface. However, if water pressure is assumed to be on the ground surface, or zero m in the beginning, stress σ imposed on the skeleton of soil at depth h m from the surface will become as follows (provided that the apparent density of a unit weight of saturated soil is γ t/m^3):

$$\sigma = (\gamma - 1)h \quad \text{t/m}^2$$

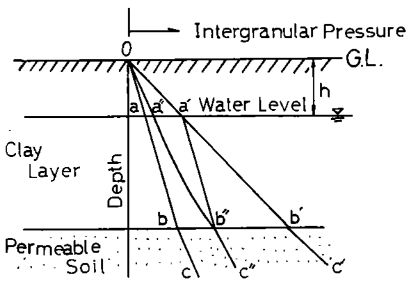

FIGURE 3. *Relation between intergranular pressure (uniformly fall of water level)*

This relation of σ and h may be represented by line oabc in figure 3. In a case where water pressure has dropped completely, stress becomes γh at the point of h m, which gives line o a′ b′ c′. When water pressure has dropped, curve $\sigma - h$ becomes ab parallel to a′b″, bc parallel to b″c″, and line o a′b″c″. However, this is the case of perfect consolidation, and under a consolidating stage, σ assumes a curve such as o a″b″ of figure 3 because the unstable pore pressure remains. If the water pressure in the permeable layer and clay deposit is maintained, curve oa″b″ gradually approaches the straight line oa′b″, but should water pressure be restored, the rate of subsidence sometimes would decreases or remains under the stage of consolidation.

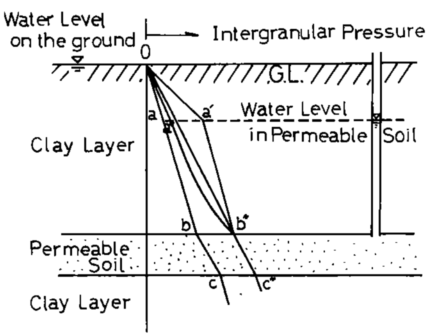

FIGURE 4. *Relation between intergranular pressure (fall of water level in permeable soil only)*

There is a supply of surface water and the water pressure in upper parts of clay deposit shows no such decrease in general cases. As shown in figure 4, therefore, if surface water remains in the state of -0 m, ob″ will be the ultimate form, which represents the curve of o a″ b″ under the consolidating stage. Hereafter, oab will be called curve σ_i and o a″ b″ curve σ_0.

What has been mentioned so far is verified also when there are a number of permeable layers and the water pressure of one of them has been lowered. In that case, the additional effective stress $\Delta\sigma$ of clay deposit increases more, the nearer we go to the permeable layer of lower water pressure. The authors' experimental results virtually proves this fact.

It is well-known that the increase of σ causes contraction to the skeleton, this is the so-called consolidation phenomenon.

The volume of such a consolidation subsidence, $\Delta\delta$, will be written for increased stress, $\Delta\sigma$, as follows,

$$\Delta\delta = H \frac{C_e}{1+e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

provided that

C_c Compression Index;

H thickness of clay layer;

e_0 initial void ratio, under σ_0

That is, the larger H, C_c and $\Delta\sigma$ are, the greater sinking will be. The alluvial soil along the coast of Ariake Bay is mostly thick and its void ratio is very large. Accordingly, apparent density is small and for the part below the underground water surface, initial effective stress, σ_0, is extremely small owing to the effect of buoyancy. For that situation, even slight additional stress, $\Delta\sigma$, will affect the subsidence considerably.

The soil in this area is divided into clay layers and permeable layers, and these two layers alternate to considerable depth. Table 1 gives representative data for some of the bore holes. The results of soil tests from number 7 boring in the central part of this area is given in figure 5, from which it is seen that there is an usually large precompressed load at the part near the 2nd permeable layer. Among the 1st, 2nd and 3rd permeable layers, the 2nd one has somewhat less permeability. In the eastern part, the 2nd and 3rd permeable layer connectes with the Omachi fault. And in the western part, for instance in number 3 boring, the 1st permeable layer is directly connected to the tertiary and there exist no layers underneath except for the 2nd clay layer. The amount of sinking, as $\sigma_i \to \sigma_0$,

Table 1 — Physical Properties of Soil

Classification		1st. Clay	2nd Clay	3rd Clay	1st Permeable Layer
Depth (m)		10.00 ~ 10.20	28.40 ~ 28.60	76.80 ~ 82.40	18.00 ~ 18.30
Specific gravity		2.60	2.71	2.67	2.71
Water content ratio(%)		125.20	51.50	17.49	5.50
Grain size	2^{mm} over	0	0	41.9	43.0
	2. ~ 0.42	0.3	3.6	18.1	18.2
	0.42 ~ 0.074	5.7	23.3	19.0	17.8
	0.074 ~ 0.005	61.5	38.6	4.5	14.0
	0.005 ~ 0.001	24.2	25.9	15.5	6.1
	0.001 under	8.3	8.6	1.0	0.9

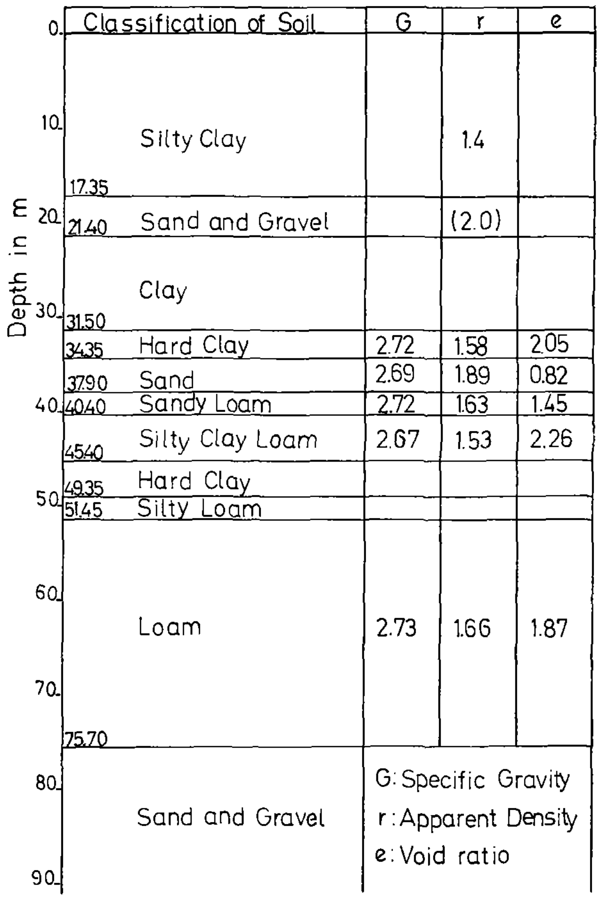

FIGURE 5. *Columner section of boring No. 7*

at number 3 is 37.6 cm by calculation. Also in number 1 boring, the 2nd clay layer is found in the layers lower than the 1st permeable layer, as shown in figure 6. It is considered from the results of boring done in the neighborhood that this geology goes on as deep as 40 m and reaches to the tertiary. If the consolidation subsidence of the 1st clay layer is evaluated by the same calculation, the result will be 144 cm, as $\sigma_i \rightarrow \sigma_0$. However, the 1st permeable layer is not connected directly to the fault and so it is not considered in regards to the subsidence due to lifting of water at coal mines.

5. CONCLUSION

Along the coast of the Ariake Bay the soft clay layer is distributed thick and widely, and at some places it has passed into the stage of spontaneous consolidation. However, there also is subsidence due to dehydration, which should not be overlooked. The consolidation of the soft clay layer is caused from lifting of underground water, as a great amount of water is pumped up from colliery through the Omachi and other faults. The phenomena of the extremely conspicuous subsidence in this area precisely indicated the fault line. On the other hand, well water for farming in the neighborhood is also drawn from the

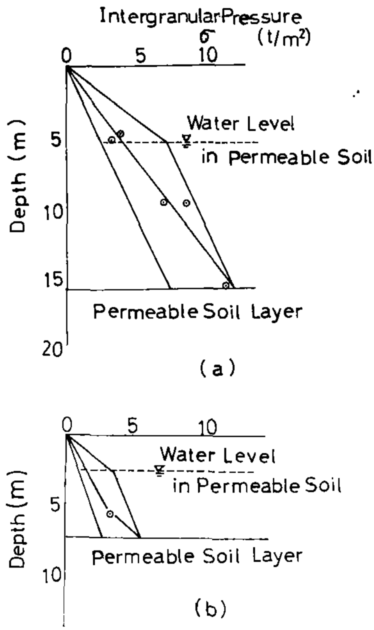

FIGURE 6. *Relation between intergranular pressure (σ) and depth (h) at boring No. 1 (a) and boring No. 3 (b)*

underground 100 m deep, and the phenomenon of land-surface sinking is apparent in the adjoining parts centering around the well. Accordingly, it is clear from experiments and the conditions of the area that consolidation subsidence can occur by the lowering of underground water pressure due to pumping from the base of thick alluvial formation, even if there is water on the surface.

SURFACE SUBSIDENCE AND SINKHOLES IN THE DOLOMITIC AREAS OF THE FAR WEST RAND, TRANSVAAL, REPUBLIC OF SOUTH AFRICA

C. A. BEZUIDENHOUT* and J. F. ENSLIN**

ABSTRACT

The gold-bearing reefs of the Far West Rand, from where nearly one-fifth of the published world annual production for 1967 was derived, underlie 1 200 metres of dolomite—the most important aquifer of the Republic.

The dolomitic ground water, which is stored by nature in separate dyke bounded compartments (a typical one covers 160 square kilometres and has a storage capacity of 700,000 megalitres) is apt to flow into the mine workings at enormous rates, depending on the number of post-dolomite faults cutting the workings and the hydrological characteristics of those faults.

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As a general policy the dolomitic ground-water compartments are dewatered by pumping and the disposal of the pumped water in such a way that it cannot return to the particular compartment and recirculate through the mines. A saving of approximately R 100 million in pumping costs will be effected and the safety of mining is increased by preventing recirculation.

Dewatering, however, results in differential subsidence of large areas and the formation of sinkholes ; certain areas have thus become unsafe for occupation. Damage to buildings caused by subsidence, and the cost of replacements, research and work to improve safety has amounted to more than R 25 million.

The geological and hydrological factors causing subsidence and sinkholes are discussed. A gravity method of delineating areas of potential subsidence and zones where the conditions are favourable for the formation of sinkholes is described.

RÉSUMÉ

Le reef aurifère du l'Extrême Rand-Occidental, d'où presque la cinquième partie de la production mondiale d'or, publiée pour l'année 1967 provient, se trouve au-dessous d'une couche de 1 200 mètres d'épaisseur de dolomite. Cette couche constitue en même temps l'aquifère le plus important de la République.

L'eau souterraine de la dolomite se trouve de façon naturelle, accumulée en compartiments limités par des dykes. Un de ces compartiments, typique pour l'ensemble, couvre 150 000 km² et possède une capacité d'accumulation de 150 000millions de gallons impériaux. L'eau souterraine est portée à s'écouler dans les travaux miniers en quantités énormes, dépendant du nombre des failles postdolomitiques, coupées par les travaux miniers et les caractéristiques hydrogéologiques des failles.

Le mode d'action général est de drainer les compartiments d'eau souterraine par des machines d'épuisement. L'eau pompée est disposée de façon à rendre impossible la réoccupation d'un compartiment drainé et la recirculation à travers la mine. En empêchant la recirculation il est possible d'économiser environ 100 millions Rands en dépense de travaux d'épuisement et en même temps d'agrandir la sécurité dans la mine.

Le drainage des mines a eu cependant comme résultat une subsidence différentielle des régions étendues et la formation de dolines. Certaines régions sont ainsi devenues dangereuses pour l'occupation. Le dommage causé par la subsidence des bâtiments, les dépenses de leur replacement, les dépenses des recherches et celles des travaux d'amélioration de la sûreté, s'élevent à plus de 25 millions Rands.

Les facteurs géologiques et hydrogéologiques qui causent la subsidence et la formation des dolines sont discutés. Une méthode de gravimétrie qui sert à tracer les régions de subsidence potentielle et les zones favorables à la formation des dolines est décrite.

When the gold mines on the Far West Rand came into production after the Second World War, it became apparent that much more water was being made here than in any of the other fields in the Witwatersrand Basin.

The cost of pumping water from the mines became an important factor in the economics of the field and the possibility of large inrushes of water into the workings presented itself as a major mining hazard.

The gold field of the Far West Rand (figs. 1 and 2) in the Transvaal covers some 750 sq. km and lies in a comparatively narrow block measuring 50 km along the strike and 15 km along the dip of the reefs. This area has become one of the most important gold-producing fields in the world. During 1967 the gold produced by the nine working mines (a tenth mine was then in the shaft sinking stage) amounted to 7,610,000 ounces or 18.8 per cent of the published world production figure of 40,500,000 ounces. The total gold produced in the Republic of South Africa that year was 30,532,880 ounces, of which 23,45 per cent was mined in this comparatively small field.

The total amount of gold produced in this field up to the end of 1968 was 80,062,566 ounces, which at the present price of of R25[1] an ounce has a value of approximately R2 billion. It is estimated that the amount of gold in the still unmined auriferous beds is approximately 240 million ounces, worth nearly R6 billion at the present price.

The gold remaining in this field is of significant economic importance and every effort will be made to mine as much of it as is economically practicable. It is, therefore, impera-

1. R1 = approximately US $1.4.

tive that the hazards of inrushes of water into the workings be countered and that pumping costs be held to a minimum.

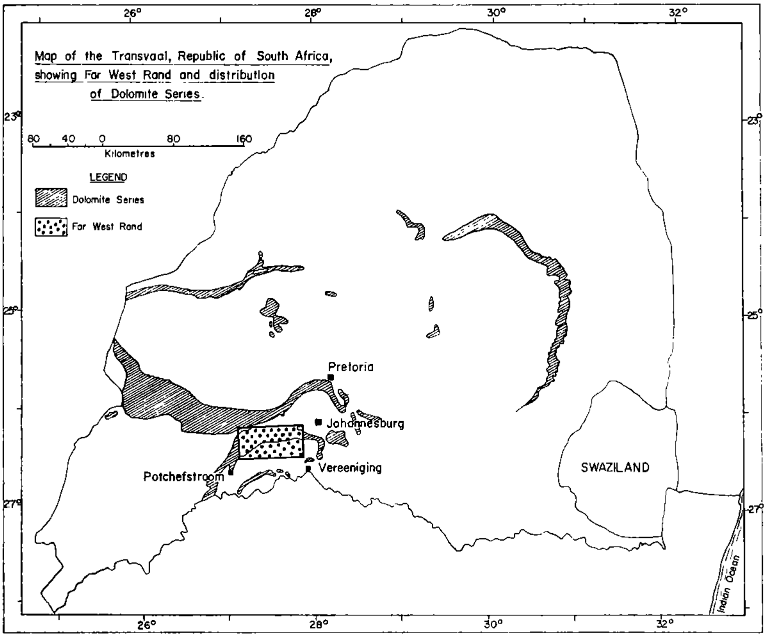

FIGURE 1.

As recently as October, 1968, an unprecedented inrush of water estimated at 365 megalitre[1] per day broke through into the workings of the West Driefontein Gold Mine when settling of the roof of a stope took place. Only by the quick action and ingenuity on the part of the responsible mining engineers was the mine saved from being completely flooded, and it is a tribute to their skill and organisation that the entire labour force was successfully evacuated from underground without a single casualty.

THE GEOLOGICAL SETTING

In the Far West Rand the auriferous reefs of the Witwatersrand System and the Ventersdorp Contact Reef which are mined, underlie the Dolomite Series of the Transvaal System. The latter system is subdivided in descending order as follows:

Pretoria Series . . . . .shale, quartzite, lava and intrusive diabase.
Dolomite Series . . . . .dolomitic limestone and chert.
Black Reef Series . . . . .quartzite and subordinate shale.

The Dolomite Series, which has a general dip of 6° to 12° to the south is about 1,200 metres thick in this area (fig. 3) and underlies the fairly flat Wonderfontein Valley which is eight to fifteen kilometres wide. Chert occurring in thin layers, mainly in the upper

1. One megalitre = 264,178 US gallons;
= 219,980 British gallons.

150 metres of the Series, and the lower quartzites of the Pretoria Series form the hills which are the southern boundary of the Valley, and rise 100 to 200 metres above the river bed. The Black Reef Series and rocks belonging to the Witwatersrand System and the Older Granite outcrop in the low hills to the north of the Valley. Sandstone, shale and tillite of the younger Karoo System form a number of outliers on the Dolomite Series.

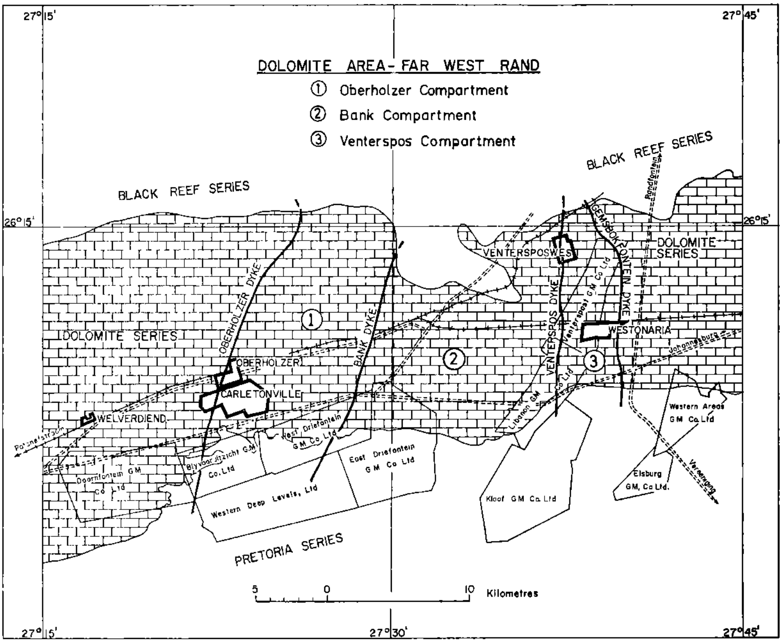

FIGURE 2.

The soil cover of the Dolomite consists of *in situ* weathered dolomite and chert, wad and transported aeolian sands of Pleistocene age.

Of the constituents of the dolomite, the carbonates of lime and magnesium dissolve slowly in dilute acidic ground-water percolating along fault planes and fissures and are removed in solution; the insoluble oxides of iron and manganese, as well as the weathered chert, remain more or less *in situ* to form a light honeycombed residue. The manganese-rich material, called wad, compacts on being dewatered and such compaction may give rise to subsidence of the surface.

GEOHYDROLOGY

The Dolomite Series, which covers about 14,000 sq. kilometres in the Transvaal, forms the most important aquifer in the Republic. It is divided into a number of separate ground-water compartments by watertight syenite or diabese dykes (figs. 2 and 3). On the Far West Rand these dykes, which intruded the sediments along tension faults, generally strike in an approximate north-south direction at right angles to the strike of the formation In each of the compartments the ground water is contained in a vast network of interconnected joints, fault planes, cavities and solution channels, as well as in the overlying weathered dolomite, chert, wad and aeolian sands. Large quantities of water are stored

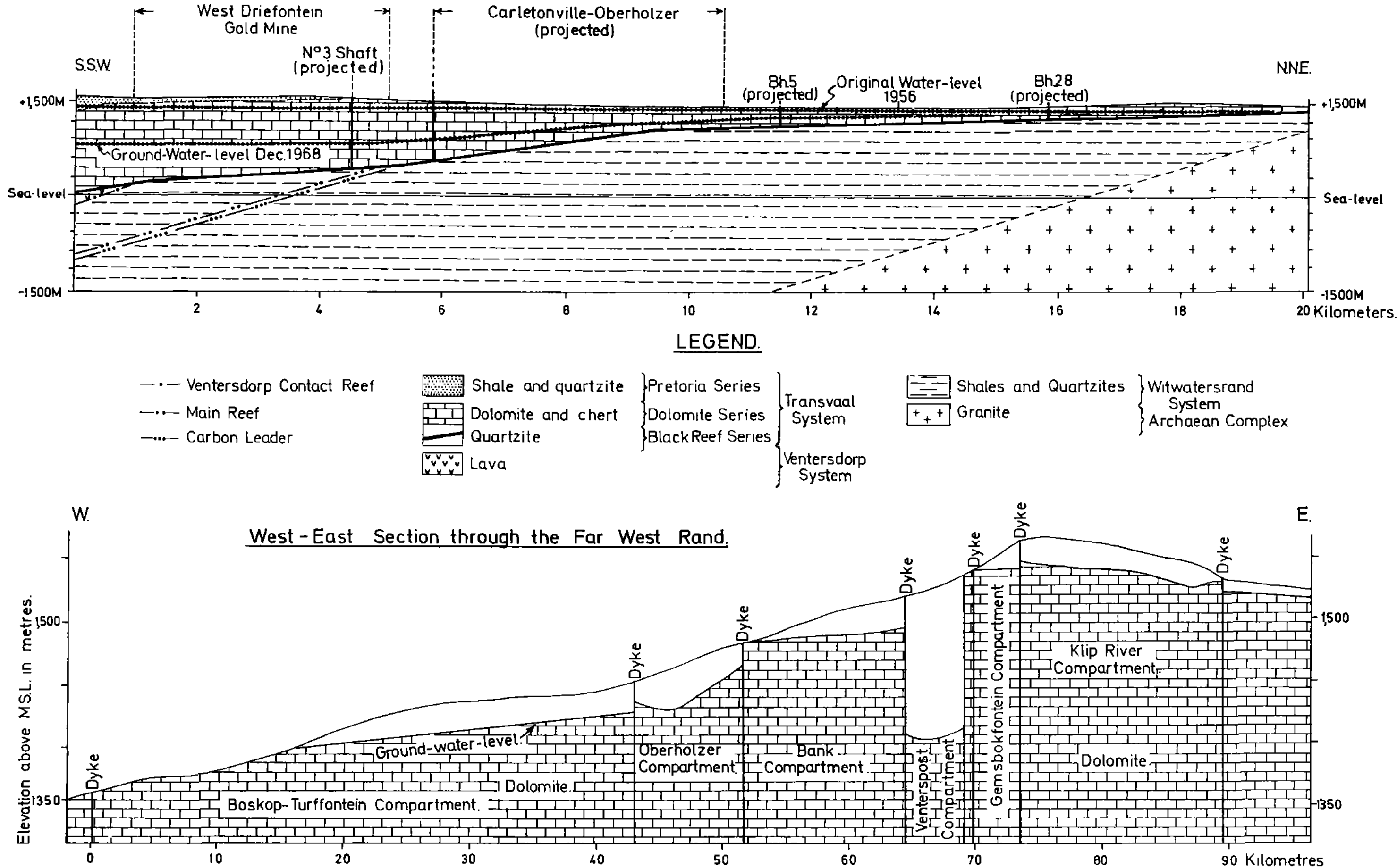

FIGURE 3. *Geohydrological sections through Dolomitic compartments*

in the unconsolidated material, in particular where the dolomite has been leached extensively and these materials fill "geological valleys" (figs. 4 and 5), which have been found to extend to depths as much as 200 metres below the surface.

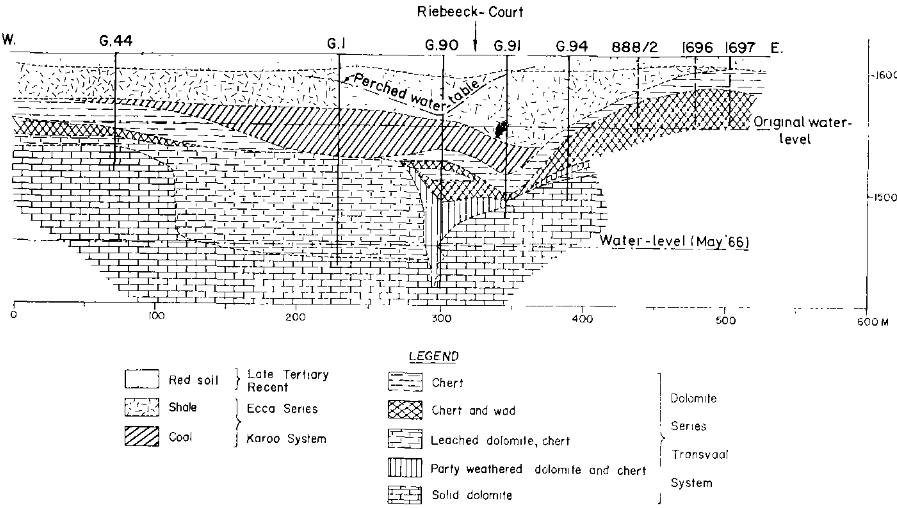

FIGURE 4. *Geological section through Riebeeck-court, Westonaria*

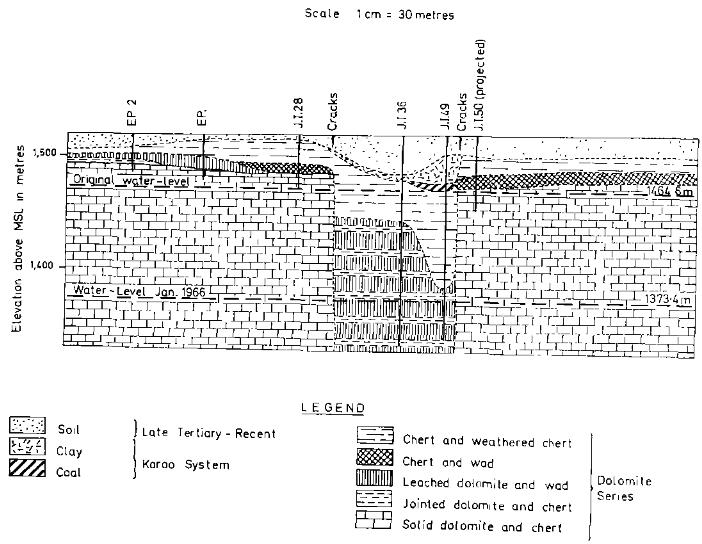

FIGURE 5. *Geological section, Kernite street*

The ground-water compartments vary in area from a few to more than 250 square kilometres. The storage capacities of the Oberholzer and Venterspost Compartments, two of the larger compartments located above the gold mines, have been assessed at 700,000 and 450,000 megalitres, respectively.

Fractures and fault planes in the Dolomite Series extend into the underlying Witwatersrand and Ventersdorp Systems and thus intersect the gold-bearing reefs. Many of these fractures, particularly the tension faults, form conduits which, when exposed in the mine workings, discharge water under great pressure from the overlying dolomitic formations. Although the ground water which flows into the mine workings at depths of 1,000 to 3,000 metres below surface is under great pressure, the fracture zones are generally fairly well sealed at those depths due to the high rock pressures and the impervious nature of the strata. The conduits, however, tend to open up as a result of doming, i.e. the settling of the roof and superincumbent rock layers above stoped areas. Doming can result in large increases in the inflow of water and may give rise to sudden inrushes of such volume to overwhelm the pumping facilities of the mine. Although such inrushes in the past have never exceeded 15 megalitres per day anywhere in this field, an unprecedented inrush in the history of mining occurred on the 26th October, 1968, when 365 megalitres per day broke through.

Although cementation has been applied extensively in the mines and inflows have been substantially decreased, the method has not been proved successful for preventing inrushes.

DEWATERING OF COMPARTMENTS

This experience confirms that under certain circumstances it is an unacceptable risk to undermine waterlogged dolomite formations. In the absence of any method of preventing the water from finding its way into the mine workings, there appears to be no alternative but to dewater the dolomite. In 1960 an Interdepartmental Committee, after intensive investigation, recommended that the mines operating in the area accept dewatering of dolomitic compartments as a necessary policy. This recommendation was based on both economic and safety considerations, although it was realised that dewatering would give rise to surface subsidence and the formation of sinkholes in certain areas, and under certain conditions. This policy was accepted and was put into effect.

Dewatering of a compartment entails not only the removal of the water contained in the dolomites, but also the disposal of that water to ensure that it cannot return. Recirculation is thus prevented and this leads to progressive depletion of the water stored in the dolomite and to the drying up of springs, the decrease of inflow into the workings, and saving in pumping costs.

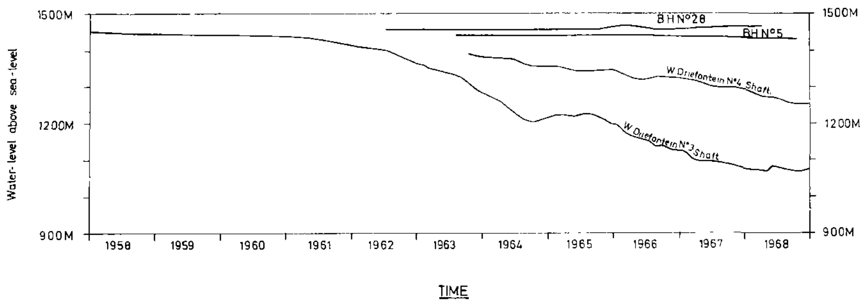

FIGURE 6. *Oberholzer compartment lowering of ground-water level, 1958-1968*

As a result of the implementation of the policy of dewatering the ground water stored in the Oberholzer and the Venterspost Compartments has already been decreased by more than 360 and 320 thousand megalitres, respectively; the ground-water levels have dropped progressively (fig. 6) and the resulting occurrence of slow surface subsidences and sink-

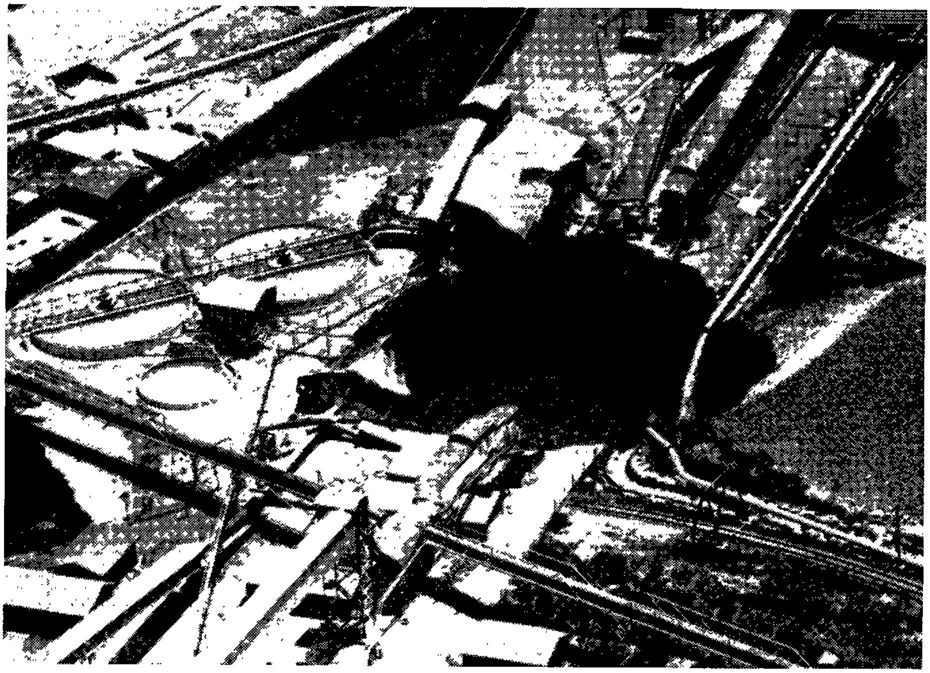

PLATE 1. *Sinkhole at Crusher Plant, West Driefontein Gold Mine*

PLATE 2. *Sinkhole at Blyvooruitzicht*

holes imposes a real threat to the development of the Waterfontein Valley and the safety of its inhabitants.

Development of the Wonderfontein Valley as an irrigation farming area started more than 100 years ago. The water supplies were obtained from a number of dolomitic springs, representing the overflows from the dolomitic compartments. When gold mining started, two towns—Westonaria and Carletonville—were established on the dolomite in the valley. At present the populations of these towns are 35,000 and 90,000, respectively. The towns were located in areas where no sinkholes had occurred or subsidences taken place previously and which were considered safe for development. In addition, most of the surface structures of the mines, including offices and crushing and reduction plants were built on dolomite.

It was only as dewatering progressed that instability of the surface became apparent. As a result of subsidences and the possibility of catastrophic sinkholes forming, certain areas were evacuated and many of the mining plants rebuilt on safe areas off dolomite. The direct expenditure in research, application of safety measures, compensation for damage and loss of water supplies and the rebuilding of mining plants on safe areas, has been estimated at R25 million.

When the phenomenon of surface subsidence was first noticed in 1959, it became a matter of urgency that the areas which were subject to subsidence or to the formation of sinkholes be delineated. The real danger to the inhabitants of the area was emphasized when in December, 1962, a sinkhole opened up on the West Driefontein Mine property and engulfed a three-storey crusher plant with the loss of 29 lives (plate 1) and again in August, 1964, when two houses and parts of two other houses disappeared into a sinkhole on Blyvooruitzicht Township with the loss of five lives (plate 2). These catastrophies imposed on the authorities an urgent duty to delineate safe and unsafe areas and to evacuate all suspect areas.

Major research efforts were initiated both by the Chamber of Mines and by the Government into the mechanics of the phenomenon and towards the development of practical methods of delineating safe areas. The Chamber mobilised a team of scientists to investigate direct methods of locating underground voids which could give rise to sinkholes. All known geological and geophysical methods, including magnetic, electromagnetic, resistivity, radio and seismic refraction and reflection methods, were tested. The problem, however, proved too complicated for direct solution by those methods.

The Geological Survey Division of the Department of Mines then turned to the extensive application of gravity methods for delineating potentially dangerous areas. Detailed gravity surveys, supplemented by boreholes drilled for interpretation purposes at strategic points, were carried out and deductions and interpretations were made as to the geological and geohydrological conditions which must pertain concurrently for slow subsidence to take place or for sudden sinkholes to form.

THE NORMAL DOLOMITE SINKHOLES

The occurrence of sinkholes is well-known in most dolomite areas in South Africa. In the course of geological time, the solution and removal of the carbonates of magnesium and calcium by weakly acidic ground water percolating along fissures and fault zones form cavities and channels. Such cavities may be filled with unconsolidated material or may remain empty below a roof of more resistant rock. Should the roof of such a cavity be linked to the surface by a widened fissure, unconsolidated material overlying the dolomite may be carried down into the cavity, leaving a void with an arched and unstable roof. If the equilibrium is disturbed the arched roof collapses and a sinkhole forms (fig. 7).

Observations have proved that, with very few exceptions, such sinkholes are formed as a direct result of abnormal concentrations of water at or very near to the surface, caused

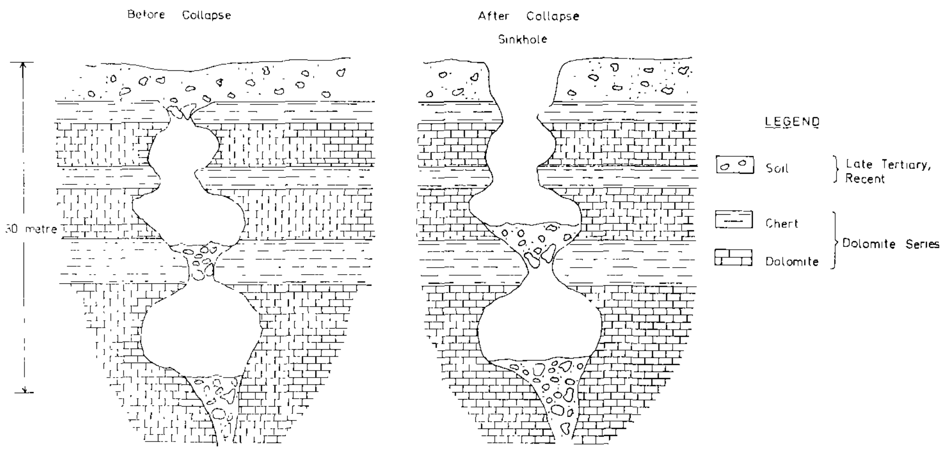

FIGURE 7. *The formation of sinkholes by collapse*

by broken sewage or water pipes, storm water, leakage from irrigation canals and even by overwatering of gardens. When a new township is being developed conditions are more favourable for abnormal concentrations of water, and sinkholes do in fact occur more frequently.

Such concentrations of water can, however, trigger sinkholes only at points where the following phenomena obtain to create a dangerous situation:

1. There must be a void or cavity into which the roof can fall.
2. The fissure must form a bottleneck of dimensions which can be bridged by a self-supporting arch of the overlying unconsolidated material. The span of the fissure where such conditions develop is seldom more than 10 metres.
3. Depending on the size of the cavity, the solid abutments of the bottleneck must be reasonable near the surface, otherwise the cavity will be filled by volume increase of the caving material and block the cave-in before the surface is reached. The cover of unconsolidated material is generally less than 15 metres.
4. The bridge must be fairly unstable and above the ground-water table, otherwise seepage of surface water will not cause its collapse.

It must be noted that the ground-water or *the lowering of the ground-water table* plays no part in the triggering of such a sinkhole. In fact, the ground-water table is generally at great depth below the bottleneck which is bridged by the unconsolidated material.

SLOW SUBSIDENCE AND SINKHOLES CAUSED BY DEWATERING

The dewatering of dolomitic compartments by the mines on the Far West Rand gave rise to two separate subsidence phenomena:

Firstly: slow subsidences which take place when the ground water is lowered and compaction of unconsolidated material occurs; and

Secondly: the sinkholes which occur when arches, below the ground-water table but otherwise similar to those which give rise to the normal type of sinkhole, collapse when the structure is weakened by the dewatering.

Slow subsidencf

As already mentioned the products of weathering of dolomite—wad, weathered chert, semi-weathered dolomite and other impurities—tend to form a honeycombed structure which has a very low bearing-strength and compacts on being dewatered.

In the Far West Rand the subsurface of solid dolomite which is covered by its weathered products and soil, and often by inliers of younger Karoo beds, is rather irregular and forms an undulating plateau interrupted by geological valleys that vary in depth and width from less than 30 metres to more than 200 metres. These valleys developed mainly as a result of the seepage of ground-water along the numerous post-Transvaal fault planes and the large scale leaching and solution of the dolomite along those zones over millions of years. Consequently the leached zones widened and ultimately became so weak that the overlying layers of rock, the horizons of chert forming the upper horizons of the Dolomite Series, the quartzite and shale of the Pretoria Series and occasionally Karoo sediments slumped into the valleys on top of the honeycombed mass of wad and chert.

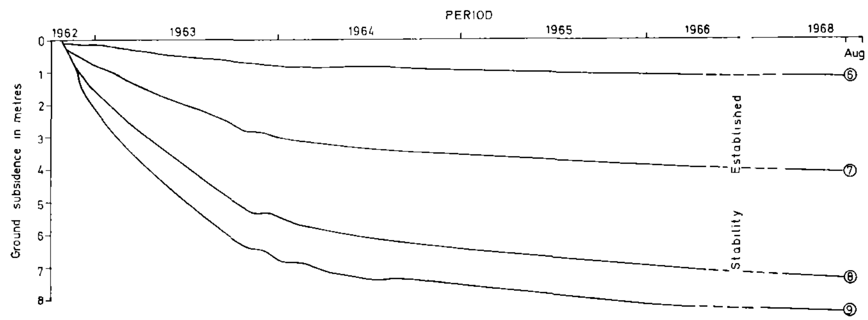

FIGURE 8. *Time subsidence curves at four observation points along Caledon street (Schutte's depression)*

These valleys which are generally below the normal water table are, therefore, partly filled with unconsolidated material which, on being dewatered, compacts and gives rise to surface subsidence. The degree of subsidence depends on the thickness and percentage of compacting material, which varies laterally, giving rise to differential subsidence, which causes large cracks at the surface. The areas of subsidence can generally be outlined by the very prominent cracks which form along the line where subsiding soil tears away from that on an adjoining stable area. The time lag between the lowering of the water-table and the surface subsidence, where observed, has been fairly short, and the total subsidence in the different valleys has varied from a few inches to more than 30 feet (fig. 8, plate 3). Subsidences of one to five feet are very common.

SINKHOLES TRIGGERED BY DEWATERING

A study of the sinkholes which formed in the Far West Rand since mining operations commenced showed very clearly that sinkholes formed concurrently with the lowering of the water table in areas which had been remarkably free from sinkholes before. These areas fall under three groups:

1. Those where the original water table was less than 15 metres below, and less frequently within 30 metres of the surface.
2. The scarp zones bordering on the deep valleys.

PLATE 3. *Slow surface subsidence, Schutte's Depression, Carletonville*

3. In narrow valleys, i.e. valleys which are deeper than they are wide. Valleys which are more than 100 metres wide can still be classified as narrow valleys if they narrow down at great depth to a width which permits blockage and bridging of the unconsolidated material to occur. The bridge will be weakened and may collapse when dewatering takes place. Such valleys give rise to sinkholes of larger dimensions than would normally be expected to occur—the largest one has a diameter of 80 metres at the surface, and a depth of 50 metres—thus constituting a much greater hazard. Sinkholes are not formed in wide valleys, i.e. those which are wider than their depth to solid rock, as compaction in this type of valley leads to slow subsidence at the surface.

It has also been observed that sinkholes, which occurred within the geological times and were subsequently filled by aeolian sands tend to redevelop when the bridging at depth collapses (plate 4).

These sinkholes are very often triggered by earth tremors caused by rockbursts in the mines.

DELINEATING POTENTIAL SUBSIDENCE AND SINKHOLE AREAS

From the foregoing it is clear that, in order to delineate those areas which would be subject to slow subsidence or which would be potential sinkhole areas when dewatering takes place, the subsurface contours of the solid dolomite relative to the depth of the original ground-water table, have to be determined.

As the specific gravity of solid dolomite (2.85) is much higher than that of the unconsolidated material which covers the dolomite and which fills the valleys (1.7-2.4) a gravity

PLATE 4. *Sinkhole, Kaolin Street, Carletonville*

method was considered to hold the most promise and detailed gravity surveys of the compartments which are being dewatered, have been carried out. Bouguer values were calculated and further corrections were applied to eliminate regional anomalies and to reduce the zero gravity contour to correspond to lines along which the subsurface of solid dolomite coincides with the original ground-water table. Positive gravity contours thus indicate areas where the solid dolomite is above the original ground-water table, i.e. areas which will not be affected by dewatering. Negative gravity contours indicate where the original ground-water table is above the solid dolomite, i.e. areas which will be subject to slow subsidence with dewatering; and the zones with the closely-spaced gravity contours indicate the scarps adjoining the valleys.

By more sophisticated analysis and interpretation of the gravity contours it is possible to delineate the zones which are potential sinkhole areas, with fair accuracy. The accuracy of the interpretations is further increased by drilling boreholes at strategic points.

It has thus been possible by the scientific interpretation of data obtained by gravimetric surveys supplemented by boreholes to delineate safe and potentially unsafe areas and to advise accordingly.

It will also be possible to plan any new mining areas where conditions are similar to those encountered on the Far West Rand and thus to kerb the large expenditure on damage caused by subsidence and sinkholes.

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DISCUSSION

Intervention of Mr. Owen G. INGLES (Australia):

Question:

Is the gravity method suitable for determining the alignment of proposed roads and railways so as to avoid the possible loss of heavy construction equipment trafficking a potentially cavernous limestone area?

Reply by Dr. ENSLIN:

Yes, the gravity method has been found very useful for the alignment of roads and railway lines.

The potentially dangerous areas are delineated on the special gravity contour map of the dolomitic area and the safe alignment of the road or railway line can be determined.

It must, however, be noted that because the potentially dangerous zones are closely linked with post-Dolomite fault zones, which may cut right across the dolomitic area, it could be very difficult to route a road or railway line without crossing a danger zone.

Intervention by Dr. Manuel N. MAYUGA (USA):

Question:

I notice that you have a substancial loss of lives and properties as a result of subsidence. Do I understand that the mines takes responsibility or the liability for all these damage and evacuation and all the liabilities involved?

Reply by Dr. ENSLIN:

The permits for dewatering the Oberholzer and the Venterspost Compartments were issued by the Department of Water Affairs after agreement had been reached between the Government and the Gold Mines that the Gold Mines concerned would be responsible for paying compensation for financial loss due to loss of water, evacuation of properties, and damage due to subsidence and sinkholes caused by the dewatering.

DAMAGE TO IRRIGATION POND DUE TO MINING SUBSIDENCE

T. NISHIDA[1] **and K. GOTO**[2]

Abstract

There seems to be very little quantitative information about mining damage to the structures on the surface. The ponds which exist in the Kyushu coal fields, and have been damaged due to mining subsidence, were investigated concerning the time, point and magnitude of damage, the length of dam, the geological condition and the mining map, and were compared with the results of the theoretical calculation of the surface strains. The following became clear after the comparison:

1. The position of the dam within tension zones in both the directions of dam axis and flow axis affects the amount of mining damage to the pond;
2. Increasing the magnitude of tensile strain on the dam by more than 2 mm/m a year causes leakage or exhaustion of pond water;
3. The effect of local geology on mining damage is of importance.

Résumé

Il semble qu'il n'existe que peu d'informations quantitatives sur les dommages miniers aux structures à la surface. Les étangs situés dans les terrains houillers de Kyushu qui sont endommagés par les affaissements miniers sont étudiés quant au temps, aux endroits et à la grandeur des dommages ; la longueur du barrage, les conditions géologiques et la carte minière sont comparées avec les résultats du calcul théorique de la tension à la surface.

Les résultats obtenus sont les suivants :

1. La position du barrage dans une zone d'extension dans les deux directions des axes du barrage et du courant affecte la grandeur du dommage minier occasionné à l'étang.
2. Si la tension au barrage est plus grande que 2 mm/m par année, il y a production de fuites ou épuisement d'eau de l'étang.
3. Les effets de la géologie locale sur le dommage minier sont importants.

INTRODUCTION

Mining subsidence causes damage to structures on the surface, but the quantitative limiting value of mining damage to the structures is not evident and may be affected by local geological condition. There exists many, but not large, irrigation ponds in the Kyushu coal fields which have earth dams. Those ponds, damaged because of mining subsidence, have been repaired. The geological conditions, mining maps, and points and magnitude of damage of such ponds were investigated and compared with the results of the theoretical calculation of the surface strains.

CALCULATION METHOD USED OF SURFACE STRAINS

Both strains of extension and of compression occur on the ground surface caused by the extraction AB of a horizontal coal seam which must affect maximum subsidence at a given point P (see fig. 1). The predicting method used here for tension and compression at an arbitrary point at the ground surface is that of one of the authors, T. Nishida. His method uses the influence circle divided into sections, as in figure 2. The radius of the circle (fig. 1) is $h. \cot \gamma$ (h = depth of seam under surface, γ = limiting angle), the length

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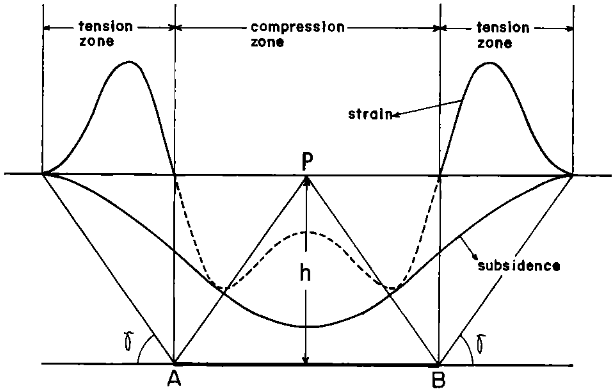

FIGURE 1. *Tension zone and compression zone due to mining subsidence*

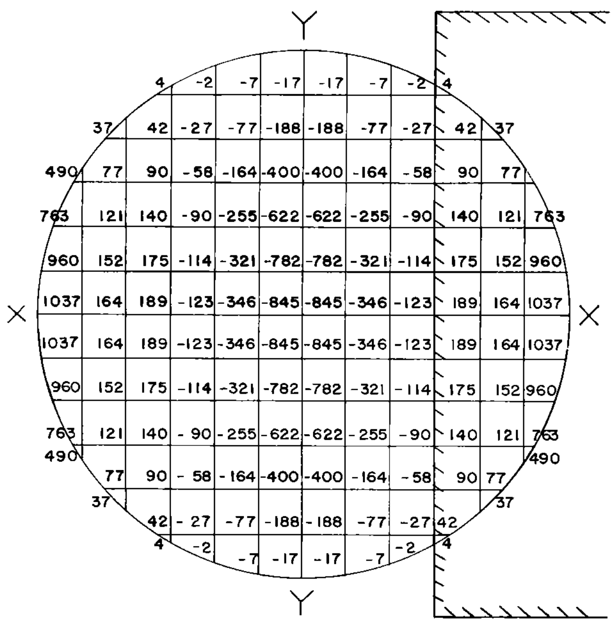

FIGURE 2. *Influence circle divided into sections (Nishida's method)*

of each side of a section is one sixth of the radius, and each section has its own strain coefficient of effect. In calculating, the influence circle is placed on the mining map having the same scale as the circle, and its center and the X-X axis are made to coincide with the picture of the surface point and with the direction intended to calculate, respectively. Adding up the coefficient of effect of the excavated partial area (positive and negative coefficients cancel each other), the strain V', then given by the formula

$$V' = 0.289 \frac{ma}{h} e \cdot 10^{-4}$$

where

m thickness of seam;
a sinking factor;
h depth of seam under surface;
e the sum of strain coefficient of effect.

If the sign of V' is positive or negative, the strain is tensile or compressive, respectively, in the direction of X-X axis. If the extracted area is as shown by the shaded portion in figure 2, and if m = 100 cm, a = 0.90 and h = 200 m, the strain

$$V' = 0.289 \times \frac{1{,}000 \times 0.90}{200} \times 0.888 = 1.15\ mm/m\ (tensile)$$

THE POSITION OF THE POND RELATIVE TO THE EXTRACTED AREA ON THE MINING MAP

The position of the pond investigated relative to the extracted area on the mining map is classified as in figure 3.

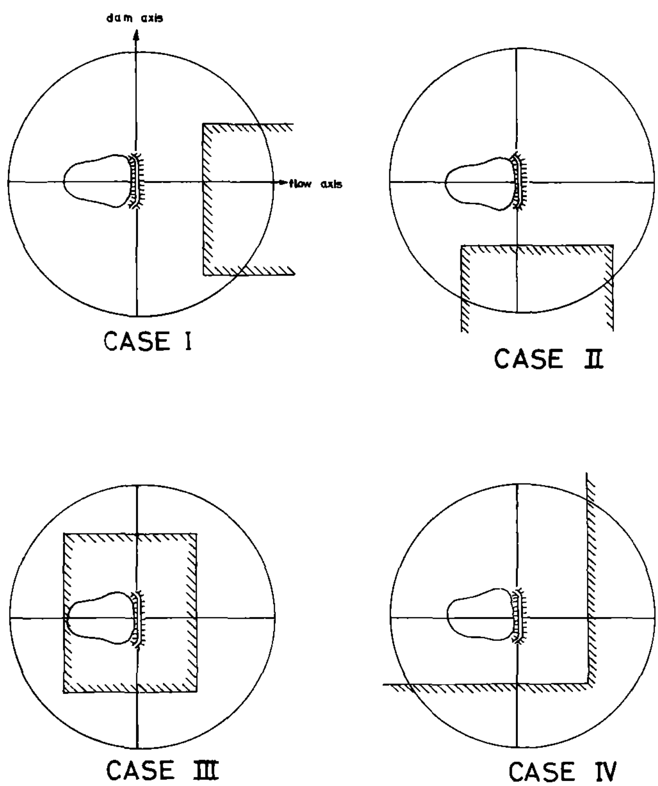

FIGURE 3. *Position of pond relative to the extracted area*

Case I: The center of the dam is within the tension zone in the direction of flow axis and within the compression zone in the direction of dam axis.

Case II: The center is within the tension zone in the direction of dam axis and within the compression zone in the direction of flow axis.

Case III: The center is within the compression zones in both directions.

Case IV: The center is within the tension zones in both directions.

And the next case is added.

Case V: The pond bottom is broken due to caving-in which occurs at the shallow depth of extraction.

PONDS INVESTIGATED

The number of the ponds investigated is 40. All of these have earth dams and are not large, that is, the volume of water held of the largest dam is less than 10,000 cu. m. They are classified roughly according to the length of dam as follows: less than 50 m—17 ponds, 51 through 100 m—17 ponds, 101 through 200 m—3 ponds, and longer than 200 m—one pond. The extraction of coal, under areas of large and important ponds—such as those for service water, is regulated by law. Ponds are classified in table 1 according to the magnitude of damage. Table 2 shows the surface strains of the ponds calculated according to T. Nishida's method, and other data pertinent to description of the ponds.

TABLE 1. Classification of ponds affected according to the magnitude of damage

Symbol		Magnitude of Damage	Number of Ponds
D	I	no damage, no leakage	14
D	II	small leakage from irrigation pipe in dam	11
D	III	leakage caused by crack in dam body	3
D	IV	leakage caused by cracks at pond bottom	5
D	V	complete drainage caused by cracks at pond bottom	7

Table 2 schows the surface straine of the ponds calculated according to T. Nishida's method, and other data pertinent to description of the ponds.

TABLE 2. The results of pond investigations

Name of coal field	Calculated Strain in the Direction indicated		No. of position	Magnitude of Damage	Length of dam	Depth of Seam	Amount of Subsidence	Note of Geology
	Dam axis mm/m	Flow Axis mm/m	Case		m	m	cm	
A	+0.39	+0.67	IV	D III	49	250	203	
B	+0.67	−2.13	II	D 1	70	300	134	
B	+0.45	+2.16	IV	D II	100	240	131	
B	+0.64	+1.94	IV	D V	25	300	190	
B	—	—	V	D V	35	20		
C	+0.44	−0.05	III	D I	90	375	8	
C	−0.06	−0.33	III	D I	90	430	49	
C	+0.51	+0.10	IV	D IV	140	215	28	
C	+0.16	+0.18	IV	D IV	45	440	153	
C	−0.24	−0.50	III	D I	140	380	39	
C	+0.51	−0.58	II	D II	75	100	85	
D	+0.93	+0.71	IV	D IV	60	180	64	ss

(Continued see next page).

TABLE 2. The results of pond investigations. (Continued)

Name of coal field	Calculated Strain in the Direction indicated		No. of position	Magnitude of Damage	Length of dam	Depth of Seam	Amount of Subsidence	Note of Geology
	Dam axis mm/m	Flow Axis mm/m	Case		m	m	cm	
D	−0.59	+1.64	I	D II	60	200	38	ss
D	−1.58	−1.21	III	D I	40	240	105	ss
D	−2.72	−1.40	III	D I	40	210	135	ss
E	−3.28	−0.80	III	D I	20	300	85	clay
E	−1.45	−0.71	III	D I	53	400	65	clay
F	+0.22	+0.14	IV	D V	45	260	58	fault
F	+1.52	−0.90	II	D V	100	260	92	fault
F	+1.62	+0.46	IV	D V	50	260	96	fault
G	+0.50	+1.26	IV	D II	470	275	104	clay
G	+0.81	+1.16	IV	D II	170	360	171	clay
H	−0.05	+0.51	I	D I	100	425	6	clay
H	+0.23	−0.24	II	D I	20	400	23	clay
H	−0.03	+0.49	I	D II	40	460	27	clay
I	+0.28	+0.38	IV	D III	53	160	64	clay
J	−0.82	−0.52	III	D II	15	550	323	sh
J	−1.52	−2.59	III	D I	30	390—550	255	sh
J	+2.92	+1.44	IV	D V	30	380	350	sh
J	+0.47	−1.35	II	D I	40	80—340	247	sh
J	−1.31	+1.98	I	D V	15	230	417	sh
K	−0.88	−0.81	III	D II	170	630	127	
K	+0.15	+0.39	IV	D II	95	500	85	
K	+1.18	+0.39	IV	D IV	90	250	71	
L	+0.85	+1.49	IV	D I	90	280	42	
L	−1.87	−0.78	III	D II	40	390	154	
L	+2.17	+0.60	IV	D III	80	370	113	
L	+0.26	+1.64	IV	D IV	70	170	44	
L	−0.55	−2.27	III	D I	40	140	122	
L	+0.60	−0.40	II	D II	90	240	33	

TABLE 3. The relation between the magnitude of damage and the position of dam relative to extracted area

Magnitude of Damage	Case Number				
	Case 1	Case II	Case III	Case IV	Case V
D—I	1	4	8	1	0
D—II	2	2	3	4	0
D—III	0	0	0	3	0
D—IV	0	0	0	5	0
D—V	1	1	0	4	1

CONSIDERATION AND CONCLUSION

The relation between the magnitude of damage and the relative position of dam to the excavation is shown in table 3.

The following may be concluded from the table:

(1) The magnitude of damage to a dam is small in the case where the position of the dam is within the compression zones in both the direction of dam axis and of flow axis, if the compressive strain is larger than 3.5 mm/m.
(2) The pond is affected heavily by mining subsidence in the case where the position of dam is within the tension zones in both directions.
(3) Tensilestrain larger than 1.5 mm/m in the direction of the flow axis affects the growth of damage to drainage pipe in a dam.
(4) If the magnitude of tensile strain in the dam body or at the bottom of the pond is larger than 2 mm/m a year, cracks may appear and cause leakage or complete drainage from the pond.
(5) A thick clay layer in the pond bottom prevents leakage of pond water.
(6) If a fault exists near the surface, there is a greater possibility of a crack appearing than in an undisturbed area.

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DISCUSSION

Intervention by Dr. O. G. Ingles (Australia):

Question:

1. With reference to your table, minor damage near pipes appears to me to be randomly distributed (in the strict statistical analysis sense). Have you examined the statistical distribution of your data?
2. With regard to tensile strain, I have found that the tensile strains in excess of about 0.2% are liable to open microtension cracks in soils, and this is also observed by Peremy in France, so it agrees with leakage developing beyond such strains.

Reference: Ingles, O.G. (1969). "Fracture Phenomena in Stabilized Clays", *Proc. 2nd Aust. Conf. Mechanics of Structure and Materials*, 15-1.

Peremy, R. (1965). "Sur les microfissures de roches soumises a des efforts uniaxiaux", *C.R. Acad. Sci.*, 260, 1209.

Answer of Prof. Nishida:

As we are dealing with deep mining, the sinkholes are very rare case and we have only one example. As to the second point, your observation on the tensile strain of dam wall, you may be right. Only I have to make it clear that our data is a result of calculation not actual measurements.

EFFECTS OF MINING SUBSIDENCES ON THE GROUND WATER AND REMEDIAL MEASURES

Prof. Dr. B. WOHLRAB (Germany)

ABSTRACT

The underground mining as a local economic activity has an effect upon with its natural the lithological, hydrological and biological sphere. Therefore, as a rule, changes in the natural water utilization are inevitable. The most important effects in this regard will be explained. Moreover, the special problems of draining the areas of land subsidences caused by mining will be discussed. The difficulties, limits and possibilities of remedial measures in this regard will be discussed by means of models. The report ends with some principles of draining the areas of land subsidences caused by mining.

RÉSUMÉ

L'exploitation minière souterraine est liée à des problèmes de nature lithologique, hydrologique et biologique. Il en résulte des modifications dans l'utilisation de l'eau. Les effets les plus importants en sont exposés. Cependant, les problèmes particuliers du drainage des aires affaissées par l'exploitation minière sont pris en considération. Les difficultés, les limites et les possibilités des remèdes utilisés à cet égard seront discutées à l'aide de modèles. Le rapport se termine par la présentation de quelques principes concernant le drainage des zones affaissées par l'exploitation minière.

1. COMPONENTS OF GROUND MOVEMENT BY UNDERGROUND MINING

First it is necessary to make clear briefly the causal processes of such ground movements. This general view is mainly confined to conditions as they are present in hard coal mining in the Ruhr district (Federal Republic of Germany). Figure 1 shows in a simplified and

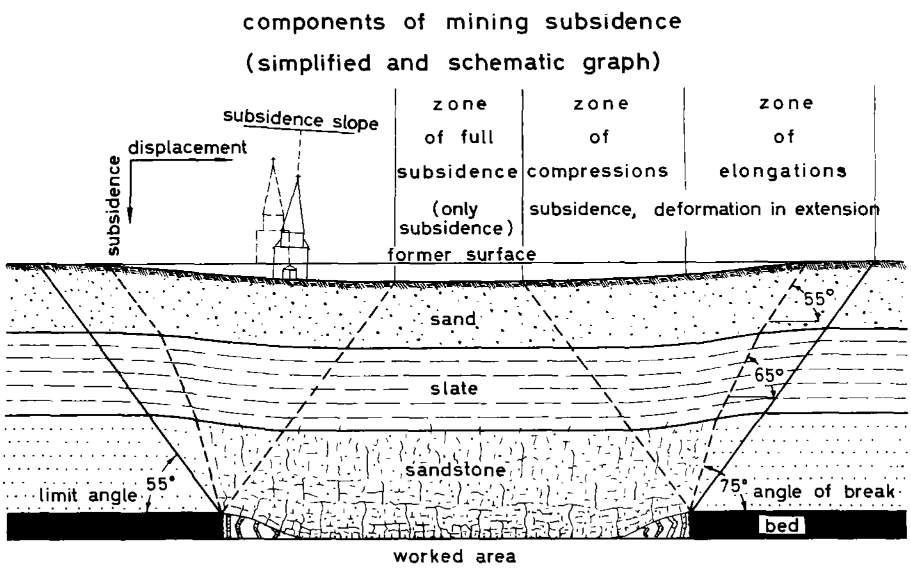


FIGURE 1.

schematic way how the hanging walls have fallen into the cavities caused by the extraction without stowing or packing. The so-called 'angle of break' (Niemczyk) depends on petrographic peculiarities, and on stratigraphic and tectonic conditions of the specific hanging formations as can be seen from figure 1: sandstone 75°, slate 65°, and sand 55°. The ground movements extend beyond the breaking edge. The so-called 'limit angle' leading to the extreme border of subsidence, in the figured case, amounts to 55°. Under a thick layer of overburden the upper strata are deformed according to their plasticity, whereas those hanging directly above the coal-drift fall down.

The effects of ground movements on the surface can be divided into the components: subsidence, subsidence slope, displacement and linear deformation. This deformation appears as a shortening or as an extension. In the centre of a greater mining area there is the zone of full subsidence (fig. 1). Besides the full subsidence itself which decreases towards the through edges, on both sides of this zone appear subsidence slope and displacements. Moreover in the zone of lenght extension elongations appear, and in the zone of lenght shortening compressions appear, resulting from the push to the trough centre. The movements in the hanging formations and the deforming of the surface are more or less dependent on number, thickness, and the tectonics of the worked deposits. Moreover they depend on the direction of the coa mining, and on the kind of stowed, if any, used.

2. DISORDERS OF WATER UTILIZATION AS A RESULT OF GROUND MOVEMENTS

The effects of ground movements appear very often—mainly in flat country—as disorders of the hydrologic conditions. For instance, if a scarcely workable, solid stratum forms the ground-water bed of an upper water-bearing layer, it can rip deeply in the elongation zone as can be seen from figure 2. The consequences are the lowering of the ground-water level (Koehne). Sometimes this can effect a troublesome inrush of water into the underground

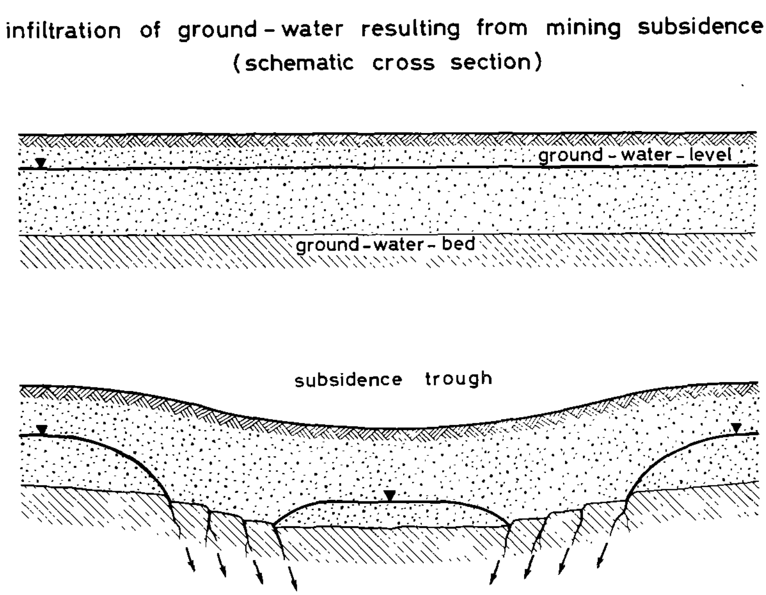


FIGURE 2.

workings, which requires a greater effort for the pit-work (Oberste-Brink). Often this impairs the water utilization.

In case the ground-water bed is thick and plastic enough to withstand the elongations, subsidences can effect a reduction of ground-water level in reference to ground elevation, or even a cropping out of the ground water on the surface (fig. 3). If, at the same time, in the subsidence area the natural flow of water becomes impaired or is stopped, an expanse of water arises resulting in swampy neighbouring districts (fig. 4, 5).

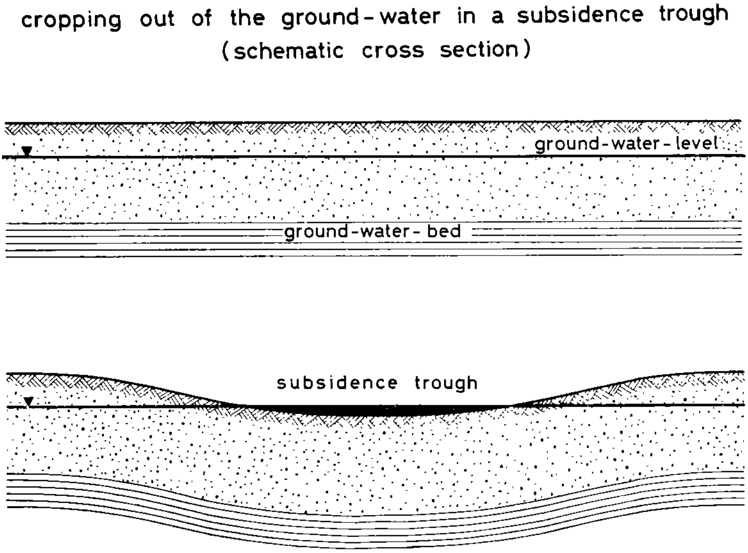


FIGURE 3.

Corresponding to the local morphological conditions, mining subsidences can have an important influence on the water cycle, and thereby also on the cultivation of land. In addition to the subsidences, the other components of ground movements also disorder the water utilization directly or indirectly. Elongations, compressions, displacements and subsidence slope are a disadvantage for hydraulic engineering, as for drainage and irrigation works.

3. REMEDIAL MEASURES

The provision of discharge and drainage for subsidence areas is an espacially important task for the economic continuation and development of the mining fields of the flat country. The provision of discharge and the disposal of waste water in the Ruhr district were delegated to the water associations which were founded especially for this purpose. The dimension and importance of the measures taken by these association can be seen in the size of one area: the catchment areas of the pump works for artificial drainage, the so-called polder, have a size of about 20% of the drainage basin of the Emscherriver, and are more than 170 square km. Great parts of these areas would be submerged, unless the inflow was artificially raised at the lowest level. When in 1945 the pump works stopped, many such areas were submerged (Ramshorn).

A sufficient discharge is a necessary adjunct to areal drainage for the agricultural and forestry areas in the subsidence districts. The difficulties, possibilities, and limits of the measures necessary for this purpose shall be explained by some examples.

Winter 1952

FIGURE 4. *Subsidence area caused by mining in the northeastern part of the district Niederrhein-Westfalen*

First we deal with some cases in which the disorders can be resolved by relatively simple measures.

A subsidence trough without any drainage was formed on a formerly almost flat plateau with a deep ground-water level which was more than 5 m below the surface (fig. 6). The soil consists of loess or loam over porous sands, gravels or split-gravels. The water on the surface, resulting from heavy rain or melting snow, flows together at the lowest stratum on the surface of the subsidence trough. Here it becomes more or less continuous impounded water, according to the permeability of the loam.

It is most practical to drain this temporarily impounded water by an absorbing well, because this particular case the underground sand and gravel is permeable enough. The well drainage can be accelerated by some flat mole drains.

An absorbing well is out of the question if the underground of the loam stratum consists of impermeable or hardly permeable rocks (fig. 7). In this case the flowing surface water must be drained off by a drain-pipe from the subsidence trough to the nearest receiving stream. In the centre of the subsidence trough, it is practical to fill the drain with permeable material (fig. 7).

It is much more difficult to drain subsidence troughs of flat areas which have little natural discharge and ground water close below the surface. Figure 5, for instance, shows a water course in which the surface water flowed together in the subsidence trough and flooded a wider area with the out-flowing ground water, which turned its borders swampy. The drainage of such areas will be explained by means of schematic cross sections, longitudinal sections, and ground plans. Figure 8 shows a natural depression with a water course which flows in meanders through the centre of the ground plan shown. The water course has a sufficient and comparatively regular slope (longitudinal section C-D, fig.8). The cross-section (A-B) shows that the water course and the ground water coincide with each other. Moreover, a superficial site of the ground-water level can be seen in the depression. This area is affected by mining subsidence in such a manner that the zone of full

Summer 1962

subsidence is situated in the upper left edge of the ground plan (dotted line level of equal sinking: 1.5 m, etc.; fig. 8, ground plan). The mining subsidence ends in the lower right edge of the plan (line of equal sinking: 0 m). In this way a different relief develops (new

FIGURE 5. *Flooded subsidence trough, as a consequence of underground mining*

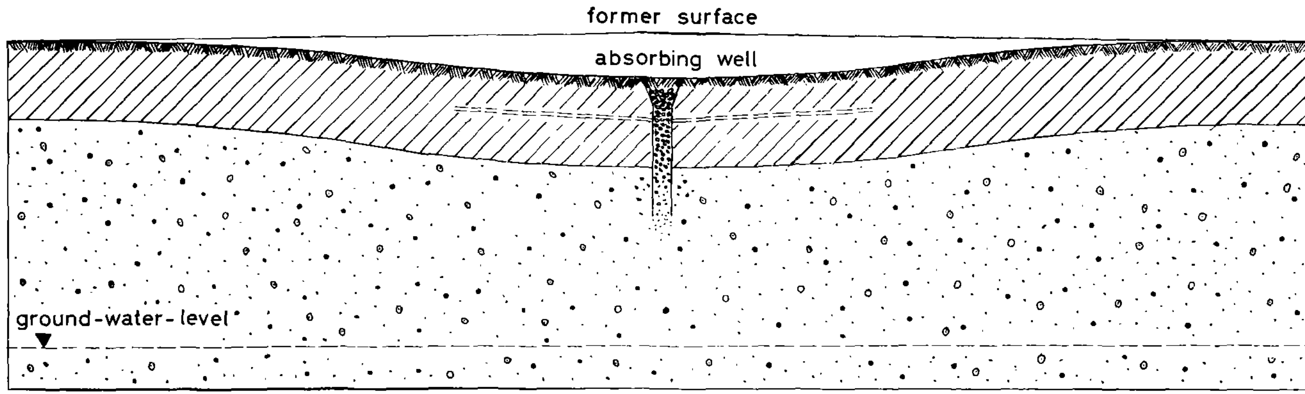


FIGURE 6.

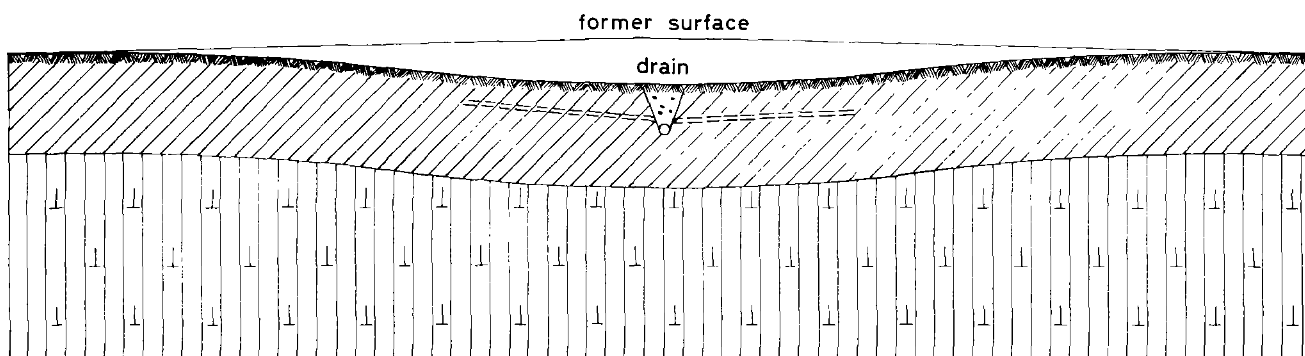
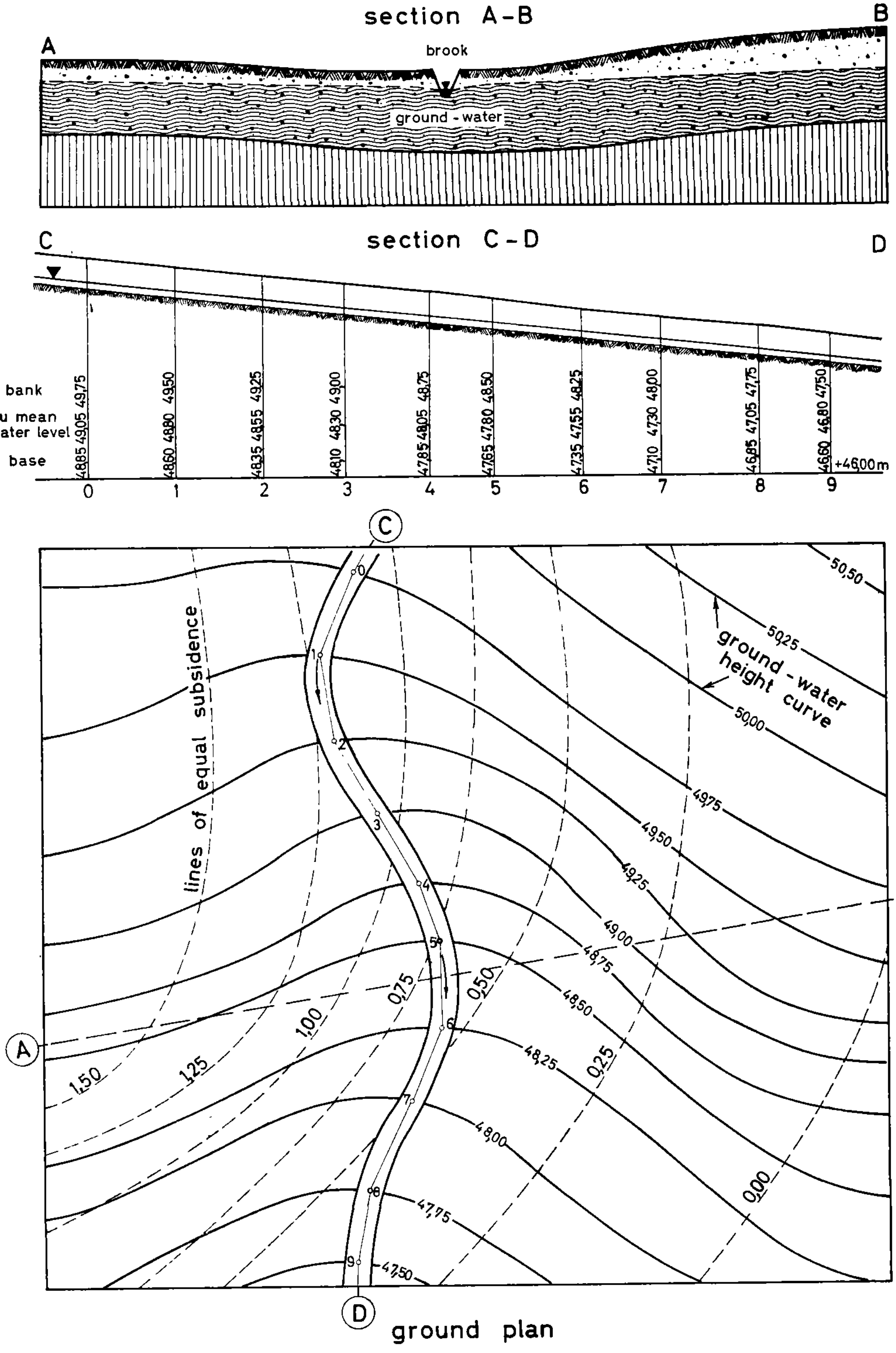


FIGURE 7.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lines of equal subsidence
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ground plan

FIGURE 8.

contour lines: ground plan of fig. 9). The disordered area has in A a drainless basin which extends to the water course and hinders its free discharge (longitudinal section C-D in fig. 9). In the basin the ground water comes to the surface, and forms a coherent sheet of water with swampy neighbouring areas (cross-section A-B and grid-system in the ground plan of fig. 9). The receiving stream, too, is touched by the edge of the subsidence basin, but not strongly enough the completely stop the discharge (longitudinal section C-D).

In spite of the changed morphologic conditions this area can be drained sufficiently by building a receiving stream which flows into the lower course of the brook (outside of the ground plan of fig. 10). At the same time the brook bottom must be leveled and the brook profile must be planked for a sufficient length (longitudinal section C-D in fig. 10) to prevent backing-up of the surface water from the subsidence trough. A drainage system to the receiving stream must provide the conditions that will bring the flooded area back into a regular state and use (ground plan and cross section A-B in fig. 10).

An artificially created flow of water and a suitable ground-water level for the subsidence area can be successfull in the long run only if the ground movements stop, If ground movements continue, you cannot avoid building a pumping station. In spite of the artificially created flow of water, with or without a pumping station, however, drainage of subsidence troughs will remain difficult so long as ground movements go on. In this regard principles can be established for the following three eventualities:

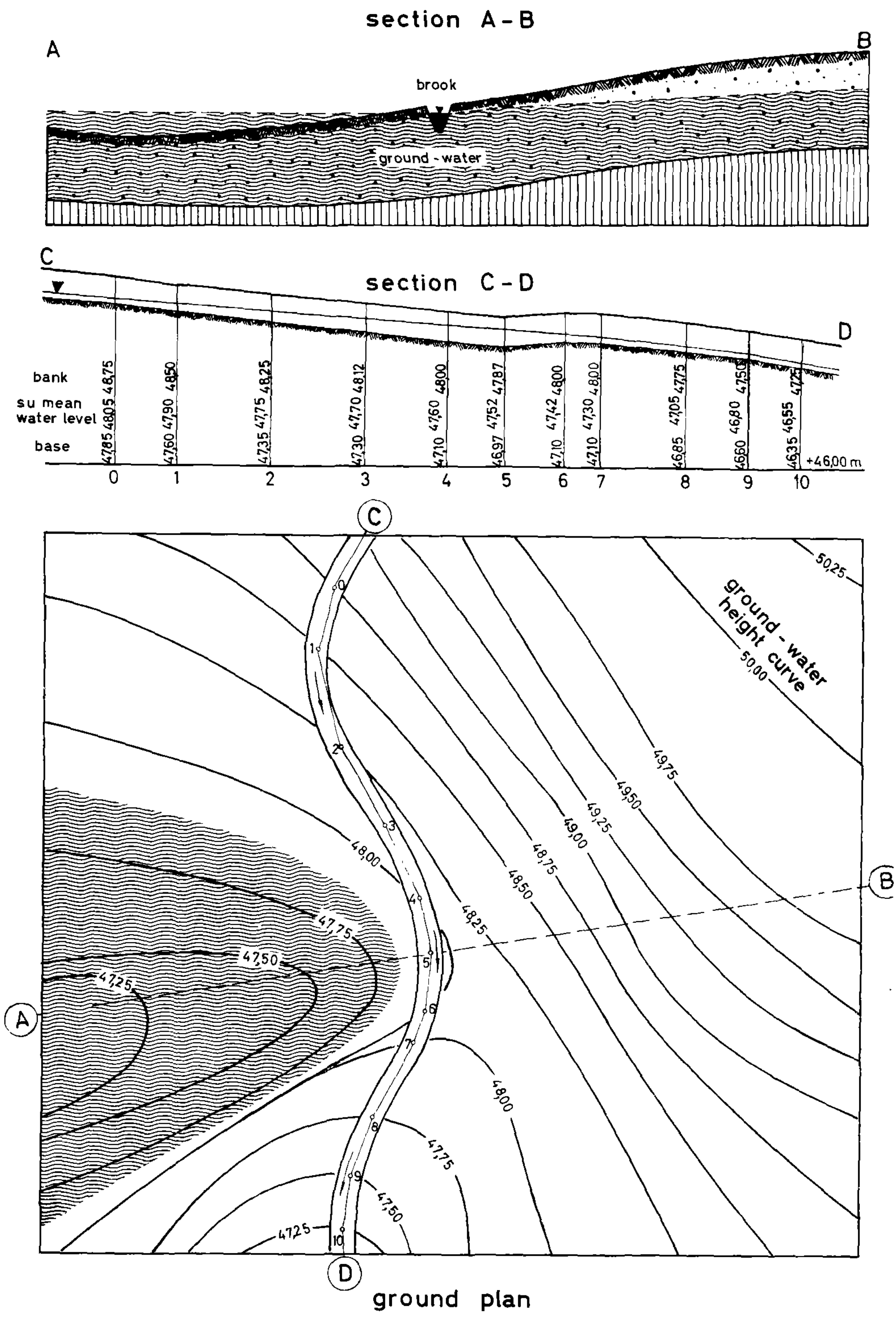
a) It is very easy to plan and to put into effect the drainage measures, if mining leaves the concerned area and the ground movements stop. In this case the drainage can be the same as in those areas which have a natural excess of water. If this is ground water, special care must be taken in regard to foreign water from neighbouring areas, and to possible artesian water as a consequence of lasting disorders of the hydro-geological conditions. In addition, old drains have to be carefully checked (for changed slope) and possibly closed.

b) If there is not as yet a stop of the ground movements in time and dimension, if especially the future conditions of slope and discharge are very uncertain, there must be a temporary draining in a way that does not intensify possible future disorders. In this case one must generally avoid drain-pipes. Apart from open ditches which show further ground movements as they begin mole drains are possible as branch drains in heavy soil.

c) If the ground movements have not yet come to a stop, and if it is certain either that only subsidences are to be expected, or that displacements, compressions, and elongations will be of comparatively small dimension and the subsidence will not change the slope conditions, systematic drainages can be made. The single arterial drainage should be minimal, and the main collector drains and branch drains should be as short as possible, in order to have a better surface localization of actual disorders. An exact lay-out with reliable and fixed observation points in the grounds and a corresponding altimetry are of special importance in this regard.

CONCLUSION

Underground mining as an economic activity is necessarily an intervention in part of the earth with its natural well-established balance of the lithosphere, the hydrosphere, and —if the soil is included—of the biosphere. Changes of the water utilization are normally inevitable. The most important effects in this regard have been described. Moreover, entering into this are special problems of the drainage of mining subsidences. The difficulties, possibilities, and limits of the measures necessary for this purpose have been dealt with by means of examples. The paper ends with some principles for the drainage of mining subsidences.


section A-B
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brook
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section C-D
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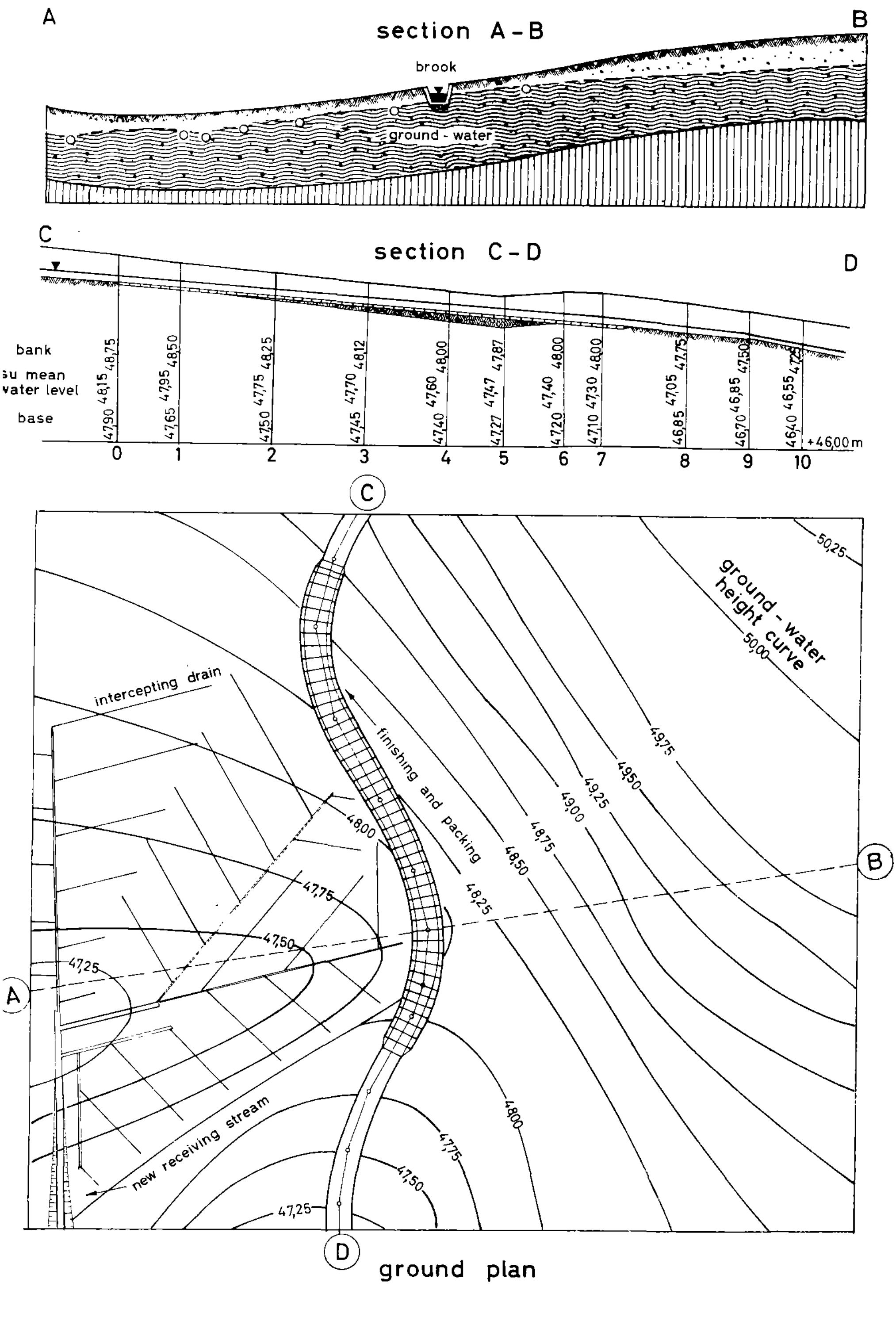
ground plan

FIGURE 9.



A
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section A-B
brook
ground - water
C
section C-D
D
bank
su mean
water level
base
+46,00 m
0
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2
3
4
5
6
7
8
9
10
C
ground - water height curve
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49,25
49,00
48,75
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48,25
48,00
47,75
47,50
47,25
intercepting drain
finishing and packing
new receiving stream
B
A
D
ground plan

FIGURE 10.

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PREDICTION OF HORIZONTAL MOVEMENTS DUE TO SUBSIDENCE OVER MINED AREAS

Kenneth L. LEE[1] and Michael E. STRAUSS[2]

ABSTRACT

Some interrelations between vertical and horizontal movements in ground subsidence problems are summarized. A case history of horizontal movements which develop as a result of subsidence over a sulphur mining area is discussed. The finite element method of analysis is shown to give a good prediction of the correct nature and magnitude of both vertical and horizontal movements resulting from this mining subsidence problem.

RÉSUMÉ

Certaines relations entre les mouvements horizontaux et verticaux concernant les problèmes d'affaissement sont résumées. Le cas historique des mouvements horizontaux qui se développent à cause de l'affaissement à l'intérieur d'une superficie minière de roches sulfurées est discuté. La méthode d'analyse par éléments finis est utilisée pour donner une bonne prédiction de la forme correcte et des longueurs des mouvements verticaux et horizontaux résultant des problèmes d'affaissements miniers.

INTRODUCTION

Removal of material from below the ground, whether it be water, oil, gas, or solids, frequently results in subsidence of the ground surface unless special precautions are taken to prevent this from occurring. Not only do points on the surface move vertically down, but in many cases they also move laterally. Many engineering structures founded on or

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near the ground surface are much more sensitive to horizontal movements than to vertical movements. However, methods for predicting the nature and magnitude of horizontal movements which accompany ground subsidence are not well defined at present. This paper suggests a method for predicting both the vertical and horizonatl components of ground movement and illustrates the method by applying it to an actual case study of horizontal movements caused by subsidence over a sulphur mining area.

A comprehensive review of more than a dozen field examples, where horizontal movements are known to have accompanied land subsidence, has been presented by Lee and Shen (1969). These include subsidence due to mining, withdrawl of oil, water, and gas, and settlement due to consolidation of an underlying compressible layer of soil. A review of these examples indicated that two key features were always present in every case of subsidence which was accompanied by horizontal movements. On the other hand, several examples of vertical subsidence were cited in which there appeared to be no significant horizontal movements. In these cases one of the key features was absent.

These two key features which appear to be essential in order for horizontal movements to develop along with vertical subsidence are: first, the seat of major settlement must be located at some significant depth below the ground surface; and second, there must be significant differential vertical movement across the subsiding area. These two features are illustrated on figure 1 by the idealized example of subsidence due to consolidation of a deep-seated layer of compressible soil capped by an upper layer of relatively stiffer material. This upper material does not compress to cause the subsidence but merely moves

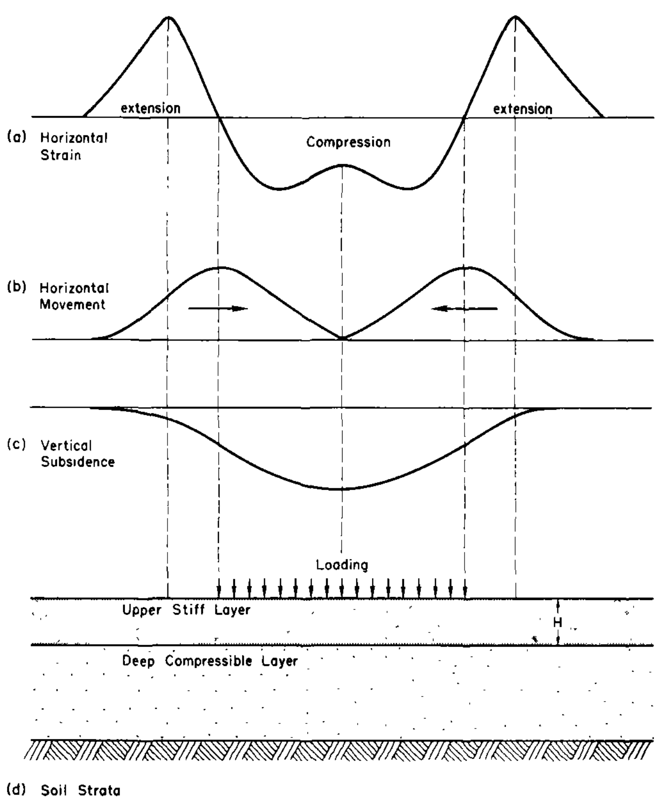

FIGURE 1. *Idealized Surface Movements at a Subsiding Area*

down to occupy the space created by the lower layer of soil as it compresses due to some external or internal cause.

Subsidence due to underground mining would also fit the same analogy shown in figure 1. In that case the subsidence would be caused by removel of material from a deep zone, and again the overlying strata simply moves down to fully or partially occupy the void left by the mining operation.

The idealization shown on figure 1 also illustrates a summary of several important interrelations between soil profile, subsidence and horizontal movements which have been repeatedly observed from actual subsidence areas where horizontal movement data have been obtained.

1. There is no horizontal movement at the point of maximum subsidence, nor are there any horizontal movements at considerable distances beyond the subsidence zone.
2. At each location the direction of horizontal movement is toward the zone of maximum settlement.
3. The point of maximum movement corresponds to the point of steepest slope of the vertical subsidence profile, and the horizontal strain at this point is zero
4. The horizontal strain over much of the central part of the subsidence area is compressive, and extension strains develop near the outer edges of the subsidence zone.
5. The point of maximum horizontal strain is located at the point of steepest of the horizontal movement curve.

These above-stated interrelations are, of course, only approximate and real cases would never be so symmetric and so well defined as indicated on figure 1. However, the data from most real cases agree very well with these idealizations.

Grant (1954) has proposed that the mechanism which leads to horizontal movements may be analogous to the deformation mechanism of a thick plate which deforms to the shape of the subsidence bowl. In the analogy this thick plate corresponds to the upper layer of stiff soil shown in section on figure 1d. Based on an analogous comparison between the deformation mechanism of a bending beam to that of the stiff upper layer, a semiempirical relationship was suggested by Hardy, Ripley, and Lee (1961) and later modified by Lee and Shen (1969) to relate the horizontal surface movements m to the thickness of the upper stiff stratum H and the vertical subsidence profile:

$$m = kH \tan \alpha \tag{1}$$

where

$\tan \alpha$ the slope of the surface subsidence profile at any point;
k an empirical unitless constant between 0 and 1.0.

For several cases it was found that $k = 2/3$ gave good agreement between the observed horizontal movements and those predicted by Equation [1]. However, in all of these cases the edges of the deep-seated subsidence zone were not as abrupt as in the case of a mined out ore body where the cavity causing the subsidence comes to a sudden end at the edge of the mine excavation.

In addition to the semiempirical relationship of equation [1], Lee and Shen (1969) also illustrated that food predictions of horizontal movements could be obtained by means of the finite element method of analysis (FEM). The details of this powerful analytical method are fully described elsewhere (cf. Clough, 1965; Wilson, 1963, 1965).

As an illustration of the application of these two methods, a subsidence model was constructed, and the measured movements were compared with those prodicted by equation [1] and by the finite element method. The model was a beam of granular soil formed in a box 93 in. long by 24 in. deep. The bottom was flexible and could be moved to assume any desired shape. Special precautions were taken to eliminate the effect of friction between the soil and the walls of the box as the soil subsided. The experiment was conducted by deforming the bottom of the soil beam to correspond to one-half of a typical subsidence profile. The corresponding surface movements were then measured.

A shematic view of this soil beam model is shown on figure 2a only one-half of an assumed symmetrical subsidence profile was modeled. The horizontal movement of points on the surface of the soil corresponding to a particular subsidence profile are shown on figures 2b and 2c. Also shown are the corresponding horizontal movements predicted by equation [1] and by the finite element method. Either of the two prediction methods gave satisfactory results for this case.

In addition, Lee and Shen (1969) have also used both prediction methods to study the horizontal movements resulting from subsidence of earth dams, and again found satisfactory agreement. However, in all of these cases the edges of the deep-seated subsidence were not abrupt, as for example in the case of mining subsidence. Therefore, in order to investigate the possible application of either of these methods to mining subsidence problems, the following study was undertaken.

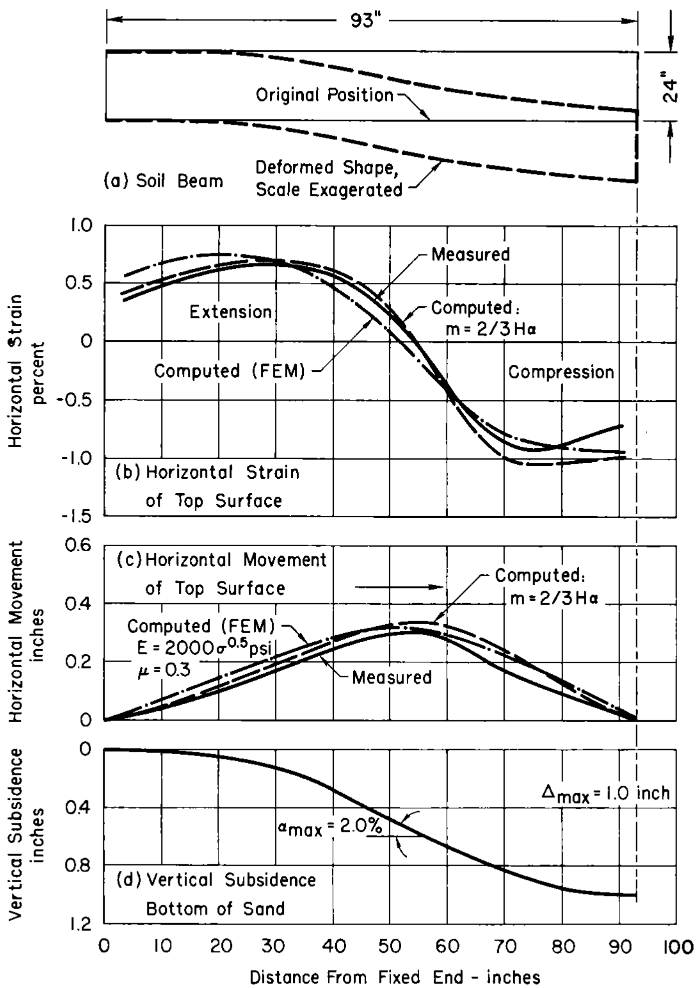

FIGURE 2. *Measurement and Computed Horizontal Movements in Model Beam*

APPLICATION TO MINING SUBSIDENCE

A case history of ground subsidence caused by mining sulphur in the gulf coast region of Texas has been described by Deere (1961). At this site sulphur was produced from depths of from 1300 to 1500 feet below the surface. The material overlying the producing zone was described as "typical coastal plain unconsolidated sediments of Tertiary to Recent age, slightly cemented to uncemented sands, gravels, clays and clay shales often with limey zones and concretions." The sulphur was extracted through numerous wells drilled through these soils. The materials below the producing zone consisted of gypsum, anhydrite and brecciated limestone, but had little, if anything, to do with the pattern of surface subsidence, and they were not included in any of the theoretical studies.

Shortly after the mining operations commenced, vertical and horizontal movements began to be noticed, and a survey system was quickly set up to measure these deformations However, a small amount of initial movement had already developed before the measuring

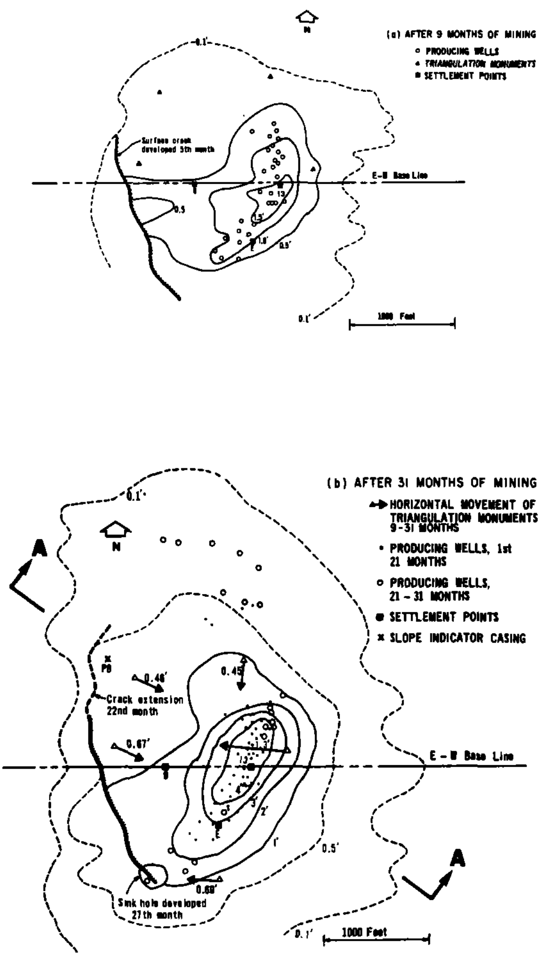

FIGURE 3. *Plan Views Showing Contours of Subsidence and Horizontal Movements of Sulphur Mining Area After 9 and 31 Months of Mining (After Deere)*

systems had been completely installed, both these early, unmeasured movements were thought to have been relatively small. In the theoretical analysis the measured movements between two periods of time were compared with the theoretical movements for the same conditions, and thus the absence of a zero reference was of no importance to this study.

A plan view of the mining area indicating the location of the producing wells, and the vertical subsidence which had developed after nine months of production, is shown on figure 3a. The maximum vertical surface settlement amounted to about 1.5 ft., and the contours of equal settlement define a subsidence bowl extending about 2000 ft. in all directions from this zone of maximum settlement, and considerably beyond the zone of producing wells. Five months after mining began, a crack developed on the west side of the area which ran more or less parallel to the settlement contours.

The mining operation was continued by extending the zone of producing wells outward from the center. This was accompanied by a continuation of the vertical settlements, surface horizontal movements, and extension of the crack. The conditions after thirty-one months of production are illustrated in figure 3b. By this time the magnitude and extent of the zone of vertical settlement had enlarged considerably. The time settlement data for three typical settlement points are shown in figure 4 and indicate that the settlement occurred in a more or less steady manner. The magnitude and direction of the horizontal movements which developed between the ninth and thirty-first month are shown in figure 3b by vector arrows from the survey monuments. These movements are generally directed toward the zone of maximum subsidence, although one point in the south appears to have been influenced by a nearby sinkhole which developed during the twenty-seventh month.

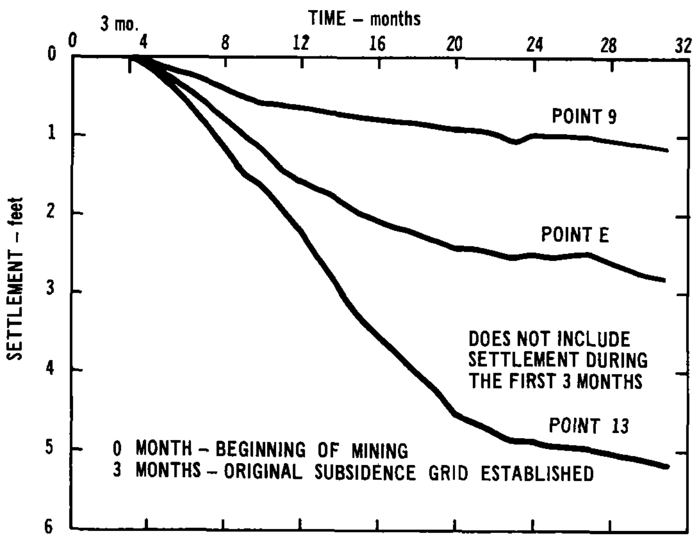

FIGURE 4. *Time Settlement Curves for Three Points Within the Subsiding Zone of the Sulpher Mining Area (After Deere)*

During the interval of time between the ninth and thirty-first month, the crack had extended a considerable distance to the north. Moreover, there was an abrupt change in the settlement contours across the crack indicating that it probably extended to a considerable depth. A slope indicator P8, which had been installed within the subsidence bowl about 160 ft. from the crack zone indicated the location of a definite shear zone at a depth of 77.5 ft. This suggested that the crack probably extended as a fault all the way from the surface down to the zone of sulphur production as shown in figure 5.

The first objective of the theoretical study was to see if the finite element method of analysis would lead to any information which might suggest that a deep crack or fault could be expected to develop. An axisymmetric FEM program (Wilson, 1965) was used with a grid of rectangular elements over a section of ground 6000 ft. radius and 1500 ft. deep. It was assumed that the mining operations were restricted to a narrow, level zone at a depth of 1500 ft. below the surface as shown on figure 6. In the analysis, gravity was assumed to be zero. The physical mining operation, which in reality involved removing vertical support across the producing zone, was simulated in the FEM analysis by applying a tension stress across the producing zone equal to the overburden pressure.

The soil above the producing zone was essentially a granular material, and for calculation purposes was assumed to be completely homogeneous and isotropic. Since the overburden material was essentially granular soil, the modulus was assumed to increase with depth as defined by $E = 2.4 \times 10^4 \sigma_0.^5$ psf where σ is the vertical overburden pressure in psf. This type of variation of modulus of granular soil with confining pressure is typical of many data in the author's files. A value of 0.3 was assumed for Poisson's ratio.

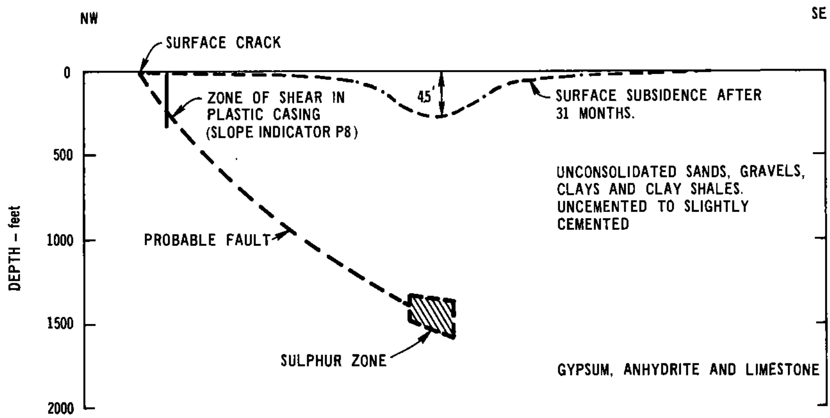

FIGURE 5. *Section AA Showing Sulphur Zone, Probable Fault Location, and Surface Subsidence After 31 Months (After Deere)*

Subsidence during the first few months was assumed to have occurred without a fault developing, and therefore no provision was made for a fault in the FEM analysis for these early movements. The analysis was made for an assumed width of producing zone of 250 ft The horizontal strains computed by the FEM along the surface, and at two depths within the overburden soil are shown in figure 6. The form of these curves is similar to that shown in figure 1 which summarizes observed movements at many locations. There is a compression zone in the center which is surrounded by a zone of extension.

If a crack or a fault were to develop, it seems reasonable to assume that it would probably develop at regions where the horizontal extension strain was the greatest. As shown in figure 6, the line joining these points of maximum horizontal extension strain at various depths is in close agreement with the assumed position of the fault.

Prior to the development of the fault, the study was treated as a symmetrical problem. Since nothing is completely uniform or symmetrical in nature, it is not surprising that a fault developed only on one side, where no doubt the soil was either slightly weaker, or the stresses were slightly greater than elsewhere. As soon as a fault developed, the problem no longer would be even approximately symmetrical. Since the resistance to deformation along the fault would probably be much less than in the surrounding intact soil, the presence of the fault would be expected to be very important in governing all of the subsequent movements.

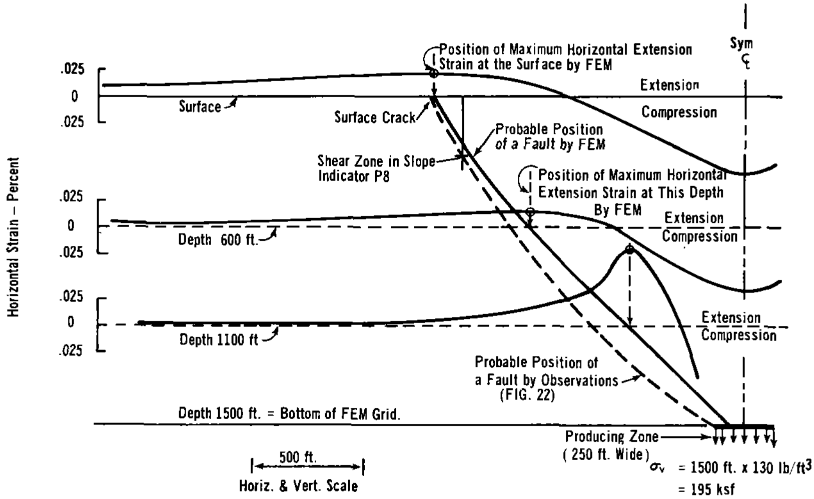

FIGURE 6. *Horizontal Strains Computed by the FEM in Material Above the Sulphur Producing Zone Along Section AA*

Therefore, to study the deformations which occurred after faulting (which included most of the observed deformations) a new FEM grid and a plane strain program (Wilson, 1963) were set up. The fault was simulated by a line of thin elements with a modulus of only 1/50th of the modulus of the surrounding soil. The basic element in the large grid was 150 ft. sq. The element in the fault zone was 30 ft. wide by 150 ft. long. This is a fairly wide fault zone, but the width was required in order to have a conveniently workable FEM grid. Since the study was concerned mainly with the over-all movements rather than the particular deformations within and adjacent to the fault, it was felt that this method of simulating a fault zone was satisfactory for the purpose intended.

The vertical subsidence profile along the E-W base line computed by the FEM is compared to the actual observed vertical subsidence profile in figure 7, although there are obvious differences in the two curves, the general forms are encouragingly similar. Both curves show a large differential settlement across the fault. The amount of maximum subsidence in the zone of major settlements are also similar for both curves.

It was therefore of interest to extend the calculations to the conditions after thirty-one months at which time larger settlements had developed, and considerable surface horizontal movement data had been obtained.

As shown on figure 3, the producing zone had been enlarged considerably between the ninth and the thirty-first month. Although most of this extension was in the north-south direction, there was probably also some extension in the east-west direction as well. Therefore, for the purpose of analysis it was assumed that by the thirty-first month the width of the producing zone along the E-W base line was about 450 ft. The theoretical movements after thirty-one months were computed on this assumption. The movements which had been computed for a production zone width of 250 ft. as shown on figure 7 simulating the conditions after nine months, were subtracted from this thirty-one month data to obtain the movements within the interval nine to thirty-one months. These computed movements are shown on figure 8. The observed movements between nine and thirty-one months are also shown on figure 6 for comparison. Again in spite of some

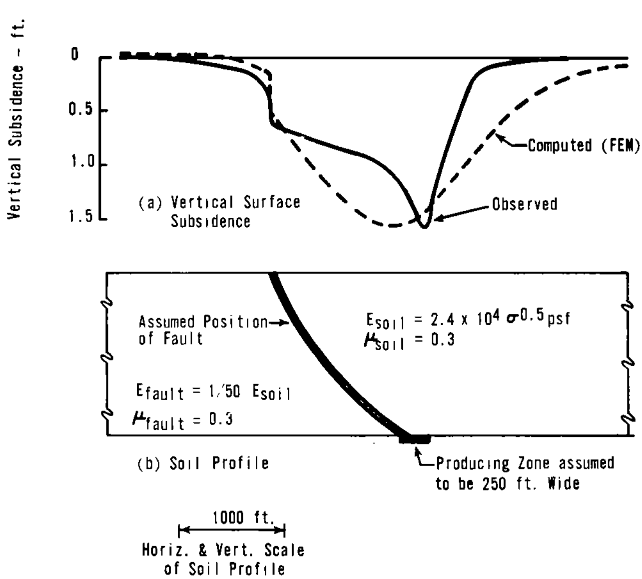

FIGURE 7. *Observed and Computed Vertical Surface Subsidence Along the E-W Base Line After 9 Months*

discrepancies in the details the general agreement is encouraging, both as to horizontal as well as vertical movements.

In each case the observed settlement profile was narrower and steeper than the computed profile. Various combinations of modulus, Poisson's ratio, and width of producing zone were tried to see if a better agreement could be obtained. The effect of varying these conditions resulted in different absolute magnitues of the computed theoretical movements, but the general form was always the same. The factor which had the greatest influence on the magnitude of movements was the assumed width of the producing zone. However, the widths of the producing zone which are required to give good agreement with the observations do not appear to be unreasonable from the known surface position of the wells as shown on figure 3. The fact that the computed subsidence profile is not as steep as the observed profile suggests that perhaps the properties of the overburden soil may be somewhat different than the simple values which were assumed, and that they might vary significantly over and around the producing zone.

It is of interest that a small amount of vertical uplift occurred at the outer edges of the subsidence zone, especially on the side adjacent to the fault. The amount of this observed uplift was only of the order of about 1 in. at the maximum, and therefore hardly shows up on the scale used in figures 7 and 8. It is further interest, however, that in the FEM of analysis a small uplift of about this same order of magnitude was also computed for about the same zones that uplift was actually observed.

In addition to the FEM studies, an attempt was made to compute the horizontal movements using equation [1]. This attempt was not entirely successful. It was not immediately obvious what values of H and k would be appropriate for this type of problem. Using H = 1500 ft., k = 2/3, and tan α equal to the slope of the observed surface subsidence profile. The computed horizontal movements were of the order of 5 to 10 times greater than the observed movements. However, the extent and the direction of these computed horizontal movements were in good agreement with the observed data.

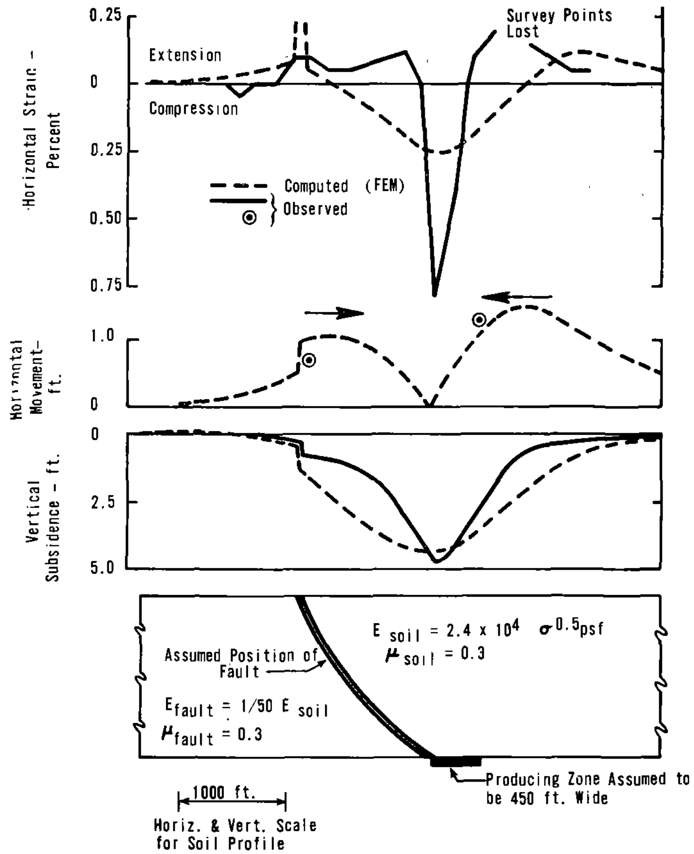

FIGURE 8. *Observed and computed Surface Movements Along E-W Base Line for Period 9 to 31 Months*

CONCLUSIONS

This study has reviewed some qualitative and quantitative interrelations between vertical subsidence, geological conditions and resulting horizontal movements. A simple previously-suggested emperical expression which has been found to be useful in predicting both the nature and magnitude of horizontal movements for some subsidence problems has been found to be only of qualitative value in predicting horizontal movements for this problem. However, the finite element method of analysis has been found to be useful in predicting the nature, extent and magnitude of both vertical and horizontal movements. Some discrepencies exist. However, with a better knowledge of the soil properties and boundary conditions it is probable that the accuracy of the predictions may be further improved.

ACKNOWLEDGMENTS

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DISCUSSION

Intervention by Mr. Ben E. LOFGREN (U.S.A.)

Question:

1. In your sand model, in order to get tension stresses, do you have to have tension stress right within your model?
2. In the actual field mine, did you have a groundwater condition with groundwater gradients towards the excavation which might also be involved in your system?

Answer of Prof. LEE:

I will answer the second question first. I do not know. We obtained the data for thus mine problem from a paper by Don Deere, Ref. 2; it did not mention whether there were groundwater gradients or not.

With regard to the first question, as shown on Fig. 6, we applied a vertical tension stress at the level of the mine, and neglected all the pre-existing gravity stresses due to the overburden material. Thus we were only studying the effect of a stress change. If we would have added the overburden pressure, then the net effect of our applied stress would have been to reduce the vertical pressure at the mine from the value of the overburden pressure down to zero. However, since we were dealing with an assumed linear elastic material, and were only concerned with strains or deformations rather than with stresses, the final resulting changes in strain or deformation were the same whether gravity stresses were included or not. Since it was easier to neglect them, they were neglected. However, if instead of horizontal strains and deformations as shown on the figures, it would have been of interest to compute stresses, then the gravity stresses would have had to be applied also, or superimposed on the stresses which were computed ignoring gravity.

SUBSIDENCE OF ORGANIC SOILS IN THE U.S.A.[1]

John C. STEPHENS and William H. SPEIR[2]

ABSTRACT

Organic soils subside when drained: by shrinkage from drying, loss of groundwater bouyancy, compaction, wind erosion, burning, and biochemical oxidation. Relative loss due to each causative factor depends on soil origin, climate, and land management.

Investigations in the U.S.A. by ARS show shrinkage rate proportionate to drainage depth—the lower the water table, the greater the subsidence. Level surveys at 5-year intervals from 1913 to 1968 have established the pattern of subsidence in the Florida Everglades; viz., initially rapid, mainly from shrinkage and compaction, then declining to a continuous steady rate, primarily from oxidation, until underlying mineral material is reached. Arable peats have averaged sinking 3 cm per year. Predictive studies indicate Everglades peats will be too shallow for agricultural use by 2 000 A.D.

Under similar drainage, organic soils subside faster in warm climates, and lowmoor faster than highmoor peats. To conserve organic soils, water tables should be kept as high as crop and field requirements permit.

RÉSUMÉ

Le sols organiques se tassent quand ils sont asséchés par rétrécissement dû au séchage, par réduction des quantités de fluides, par compactage, par érosion due au vent, par combustion et par oxydation biochimique. La perte relative due à chaque facteur causatif dépend de l'origine du sol, du climat et de l'entretien de la terre.

Des investigations aux Etats Unis par l'ARS montrent une vitesse de tassement proportionelle à la profondeur d'assèchement — plus le niveau de l'eau est bas, plus le tassement est grand. L'examen des niveaux à des intervalles de 5 ans de 1913 à 1968 ont établi le mode de tassement dans les " Everglades " de Floride ; rapide au début, principalement dû au rétrécissement et au compactage, se ralentissant ensuite à une vitesse continue, principalement due à l'oxydation jusqu'à ce que le matériel minéral se trouvant en dessous soit atteint. Les tourbes cultivables se sont en moyenne tassées de 3 centimètres par année. Des études de prédiction indiquent que les tourbes des Everglades ne seront plus assez profondes pour usage agricultural vers l'an 2000 A.D.

Sous l'action d'assèchements semblables, les sols organiques se tassent plus rapidement dans des climats chauds et plus rapidement dans les tourbes des plateaux marécageux que dans les tourbes des bas fonds et plaines marécageuses. Pour conserver les sols organiques, les niveaux d'eau devraient être maintenus aussi haut que les cultures et exigences des champs le permettent.

INTRODUCTION

All organic soils lose surface elevation, or subside, when drained. Known causes, apart from possible regional tectonic earth movements are 1) shrinkage due to desiccation; 2) consolidation by loss of the bouyant force of groundwater, and loading, or both; 3) compaction with tillage; 4) wind erosion; 5) burning; and 6) biochemical

1. Contribution from Southern Branch, Soil and Water Conservation Research Division, Agricultural Research Service, USDA, in cooperation with University of Florida Agricultural Experiment Station, and Central & Southern Florida Flood Control District. presented at the Symposium on Land Subsidence; IASH Proc., Tokyo, Japan; Sept. 1969.
2. Research Hydraulic Engineer and Research Investigations Leader, Watershed Engineering, ARS-SWC, Athens, Ga.; and Hydraulic Engineering Technician and Project Leader, ARS-SWC, Fort Lauderdale, Fla.

NOTE: Numbers in parentheses indicate references cited.

oxidation. A knowledge of the rate of settlement and the relative influence of each of the causative factors is essential for the economic development and management of agricultural peats.

SUBSIDENCE RATES

In the Everglades of Florida (USA), the arable organic soils have an average subsidence rate, after initial settlement, of 1 inch (3.05 cm) per year. Concrete monuments, with their tops originally flush with the ground surface and set directly on the solid underlying rock some 45 years ago, now protrude some 5 ft. above ground and furnish visual evidence of soil loss. Figure 1 shows such a monument set at the Everglades Agricultural Experiment Station in 1924 as it appeared in 1968, after $5\frac{1}{4}$ ft. (1.6 m) of subsidence had occurred.

In the organic soils of the Sacramento-San Joaquin Delta of California (USA), average rate of subsidence is reported as slightly over 3 inches (7.6 cm) a year [17]. These soils are tule-reed peats.

In Michigan (USA), average losses in surface elevation on organic soils measured over 5-year period at 13 sites were 0.30 ft. (9.16 cm) [2]. Near Hennepin County, Minnesota, the total settling varied from 0.5 ft. (15.1 cm) to about 1.1 ft. (33.6 cm) with subsidence approximately proportionate to water-table depths.

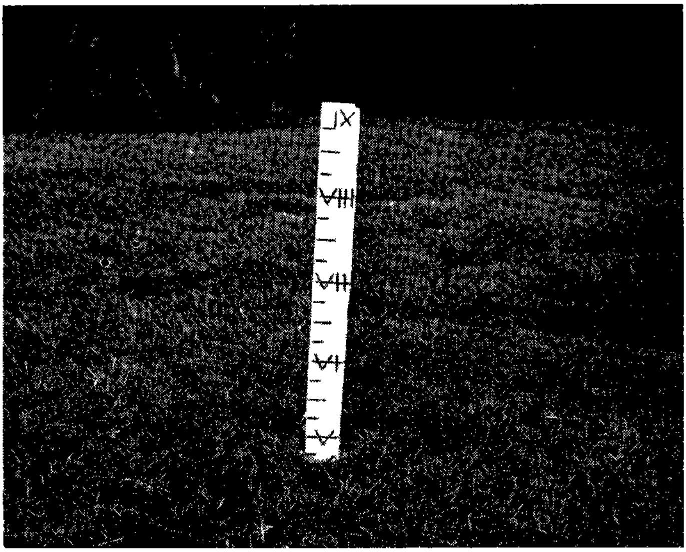

FIGURE 1. *The concrete monument at the Everglades Experiment Station showing subsidence losses between 1924 and 1968*

INFLUENCE OF CLIMATE, PEAT ORIGIN, AND GROUNDWATER DEPTH ON SUBSIDENCE RATES

Investigators generally agree that subsidence rates for organic soils are positively correlated with groundwater depth—the higher the water table, the lower the subsidence rate. There is, however, no general agreement on the proper depth at which water tables should be held for good land management; or on the relative effect of each of the factors that cause subsidence. Most researchers in the U.S.A. and England believe that deep drainage of organic soils promotes destruction of organic matter, eventually leading to the loss of the organic bed. Several Russian researchers, however, regard deep drainage drainage as a soil-forming process that improves fertility and transforms the underlying subsoil layers into cultivable soil. We believe that both viewpoints are valid when local soil conditions, crop needs, and yield objectives are considered.

To obtain an idea of the rate and amount of future organic soil subsidence under various drainage and climatic conditions, brief reviews of some research findings are cited.

Netherlands Findings:

A Netherlands study was made in the polder Mastenbroek to determine the relative effect of the various components constituting total subsidence [9]. The polder was reclaimed in 1913 and the organic soil covered by a clay layer 8 to 12 inches (20 to 30 cm) thick. The area is used as permanent pasture. Peat depths, excluding the clay cover, varied from 53 to 94 inches (1.35 to 2.40 m). The total subsidence for 50 years ranged from 4 to 24 inches (0.10 to 0.65 m), depending on groundwater level. Total subsidence was considered to be composed of settlement, oxidation, and shrinkage.

The settlement was calculated from the bulk densities of the subsided and unsubsided profiles. Oxidation losses were computed as the difference in total weight of organic matter in the subsided and unsubsided profiles. Shrinkage was determined as the residual term.

Of the total subsidence, settlement accounted for 30%, oxidation for 20%, and shrinkage for 50%. From 80 to 90% of the total soil subsidence occurred above the water table.

USSR Findings

Skoropanov [10, 11] gives an account of peat subsidence in Belorussia at the Minsk bog experiment station. These soils were developed as sedge, sedge-cane, and wood-cane peat, and are classed as lowmoor peats having a pH of 5.6. Minsk is near latitude 54° N. Frost-free days for peat-bog soils at Minsk average 123 days.

When the water table was at 47 to 69 inches (120 to 150 cm), the amount of organic matter decomposed annually was about 5 tons/ha (12.35 tons/acre), producing approximately 254 to 265 lb. (115 to 120 kg) available nitrogen. However when the water table depth was at 28 to 31 inches (70 to 80 cm), the amount of decomposed organic matter did not exceed 2.5 tons, containing 132 lb. (60 kg) of nitrogen, a quantity insufficient for high crop yields.

In 1961, thickness of the Minsk peat deposit did not exceed 3.28 ft. (1 m), whereas it was 6.56 ft. (2 m) at the beginning of the cultivation in 1914 [11]. Research under Belorussian conditions showed that oxidation accounted for 13 to 14% of the decrement on two plots, which had been cultivated for 46 years; and for about 5% on another plot, which had been drained but not cultivated [10]. Skoropanov contends that oxidation should not be considered as destruction of soil, but instead, represents the process of soil formation.

According to Ivitskii [3], deep depression of the water table intensifies decomposition of the organic matter by mineralization; and that, although this process is an integral aspect of soil fertility, excessive drainage is undesirable. He recommends that the drainage norm for intertilled crops and grains for low-bog soils be at least 24 inches (60 cm), but not over 69 inches (150 cm) for the presowing period. During the growing season, he recommends an optimum norm of 47 to 51 inches (120-130 cm), with maximum depth not exceeding 87 to 102 inches (220-260 cm).

USA Findings

Laboratory studies. — The biochemical aspect of peat decomposition is indicated by the laboratory studies of Waksman and Purvis [15]. Peat types, such as lowmoor, highmoor, forest, and sedimentary, differ in their chemical composition, in the nature of the microbial flora inhabiting them, and in the rate of attack by micro-organisms. Thus, peats vary considerably in the rate of decomposition, depending on different environmental conditions and treatment.

Waksman found for samples of Florida lowmoor peat (those with pH's from neutral to basic) that about 15% of the total (dry weight) was decomposed at 28 °C. in 18 months, most of which could be accounted for as CO_2 gas. The optimum moisture content for decomposition was 50 to 80% of the total moist peat. Above and below this moisture range the rate rapidly diminished. Wet and dry cycles geatly stimulated peat decomposition as compared to constant moisture.

In related studies [16], oxidizing bacteria were found to be most numerous in drained lowmoor peats and least numerous in the extremely acid highmoor peats. However, when the acid peats were limed, manured, and put under cultivation, the micropopulation increased to about that of the lowmoor peats under similar drainage, and decomposition rates also increased to about that of the lowmoor peats.

Cold climate retards the activity of micro-organisms. Waksman found that organisms causing decomposition were perceptibly active only when soil temperatures remained above 40 °F. (5 °C.). Jenny [4] noted that microbial activity generally doubled for each 10°C. increase in soil temperature.

Water table studies. — At the University of Florida Everglades Agricultural Experiment Station in 1934, a field of Everglades peat was divided into eight blocks, each 100×240 ft (30.5×73.4 m) and provided with a system of ditches, underdrains, and check dams, so that the water table could be held at any depth desired [1, 14]. Water levels ranging from 12 to 36 inches (30.5 to 91.4 cm) below the surface, and varying about 6 inches (15.2 cm) between blocks, were established in 1936 and held at the same depths until 1943. Surface levels were rerun and soil samples were taken annually from each block to determine elevation losses and any compaction changes.

The rate of subsidence was found to be dependent upon the depth to the water table — the higher the water table, the lower the soil loss. Density and mineral-content determinations showed that all soil losses took place above the water table. By collecting the soil gases, Neller [8] found that the production of carbon dioxide (CO_2), a function of oxidation, was directly related to the amount of soil loss.

At the Purdue University Muck Crop Experiment Station, near Walkerton, Indiana, on controlled plots similar to those at the Everglades Experiment Station, the rates of subsidence were about half those for Florida [6, 7]. This difference can be explained when water-table treatments are considered. Prescribed water tables were held year-round in the Florida experiments, while in the Indiana experiments the lowered water tables were held only during the crop year, from May to September. Furthermore, during the winter season the Indiana plots were exposed to freezing temperatures.

Significant results from the Florida and Indiana water-table studies on subsidence rates are summarized in figure 2.

FLORIDA EVERGLADES

The Everglades peat and muck soils were formed in a shallow trough of limestone, approximately 100 miles (161 km) long and 40 miles (64 km) wide, extending southward from Lake Okeechobee to the sea [12]. In past historic times, the Lake overflowed its southern and eastern rims during the rainy season each year. This overflow, plus an

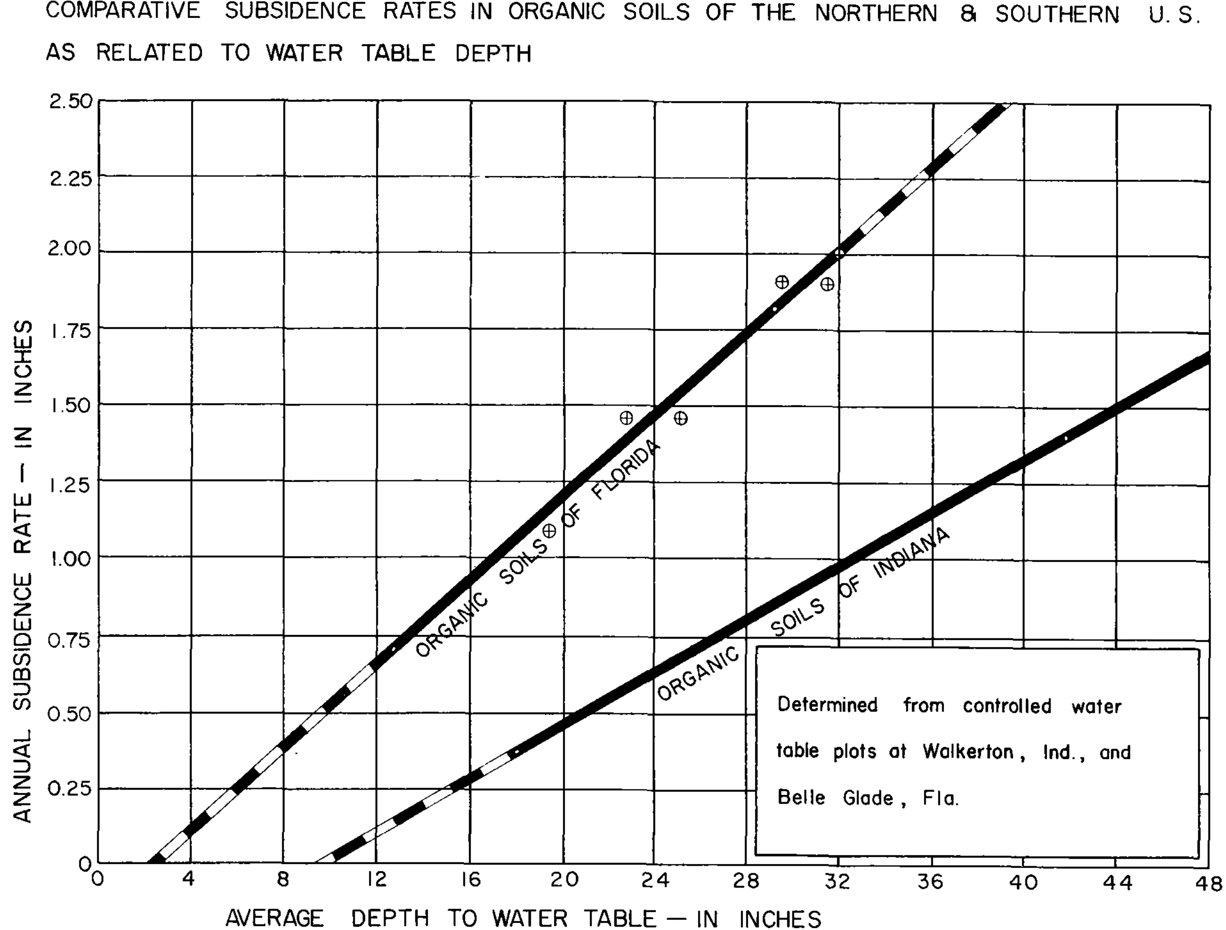

FIGURE 2. *This shows that the rate of subsidence is dependent upon the depth to the water table in organic soils. The greater the depth to the water table, the greater the subsidence rate. Data from Indiana, based on drainage depth for crop year (May to September); for Florida, drainage depth was held year-round.*

annual rainfall of about 60 inches (153 cm), inundated the entire Everglades basin and provided a suitable environment for building up the present complex of organic soils.

Near the shore of the Lake, silt, clay, and colloids were carried in suspension to intermix with the remnants of plants to form muck, and occasional interfingering layers of peaty-muck. When reclaimed, these soils became excellent croplands. Most of the other arable Everglades soils were derived from emergent reed and sedgelike plants, principally sawgrass (Cladium jamaicensis). Today, these lowmoor-type sedge peats, together with the mucks, comprise the agricultural soils of the Everglades.

Beginning about 1906, anaerobic conditions in the Everglades were upset by the installation of canals leading from Lake Okeechobee to the sea. The water balance was disturbed to the extent that aerobic conditions prevailed. Thus, the process of peat accumulation was reversed and subsidence began. Drainage was intensified in the following years and the original hydrology of the Everglades region has been greatly altered.

Subsidence studies in the Everglades were instituted soon after drainage works were started. Several methods of investigation have been used. First, the drop in land elevation has been periodically measured along selected profile lines. Second, the original soil depths were compiled from the first drainage surveys and compared with those obtained from later surveys to determine the total subsidence over the whole of the arable soils of the upper Everglades, which comprise about 1,000 square miles (2,590 sq. km). Third, water table plots, as previously described, were employed.

Subsidence Line Studies

The first elevations of surface profiles along selected subsidence lines in Florida were originated in 1913. Additional lines were established in the early 1930's at the Everglades Experiment Station, where land use and treatment could be controlled. Some 15 lines altogether have been established, and about 11 are still being surveyed. Those discarded have been negated by construction; or, have subsided until the organic surface has disappeared and the underlying mineral material exposed.

The original subsidence lines were usually 1 mile (1.61 km) long. Elevations were established on permanent bench marks set in bedrock near the point of origin. Loss in elevation has been determined by periodically resurveying these lines and taking the average of the elevations over the length of the line as the ground surface elevation at the time of the survey.

Three curves, which show the characteristic sequence of observed subsidence in the soils of the upper Everglades, are shown for profile lines *A*, *B*, and *C* in figure 3.

Line *A* is located on peaty muck soil, which had gravity drainage prior to the installation of pumps in 1927. Very fast sinkage occurred during the first 5 years after initial drainage. As the ground elevation fell to a level where gravity drainage was not effective in lowering the water table, the subsidence rate leveled off. When the water table was lowered by pumping in 1927, the subsidence rate increased because of better drainage and also initial tillage. After initial tillage, a relatively steady loss of about 1 inch (2.54 cm) per year has resulted under pumped drainage. This line is located normal to the North New River Canal, a major drain, below South Bay Locks near the 1-Mile Post.

Line *B* is also on peaty muck soil, located 280 ft. (85.5 m) north of the Bolles Canal, a major drain at Okeelanta. This land had gravity drainage until 1942, when pumps were installed. The line runs parallel to the main drainage canal instead of normal, as in the case of line *A*. Historically, line *B* has had better drainage than line *A*. Total subsidence has been 7.94 ft (2.42 m) for line *B*, and 6.21 ft. (1.89 m) for line *A*. These curves show the characteristic pattern: initial susbidence, a decreasing rate under gravity drainage as the ground sank and drainage was impaired, and the consequent increase in subsidence rates after the installation of pumps.

Line *C* was located on Everglades peat at the Everglades Experiment Station in 1934, and is representative of land that has had its initial subsidence from drainage and cultivation. It has been planted to truck crops continuously with controlled drainage for the period of observation, and probably represents average subsidence conditions — 1.05 inches (2.67 cm) per year — on lands planted to truck crops in the Everglades.

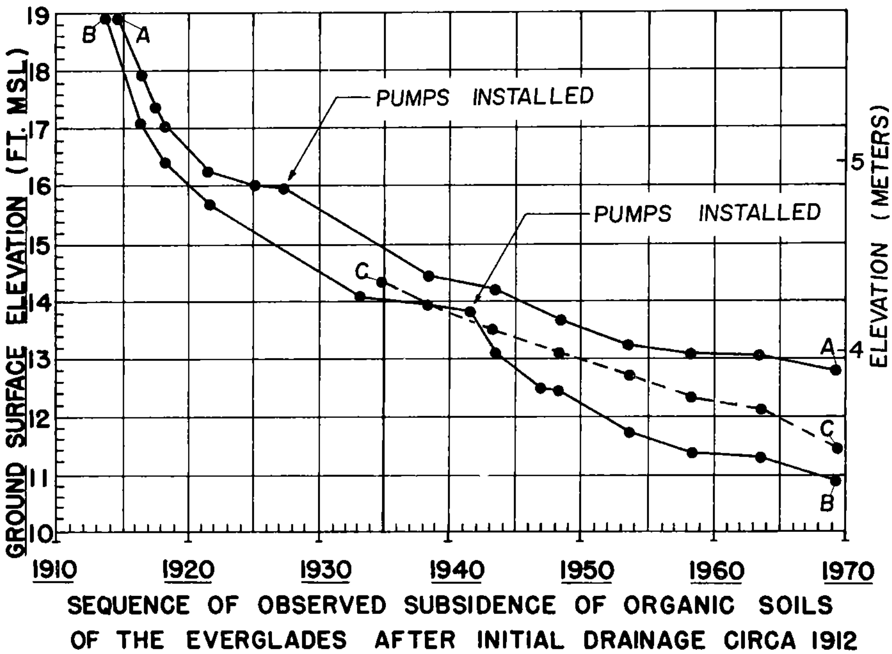

FIGURE 3. *Sequence of observed subsidence of organic soils in the Everglades after initial drainage, circa 1912*

These three sample curves are sufficient to show the pattern of subsidence in the organic soils of the Everglades.

In general, there is severe sinkage immediately after drainage, which is attributed primarily to shrinkage due to desiccation. Oven-dried samples lose approximately 70 to 80% of their original weight and 60% of their original volume. On this basis, air-dry peats are estimated to lose not over 20 to 30% of volume by descciation within the drained zone.

The first tillage results in further loss of elevation by compaction that reduces pore space. After 5 years of tillage the top 18 inches (45.7 cm) increases in density to about double that of the peat underneath. Simultaneously, the color blackens, the seepage rate decreases, and the top layer changes into a mucky, amorphous mass. As drainage continues the rate of subsidence levels off to a more or less steady rate, dependent upon water levels.

There seems little doubt that after the initial rapid sinkage, the slow, steady subsidence is due in large part to oxidation, which in turn is related to the rate of air movement in the drained zone. The internal structure of the sedge-peat, unlike sedimentary peat, resists consolidation to a great extent. Sedge peats develop in place in shallow water, while sedimentary peats develop from settlement of surface vegetation in deeper water. Whenever the surface of the peat soils of the Everglades are sealed, the subsidence rate decreases materially. Thus, paved roads, tennis courts, and concrete floor slabs in the area, which were constructed 10 or more years ago, now stand a foot or more above the immediately surrounding lands.

Core samples taken from virgin peat that was drained, but left untilled, showed that the aerated top zone developed a spongy open texture. This condition appears to allow free air movement because when such soil is later tilled, the loss in depth is greater than adjacent cultivated fields, provided both have had equal drainage.

It has been noted that grazed sod fields are sinking at a slightly slower rate than tilled lands under the same drainage. At the Everglades Experiment Station, the annual rate of subsidence for the past three decades has been 0.84 inches (2.13 cm) for St. Augustinegrass pasture, and 1,05 inches (2.67 cm) for cultivated land in truck crops, with the same drainage. When the somewhat higher surface elevations of the pasture land were corrected for difference in density, however, the difference in loss of soil weight between the two was not significantly different [13].

Area Subsidence Studies

All of the early investigations made at specific locations in the Everglades pointed to the certainty of soil subsidence associated with drainage of organic soils. Two questions then arose. First, are the findings applicable to the whole Everglades area? Second, if so, what is the future outlook for agriculture in the Everglades?

To answer these questions, Stephens and Johnson [14] in 1951 assumed that "the study of the past is the key to the future." Isopachous charts were prepared from surveys made in 1912, 1925, 1940 and 1950 showing the depths of the organic soils for various times, since initial drainage, for the upper Everglades area. A cross-sectional view through this area, section A-A, was prepared from these data and is shown in figure 4. Predicted ground elevation for the years 1970 and 2000 are also shown.

"Subsidence valleys" several miles wide exist ar drainage canals. Valley depth is greatest where drainage was best. The loss of soil in the areas studied amounted to 40% of the total volume between the first surveys in 1912 and the 1950 surveys.

Concerning the future outlook for agriculture in the Everglades, it was predicted in 1951 that, assuming the same subsidence pattern in the future, the total volume loss in cross-sectional area from predrainage days would be:

Year	*Total Volume of Soil Lost*
1912	0 (base datum)
1940	32% (measured)
—	—
1970	66% (estimated)
2000	88% (estimated)

Another area-wide soil-depth survey in the upper Everglades was made in 1965-66 by a private consulting engineer. Speir, who made a survey of the same area in 1941-42 for a USDA report [5], furnished data that enabled the 1965-66 survey to be compared with the 1941-42 ground levels, and estimated that the subsidence, which had occurred in the 25-year period, would be 33 inches (83.8 cm). The new surveys, which were extensive, found the actual average subsidence to be 33.5 inches (85.1 cm).

DISCUSSION

The drying, consolidation, and compaction by tillage that lead to organic soil sinking by shrinkage are each largely a one-time, non-recurring phenomenon. Wind erosion

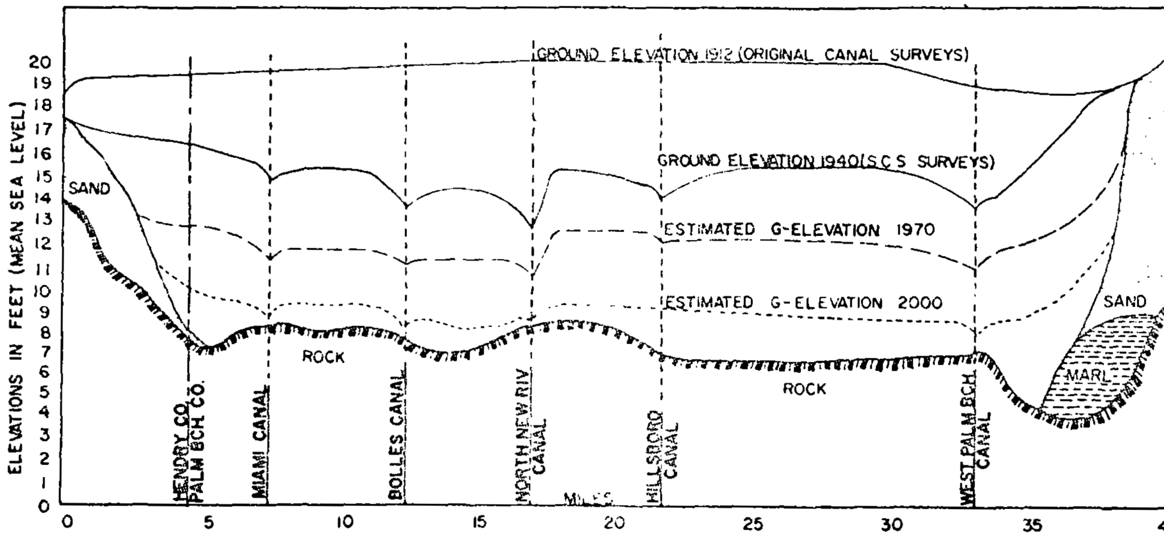

FIGURE 4. *Cross-sectional view throught the upper Everglades showing original surface elevation, and the elevation in 1940 as shown by topographic surveys. Estimated ground elevations for the years 1970 and 2000 are also shown (after Stephens and Johnson)*

and burning may reoccur periodically. Oxidation, however, is a continuous process as long as temperature, pH, and aeration are conducive to biochemical action.

Since 1951, several investigators who have studied peat subsidence in cooler climates have suggested that the role of biochemical oxidation was overstressed in the Everglades reports. They concluded that compaction and desiccation were the major factors in subsidence and implied that biochemical losses of organic soils from deep drainage were relatively unimportant as compared to the benefits derived.

Thus, it is appropriate that subsequent studies in the Everglades made since 1951 be analyzed to determine if the predictions then made are still valid. Namely, "with continued subsidence, by 1990 much of the present area of organic soils in the upper Everglades will probably be too shallow in depth to support a paying agriculture, and, by the turn of the century, most of the area will have subsided to the point of wide-scale abandonment" [12].

While sporadic fires have accounted for some spectacular losses in the Everglades, the total effect has probably been less than 5% for the region, and wind erosion has been practically nil.

In the late 1950's, we made additional investigations to determine the causes of subsidence as related to physical, chemical, and biological soil properties. Bulk densities and organic matter content were determined for various depths from 252 core samples taken from nine test tracts at the Everglades Experiment Station. The average depth of organic soil at the time was 46.41 inches (1.18 m), of which 43.20 inches (1.10 m), or 93.4%, was pore space.

The depth distribution in January 1959 of the ratio of voids to solids is shown in figure 5-A, and the bulk densities in figure 5-B. The original profile as reconstructed from past records is superimposed, which extends the profile to elevation 19 ft., circa 1912. In figure 5-A, the porosity of the undisturbed soils below the water table is 96.02%.

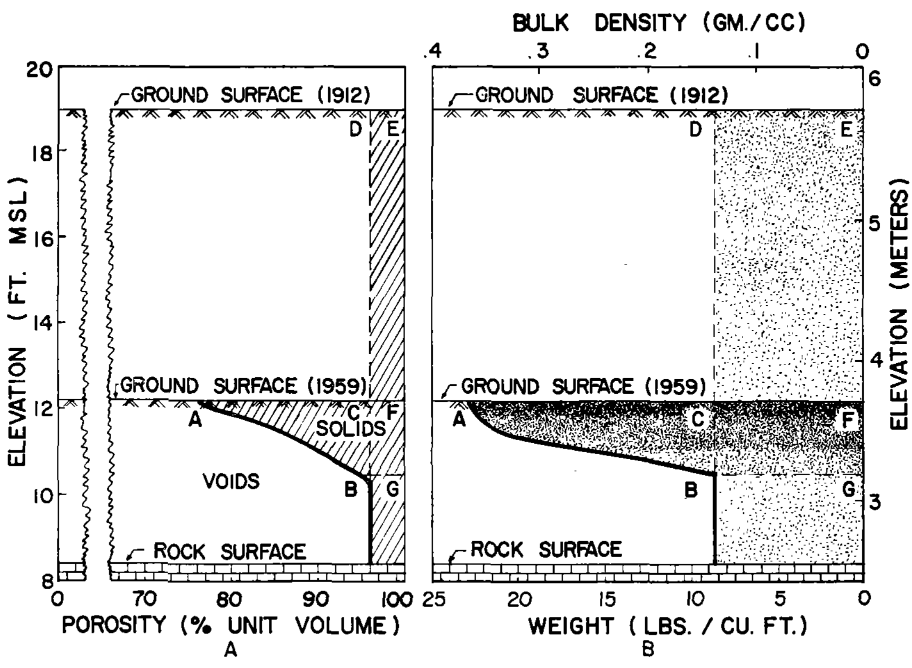

FIGURE 5. *Changes in porosity and bulk density of the top 18-inch (45.1 cm) stratum of organic soils after a half century of drainage*

At a depth of 18 inches (45.7 cm) below ground level, the porosity begins to decrease reaching a minimum of 78% at ground surface. The weighted-average porosity in the top 18 inches is 87.77%.

Analyzing the change in solid-voids ratio shown in figure 5-A, the weighted-average porosity of the top 18 inches decreased from 96.02% to 87.77% for an increase in the volume of solids from 3.98% in 1912 to 12.23% in 1959. This is an increase of 8.25% due to soil compaction, or a ratio of 2.07 to 1 increase in the original solids in an 18-inch (45.7 cm) vetical depth. This ratio multiplied by 1.5 ft. (45.7 cm) amounts to 3.10 ft. in terms of the original soil depth; or 3.10/6.8 = 45.6% of the subsidence accounted for on a change in volume basis; which leaves 54.4% due to oxidation.

In figure 5-B, the weight of the soil per cubic foot is shown to be 8.70 lb. below 18 inches (45.7 cm), increasing to 22.66 lb. near ground surface. Considering a 1-sq.-ft. column of soil, the total weight in the top 18-inch zone of densification is 25.80 lb.

This is an increase of 12.75 lb. in the area *A*, *B*, *C* over the original weight of 13.05 lb. in the top 18 inches, which is the area *B*, *C*, *F*, *G*. This 12.75-lb. increase in weight due to densification represents the residual accumulation from the subsidence of 6.8 ft. (2.07 m) (area *C*, *D*, *E*, *F*) of soil that was lost from 1912 to 1959, which is assumed to have originally weighed 59.16 lb. Thus, the fraction of subsidence that occurred from densification (desiccation and compaction was 12.75/59.16 = 0.215. Since no fires have burned over this area, the complement of this fraction, or 78.5%, is apparently due to biochemical oxidation.

Thus, oxidation losses are computed to be approximately 75% using changes in bulk density, and 55% using changes in soil porosity. In using both methods, it was assumed that the original soil profile was homogenous from top to bottom.

It is of interest to compare Skoropanov's finding of 13 to 14% subsidence by oxidation at Minsk with the Everglades findings, taking into account differences in monthly soil temperatures. Assuming that significant biochemical activity starts at 5 °C and doubles for each 10 °C rise, then, all other conditions being equal, the rate of biochemical activity in the Everglades should be 4.7 times greater than at Minsk. Thus, the 13% oxidation rate at Minsk would be increased, hypothetically, to 61% in the Everglades. This gives reasonably close support to the values of 55% and 75% found in the Everglades studies.

CONCLUSIONS

First, a study of the 11 subsidence profile lines with continuous records showed that there has been no diminution of subsidence rates since 1950, as evidenced by the resurveys of 1953, 1958, 1963, and 1968. Second, the area-wide soil-depth survey of 1965-66 showed the area continued to subside at the predicted rate. Third, the analyses of the 1959 field and laboratory studies of samples taken at the Everglades Experiment Station showed oxidation to account for at least 50% of soil loss.

Thus, the Everglades studies, considered altogether, show that there has been no apparent change in the rate of subsidence that would justify a revision of the 1951 predictions concerning the fate of the upper Everglades agricultural area.

In conclusion, to conserve the life of organic soils, it is advisable to hold the water table as high as crop requirements and field conditions will permit. It should be kept in mind that both crop requirements and field conditions will vary with the crop, climate, and organic soil type.

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COMPRESSION OF THE PEAT-BOGS AFTER DRAINING

A.I. MURASHKO

ABSTRACT

Peat-bogs cover one fourth of the total territory of Byelorussia or 4.5 million hectares. 70-80 thousand hectares of bogs are drained annually in the Republic for agricultural purposes. Constructing canals and tile drainage, roads and other constructions on the bogs is impossible without forecasting and taking into consideration the peat compression as a result of draining. Peat compression results in diminishing of drain and canal depths, deformation of canal sections, alternation of slopes of drain lines and relief.

The report describes the principal regularities of the peat compression process. It occurs unevenly in the depth of layer, in the space between the canals, and in time. Methods are described for calculating peat compression, developed from solution of the differential equation developed by the author and utilization of data of long-termed experiments.

The formulae, presented here, take into account thickness of peat layer and its density, characteristics of draining net system and duration of draining. They make it possible to calculate peat compression at any point of a peat-bog surface for any space of time. E.g. compression of peat-bog surface along the alignment perpendicular to canal axis may be calculated from the equation

$$S_{n,x} = AH_0 \{1-\exp[-(h-\Psi)(a+bt)]\}$$

$$\Psi = (h-z-h_0) \frac{\ln \frac{2(x-mh)}{b}}{\ln \frac{E-2mh}{b}}$$

where:

A quotient of density, which depends on the volume weight of the solid matter, humidity and degree of decomposition of peat (it is found in the nomogram, presented in the report);
H_0 thickness of peat layer prior to draining, m;
h canal depth, m;
t duration of drainage, *yr;*
z depth of water table halfway between the canals (normal drainage), m;
h_0 depth of water in the canal, m;
E distance between the canals;
b width of the canal bottom;
x distance of the surface point from the canal axis, m;
m laying the slopes of trapezoidal canal;
a quotient of the compression rate in the first year of draining, 1/m;
b quotient of the compression rate for the following years, 1/m.yr.

For the climatic conditions of the BSSR and the Soviet Baltic Region the values of quotients are as follows:

$a=0.07(1/m)$, $b=0.006(1/m.yr)$ for low-lying bogs, and
$a=0.065(1/m)$, $b=0.009(1/m.yr)$ for intermediate bogs.

Methods for calculating peat compression in drained peat-bogs, presented in the report, make possible the correct designing of drainage systems to provide for long-termed efficient functioning.

RÉSUMÉ

La tourbe couvre un quart de l'étendu totale de la Biélorussie ou 4,5 millions d'hectares. 70 à 80 Ha en sont drainés annuellement dans la république pour des fins agricoles.

La construction de canaux et le drainage par des tuyaux en terre cuite, de routes et d'autres ouvrages sur la tourbe n'est possible qu'on faisant des prévisions (et en les tenant en considération) de la compression sous l'action du drainage. La compression de la tourbe diminue les profondeurs de drains et canaux, déforme leurs sections et altère les pentes des drains et le relief.

Le rapport décrit lcs lois de la compression de la tourbe. Elle se produit d'une façon inégalement répartie en profondeur de la couche, dans l'intervalle entre les canaux et dans le temps. Des méthodes de calcul de la compression sont indiquées, développées en partant de l'équation différentielle établie par l'auteur et en utilisant les données d'une longue expérience.

Les formules présentées tiennent compte de l'épaisseur de la couche de tourbe et de sa densité, des caractéristiques du réseau de drainage et de sa durée. Elles permettent de calculer la compression de la tourbe en un point quelconque de sa surface le long d'un alignement perpendiculaire à l'axe du canal par la formule

$$S_{n,x} = AH_0\{1-\exp[-(h-\Psi)(a+bt)]\}$$

$$\Psi = (h-z-h_0)\,\frac{\ln\dfrac{2(x-mh)}{b}}{\ln\dfrac{E-2mh}{b}},$$

dans laquelle :

A est la fraction de densité qui dépend du poids du volume de la matière solide, de l'humidité et du degré de décomposition de la tourbe (voir le nomogramme dans le rapport) ;
H_0 est l'épaisseur en m de la couche de tourbe avant drainage ;
h est la profondeur du canal en m ;
t est la durée du drainage en années ;
z est la profondeur en m de la nappe phréalique à mi-chemin entre les canaux ;
h_0 est la profondeur de l'eau dans le canal en m ;
E est l'entredistance des canaux ;
b est la largeur du canal au fond ;
x est la distance du point considéré à l'axe du canal, en m ;
m est la pente du canal trapézoïdal ;
a est la fraction du taux de compression pour la première année du drainage, 1/m ;
b cette fraction pour les années suivantes (1/m année).

Pour les conditions climatiques de la Biélorussie et la région baltique de l'U.R.S.S. les valeurs des fractions en question sont les suivantes :

$a = 0{,}07$ (1/m), $b = 0{,}006$ (1/année) pour des tourbières basses et
$a = 0{,}065$ (1/m), $b = 0{,}009$ (1/m année) pour des tourbières intermédiaires.

Les méthodes de calcul de la compression exposées dans le rapport donnent la possibilité d'établir un projet correct de système de drainage pouvant fonctionner efficacement durant de longues années.

The territory of the Byelorussian Soviet Socialist Republic (the BSSR) is situated between 51°10′ and 56°10′ of the North latitude and 23°11′ and 32°46′ of the East longitude. Though the latitudinal and longitudinal ranges are not very great, the climate of the different parts of the BSSR has its peculiarities, chiefly due to the geographical position of its territory. The climate of the BSSR is influenced by the sea climate (in the West) and continental climates (in the East). The North and the South parts of the country also influence the climatic conditions.

The total area of marshlands in Byelorussia covers almost 35 per cent of its territory. Bogs pieces of wet spongy ground covered with a layer of organic matter (peat) more than 30 cm thick—make up 23 per cent of the territory of the Republic, which is equal to 4.5 million hectares (ha).

There are three types of bogs in Byelorussia, they are low-lying, intermediate, and elevated.

LOW-LYING BOGS

Are bogs with a concave or flat surface. The vegetation that grows on the low-lying bogs is sufficiently provided with mineral substances, the latter coming in either from mineral ground by capillary infiltration of water or carried in by soil and surface waters.

Low-lying bogs are differentiated according to the prevailing vegetation: woodless herbaceous and wooded types. Woodless herbaceous low-lying bogs may be covered with reeds, horse-tails, canes, carex, Typha, or mosses (Hypnoceae and Sphagnum). These bogs appear most often when lakes, river-beds, and flood-lands are overgrown with reeds, canes, horse-tails, etc. Rich mineral nourishment is characteristic of reed, cane and horse-tail bogs. Peat is being deposited as a result of annual dying off of overgrowth of the vegetation and their incomplete decomposition, but accumulation of peat leads to decrease in dampness and mineral content.

For wooded low-lying bogs alder (Almus Glutinosa) and birch (Betula Pubescens) are characteristic. Alder low-lying bogs appear in the area of percolation of the ground waters saturated with lime and phosphorus or in places flooded for short recurrent periods of time with waters rich in silt. Alder bogs are periodically heavily flooded and sufficiently supplied with mineral substances. The peat-layer in them usually is not thick—from 0.5 to 1 m. The percentage of nitrogen in the peat in alder bogs reaches 3 to 4, thus drained alder bogs are very valuable for cultivation.

The location, and water and mineral conditions of birch bogs are similar to those of alder bogs. The former differ from the latter in only one respect, they are permanently filled with water and are not so rich in mineral salts, especially in lime. The proportion of decomposition of the upper peat layer is also smaller. The peat in these bogs has fine fibres, it contains birch-carex, birch, and grass, with an admixture of reed, horse-tail and mosses. Its mineral content reaches 9 to 12 per cent.

The cultivation of the birch bogs is hampered by an abundance of tussocks, tree remnants, and stumps both in the peat layer itself and on the surface of it. When cultivating birch bogs it is necessary as a rule to use potash and phosphorus fertilizers.

INTERMEDIATE BOGS

Occupy an intermediate position between low-lying bogs, which are sufficiently supplied with mineral substances, and elevated bogs, which are extremely poorly supplied with them. Intermediate bogs are wooded bogs, with the predominating tree being the mine tree-whose height reaches 15 to 20 m. Occasional birch trees are an indication of conversion of wooded low-lying bogs into intermediate bogs. Predomination of sphagnous mosses distinguishes intermediate bogs from low lying ones. The peats of the intermediate bogs are characterised usually by a low degree of decomposition. Peat mineral content reaches 4 to 6 per cent. Nitrogen content also is low. That is why the quantity of basic mineral elements necessary for growing agricultural plants on these bogs is insufficient.

ELEVATED BOGS

Are bogs with elevated or bulging surface. Accumulation of the organic mass of peat leads to the general raising of a bog surface. Due to the thick layer of peat, the entry of mineral elements from the mineral bottom of a bog is blocked. Furthermore, the bulging of the surface not only prevents the deposition of outside mineral salts, but also facilitates their being washed out since water flows down from the elevated part of a bog to its margins. The vegetation growing on the bulging parts of elevated bogs is content with only those mineral salts that rainfall provides it with. That is why elevated bogs are also called air-fed bogs. Only plants with a minimum need of mineral salts can grow on these bogs. Thus, the best way to utilize them is as an industrial peat supply.

The chemical composition and physical properties of the basic types of bogs are shown in table 1.

The utilization of peat for practical purposes has been attracting the attention of a wide range of specialists who are studying it as a natiral resource. In some countries bogs are used extensively as a fertilizer or fuel. In addition, important research in the field is being carried on in the Byelorussian Soviet Socialist Republic. More than a million hectares of bogs have already been turned into arable land and have high crops yields. Unheard of large-scale land-reclamation measures have been worked out and adopted by the May (1966) plenum of the Central Committee of the CPSU. By 1970, it is planned to drain 1,550,000 ha of marshlands in Byelorussia, and in 1970-1975 the area will increase to 2,200,000 ha. Collective and state farms in Byelorussia constantly raise good crops on the drained bogs irrespective of climate conditions—grain yields have been 30 and more metric centners per hectare, potatoes 200-300 metric centners per hectare, sugar-beets 301-350 metric centners per hectare, grass (hay) 80-90 metric centners per hectare. Before draining, the yield on these lands was not more than 1 to 8 metric centners per hectare for corax hay.

SINKING OF PEAT LAYER AFTER DRAINAGE

PRINCIPAL PROPOSITION

Solid peat particles are suspensed in water and their mutual pressure, P_b, at a depth, l, from the surface is defined by the relation,

$$P_b = \frac{\Delta - \Delta_0}{1+\varepsilon_0} l, \tag{1}$$

where:

Δ specific weight of soil particles, gm/cc;
Δ_0 specific weight of water, gm/cc;
ε_0 porosity quotient;
l layer thickness, cm.

The mean values of these parameters for undrained bogs vary in the following limits: $\Delta = 1.50$ to 1.65 gm/cc, $\varepsilon_0 = 10$ to 15. Thus, for the mutual pressure of solid particles at one meter depth from the surface of an undrained bog we get 4 to 6 gccm.

The sinking of the water level caused by a drainage system, evaporation, and transpiiration drastically alters the static conditions in a peat-bog and the evolution of soil processes. Since the suspending force is removed, the solid soil particles of the upper dry layer of the peat-bog press the lower ones with all their weight plus the weight of the moisture retained by them. In this case, the pressure, P, of solid particles deposited below the water level has the form,

$$P = \gamma \mathrm{h} \tag{2}$$

where:

γ voluminal weight of the ground, gm/cc;
h depth of the water table from the surface, cm.

If the water level falls by one meter and the volume weight of the peat is 0.8 to 0.9 gm/cc, the pressure among solid particles at this depth will be 80 to 90 gm/cc.

TABLE 1.

Soils	Degree of decomposition of organic matter	Mineral content in completely dry soils	Voluminal weight	Moisture absorption for complete dry soil	Acidity of water extract	General No	P_2O_5	K_2O	CaO	P_2O_3
						for completely dry soil				
Elevated	5-30	1.3- 5.8	0.04-0.08	600-1200	2.6-4.2	0.56-2.0	0.03-0.25	0.01-0.25	0.1 -0.48	0.03-1.4
Low-lying	15-60	7.5-17.0	0.11-0.23	460- 870	4.8-7.0	1.6 -4.0	0.1 - 0.4	0.02-0.3	1.2 - 6.8	1.2 -7.2
Intermediate	10-45	5 -10	0.11-0.16	350- 950	2.8-5.3	1.4 -2.5	0.02-0.35	0.05-0.2	0.15 - 2.5	0.9 -4.7

For draining bogs, this pressure is transferred onto the whole peat layer below since it can be considered as a load equally distributed over a semi-space. Thus, pressure among solid particles increases 15 to 20 times and leads to compression of the total peat layer.

On the other hand, the upper dried peat layers constitute a three-phase dispersion system consisting of solid particles of different shape and size, and the space between them is filled with water and air. The presence of air and water in a peat layer accounts for the formation of water lenses and, consequently, there appears a capillary force. σ, whose value is expressed by the Laplace formula,

$$\sigma = 2\frac{\alpha}{z}, \tag{3}$$

where:

α surface tension of water, equal to 0.000075 kgcm;

r capillary radius, cm.

Capillary pressure reaches large values and it facilitates the compaction of the particles still more the more the water is evacuated and the smaller the soil particles are.

Airing of the dried upper layers of a peat deposit also leads to the intensification of microbiological processes and the replacement of anaerobic processes by aerobic ones, which causes the intensification of decomposition of the peat organic matter and its mineralization. This, results in the compaction of soil particles and their closer packing. The upper layers of a peat deposit are compressed under the influence of gravitational, capillary, and molecular forces, and their mechanical compression and mineralization of the organic matter of peat. The compression of the lower layer mainly takes place under the influence of additional pressure of the upper one which before draining was in a suspensed state.

Compression of peat under the influence of draining spreads through the whole depth of a bog from its surface to its mineral bottom. This results in lowering of the peat-bog surface and sinking of canal bottoms.

Compression varies with depth. According to the nature of the development of the process, three zones may be distinguished: the upper zone, lying above the ground-water table; the middle zone, undergoing periodic wetting, and the lower zone, lying below the ground-water table.

The upper layers undergo compression at a considerably higher degree than the lower ones. It is accounted for by the fact that they are more liable to drying and mineralization. These processes do not take place in the layers lying below the ground-water table, and consequently the compression of peat in them develops only as a result of the growing pressure from above. Therefore, compression is not great (fig. 1).

The main cause of peat compression is the fall of the water table as a result of drainage. The fall of the water table develops irregularly depending upon the distance from canal and drains; it reaches maximum values near the canal and gradually diminishes towards the middle. Therefore, compression along an alignment perpendicular to drains develops unevenly. Compression reaches maximum values in the canal zone 10 to 13 m wide, compression is minimal half way between drains.

Compression of peat increases unevenly with time. The main compression occurs in the first year after draining. In the following years the increase of compression is comparatively gradual and it slowly diminishes during a long period of time. Compression in the upper layers develops more intensively. That is why the canal depth diminishes permanently.

The principal factors for compression are peat density, depending on volume weight of the solid matter and the humidity; the degree of decomposition; the thickness of the peat layer; the depth of fall of the water table as a result of draining; the character and duration of agricultural usage of the drained peat-bogs.

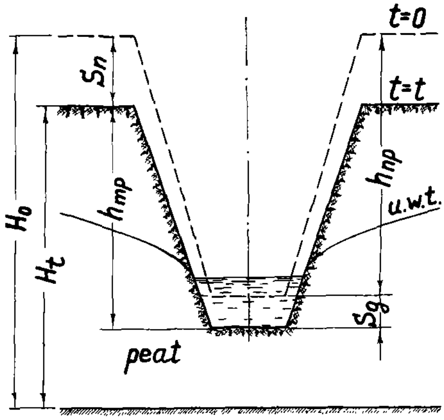

FIGURE 1.

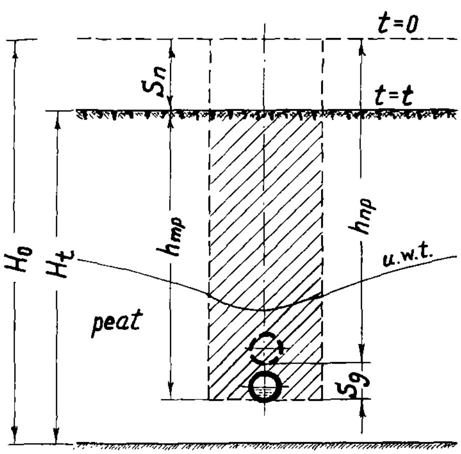

FIGURE 2.

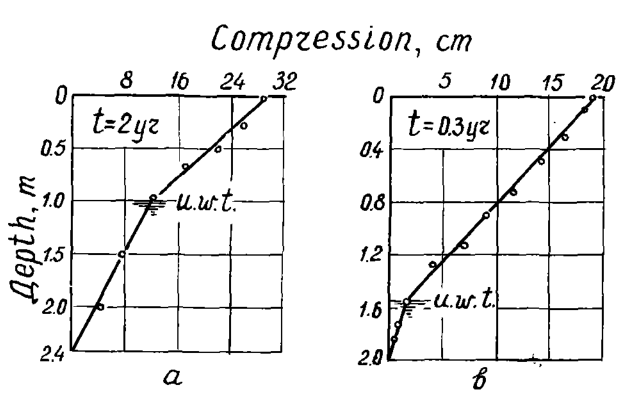

FIGURE 3. *Compression of peat:* a) "*Ponya*" *peat-bog,* b) "*Bobrik*" *peat-bog,* t = *drainage period, years; u.w.t.* = *usual level of the water table*

The data in table 2 give a definite idea of the surface compression of peat bogs, as a result of draining and cultivation.

TABLE 2. Compression Data for Bogs

Alignment	Original thickness of peat layer, m	Compression, cm			Mean annual compression, cm	
		Mean along alignments	near canal	half-way between canals	near canal	half-way between canals
"Maryino"	Marshland	for 30 years (1929-1959)				
D	2.29	72.0	90	101	3.00	1.80
B	3.67	80.4	95	64	3.14	2.10
E	4.09	90.9	112	101	3.70	3.36
	1.26	49.0	67	34	2.20	1.13
A	3.65	79.2	—	—	2.70	—
Minsk Bog Station		for 47 years (1913-1960)				
I	1.05	61	—	—	1.30	—
II	2.05	136	—	—	2.90	—

The data in table 2 show that on "Maryino" marshland, where the thickness of the peat layer before draining was 2.8 m, mean compression for four alignments was 91 cm, and at the distance of 45 m from the canals (the distance between open canals is 90 m) was 63 cm, which is considerably less.

CALCULATION OF SURFACE COMPRESSION OF A BOG

The process of the surface compression of a bog with time may be described by the following differential equation:

$$-\frac{\mathrm{d}H}{\mathrm{d}t} = \lambda h H, \tag{4}$$

where:

— $\mathrm{d}H/\mathrm{d}t$ the speed of compression in meters per year (the negative sign shows that compression decreases with time).

H the depth of the bog, m;

h the depth of the drain, m;

t the duration of draining, years;

λ "the constant" of compression, the quotient which depends on the physical properties of peat, 1/m per year.

The formula foı calculating surface compression of low lying bogs at the borders of open canals and near drains, which results from integration of equation (4) and utilization of experimental data, has the form:

$$S_n = AH_0\{1-\exp[-h(a+bt)]\} \tag{5}$$

where:

A quotient of peat density, which depends on the volume weight of the solid matter;
H_0 thickness of peat layer before draining, m;
a and b experimental quotients ($a = 0.07\ m^{-1}$; $b = 0.06\ m^{-1}$, year^{-1}).

For practical calculation using equation (5), it is necessary to know the thickness of the peat layer before draining and the volume weight of solid matter, and to define the drain depth sufficient for successful agricultural production or other purposes.

In designing draining systems, the value of t in equation (5) should be for draining operation of the project, or time for repairing or reshaping the drainage system. For bogs where wooden drains are installed, the operation time of which is 15 to 20 years, the compression should be calculated taking into account half of this term and assuming that the mean depth of the ground water will be slightly higher for the first half of the operation period and slightly lower for the second half of the period than the optimum depth. For ceramic drains, with operation terms of 40 to 50 years, the calculation value of the compression should be defined for t = 20 to 25 years. For a drainage system, t is assumed to be equal to the time after which repairs or reconstruction of the drainage system is planned, that is approximately for 5 to 10 years.

Equation (5) is correct when $t \geqslant 1$ and the constant depth of water in drains is h_0 = 5 to 40 cm. If the designed depth of water in the canal is assumed to be more than the above mentioned value, the canal depth h should be reduced by the value of the excess. For example, if h = 2.5 m and h_0 = 1.0 m, than the value of h = 2.5 − (1.0—0.4) or 1.9 m, which should be used in equation (5). This assumption is based on the fact that when the water table of the bog is constantly near the canal borders the compression of peat will not take place.

Values of peat layer density quotient A are defined by the value of volume weight of solid matter according to the nomogram (fig. 2). The magnitude of volume weight of perfectly dry peat should be defined by selection of samples with intact structure taken through the whole depth of the peat-bog. In case the peat layer is inhomogeneous, such as when there are different layers with thickness of l_1, l_2, ..., l_n with different density

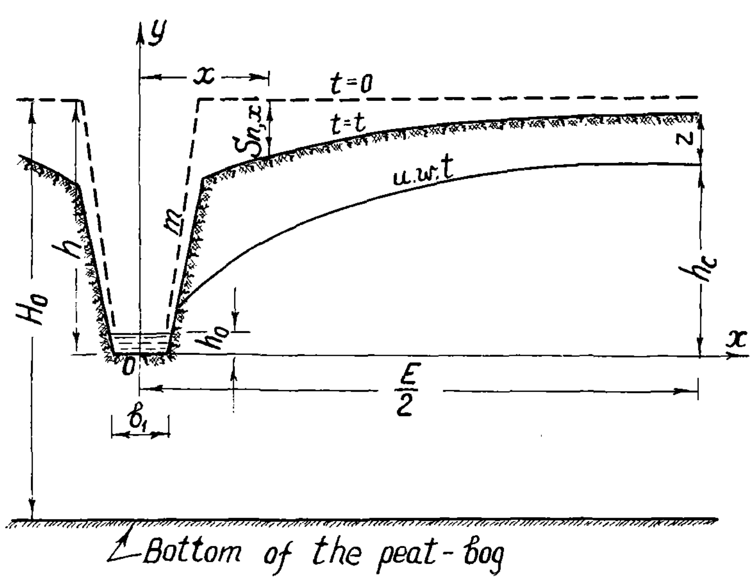

FIGURE 4.

through, the depth of the bog, then the volume weight of the solid matter should be defined as a "mean weighted value".

$$\delta = \frac{\delta_1 l_1 + \delta_2 l_2 + \ldots + d_n l_n}{l_1 + l_2 + \ldots + l_n}. \tag{6}$$

Since under production conditions it is rather difficult to define the weight of solid matter of an undrained layer, it is possible to choose values of A by means of the value of natural humidity, W, and the degree of decomposition, R. Determination of W and R is made by selecting samples with disturbed structure through the whole peat thickness. The monogram (fig. 2) shows also the method of defining the quotient of A by these factors.

CALCULATION OF COMPRESSION OF BOTTOMS OF DRAINS AND CANALS

Because, compression develops throughout the whole thickness of the peat layer under the influence of draining, the choice of layout for canals and drains and the design of their longitudinal section must be made with due regard to future compression of peat below their bottoms. Compression of the bottoms of drains and canals also influences the change in the drainage system depth.

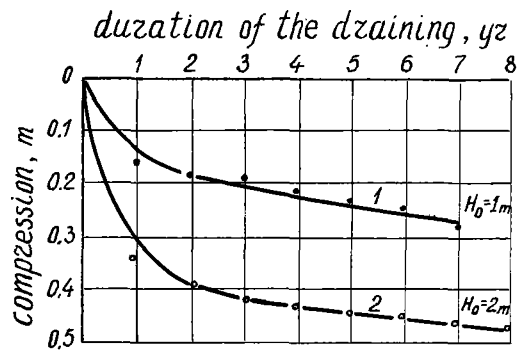

FIGURE 5.

The process of compression of the bottoms of drains and canals in low lying bogs with time (4) is described by the following relation.

$$S_g = A(H_0 - h)\{1 - \exp[-h(c + dt)]\} \tag{7}$$

where

c and d are experimental quotients
($c = 0.021\ \mathrm{m}^{-1}$, $d = 0.05\ \mathrm{m}^{-1}$, year^{-1});

(H_0—h) the depth of peat below drain bottom, m;
and the other symbols are the same.

The choice of calculating factors A and t is made in the same way as for calculating the surface compression of bogs. Equation (7) is correct when $(H_0 - h) > 0$ and $t \geqslant 1$.

THE CHOICE OF DESIGNED DRAIN DEPTH

Calculation of designed depth of drainage canals and drains proceeds with the assumption that they should have a depth after compression which should provide the drainage rate necessary for growing agricultural plants.

Because the compression of both the bog surface and drain bottom occurs during drainage, these magnitudes should be considered separately. Surface compression of bogs will lessen the depth when laying the draining system whereas bottom compression will increase it. Thus, the designed depth of drain is defined according to the following expression:

$$h_{np} = h_{mp} + S_n - S_g, \tag{8}$$

where:

h_{np} designed drain depth, m;
h_{mp} drain depth required by technical specifications after compression of peat, m;
S_n surface compression of the bog by the time of complete draining, m;
S_g compression of drain bottom by the same time, m.

The determination of design depth of drains is deduced by the method of consecutive approximation.

CALCULATION OF SURFACE COMPRESSION OF THE BOG AT ANY DISTANCE FROM DRAINS

The preceding sections presented the methods of calculation of compression of peat at the borders of drainage canals and near drains, which are necessary chiefly for choosing their design depth. That was for maximum compression.

The primary cause of compression of peat is the fall of water table in the bog. The magnitude of compression, all other conditions being even, is proportional to the depth of the fall of water table. Compression of peat reaches its maximum near drains and its minimum between them. It is proportional to the stable curve of depression. As a result along the alignment perpendicular to the drains and moving away from the latter compression is regular.

A change of the original landscape of the bogs develops as a result of uneven compression at some distance from the drains. The investigations have proved that as a result of continued draining considerable dislocation of horizontal lines occurs, and in some cases their turning reaches 90°. Therefore, when it is necessary to forecast the bog landscape after drainage, the alteration of peat-bog slopes, and—as it will be shown below—for forestalling the alteration of drain slopes, it is important to calculate the relationship between compression and distance from drains.

Surface compression of low lying peat bogs along the alignment perpendicular to the canal axis may be defined with the help of the following mathematical relations:

$$S_{n,x} = AH_0\{1-\exp[-(h-\Psi)(a+bt)]\}, \tag{9}$$

where:

$$\Psi = (h-z-h_0)\frac{\ln\dfrac{2(x-mh)}{b}}{\ln\dfrac{E-2mh}{b}}; \tag{10}$$

z	depth of the water table half-way between the canals (normal drainage), m;
h_0	depth of water in the canal (stable), m;
E	distance between canals, m;
b	width of the canal (drain diameter), m;
X	distance of the point on the bog surface from the canal axis, m;
m	laying the slopes of trapezium canal.

The other symbols are the same. The scheme is given in figure 3.

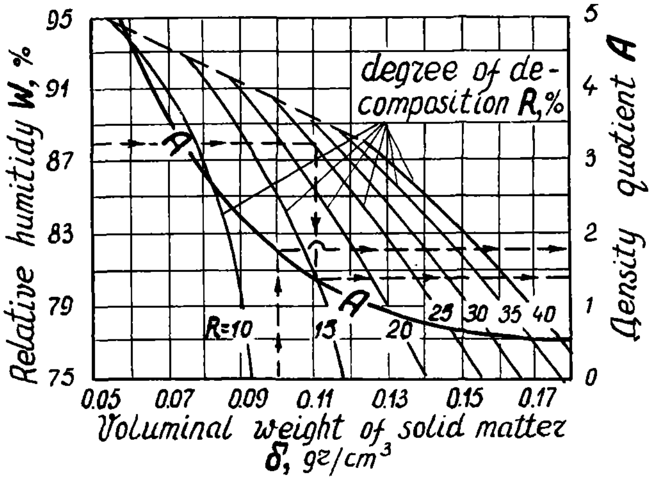

FIGURE 6. *Nomogram for choosing peat bog density quotient A*

CONCLUSION

All practical calculations for defined compression of peat-bogs caused by drainage should be carried out considering the actual conditions of the drained object. Such factors as operating term of the construction, their durability, the presence or absence of previous drainage, etc., should be taken into consideration. When prospecting for bogs, it is necessary to pay great attention to defining the physical constants of peat—its thickness, humidity, density, degree of decomposition, etc.

A correct approach to calculations of compressions allows us to choose the reasonable parameters of drainage system and in many cases produces considerable economic results

VISCO-ELASTIC THEORY OF THE DEFORMATION OF A CONFINED AQUIFER

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ABSTRACT

The author derived the dynamic theory for the deformation of a granular solid saturated with a liquid, assuming that the liquid filling up the pore space is a Newtonian viscous fluid and the skeleton constituted by solid particles is a linear visco-elastic solid. The theory consists of three fundamental equations, that is, the equations of motion of liquid and skeleton and the equation of continuity between the particles and liquid. In a case where the particle and liquid are taken to be incompressible and the deformation of soil is a quasi-static process, these equations are accepted as the theory of three-dimensional consolidation including Terzaghi's well-known equation as a special case and are also recognized as the basic equations of motion of confined ground water in a visco-elastic aquifer. A theoretical example will be shown for rheological deformation of a infinite confined aquifer with uniform thickness caused pumping up water at a constant rate.

RÉSUMÉ

L'auteur présente la théorie dynamique de la déformation d'une couche solide granulaire saturée par un liquide, en supposant que le liquide remplissant les pores soit newtonien et que le squelette constitué par les particules solides soit un solide linéairement visco-élastique. La théorie consiste en trois équations fondamentales qui sont les équations du mouvement du liquide et du squelette et l'équation de continuité entre les particules et le liquide. Si on suppose les particules et le liquide incompressibles et si on admet que la déformation du sol constitue un processus quasi statique, ces équations sont acceptées comme constituant celles de la théorie de la consolidation à trois dimensions en comprenant l'équation bien connue de Terzaghi comme un cas spécial. Ces équations constituent les équations de base du mouvement de l'eau artésienne dans un aquifer visco-élastique. Un exemple théorique sera montré pour la déformation rhéologique d'un aquifer artésien infini d'épaisseur uniforme dans le cas de pompage de l'eau à un taux constant.

INTRODUCTION

It is well known that soil mechanics has made great advances since the conception of pore pressure (Hydrostatische Überdruck) proposed by K.V. Terzaghi. He considered that the soil particles, being more or less bound to each other by an attracting force, constitute a skeleton of soil with elastic properties and that the skeleton supports the external burden together with the assistance of the water filling up the pore space between the particles. He successfully solved the settlement of the soil layer with the idea that a contraction of soil depends on the rate of squeezing out of pore water, which neccessarily brings about the decrease of pore pressure. But he treated only a one-dimensional problem under constant load with a quasi-static method.

In October 1941, M.A. Biot (1) published the theory of three dimensional consolidation and developed the treatment of soil deformation for any arbitrary load variable with time. Recently, in September 1963, M. Mikasa (2) published the useful theory of soft layer consolidation showing many suitable examples, especially taking account of finite strain. But they treated only a one dimensional and quasi-static problem in the same way as Terzaghi.

Recently in soil mechanics, much attention has been paid to the correspondence of soil deformation caused by a vibrating agency in connection with the effective performance of engineering construction or safe protection from heavy damage, such as from an earthquake. It may be clear that the theory of consolidation must be improved and made into a dynamic one for the above requirements. We shall now derive in this paper the dynamic theory of consolidation, considering the rheological properties of soil.

1. Derivation of fundamental equations

Soil particles constitute the skeleton of the soil matrix, pushing and rubbing each other's contact portions against an external burden. Owing to the complexity of its structure, however, one could not expect the direct treatment of forces acting on each particle. In the same situation, it would also be quite impossible to deal quantitatively with the motion of pore water attending to tortuous and irregular pore space. Therefore, we are obliged to consider the representation of motion averaged over a volume element of soil, which is taken to be large enough compared to the size of the pores so that it may be treated as homogeneous and at same time small enough compared to the scale of macroscopic phenomena in which we are interested so that it may be considered as infinitesimal in the mathematical treatment. It will be sufficient in soil mechanics to consider the average conditions over the volume of soil in the above sense.

(a) *Equation of motion of pore water*

The motion of water in pores is governed by the hydrodynamic equation of viscous fluid. We regard the pore water as a Newtonian fluid and denote by $\mathbf{V}$ (V_1, V_2, V_3) the particle velocity of pore water. The equation of motion of pore water is expressed by

$$\frac{\mathrm{D}\mathbf{V}}{\mathrm{D}t} = \mathbf{X} - \frac{1}{\rho}\operatorname{grad} p - (\tfrac{1}{3}\eta - \kappa)\operatorname{grad}\theta + \eta\nabla^2\mathbf{V}, \tag{1.1}$$

where

t is time, $\mathbf{X}$ is external body force, ρ and p are the density and pressure of water, respectively, θ is the divergence of water flow, and η and κ are the kinematic viscosity of shear and bulk respectively; the dependences of which on density ρ are assumed to be slight.

Consider a unit volume of the soil matrix in the sense stated above. Integrating equation (1.1) over the pore space σ of the unit volume and using the following notations

$$\mathbf{U} \equiv \iiint_\sigma \mathbf{V}\,\mathrm{d}v \qquad P \equiv \frac{1}{\sigma}\iiint_\sigma p\,\mathrm{d}v, \tag{1.2),(1.3}$$

we have

$$\frac{\mathrm{D}\mathbf{U}}{\mathrm{D}t} = \sigma\mathbf{X} - \frac{\sigma}{\rho}\operatorname{grad} P - (\tfrac{1}{3}\eta - \kappa)\iiint_\sigma \operatorname{grad}\theta\,\mathrm{d}v + \eta\iiint_\sigma \nabla^2\mathbf{V}\,\mathrm{d}v, \tag{1.4}$$

where $\mathbf{U}$ is called "Darcy's velocity" or "specific flow rate" and P is the "pore pressure" proposed by Terzaghi.

The pore pressure P is generally taken to be thermodynamic pressure and is determined by the density and temperature of water. In constant temperature, we can write

$$\log\frac{\rho}{\rho_0} = \beta(P - P_0), \tag{1.5}$$

where β is called isothermal compressibility. In the foregoing, ρ_0 and P_0 are the density and pressure in some reference state, say a state at rest. In the case where external body force $\mathbf{X}$ is gravitational force, it is convenient to introduce the quantity φ which is termed the "piezometric head"

$$\varphi \equiv \frac{P}{\rho g} + x_3, \tag{1.6}$$

where x_3 – axis is taken as positive upward.

Appropriate expression to the last term in equation (1.4) is done by refering to Darcy's law governing the flow of water in a porous medium. He postulated the viscous force acting on water to be proportional to the flow velocity and introduced, as the proportional constant, physical quantity k which is called the coefficient of permeability of the soil, and according to his expression, the viscous force per unit volume of soil is written by

$$\mathbf{F} \equiv \eta \iiint_{\sigma} \nabla^2 \mathbf{V}\, dv \equiv -\frac{\sigma g}{k} \iiint_{\sigma} \mathbf{V}\, dv$$

In the next paragraph we shall treat the motion of soil particles together with the flow of pore water, and so it may be reasonable to assume that the viscous force **F** wäs proportional to the relative motion of water to soil particles, viz. $(\mathbf{V} - \partial\mathbf{u}/\partial t)$ where **u** is the mean displacement of soil particles as seen latter. From our assumption, it is possible to express its force as follows

$$\mathbf{F} \equiv -\frac{\sigma g}{k} \iiint_{\sigma} \left(\mathbf{V} - \frac{\partial \mathbf{u}}{\partial t}\right) dv = -\frac{\sigma g}{k}\left(\mathbf{U} - \sigma \frac{\partial \mathbf{u}}{\partial t}\right). \tag{1.7}$$

Pore water is regarded to be almost incompressible in engineering practice. In this case, inserting the expressions (1.5), (1.6) and (1.7) into equation (1.4) and neglecting the inertia part of acceleration,

$$\frac{\partial \mathbf{U}}{\partial t} + \sigma g \left\{ \operatorname{grad} \varphi + \frac{1}{k}\left(\mathbf{U} - \sigma \frac{\partial \mathbf{u}}{\partial t}\right)\right\} = 0. \tag{1.8}$$

Equation (1.8) is, of course, reduced to Darcy's law if the soil particle has no motion and the flow of water is not accelerated.

(b) Equation of motion of soil skeleton

We shall now pay attention to the motion of the soil skeleton. Terzaghi, Biot and Mikasa assumed the elastic isotropy of stress-strain relations for the soil skeleton. While we also accept the isotropy in order to avoid the trouble of mathematical presentation, we had better give up the elastic property of soil with reference to the results of many investigations postulating the nonelastic deformation of soil 3). On the other hand, it may be clear that the non-linear relation between stress and strain make it difficult to analyse the deformation quantitatively. We now regard it tentatively as a linear viscœlastic relation.

Consider again the volume element in the sense stated previously and take the average over the actual displacement **v** of soil particle contained in that volume. We define it as the displacement of skeleton **u**, that is

$$\mathbf{u} \equiv \frac{1}{1-\sigma} \iiint_{(1-\sigma)} \mathbf{v}\, dv. \tag{1.9}$$

Supposing that the difference (**v**-**u**) produces only a minor effect on the stress on the skeleton viz. effective stress and assuming the strain to be infinitesimally small, the strain on the skeleton is given by tensor (e_{ij})

$$e_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right) \quad (i, j = 1, 2, 3). \tag{1.10}$$

Corresponding to this, the effective stress (σ_{ij}) is exerted on the skeleton. According to the assumption of linear visco-elasticity, stress σ_{ij} is represented, as positive in compression ([4]), by

$$\left(1 + \sum_{p=1}^{n} \gamma_p \frac{\partial^p}{\partial t^p}\right)\sigma_{ij} = -\lambda\left(1 + \sum_{p=1}^{l} \alpha_p \frac{\partial^p}{\partial t^p}\right)\delta_{ij}\Theta - 2\mu\left(1 + \sum_{p=1}^{m} \beta_p \frac{\partial^p}{\partial t^p}\right)e_{ij}, \tag{1.11}$$

where δ_{ij} is Cronecker's notation and Θ is the dilatation of the soil skeleton, that is

$$\Theta = \frac{\partial u_1}{\partial x_1} + \frac{\partial u_2}{\partial x_2} + \frac{\partial u_3}{\partial x_3}. \tag{1.12}$$

Introducing the operations with respect to time

$$\mathscr{L} \equiv \lambda \frac{1 + \sum_{p=1}^{l} \alpha_p \frac{\partial^p}{\partial t^p}}{1 + \sum_{p=1}^{n} \gamma_p \frac{\partial^p}{\partial t^p}}, \qquad \mathscr{M} \equiv \mu \frac{1 + \sum_{p=1}^{m} \beta_p \frac{\partial^p}{\partial t^p}}{1 + \sum_{p=1}^{n} \gamma_p \frac{\partial^p}{\partial t^p}}, \qquad (1.13), (1.14)$$

we can write the stress force per unit cubic element of soil as follows

$$\frac{\partial \sigma_{ij}}{\partial x_j} = -(\mathscr{L}+\mathscr{M}) \frac{\partial \Theta}{\partial x_i} - \mathscr{M} \nabla^2 u_i. \tag{1.15}$$

Furthermore, the skeleton is pushed by the pore pressure of surrounding water. This pressure action $\mathbf{f}_1$ may not produce any shearing strain by reason of the assumed isotropy and will be expressed by

$$\mathbf{f}_1 = \iint_{(1-\sigma)\,\mathrm{surf}} p\mathbf{n}\, \mathrm{d}a = - \iiint_{(1-\sigma)} \operatorname{grad} p\, \mathrm{d}v = -(1-\sigma) \operatorname{grad} P \tag{1.16}$$

per unit volume of soil, where $\mathbf{n}$ is the outward unit vector normal to the surface element d a of solid space.

In addition to above forces, the skeleton tends to be dragged by the flow of pore water in its direction through the reaction of the viscous force acting on the pore water. This drag force $\mathbf{f}_2$ will be expressed by

$$\mathbf{f}_2 = -\rho \frac{\sigma g}{k} \left(\sigma \frac{\partial \mathbf{u}}{\partial t} - \mathbf{U} \right). \tag{1.17}$$

We can thus establish the equation of motion of the soil skeleton, that is

$$\begin{aligned} (1-\sigma)\rho_s \frac{\partial^2 \mathbf{u}}{\partial t^2} &= (1-\sigma)\mathbf{X} + \nabla \cdot \sigma + \mathbf{f}_1 + \mathbf{f}_2 \\ &= (1-\sigma)\mathbf{X} + (\mathscr{L}+\mathscr{M}) \operatorname{grad} \Theta + \mathscr{M}\nabla^2 \mathbf{u} - (1-\sigma) \operatorname{grad} P + \rho \frac{\sigma g}{k} \left(\mathbf{U} - \sigma \frac{\partial \mathbf{u}}{\partial t} \right) \end{aligned} \tag{1.18}$$

where ρ_s is the density of soil particles and $\mathbf{X}$ is an external body force. In almost all cases with which we are concerned, $\mathbf{X}$ is a gravitational force. Expressing the vertically upward unit vector by $\mathbf{k}$, we have

$$\begin{aligned} (1-\sigma)\rho_s \frac{\partial^2 \mathbf{u}}{\partial t^2} = -(1-\sigma)\rho_s g \mathbf{k} + (\mathscr{L}+\mathscr{M}) \operatorname{grad} \Theta + \mathscr{M}\nabla^2 \mathbf{u} \\ -(1-\sigma) \operatorname{grad} P + \rho \frac{\sigma g}{k} \left(\mathbf{U} - \sigma \frac{\partial \mathbf{u}}{\partial t} \right). \end{aligned} \tag{1.19}$$

(c) *The equation of mass continuity*

Finally, we shall derive the equation of mass continuity per unit volume of soil. Suppose that a skeleton in any volume of soil had porosity σ_0 at an instance of no dilatation $\Theta=0$, and that the particles in it had density ρ_{s0} at that time. Because the skeleton under consideration is to be framed by the same particles at any instance, the mass of the skeleton must be conserved, that is

$$\rho_s(1-\sigma)(1+\Theta) = \rho_{s0}(1-\sigma_0), \tag{1.20}$$

since the dilatation Θ represents the volume increase of soil skeleton per unit initial volume and ρ_s is the density of particles at dilatation Θ. The volume of the skeleton is varied with time by external force and consequently, porosity σ is also varied. Equation (1.20) gives us the relation between their time rates. Because we are dealing with soil fully saturated with pore water, the change of pore volume results in the flow of pore water into or out of the volume element. This situation is represented by

$$\frac{\partial}{\partial t}(\sigma\rho) = -\operatorname{div}(\sigma\rho\mathbf{V}) = -\operatorname{div}(\rho\mathbf{U}). \tag{1.21}$$

where ρ is the density of water; $\mathbf{V}$ is the particle velocity of water and $\mathbf{U}$ is the specific flow rate as defined previously.

Combining equation (1.20) with equation (1.21), we make

$$\frac{\partial}{\partial t}\{(1-\sigma)\rho_s+\sigma\rho\} = \frac{\partial}{\partial t}\left\{\frac{(1-\sigma_0)\rho_{s0}}{1+\Theta}\right\} - \operatorname{div}(\rho\mathbf{U}) \tag{1.22}$$

This is an equation which we expected to derive. Relation of ρ_s to ρ_{s0} may be obtained from the consideration of the compression of soil particles due to the effective stress σ_{ij} and pore pressure P, although we have little knowledge about it at present. However, in general, the compressibilities of soil particles and water are small. Assuming both densities to be constant, we rewrite equation (1.22) as

$$(1-\sigma_0)\frac{\partial\Theta}{\partial t} + \operatorname{div}\mathbf{U} = 0 \tag{1.23}$$

with good approximation neglecting the small quantity of order Θ^2.

Approximating the values σ in equations (1.8) and (1.19) to the value σ_0 after equation (1.23) and summarizing the fundamental equation in the case where the external body force is only gravitational force and the soil particles and pore water are incompressible, we have

$$(1-\sigma_0)\rho_{s0}\frac{\partial^2\mathbf{u}}{\partial t^2} = -(1-\sigma_0)(\rho_{s0}-\rho_0)g\mathbf{k}+(\mathscr{L}+\mathscr{M})\operatorname{grad}\Theta+\mathscr{M}\nabla^2\mathbf{u}$$

$$-(1-\sigma_0)\rho_0 g\operatorname{grad}\varphi+\rho_0\frac{\sigma_0 g}{k}\left(\mathbf{U}-\sigma_0\frac{\partial\mathbf{u}}{\partial t}\right) \tag{a}$$

$$\frac{\partial\mathbf{U}}{\partial t}+\sigma_0 g\left\{\operatorname{grad}\varphi+\frac{1}{k}\left(\mathbf{U}-\sigma_0\frac{\partial\mathbf{u}}{\partial t}\right)\right\}=0 \tag{b}$$

$$(1-\sigma_0)\frac{\partial\Theta}{\partial t}+\operatorname{div}\mathbf{U}=0, \tag{c}$$

where

$$\Theta = \frac{\partial u_1}{\partial x_1}+\frac{\partial u_2}{\partial x_2}+\frac{\partial u_3}{\partial x_3}, \qquad \varphi = \frac{P}{\rho_0 g}+x_3 \tag{d), (e}$$

$$\mathscr{L} \equiv \lambda \frac{1+\sum_1^l \alpha_p \frac{\partial^p}{\partial t^p}}{1+\sum_1^n \gamma_p \frac{\partial^p}{\partial t^p}}, \qquad \mathscr{M} \equiv \mu \frac{1+\sum_1^m \beta_p \frac{\partial^p}{\partial t^p}}{1+\sum_1^n \gamma_p \frac{\partial^p}{\partial t^p}}. \tag{f),(g}$$

In the remainder, we shall omit a subscript "$_0$" from the respective notations.

2. Derivation of fundamental equations of subsidence

In this section, we shall seek the fundamental equations for the deformation of confined aquifer caused by ground water flow.

Let us first examine a static equilibrium state of the skeleton with a steady flow of pore water. From equations (a), (b) and (c), we have

$$0 = -(1-\sigma)(\rho_s-\rho)g\mathbf{k}+(\mathscr{L}+\mathscr{M}) \text{ grad } \Theta+\mathscr{M}\nabla^2\mathbf{u}-\rho g \text{ grad } \varphi . \tag{2.1}$$

Now, we divide the displacement $\mathbf{u}$ into two parts, $\mathbf{u}_{01}$ and $\mathbf{u}_{02}$ which satisfy the equations

$$0 = -(1-\sigma)(\rho_s-\rho)g\mathbf{k}+(\mathscr{L}+\mathscr{M}) \text{ grad } \Theta_{01}+\mathscr{M}\nabla^2\mathbf{u}_{01} \tag{2.2}$$

$$0 = (\mathscr{L}+\mathscr{M}) \text{ grad } \Theta_{02}+\mathscr{M}\nabla^2\mathbf{u}_{02}-\rho g \text{ grad } \varphi , \tag{2.3}$$

respectively, where

$$\Theta_{01} = \text{div } \mathbf{u}_{01}, \qquad \Theta_{02} = \text{div } \mathbf{u}_{02} \tag{2.4),(2.5}$$

Equation (2.2) expresses that $\mathbf{u}_{01}$ is the displacement caused by the apparent weight of the soil skeleton in water without external load and equation (2.3) means that if $\mathbf{u}_{02}$ is uniform in the entire body of soil, grad φ, consequently $\mathbf{U}$, is zero, say conversely, if pore water flows, the skeleton of the soil must be strained to that extent.

When external load and or piezometric head φ vary with time after the initial state, the strain on the skeleton and the velocity of flow begin to leave the static state. We proceed to investigate the unsteady motion.

Dissolving the quantities $\mathbf{u}$. Θ, $\mathbf{U}$ and φ into the static part $\mathbf{u}_0 \equiv \mathbf{u}_{01}+\mathbf{u}_{02}$, $\Theta_0 \equiv \Theta_{01}+\Theta_{02}$ $\mathbf{U}_0$ and φ_0 and respective deviations $\mathbf{u}'$, Θ', $\mathbf{U}'$ and φ' from static one, we have for the deviating parts

$$(1-\sigma)\rho_s \frac{\partial^2 \mathbf{u}'}{\partial t^2} = (\mathscr{L}+\mathscr{M}) \text{ grad } \Theta'+\mathscr{M}\nabla^2\mathbf{u}'-(1-\sigma)\rho g \text{ grad } \varphi'+\rho\frac{\sigma g}{k}\left(\mathbf{U}'-\sigma\frac{\partial \mathbf{u}'}{\partial t}\right) \tag{2.6}$$

$$\frac{\partial \mathbf{U}'}{\partial t}+\sigma g\left\{\text{grad } \varphi'+\frac{1}{k}\left(\mathbf{U}'-\sigma\frac{\partial \mathbf{u}'}{\partial t}\right)\right\}=0 \tag{2.7}$$

$$(1-\sigma)\frac{\partial \Theta'}{\partial t}+\text{div } \mathbf{U}' = 0. \tag{2.8}$$

We can now derive the equation for only the dilatation Θ in the following manner. In the remainder, let us omit the prime on each quantity. Taking the divergences of equations (2.6) and (2.7) and time derivative of equation (2.8)

$$(1-\sigma)\rho_s \frac{\partial^2 \Theta}{\partial t^2} = (\mathscr{L}+2\mathscr{M})\nabla^2\Theta-(1-\sigma)\rho g\nabla^2\varphi+\rho\frac{\sigma g}{k}\left(\text{div } \mathbf{U}-\sigma\frac{\partial \Theta}{\partial t}\right) \tag{2.9}$$

$$\frac{\partial}{\partial t}\operatorname{div}\mathbf{U}+\sigma g\left\{\nabla^2\varphi+\frac{1}{k}\left(\operatorname{div}\mathbf{U}-\sigma\frac{\partial\Theta}{\partial t}\right)\right\}=0. \tag{2.10}$$

$$(1-\sigma)\frac{\partial^2\Theta}{\partial t^2}+\frac{\partial}{\partial t}\operatorname{div}\mathbf{U}=0. \tag{2.11}$$

Combining equation (2.10) with equations (2.8) and (2.11),

$$\nabla^2\varphi=\frac{1-\sigma}{\sigma g}\frac{\partial^2\Theta}{\partial t^2}+\frac{1}{k}\frac{\partial\Theta}{\partial t}. \tag{2.12}$$

Substituting from equations (2.8) and (2.12) into equation (2.9),

$$(1-\sigma)\left\{\rho_s+\frac{1-\sigma}{\sigma}\rho\right\}\frac{\partial^2\Theta}{\partial t^2}+\frac{\rho g}{k}\frac{\partial\Theta}{\partial t}=(\mathscr{L}+2\mathscr{M})\nabla^2\Theta. \tag{2.13}$$

This is the equation we want to derive.

As the particular case which interests us, we shall pick out the quasi-static motion. Neglecting the accelerated terms in equations (2.12) and (2.13), we find

$$\frac{\partial\Theta}{\partial t}=k\nabla^2\varphi=\frac{k}{\rho g}(\mathscr{L}+2\mathscr{M})\nabla^2\Theta. \tag{2.14}$$

Differentiating the right part of equation (2.14) with respect to time and inserting the left part of equation (2.14) into this, we make

$$\nabla^2\left[\frac{\partial\varphi}{\partial t}-\frac{k}{\rho g}(\mathscr{L}+2\mathscr{M})\nabla^2\varphi\right]=0. \tag{2.15}$$

This may be regarded as the basic equation of three dimensional consolidation. It is worthy of note that if $\varphi_1(t, x_i)$ is a solution of equation (2.15), $\varphi_1(t, x_i)+F_1(x_i)+F_2(t)$, where $\nabla^2F_1=0$, is also the solution within the limits of quasi-static transition, in physical words, the progress of consolidation is not affected by the non-divergent flow of pore water and/or the gradual change of the water head in the entire region of the soil medium. When the non-divergent flow has already been included in the static part of piezometric head considered previously, equation (2.15) is reduced to

$$\frac{\partial\varphi}{\partial t}=\frac{k}{\rho g}(\mathscr{L}+2\mathscr{M})\nabla^2\varphi\ . \tag{2.16}$$

In the case of the elastic skeleton, operations $\mathscr{L}$ and $\mathscr{M}$ are reduced to Lame's constant λ and μ, respectively. Let us examine the special case of a column of soil supporting a load and confined in a rigid cylinder so that no lateral expansion can occur. Equation (2.16) is then rewritten as

$$\frac{\partial\varphi}{\partial t}=\frac{k}{\rho g}(\lambda+2\mu)\frac{\partial^2\varphi}{\partial x_3^2}.$$

By comparing this with Terzaghi's well-known equation, we can see the relation of operations $\mathscr{L}$ and $\mathscr{M}$ to the value a which is termed the coefficient of compressibility in soil mechanics, ([5])

$$\mathscr{L}+2\mathscr{M}=(1+e)/a=1/(1-\sigma)a.$$

Equation (2.14) is taken as the basic equation for subsidence related closely to the flow of confined ground water through the visco-elastic aquifer. We shall demonstrate it here in a simple example.

3. Example

We consider the deformation of a confined aquifer caused by pumping up the ground water at a constant rate in a laterally infinite aquifer with a uniform thickness, as seen in figure 1.6).

Assuming that, at the initial state, the confined water had no flow and the aquifer was in equilibrium under the overburden load in gravitational field, we have the equation for

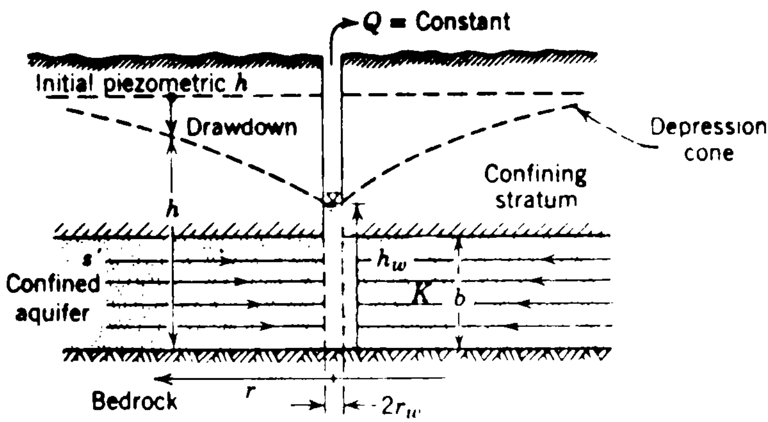

Figure 1. *Radial flow to a well completely penetrating an infinite confined aquifer with a uniform depth b*

the quasi-static motions of soil skeleton and ground water

$$\frac{\partial \Theta}{\partial t} = \frac{k}{\rho g}(\mathscr{L}+2\mathscr{M})\nabla^2 \Theta = k\nabla^2 \varphi. \tag{3.1}$$

Now, supposed that the aquifer deforms only in vertical direction and the flow of water is uniform in vertical cross-section and that the upper and lower boundary surfaces of aquifer are not leaky, we can put on initial and boundary conditions

$$t = 0; \quad \Theta = 0, \quad \varphi = 0, \quad w(\equiv u_3) = 0 \tag{3.2}$$

$$t > 0; \quad -2\pi bkr\frac{\partial \varphi}{\partial r} = Q, \quad \text{at} \quad r = r_w \tag{3.3}$$

$$\Theta = 0, \quad \frac{\partial \varphi}{\partial r} = 0, \quad \text{at} \quad r = \infty \tag{3.4}$$

$$\frac{\partial \varphi}{\partial z} = 0, \quad w = 0, \quad \text{at} \quad z = 0, \quad \frac{\partial \varphi}{\partial z} = 0 \quad \text{at} \quad z = b, \; (3.5), (3.6)$$

where b and k are the thickness and permeability of aquifer, respectively, r_w is the radius of pumping well, Q is pumping rate of water and the surfaces $z = 0$ and b are the lower and upper boundary surfaces of aquifer, respectively.

From the conditions (3.5) and (3.6), we can regard that the quantities φ and Θ are independent of z and that the amount of subsidence is obtained by $w\ (z = b) = b\Theta$.

Reducing the equation (3.1) to ordinary differential equation by Laplace transformation,

$$pV_\Theta = \frac{k}{\rho g}\{\mathscr{L}(p)+2\mathscr{M}(p)\}\nabla^2 V_\Theta = k\nabla^2 V_\varphi \tag{3.7}$$

with

$$r = r_w; \quad -2\pi bkr\frac{\partial V_\varphi}{\partial r} = \frac{Q}{p} \tag{3.8}$$

$$r = \infty; \quad V_\Theta = 0, \quad \frac{\partial V_\varphi}{\partial r} = 0, \tag{3.9}$$

where

$$V_\Theta = \int_0^\infty e^{-pt}\,\Theta\,dt, \quad V_\varphi = \int_0^\infty e^{-pt}\,\varphi\,dt. \qquad (3.10), (3.11)$$

Let us now consider the deformation in a Voigt model as a typical visco-elasticity. In this case, the operation $\mathscr{L}+2\mathscr{M}$ is expressed by

$$\mathscr{L}(p)+2\mathscr{M}(p) = (\lambda+2\mu)\frac{1+cp}{1+\gamma p}, \quad \text{where} \quad c \equiv \frac{\alpha\lambda+2\beta\mu}{\lambda+2\mu} \tag{3.12}$$

and by usual notations shown in figure 2, the quantities λ, μ, γ and c are represented as

$$\lambda+2\mu = \frac{E_1E_2}{E_1+E_2}, \quad \gamma = \frac{\eta_2}{E_1+E_2} \quad \text{and} \quad c = \frac{\eta_2}{E_2} \tag{3.13}$$

with notice that $1/c < 1/\gamma$. We have the solutions of equation (3.7)

$$V_\varphi = \frac{\sqrt{\tau}\,Q}{2\pi br_w}\sqrt{\frac{1+cp}{p^3(1+\gamma p)}}\;\frac{K_0\left(\frac{\sqrt{\tau}}{v}rz\right)}{K_1\left(\frac{\sqrt{\tau}}{v}r_w z\right)} \tag{3.14}$$

$$V_\Theta = \frac{\sqrt{\tau}\,Q}{2\pi vbr_w}\sqrt{\frac{1+\gamma p}{p^3(1+cp)}}\;\frac{K_0\left(\frac{\sqrt{\tau}}{v}rz\right)}{K_1\left(\frac{\sqrt{\tau}}{v}r_w z\right)} \tag{3.15}$$

where K_0 and K_1 are the modified Bessel functions and

$$v \equiv \frac{\lambda+2\mu}{\rho g}, \quad \tau \equiv \frac{\lambda+2\mu}{k\rho g} = \frac{v}{k}, \quad z \equiv \sqrt{\frac{p(1+\gamma p)}{1+cp}}.$$

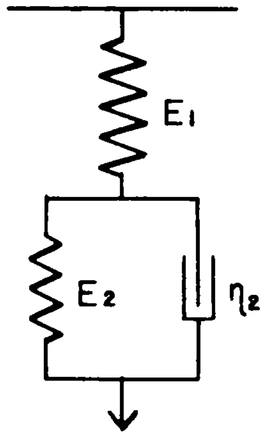

FIGURE 2. *Voigt's model for the rheological character of the aquifer*

The solutions φ and Θ are determined from V_φ and V_Θ by the use of the inversion theorem for the Laplace transformation

$$\varphi = \frac{1}{2\pi i}\int_{a-i\infty}^{a+i\infty} e^{tp} V_\varphi \, dp, \quad \Theta = \frac{1}{2\pi i}\int_{a-i\infty}^{a+i\infty} e^{tp} V_\Theta \, dp \qquad (3.16), (3.17)$$

As the functions

$$\sqrt{\frac{1+cp}{p^3(1+\gamma p)}}, \; \sqrt{\frac{1+\gamma p}{p^3(1+cp)}}, \; K_0\left(\frac{\sqrt{\tau}}{v} rz\right) \text{ and } K_1\left(\frac{\sqrt{\tau}}{v} r_w z\right)$$

have the branch points at $p = 0, -1/c, -1/\gamma$ and $-\infty$, the integrations in equations (3.16) and (3.17) are carried out, using the contour of figure 3 with two cuts on the negative real axis so that the integrants are single valued functions of p within and on the contour.

In the limit as the radius of circle Γ tends to infinity and the radii of circles δ_γ, δ_c tend to zero, the respective integrals round them can be shown to vanish. On the circle δ_0 we can find the limiting value

$$\lim_{\delta_0 \to 0} \frac{1}{2\pi i}\int_{\delta_0} e^{tp} V_\varphi \, dp = \frac{\tau Q}{4\pi v b}\left\{2C + \lim_{\delta_0 \to 0} \log\left(\frac{\tau r^2}{4v^2}\delta_0\right)\right\} \qquad (3.18)$$

as the radius δ_0 tends to zero, where $C = 0.5772\ldots$ is Euler's constant.

Because the argument of p is π and $-\pi$ on the lines L_1, L_2 and L_1', L_2' respectively and then $K_0[(\sqrt{\tau/v})rz)]$, $K_1[(\sqrt{\tau/v})r_w z]$ are expressed by the formulas

$$K_0(\pm iz) = -\frac{\pi}{2}[Y_0(z) \pm iJ_0(z)],$$

z: real, not negative

$$K_1(\pm iz) = -\frac{\pi}{2}[J_1(z) \mp iY_1(z)],$$

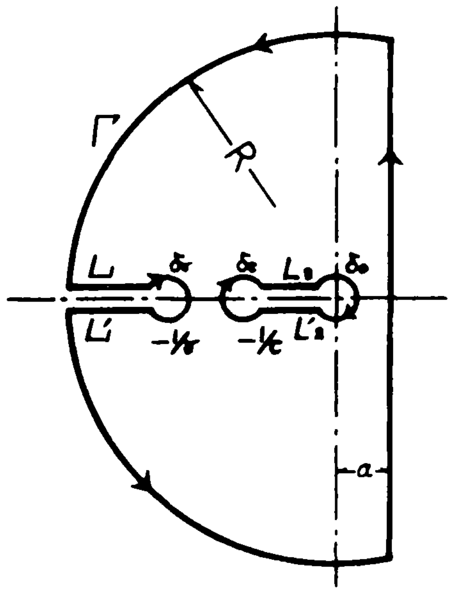

FIGURE 3. *The contour of inverse integration for Laplace transformations V_φ and V_Θ*

we get

$$\frac{1}{2\pi i}\left\{\int_{L_1} dp + \int_{L_1'} dp\right\}$$

$$= -\frac{\sqrt{\tau}Q}{2\pi b r_w}\frac{1}{\pi}\int_{1/\gamma}^{\infty} e^{-\rho t}\sqrt{\frac{1-c\rho}{\rho^3(1-\gamma\rho)}}\;\frac{J_0(r')Y_1(r_w')-Y_0(r')J_1(r_w')}{J_1^2(r_w')+Y_1^2(r_w')}d\rho,$$

$$\frac{1}{2\pi i}\left\{\int_{L_2} dp + \int_{L_2'} dp\right\}$$

$$= -\frac{\sqrt{\tau}Q}{2\pi b r_w}\frac{1}{\pi}\int_{\delta_0}^{1/c} e^{-\rho t}\sqrt{\frac{c\rho-1}{\rho^3(\gamma\rho-1)}}\;\frac{J_0(r')Y_1(r_w')-Y_0(r')J_1(r_w')}{J_1^2(r_w')+Y_1^2(r_w')}d\rho.$$

Finally, we get the solution for the piezometric head

$$\varphi = \frac{\tau Q}{4\pi v b}\left\{-2C - \lim_{\delta_0\to 0}\log\left(\frac{\tau r^2}{4v^2}\delta_0\right)\right\}$$

$$+\frac{\sqrt{\tau}Q}{2\pi^2 b r_w}\cdot\frac{1}{\pi}\lim_{\delta_0\to 0}\int_{\delta_0}^{1/c} e^{-\rho t}\sqrt{\frac{1-c\rho}{\rho^3(1-\gamma\rho)}}\;\frac{J_0(r')Y_1(r_w')-Y_0(r')J_1(r_w')}{J_1^2(r_w')+Y_1^2(r_w')}d\rho$$

$$+\frac{\sqrt{\tau}Q}{2\pi^2 b r_w}\frac{1}{\pi}\int_{1/\gamma}^{\infty} e^{-\rho t}\sqrt{\frac{c\rho-1}{\rho^3(\gamma\rho-1)}}\;\frac{J_0(r')Y_1(r_w')-Y_0(r')J_1(r_w')}{J_1^2(r_w')+Y_1^2(r_w')}d\rho,$$

where

$$r' = \frac{\sqrt{\tau}}{v}\, r \sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}, \qquad r'_w = \frac{\sqrt{\tau}}{v}\, r_w \sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}.$$

On a similar way, we can find the solution for the dilatation

$$\Theta = \frac{\tau Q}{4\pi v^2 b}\left\{-2C - \lim_{\delta_0\to 0} \log\left(\frac{\tau r^2}{4v^2}\delta_0\right)\right\}$$

$$+ \frac{\sqrt{\tau}Q}{2\pi v b r_w}\frac{1}{\pi}\lim_{\delta_0\to 0}\int_{\delta_0}^{1/c} e^{-\rho t}\sqrt{\frac{-\gamma\rho}{\rho^3(1-c\rho)}}\, \frac{J_0(r')\,Y_1(r'_w) - Y_0(r')\,J_1(r'_w)}{J_1^2(r'_w)+Y_1^2(r'_w)}\,\mathrm{d}\rho$$

$$+ \frac{\sqrt{\tau}Q}{2\pi v b r_w}\frac{1}{\pi}\int_{1/\gamma}^{\infty} e^{-\rho t}\sqrt{\frac{1-\gamma\rho}{\rho^3(1-c\rho)}}\, \frac{J_0(r')\,Y_1(r'_w) - Y_0(r')\,J_1(r'_w)}{J_1^2(r'_w)+Y_1^2(r'_w)}\,\mathrm{d}\rho.$$

When the radius of the pumping well is very small, we have

$$\lim_{r_w\to 0} J_1(r'_w) = 0,\ \lim_{r_w\to 0} r_w\, Y_1(r'_w) = -\frac{2}{\pi}\frac{r_w}{r'_w} = -\frac{2v}{\pi\sqrt{\tau}}\sqrt{\frac{1-c\rho}{\rho(1-\gamma\rho)}}$$

and then

$$\lim_{r_w\to 0} \varphi = \frac{\tau Q}{4\pi v b}\Bigg[-2C - \lim_{\delta_0\to 0} \log\left(\frac{\tau r^2}{4v^2}\delta_0\right)$$

$$- \lim_{\delta_0\to 0}\int_{\delta_0}^{1/c}\frac{e^{-\rho t}}{\rho} J_0\left(\frac{\sqrt{\tau}}{v}\, r\sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}\right)\mathrm{d}\rho - \int_{1/\gamma}^{\infty}\frac{e^{-\rho t}}{\rho} J_0\left(\frac{\sqrt{\tau}}{v}\, r\sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}\right)\mathrm{d}\rho\Bigg] \tag{3.19}$$

$$\lim_{r_w\to 0} \Theta = \frac{\tau Q}{4\pi v^2 b}\Bigg[-2C - \lim_{\delta_0\to 0} \log\left(\frac{\tau r^2}{4v^2}\delta_0\right)$$

$$- \lim_{\delta_0\to 0}\int_{\delta_0}^{1/c} e^{-\rho t}\frac{1-\gamma\rho}{\rho(1-c\rho)} J_0\left(\frac{\sqrt{\tau}}{v}\, r\sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}\right)\mathrm{d}\rho$$

$$- \int_{1/\gamma}^{\infty} e^{-\rho t}\frac{1-\gamma\rho}{\rho(1-c\rho)} J_0\left(\frac{\sqrt{\tau}}{v}\, r\sqrt{\frac{\rho(1-\gamma\rho)}{1-c\rho}}\right)\mathrm{d}\rho\Bigg]. \tag{3.20}$$

In the special case where $\gamma = 0$, we have

$$\lim_{r_w\to 0} \varphi = \frac{\tau Q}{4\pi v b}\Bigg[-2C - \lim_{\delta_0\to 0} \log\left(\frac{\tau r^2}{4v^2}\delta_0\right)$$

$$+ \lim_{\delta_0\to 0}\int_{\delta_0}^{1/c}\frac{\partial}{\partial\rho} W(\rho t)\, J_0\left(\frac{\sqrt{\tau}}{v}\, r\sqrt{\frac{\rho}{1-c\rho}}\right)\mathrm{d}\rho\Bigg]$$

$$= \frac{Q}{4\pi b k}\Bigg[-2C - \lim_{\delta_0\to 0}\log\left(\frac{\tau v^2}{4v^2}\delta_0\right) - \lim_{\delta_0\to 0} W(\delta_0 t)$$

$$+\frac{\sqrt{\tau}}{v}r\int_0^{1/c} W(\rho t)\frac{\partial}{\partial\rho}\sqrt{\frac{\rho}{1-c\rho}}\cdot J_1\left(\frac{\sqrt{\tau}}{v}r\sqrt{\frac{\rho}{1-c\rho}}\right)\mathrm{d}\rho\Bigg]$$

$$=\frac{Q}{4\pi bk}\left[-C-\log\left(\frac{\tau}{4v^2}\frac{r^2}{t}\right)+\frac{\sqrt{\tau}}{v}r\int_0^{\infty} W\left(\frac{t\xi^2}{c\xi^2+1}\right)J_1\left(\frac{\sqrt{\tau}}{v}r\xi\right)\mathrm{d}\xi\right],$$

where

$$W(x)=\int_x^{\infty}\frac{\mathrm{e}^{-\eta}}{\eta}\mathrm{d}\eta \text{ is well function and } \xi\equiv\sqrt{\frac{\rho}{1-c\rho}},$$

and also

$$\lim_{r_w\to 0}\Theta=\frac{\tau Q}{4\pi v^2 b}\Bigg[-2C-\lim_{\delta_0\to 0}\log\left(\frac{\tau v^2}{4v^2}\delta_0\right)-\lim_{\delta_0\to 0}\int_{\delta_0}^{1/c}\frac{\mathrm{e}^{-\rho t}}{\rho}J_0\left(\frac{\sqrt{\tau}}{v}r\sqrt{\frac{\rho}{1-c\rho}}\right)\mathrm{d}\rho$$

$$-c\int_0^{1/c}\frac{\mathrm{e}^{-\rho t}}{1-c\rho}J_0\left(\frac{\sqrt{\tau}}{v}r\sqrt{\frac{\rho}{1-c\rho}}\right)\mathrm{d}\rho\Bigg]$$

$$=\frac{\rho g Q}{4\pi b(\lambda+2\mu)k}\Bigg[-C-\log\left(\frac{\tau}{4v^2}\frac{r^2}{t}\right)+\frac{\sqrt{\tau}}{v}r\int_0^{\infty}W\left(\frac{t\xi^2}{c\xi^2+1}\right)J_1\left(\frac{\sqrt{\tau}}{v}r\xi\right)\mathrm{d}\xi$$

$$-\mathrm{e}^{-t/c}\int_0^1\frac{\mathrm{e}^{-(t/c)\eta}}{\eta}J_0\left(\frac{\sqrt{\tau}}{v}r\sqrt{\frac{1-\eta}{c\eta}}\right)\mathrm{d}\eta\Bigg]\quad(1-c\rho\equiv\eta)$$

$$=\frac{\rho g Q}{4\pi b(\lambda+2\mu)k}\Bigg[-C-\log\left(\frac{\tau}{4v^2}\frac{r^2}{t}\right)+\mathrm{e}^{-t/c}W\left(-\frac{t}{c}\right)$$

$$+\frac{\sqrt{\tau}}{v}r\int_0^{\infty}W\left(\frac{t\xi^2}{c\xi^2+1}\right)J_1\left(\frac{\sqrt{\tau}}{v}r\xi\right)\mathrm{d}\xi$$

$$-\frac{\sqrt{\tau}}{v}r\mathrm{e}^{-t/c}\int_0^{\infty}W\left\{-\frac{t}{c(c\xi^2+1)}\right\}J_1\left(\frac{\sqrt{\tau}}{v}r\xi\right)\mathrm{d}\xi\Bigg].$$

Using the dimension time t^* and distance r^*

$$t^*\equiv\frac{t}{c},\quad \overline{r^*}\equiv\sqrt{\frac{\tau}{c}}\frac{r}{v},$$

the amount of subsidence ζ is obtained by

$$\zeta=b\Theta$$

$$=\frac{\rho g Q}{4\pi(\lambda+2\mu)k}\Bigg[-C-\log\left(\frac{r^{*2}}{4t^*}\right)+\mathrm{e}^{-t^*}W(-t^*)$$

$$+\int_0^{\infty} W\left(\frac{t^* y^2}{y^2+1}\right) r^* J_1(r^* y)\mathrm{d}y$$

$$-\mathrm{e}^{-t^*}\int_0^{\infty} W\left(\frac{-t^*}{y^2+1}\right) r^* J_1(r^* y)\mathrm{d}y\Bigg];\ (y\equiv\sqrt{c\xi})$$

$$=\frac{\rho g Q}{4\pi(\lambda+2\mu)k}\Bigg[-C-\log\left(\frac{r^{*2}}{4t^*}\right)+\mathrm{e}^{-t^*}W(-t^*)$$

$$-\int_0^{\infty}\left\{C+\log\left(\frac{t^* y^2}{y^2+1}\right)+\sum_{n=1}^{\infty}\frac{(-1)^n}{n\cdot n!}\left(\frac{t^* y^2}{y^2+1}\right)^n\right\} r^* J_1(r^* y)\mathrm{d}y$$

$$+\mathrm{e}^{-t^*}\int_0^{\infty}\left\{C+\log\left(\frac{t^*}{y^2+1}\right)+\sum_{n=1}^{\infty}\frac{1}{n\cdot n!}\left(\frac{t^*}{y^2+1}\right)^n\right\} r^* J_1(r^* y)\mathrm{d}y\Bigg]$$

$$=\frac{\rho g Q}{4\pi(\lambda+2\mu)k}\Bigg[-C-\log\left(\frac{r^{*2}}{4t^*}\right)+\mathrm{e}^{-t^*}W(-t^*)-(C+\log t^*)$$

$$-2r^*\int_0^{\infty}\log y\cdot J_1(r^* y)\mathrm{d}y+\int_0^{\infty}\frac{2y}{y^2+1}J_0(r^* y)\mathrm{d}y$$

$$-\sum_{n=1}^{\infty}\frac{(-1)^n}{n\cdot n!}t^{*n}\int_0^{\infty}\frac{\mathrm{d}}{\mathrm{d}y}\left\{1-\frac{1}{y^2+1}\right\}^n J_0(r^* y)\mathrm{d}y$$

$$+\mathrm{e}^{-t^*}(C+\log t^*)-\mathrm{e}^{-t^*}\int_0^{\infty}\frac{2y}{y^2+1}J_0(r^* y)\mathrm{d}y$$

$$+\mathrm{e}^{-t^*}\sum_{n=1}^{\infty}\frac{1}{n\cdot n!}t^{*n}\left\{1+\int_0^{\infty}\frac{\mathrm{d}}{\mathrm{d}y}\left(\frac{1}{y^2+1}\right)^n J_0(r^* y)\mathrm{d}y\right\}\Bigg]$$

$$=\frac{\rho g Q}{4\pi(\lambda+2\mu)k}\Bigg[-2C-\log\frac{r^{*2}}{4}+\mathrm{e}^{-t^*}W(-t^*)-2\left(-C-log\frac{r^*}{2}\right)$$

$$+2(1-\mathrm{e}^{-t^*})K_0(r^*)+\mathrm{e}^{-t^*}\left(C+\log t^*+\sum_{n=1}^{\infty}\frac{1}{n\cdot n!}t^{*n}\right)$$

$$-2\sum_{n-1}^{\infty}\frac{(-1)^n}{n}t^{*n}\left\{\sum_{m=0}^{n-1}\frac{(-1)_m}{m!(m+1)!(n-m-1)!}\left(\frac{r^*}{2}\right)^{m+1}K_{m+1}(r_*)\right\}$$

$$-2\mathrm{e}^{-t^*}\sum_{n=1}^{\infty}\frac{1}{(n!)^2}t^{*n}\left(\frac{r^*}{2}\right)^n K_n(r^*)\Bigg]$$

$$= \frac{\rho g Q}{2\pi(\lambda+2\mu)k}\left[(1-e^{-t^*})K_0(r^*) + \sum_{n=1}^{\infty}\frac{1}{(n-1)!n!}\left(\frac{r^*}{2}\right)^n K_n(r^*)\sum_{m=0}^{\infty}\frac{(-1)^m {}^{*n+m}}{m!(n+m)}\right.$$

$$\left. - e^{-t^*}\sum_{n=1}^{\infty}\frac{1}{(n!)^2}t^{*n}\left(\frac{r^*}{2}\right)^n K_n(r^*)\right]$$

$$= \frac{\rho g Q}{2\pi(\lambda+2\mu)k}\left[(1-e^{-t^*})K_0(r^*) + \sum_{n=1}^{\infty}\frac{1}{n!}\left(\frac{r^*}{2}\right)^n K_n(r^*)\left\{1-e^{-t^*}\sum_{m=0}^{n}\frac{t^{*m}}{m!}\right\}\right]$$

Summarizing the result of the calculation, we have the solutions φ and ζ in a special case where

$$\mathscr{L}+2\mathscr{M} = (\lambda+2\mu)\left(1+c\frac{\partial}{\partial t}\right)$$

$$\lim_{rw\to 0}\varphi = \frac{Q}{2\pi bk}\left[K_0(r^*)+\sum_{n=1}^{\infty}\frac{1}{n!}\left(\frac{r^*}{2}\right)^n K_n(r^*)\left\{1-e^{-t^*}\sum_{m=0}^{n-1}\frac{t^{*m}}{m!}\right\}\right] \quad (3.21)$$

$$\lim_{rw\to 0}\zeta = \frac{\rho g Q}{2\pi(\lambda+2\mu)k}\left[\sum_{n=0}^{\infty}\frac{1}{n!}\left(\frac{r^w}{2}\right)^n K_n(r^*)\left\{1-e^{-t^*}\sum_{m=0}^{n}\frac{t^{*m}}{m!}\right\}\right], \quad (3.22)$$

where

$$t^* \equiv \frac{t}{c},\ r^* \equiv \sqrt{\frac{r}{c}}\ \frac{r}{v} \equiv \sqrt{\frac{\rho g r_2}{ck(\lambda+2\mu)}}. \quad (3.23)$$

4. Practical determination of proper constants of aquifer in Voigt model

We may have to find the adequate method determining proper constants of the aquifer, that is, elastic factor $\lambda+2\mu$, permeability k and retarding time c for practical utility of above solutions (3.21) and (3.22). First approximations will make it possible to us in following way.

Differentiating these with respect to tume t, we have

$$\frac{\partial\varphi}{\partial t} \simeq \frac{Q}{2\pi cbk}\frac{r^*}{2}K_1(r^*)e^{-t/c},\ \frac{\partial\varsigma}{\partial t} \simeq \frac{\rho g Q}{2\pi ck(\lambda+2\mu)}K_0(r^*)e^{-t/c} \quad (4.1), (4.2)$$

and could determine the retarding time c from the slope of each line along which the rate values of variation of head φ or subsidence ζ were plotted against time on the semi-logarithmic graphs, respectively. Moreover, from the segments on the ordinate of graphs, respectively. Moreover, from the segments on the ordinate of graphs cut off by plotted lines, $\log a_\varphi$ and $\log a_\zeta$, we obtain the relations

$$\frac{4\pi c}{Q}a_\varphi = \frac{r^*}{bk}K_1(r^*) = \frac{1}{bk}\frac{r}{\sqrt{c}}zK_1\left(\frac{r}{\sqrt{c}}z\right) \quad (4.3)$$

$$\frac{2\pi c}{Q}a_\zeta = \frac{\rho g}{k(\lambda+2\mu)}K_0(r^*) = z^2K_0\left(\frac{r}{\sqrt{c}}z\right), \quad (4.4)$$

where

$$z \equiv \sqrt{\frac{\rho g}{k(\lambda+2\mu)}} \tag{4.5}$$

So, if we are ready to use the family of curves, depending on the Bessel function $K_0(dz)$ with respect to the parameter $\alpha \equiv r/\sqrt{c}$ as seen in figure 4, we may seek easily the value of argument z in equation (4.4) fitting to calculated value $2\pi c a_t/Q$, which is regarded as the

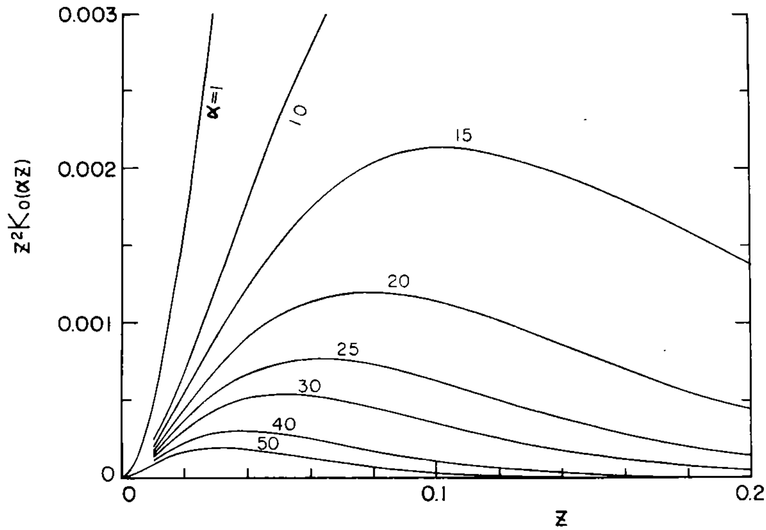

FIGURE 4. *Graph for finding the value of argument z of Bessel function fittiag to equation (4.4)*

ordinate value in this figure, because the parameter α has been specified from retarding time c and actual distance r from pumping well to observing point. When the argument z was decided, the value of permeability k will be determined by equation (4.3) in use of data for the thickness b of aquifer and subsequently the value of elastic factor $(\lambda+2\mu)$ will be also determined by eequation (4.5).

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A THEORETICAL APPROACH TO STRESS-STRAIN RELATIONS OF CLAYS

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ABSTRACT

The derivations of the stress-volumetric strain relations for normally and over-consolidated clays are described. The isometric view of the state surface defined by the stress-volumetric strain relations is given. The settlement calculation based on the stress-volumetric strain relations is proposed.

RÉSUMÉ

Les rapports contrainte-déformation de volume pour les argiles normalement consolidées et surconsolidées sont dérivés. Une vue approximative de la surface définie par les rapports contrainte-déformation de volume est décrite. Un calcul du tassement basé sur les rapports contrainte-déformation de volume est proposé.

1. INTRODUCTION

In current practice the stability analysis of soil structures is based on the equilibrium condition and the failure condition, the soils are assumed to be rigid-plastic materials which have no restraints with respect to deformation of soil elements. The engineering problems dealing with the deformation of soils are analyzed on the basis of consolidation theory or elastic theory. The theories for analyzing soil stability and soil deformation have no basic common ground.

Because of this criticism, there have been many attempts to establish the general stress-strain relation or the constitutive equation. Some of these approaches were presented in the series of papers by Roscoe *et al.* [1, 2, 3] (1958, 1963, 1963). They defined the mechanical state of a soil as a point in three-dimensional space whose coordinates consist of the principal stress difference, q, the effective mean principal stress, p, and the void ratio e. The authors would like to identify these components as the octahedral stresses τ_{oct}, σ'_m, and the void ratio e, which are herein termed parameters of state. The three-dimensional space defined by these three parameters of state is named the state space, and the points which represent the mechanical state of a soil are called state points. A change in the mechanical state of a soil results in movement of the state point. This locus of the state point, called the state path, lies on the surface defined by the equation which should be satisfied by the parameters of state of the soil. This surface is called the state surface.

The purpose of this paper is to derive the equation of the state surface. The application of the theory for the calculation of settlement of ground caused by the change of the stress state, for instance by the construction of buildings and embankment or by the change of ground-water level, will be discussed in the latter part of the paper.

2. STRESS–VOLUMETRIC STRAIN RELATIONS

The volume change of clays caused by a change in the stress state of clay elements consists of two components; one is the consolidation component that is induced by the change of hydrostatic pressure σ'_m, and the other is the dilatancy component which accompanies

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the distorsional deformation of clay elements. The infinitesimal change of void ratio by the infinitesimal change of all round pressure is represented as follows from well known $e-\ln\sigma''_m$ linear relation

$$de = -\lambda\frac{d\sigma'_m}{\sigma'_m} \tag{1}$$

where the coefficient λ is the inclination of the normal consolidation curve in $e-\ln\sigma'_m$ diagram. Shibata [4] (1963), Karube and Kurihara [5] (1966) showed the linear relation between the volume change by dilatancy and the stress ratio τ_{oct}/σ'_m. It follows that, for clays preconsolidated with the pressure of σ'_{m0}, the infinitesimal change of void ratio by dilatancy is given by

$$de = (1+e_0)\mu\left(\frac{\tau_{oct}}{\sigma'_m}\cdot\frac{d\sigma'_m}{\sigma'_m}-\frac{d\tau_{oct}}{\sigma'_m}\right) \tag{2}$$

where the coefficient μ is a soil constant and e_0 is the void ratio for the pre-consolidation pressure σ'_{m0}.

The two components of volume change (consolidation and dilatancy) are not necessarily independent of each other; therefore the total volume change induced by the stress change is not necessarily equal to the summation of consolidation and dilatancy. The appropriateness of superposition of consolidation and dilatancy is not guaranteed, but the infinitesimal volume change may be approximately given by

$$de = -\lambda\frac{d\sigma'_m}{\sigma'_m}+(1+e_0)\mu\left(\frac{\tau_{oct}}{\sigma'_m}\cdot\frac{d\sigma'_m}{\sigma'_m}-\frac{d\tau_{oct}}{\sigma'_m}\right) \tag{3}$$

A more detailed discussion and experimental background of equation (3) are given by Hata, Ohta and Yoshitani [6].

Now, writing

$$\tau_{oct} = k\sigma'_m \tag{4}$$

we get

$$d\tau_{oct} = \sigma'_m dk + k\,d\sigma'_m \tag{5}$$

and substituting into equation (3)

$$de = -\lambda\frac{d\sigma'_m}{\sigma'_m}-(1+e_0)\mu\,dk \tag{6}$$

Then, representing

$$\frac{d\sigma'_m}{\sigma'_m} = \alpha\sigma \tag{7}$$

we get

$$\sigma = \ln\sigma'_m+\text{const.} \tag{8}$$

Substituting equation (7) into equation (6)

$$de+\lambda\,d\sigma+(1+e_0)\mu\,dk = 0 \tag{9}$$

We note that equation (9) is a two variable complete differential equation. Now assuming that $dk = 0$:

$$de+\lambda\,d\sigma = 0 \tag{10}$$

and then

$$e = -\lambda\sigma - \varphi(k) \tag{11}$$

Total differential of equation (11) is

$$de + \lambda d\sigma + \varphi'(k)dk = 0 \tag{12}$$

and because equation (12) should satisfy equation (9), we get

$$\varphi'(k) = (1+e_0)\mu \tag{13}$$

and then

$$\varphi(k) = (1+e_0)\mu k + \text{const.} \tag{14}$$

We get the solution of equation (9) by substituting equation (14) into equation (11):

$$e + \lambda\sigma + (1+e_0)\mu k + \text{const.} = 0 \tag{15}$$

Substituting equations (4), (8) into equation (15)

$$e + \lambda \ln \sigma'_m + (1+e_0)\mu \frac{\tau_{oct}}{\sigma'_m} + \text{const.} = 0 \tag{16}$$

Determining the arbitrary constant in equation (16) according to the boundary condition, we get the state surface in the state space.

Now, let us consider a clay isotropically pre-consolidated under the pressure σ'_{m_0}; the void ratio for the pressure $\sigma\ _0$ is e_0. This means that the boundary condition should be represented by the point ($\tau_{oct} = 0$, $\sigma'_m = \sigma'_{m_0}$, $e = e_0$) which should be on the state surface to be defined. Substituting $e = e_0$, $\sigma'_m = \sigma'_{m_0}$, and $\tau_{oct} = 0$ into equation (16) yields

$$\text{const.} = -(e_0 + \lambda \ln \sigma'_{m_0}) \tag{17}$$

Then the state surface for a clay isotropically consolidated with the pressure of σ'_{m_0} is given by

$$e - e_0 + \lambda \ln \frac{\sigma'_m}{\sigma'_{m_0}} + (1+e_0)\mu \frac{\tau_{oct}}{\sigma'_m} = 0 \tag{18}$$

Under smoothly and monotonously increasing shearing strain the change of mechanical state of a clay is represented by the state path over this state surface. However, this surface is limited by the critical state line defined by Roscoe *et al.* [1] (1958) and by the swelling wall, which is the vertical wall on the swelling line starting from the point (σ_{m_0}, e_0) on the plane $\tau_{oct} = 0$, as shown in figure 1.

The state points of clays pre-consolidated with the pressure of σ'_{m_0} should be on the hatched part of the state surface and on the swelling wall represented by BPZD and CZS and SZPZ,S, respectively in figure 1. The initial state points of clays pre-consolidated with the pressure of σ'_{m_0} are on the swelling line PZ,S in figure 1. Under increasing shear strain, the state paths of the clays climb up along the swelling wall with increasing τ_{oct} until they reach the state surface, and then they creep the surface with the change of mechanical state. Finally they arrive at the critical state points on the critical state line and do not traverse the state surface further.

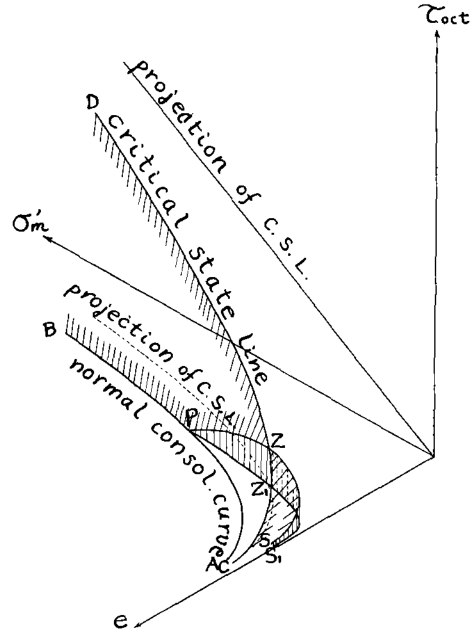

FIGURE 1. *State surface and swelling wall for isotropically consolidated clays*

If the initial state point ($\tau_{oct} = 0$, σ'_{m_i} $e = e_i$) of a clay pre-consolidated with the pressure of σ'_{m_0} is on the swelling curve, it follows that

$$e_i = e_0 - k \ln \frac{\sigma'_{m_i}}{\sigma'_{m_0}} \tag{19}$$

where the coefficient k is the inclination of the swelling curve in $e - \ln\sigma'_m$ diagram. When such a clay is sheared under undrained conditions, the initial void ratio is kept constant.

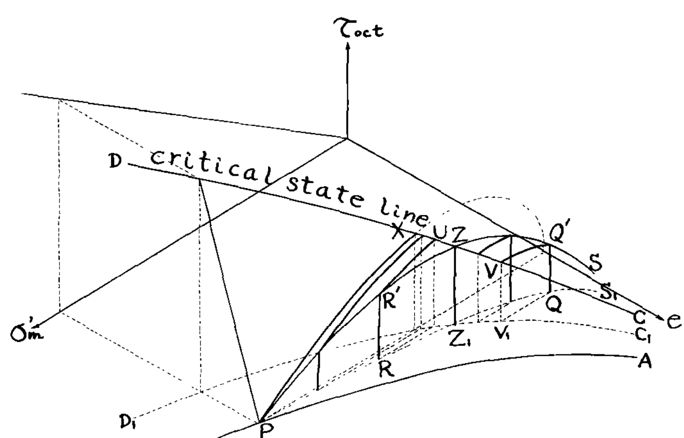

FIGURE 2. *State paths of undrained test*

Then the state path should be on the $e = e_i$ plane. The state path or the stress path which is defined as the projection of the state path on $e = 0$ plane is the section of the swelling wall and the state surface with the $e = e_i$ plane as shown by PX, RR′U, Z_1Z, QQ′V in figure 2. Only the state path, PX, of a normally consolidated clay whose initial void ratio is $e_i = e_0$ does not climb the swelling wall. The equation of the state path of a clay under undrained shear is represented by substituting $e = e_i$ into equation (18)

$$e_i - e_0 + \lambda \ln \frac{\sigma'_m}{\sigma'_{m0}} + (1+e_0)\mu \frac{\tau_{oct}}{\sigma'_m} = 0 \tag{20}$$

The height (τ_{oct}) of the vertical state path along the swelling wall is given by substituting $\sigma'_m = \sigma'_{mi}$ into equation (20)

$$\tau_{oct} = -\frac{\sigma'_{mi}}{(1+e_0)\mu}\left(e_i - e_0 + \lambda \ln \frac{\sigma'_{mi}}{\sigma'_{m0}}\right) \tag{21}$$

and substituting equation (19) into equation (21)

$$\tau_{oct} = -\frac{\lambda - k}{(1+e_0)\mu}\sigma'_{mi} \ln \frac{\sigma'_{mi}}{\sigma'_{m0}} \tag{22}$$

For the normally consolidated clay, $e_i = e_0$, and then equation (20) becomes

$$\tau_{oct} = -\frac{\lambda}{(1+e_0)\mu}\sigma'_m \ln \frac{\sigma'_m}{\sigma'_{m0}} \tag{23}$$

Equation (22) and (23) represent RR′, Z_1Z, QQ′, and PX respectively. The induced pore pressure is calculated as the difference between the total mean stress σ_m and the effective mean stress σ'_m for the same octahedral shear stres τ_{oct}.

If a clay is sheared under drained conditions, the void ratio e of the clay is given by equation (18) for the applied stresses τ_{oct} and σ'_m. If the concept of volume ratio is defined by

$$f = 1 + e \tag{24}$$

as proposed by Mikasa [7] (1963), equation (18) can be written as

$$\frac{f_0 - f}{f_0} = \frac{\lambda}{f_0} \ln \frac{\sigma'_m}{\sigma'_{m0}} + \mu \frac{\tau_{oct}}{\sigma'_m} \tag{25}$$

The left hand term $(f_0 - f)/f_0$ is the volumetric strain for normally consolidated clays and therefore equation (25) is nothing else but the stress-volumetric strain relation for normally isotropically consolidated clays. For overconsolidated clays pre-consolidated with the pressure of σ'_{m0}, equation (25) is acceptable. However, the term $(f_0 - f)/f_0$ does not represent the volumetric strain of the overconsolidated clays in a strict sense, because the initial volume ratio f_i and the initial void ratio e_i do not coincide with f_0 and e_0, respectively.

Consider a clay, normally consolidated under the pressure σ'_{m0}, with an initial state point ($\tau_{oct} = 0$, $\sigma_m = \sigma'_{mi}$, $e = e_i$) that falls on the swelling curve. If this clay sheared under the drained condition, the void ratio of the clay (whose stress state is represented by the point (τ_{oct}, σ'_m) on the plane $e = 0$) is given by the point at which a perpendicular to that plane ($e = 0$) erected at that point (τ_{oct}, σ_m') intersects the swelling wall or the state surface.

If the line representing the stress state intersects the swelling wall, the void ratio is given by

$$e = e_0 - k \ln \frac{\sigma_m'}{\sigma_{m_0}'} \tag{26}$$

independently of the initial state point on the swelling curve. In this case, the dilatancy component of volume change does not take place. The volumetric strain of an overconsolidated clay is derived from equation (26) as

$$\begin{aligned} \frac{f_i - f}{f_i} &= \frac{f_i - f_0}{f_i} + \frac{k}{f_i} \ln \frac{\sigma_m'}{\sigma_{m_0}'} \\ &= \frac{k}{f_i} \ln \frac{\sigma_m'}{\sigma_{mi}'} \end{aligned} \tag{27}$$

Equation (27) is the stress-volumetric strain relation for this case.

If the stress state line intersects with the state surface, the void ratio is given by equation (18). In this case the volumetric strain of an overconsolidated clay is given from equation (18) or (25):

$$\begin{aligned} \frac{f_i - f}{f_i} &= \frac{f_i - f_0}{f_i} + \frac{\lambda}{f_i} \ln \frac{\sigma_m'}{\sigma_{m0}'} + \frac{f_0}{f_i} \mu \frac{\tau_{oct}}{\sigma_m'} \\ &= \frac{k}{f_i} \left(\mu \frac{\tau_{oct}}{\sigma_m'} - 1 \right) \ln \frac{\sigma_{mi}'}{\sigma_{m_0}'} + \frac{\lambda}{f_i} \ln \frac{\sigma_m'}{\sigma_{m_0}'} + \mu \frac{\tau_{oct}}{\sigma_m'} \end{aligned} \tag{28}$$

Equation (28) is the stress-volumetric strain relation for this case.

Summarizing this discussion, the stress-volumetric strain relationships of clays isotropically pre-consolidated with the pressure of σ_{m_0}' are

$$\frac{f_0 - f}{f_0} = \frac{\lambda}{f_0} \ln \frac{\sigma_m'}{\sigma_{m0}'} + \mu \frac{\tau_{oct}}{\sigma_m'} \tag{25}$$

for the normally consolidated clays and

$$\frac{f_i - f}{f_i} = \frac{k}{f_i} \ln \frac{\sigma_m'}{\sigma_{mi}'} \tag{27}$$

for the overconsolidated clays whose stress states are given by

$$\begin{aligned} \tau_{oct} &\leqq -\frac{\lambda - k}{f_0 \mu} \sigma_m' \ln \frac{\sigma_m'}{\sigma_{m_0}'} \\ &= -\frac{\lambda - k}{\left(f_i + k \ln \dfrac{\sigma_{mi}'}{\sigma_{m_0}'} \right) \mu} \sigma_m' \ln \frac{\sigma_m}{\sigma_{m0}'} \end{aligned} \tag{29}$$

and

$$\frac{f_i - f}{f_i} = -\frac{\chi}{f_i}\left(\mu\frac{\tau_{oct}}{\sigma'_m} - 1\right)\ln\frac{\sigma'_{mi}}{\sigma'_{m0}} + \frac{\lambda}{f_i}\ln\frac{\sigma'_m}{\sigma'_{m0}} + \mu\frac{\tau_{oct}}{\sigma'_m} \tag{28}$$

for the overconsolidated clays whose stress states are given by

$$\tau_{oct} \leqslant -\frac{\lambda - k}{\left(f_i + k\ln\frac{\sigma'_{mi}}{\sigma'_{m0}}\right)\mu}\,\sigma'_m \ln\frac{\sigma'_m}{\sigma'_{m0}} \tag{30}$$

Using these stress-volumetric strain relations, the volume changes of clay elements under drained shear are calculated.

Equations (25), (27) and (28) may be used for approximate calculation of the volume changes of isometrically consolidated clay masses under an arbitrary stress state and under drained conditions. However, if the natural clay deposit was consolidated anisotropically, then the settlement of structures on the clay layer cannot be rigorously calculated from these equations.

For clays anisotropically pre-consolidated under the pressure σ'_{m0}, the boundary conditions of equation (16) are given by the following set:

$$\tau_{oct} = k_0\sigma'_{m0}, \quad \sigma'_m = \sigma'_{m0}, \quad e = e_0 \tag{31}$$

Substituting equations (31) into equation (16)

$$e - e_0 + \lambda\ln\frac{\sigma'_m}{\sigma'_{m0}} + (1+e_0)\mu\left(\frac{\tau_{oct}}{\sigma'_m} - k_0\right) = 0 \tag{32}$$

The state surface defined by equation (32) and the swelling wall are shown in figure 3.

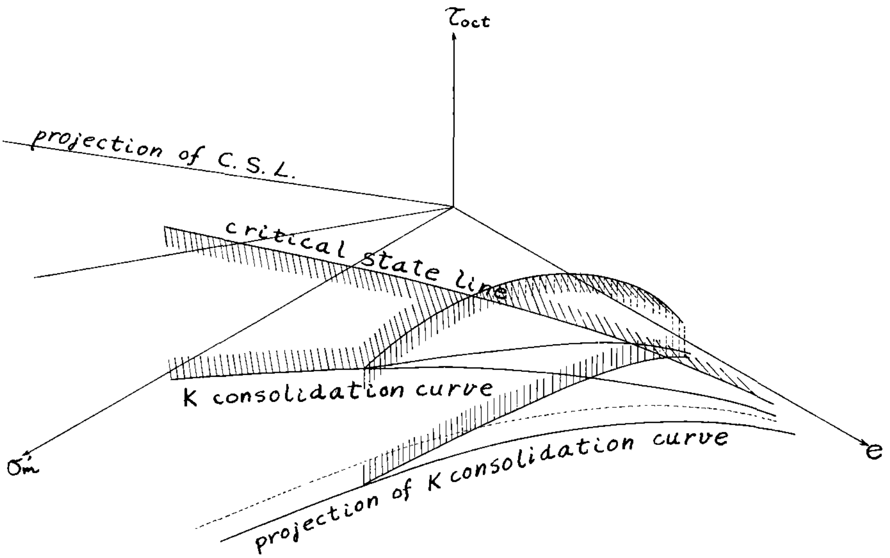

FIGURE 3. *State surface and swelling wall for anisotropically consolidated clays*

Equation (32) has not yet been supported by experiments, especially for the overconsolidated state. However, for the anisotropically normally consolidated clays equation (32) might be adaptable. The stress-volumetric strain relation for anisotropically normally consolidated clays is given by

$$\frac{f_0 - f}{f_0} = \frac{\lambda}{f_0} \ln \frac{\sigma'_m}{\sigma'_{m_0}} + \mu\left(\frac{\tau_{oct}}{\sigma'_m} - k_0\right) \tag{33}$$

3. SETTLEMENT CALCULATION

If the stress state in a clay layer before and after the loading is calculated, the volume change of the clay layer can be derived from the stress-volumetric strain relations mentioned above. However, the current theories about stress state in the ground have many inconsistencies with respect to the mechanical behaviors of clays. In spite of their incompleteness, they may be used for the calculation of the stress state in the ground in practical engineering problems concerning the deformation of a soil mass and consequent deflection of the structure.

The authors would like to show a very simple example of calculation of the settlement using the stress-volumetric strain relations mentioned above. The settlement which takes place immediately after the application of load is not dealt with. The stress increments in the ground induced by strip loading are calculated by elastic theory:

$$\begin{aligned} \varDelta\sigma_1 &= \frac{q}{\pi}(2\varepsilon + \sin 2\varepsilon) \\ \varDelta\sigma_3 &= \frac{q}{\pi}(2\varepsilon - \sin 2\varepsilon) \end{aligned} \tag{34}$$

where q is the intensity of the load and the angle ε is shown in figure 4. This result is independent of soil constants.

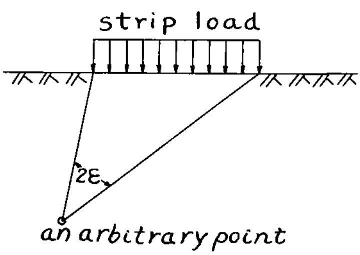

FIGURE 4. *The angle ε*

Defining the intermediate principal stress ratio as:

$$N = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \tag{35}$$

octahedral stresses are given by:

$$\tau_{oct} = \frac{\sqrt{2}}{3}\sqrt{1-N+N^2}\,(\sigma_1-\sigma_3)$$
$$\sigma_m = \frac{1}{3}\{(1+N)\sigma_1+(2-N)\sigma_3\} \tag{36}$$

and therefore stress increments are also given as:

$$\Delta\tau_{oct} = \frac{2\sqrt{2}}{3}\frac{q}{\pi}\sqrt{1-N+N^2}\sin 2\varepsilon$$
$$\Delta\sigma_m = \frac{q}{3\pi}\{6\varepsilon+(N-1)\sin 2\varepsilon\} \tag{37}$$

Karube and Harada (1967) showed that the value of N changed from about 0.2 to 0.4 during the plane strain shear of a clay. If the change of total stress state during consolidation is neglected, the approximate value of N may be chosen as 0.3 and equations (37) gives the total stress immediately after the loading and after the dissipation of the excess pore pressure.

In spite of the fact that equation (34) is based on the assumption that the ground is an isotropic homogeneous elastic material, the ground is assumed to be normally consolidated soft clay as shown in figure 5. The soil constants λ and μ are 0.2522 and 0.1203, respectively. The value of K_0 is 0.5 and therefore the value of k_0 is 0.35. Substituting $f = f_0$ into equation (33)

$$\frac{\lambda}{f_0}\ln\frac{\sigma'_m}{\sigma'_{mo}} + \mu\left(\frac{\tau_{oct}}{\sigma'_m} - k_0\right) = 0 \tag{38}$$

and then representing

$$\tau_{oct} = \tau_{oct_0} + \Delta\tau_{oct} \tag{39}$$

where τ_{oct0} is the octahedral shear stress before loading, the effective mean stress σ'_m immediately after loading is calculated for each point in the ground and therefore the pore

σ'_{10}	σ'_{30}	σ'_{m0}	τ_{oct0}	e_0
3.0	1.5	2.0	0.7	2.02
5.0	2.5	3.3	1.2	1.89
7.0	3.5	4.7	1.6	1.80
9.0	4.5	6.0	2.1	1.73

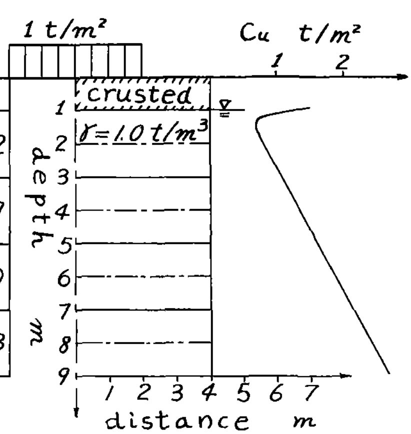

FIGURE 5. *Example for the settlement calculation*

pressure increment Δu is derived from

$$\Delta u = \sigma'_{m_0} + \Delta \sigma_m - \sigma'_m \tag{40}$$

When Δu is dissipated, the stress state in the ground is given by

$$\begin{aligned} &\tau_{oct} = \tau_{octo} + \Delta_{\tau oct} \\ &\sigma'_m + \Delta \sigma_m \end{aligned} \tag{41}$$

and substituting equations (41) into equation (33), the volumetric strain is calculated; then the settlement is given as the summation of the volume change of each layer as shown in figure 6. It should be noted that the stress ratio τ_{oct}/σ'_m cannot exceed the stress ratio at the critical state.

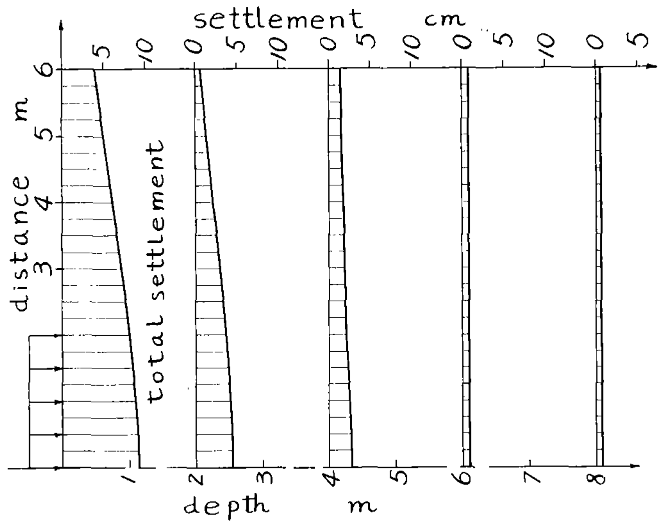

FIGURE 6. *Calculated settlement*

ACKNOWLEDGEMENTS

The authors are indebted to Professor T. Shibata for his helpful criticisms and suggestions.

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WATER PERMEABILITY AND PLASTIC INDEX OF SOILS

Yoshichika NISHIDA and Seishi NAKAGAWA

ABSTRACT

The coefficient of permeability of water in clay, on which the rate of consolidation settlement is dependent, can be approximately asked from its void ratio and plastic index through a simple formula.

RÉSUMÉ

Le coefficient de perméabilité de l'eau dans l'argile dont est dépendante la vitesse de tassement de consolidation peut être approximativement recherché par son indice des vides et son indice de plasticité par une formule simple.

INTRODUCTION

Land subsidence often is caused by the consolidation of the ground, which is due to water emerging from the soil under a load. Thus, the flow of water in the soil must have much influence on the settlement of the land surface, and it can be estimated by the coefficient of permeability of the soil. This coefficient is measured by a permeability test of the constant-head or falling-head type for the coarse-grained soils or the medium-grained soils, but such equipment is not useful for fine-grained clays. The coefficient of permeability of clay is obtained by analysis of consolidation-test results. This paper presents an experimental relationship to predict the coefficient of permeability of soils, particularly of clays, from their plasticity index, PI. The latter is one of the most simple tests, not requiring the permeability test or the consolidation test. A few comments on this idea were reported by one of the writers [1], and in this paper data on many kinds of clays are presented to confirm the relationship.

EXPERIMENT

According to many experiments, a linear relationship has already been formed between void ratio of a clay, e, and the logarithmic value of its coefficient of permeability, $\log_{10} k$, as follows:

$$e = \alpha + \beta \log_{10} k \tag{1}$$

where α and β are constants, depending on the kinds of clays, and k is in cm per sec.

Normal consolidation tests were carried out on each kind of clays and α, β and k in equation (1) were obtained by analysis of the test data according to Terzaghi's consolidation theory. Consistency tests also were carried out on many kinds of clays. Data were obtained from samples of undisturbed clays, of remolded clays, and of cation-exchanged clays soaked in different solutions. Some clays, air-dried at 20 °C and passing the 2-mm sieve, were soaked in the liquid acetete salts (Na, H, Ca, Mg, Al,) for 30 hours in order to exchange cations. The cation-exchanged clays were washed out with 80 percent methyl alcohol and air-dried. As cations in a clay are exchanged, the liquid phase around the clay particles makes a change in its characteristics and the thickness of the absorbed water. Then, the apparent size of particles is transformed and their consistency is altered.

A bentonite clay, which has high activity, was soaked in the electrolytes with different normalities in order to change the mutual forces acting between clay particles.

When clays were oven-dried at 11 °C, their consistency were different from the ones air-dried at 20 °C probably due to the change in apparent size of particles due to flocking.

TEST RESULTS

Figure 1 is the plasticity chart for the consistency tests on the clay samples used for the experiments. Table 1 shows the results of consolidation tests and consistency tests, to determine α, β, and the plasticity index. The data show that the value of α is nearly equal

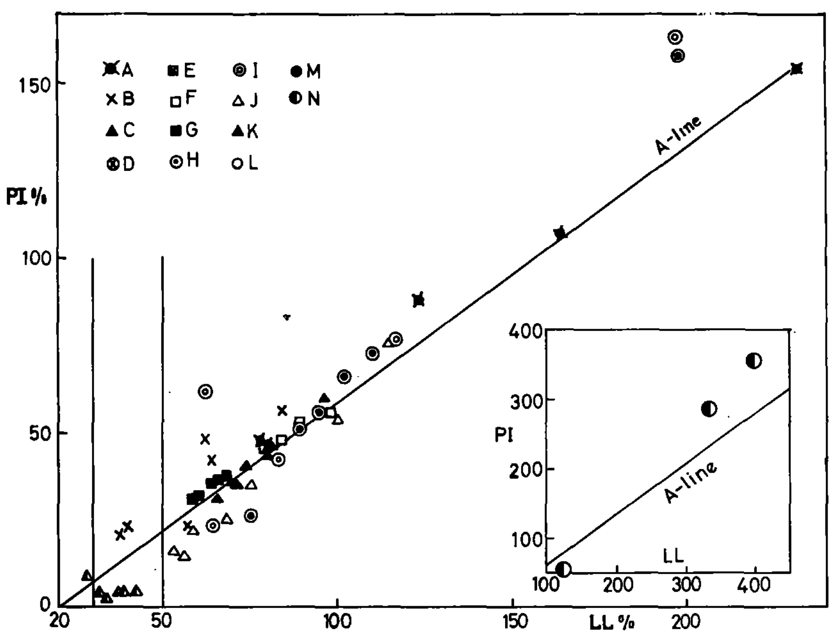


FIGURE 1. *Plasticity chart*

TABLE 1. *PI*, α and β of sample

Sample	*PI*	α	β	α/β	
A	46.9	6.2	0.6	10.3	undisturbed
	47.5	4.75	0.5	9.5	
	87.8	9.1[illegible]	0.88	10.4	
	106.8	15.0	1.7	8.8	
	154.7	13.4	1.5	8.9	
	223.1	20.1	2.1	9.6	
B	23.0	3.05	0.30	10.2	undisturbed
	32.4	3.95	0.40	9.9	
	41.5	4.84	0.48	10.1	
	47.5	3.85	0.45	8.6	
	58.4	3.10	0.25	12.4	
	20.7	2.74	0.28	9.8	

C	30.8	2.79	0.24	11.4	undisturbed
	35.3	3.64	0.38	9.6	
	36.4	3.76	0.38	9.9	
	40.0	3.54	0.33	10.7	
	43.0	3.65	0.36	10.1	
	48.9	2.52	0.22	11.5	
	60.3	4.49	0.48	9.4	
D	11.0	2.0	0.18	11.4	(Ref. 1) undisturbed vertical dir. for permeability
	14.0	2.3	0.23	10.2	
	18.0	2.5	0.25	10.0	
	21.0	2.8	0.26	10.8	
	22.0	3.0	0.27	11.1	
	23.0	2.9	0.26	11.2	
	24.0	3.1	0.30	10.3	
	30.0	3.7	0.36	10.3	
	35.0	4.0	0.39	10.3	
E	12.0	2.5	0.25	10.0	(Ref. 1) undisturbed horizontal dir. for permeability
	14.0	2.6	0.26	10.0	
	17.0	2.9	0.29	10.0	
	21.0	3.2	0.31	10.3	
	21.0	3.4	0.33	10.3	
	23.0	3.4	0.33	10.3	
	23.0	3.6	0.36	10.0	
F	55.8	6.41	0.58	11.1	Na } cation exchanged
	52.3	4.07	0.38	12.3	H
	47.9	4.80	0.41	11.7	Mg
	45.0	4.49	0.38	11.8	Ca
G	36.9	5.86	0.52	11.3	Na } cation exchanged
	35.4	4.27	0.35	12.2	H
	35.2	4.73	0.40	11.8	Mg
	31.7	4.14	0.33	12.6	Ca
	30.7	4.35	0.35	12.4	Al
H	158.0	13.8	1.33	10.4	Na } cation exchanged
	56.0	6.10	0.59	10.3	H
	65.9	6.57	0.69	9.5	Ca
	73.1	7.32	0.73	10.0	Mg
	26.0	4.08	0.35	11.7	Al
	50.8	4.86	0.44	11.1	original sample disturbed
I	42.8	7.30	0.75	9.7	H } dried in oven after cation exchanged
	22.5	4.35	0.44	9.9	Ca
	163.7	7.26	0.71	10.2	Mg
	–	3.76	0.35	10.7	Al
J	2.1	2.34	0.23	10.2	(Ref. 1) dried in oven
	4.1	2.35	0.25	9.4	
	4.5	2.58	0.22	11.7	
	4.9	2.27	0.21	10.8	
	5.2	2.53	0.22	11.5	
	8.6	2.78	0.32	8.7	

K	13.6	3.54	0.30	11.8	
	15.6	2.87	0.28	10.2	
	21.6	3.93	0.38	10.3	(Ref. 1)
	25.0	4.10	0.40	10.2	dried in oven
	34.9	4.90	0.49	10.0	
	76.2	6.23	0.36	11.1	
L	1.0	(10 β)	0.06	(10.0)	
	2.0	(10 β)	0.06	(10.0)	
	10.0	(10 β)	0.08	(10.0)	(Ref. 2)
	13.0	(10 β)	0.14	(10.0)	at denser state
	18.0	(10 β)	0.15	(10.0)	
	18.0	(10 β)	0.28	(10.0)	
	37.0	(10 β)	0.32	(10.0)	
M	1.0	(10 β)	0.27	(10.0)	
	2.0	(10 β)	0.06	(10.0)	
	10.0	(10 β)	0.16	(10.0)	(Ref. 2)
	13.0	(10 β)	0.17	(10.0)	at looser state
	18.0	(10 β)	0.25	(10.0)	
	18.0	(10 β)	0.31	(10.0)	
	37.0	(10 β)	0.50	(10.0)	
N	57.4	5.93	0.51	11.6	1 N } normality of
	28.6	15.04	1.36	11.0	0.1 N } NaCl
	35.3	28.1	2.58	10.9	0.01 N } Bentonite

to 10 times the value of β, and the coefficient β in equation [1] seems to be linear with the plasticity index. Figure 2 shows that the linear relationship between β and *PI* holds for each kind of clay. Those lines representing the relationships are parallel to each other, with a slope of about 0.01, and may be expressed for almost all kinds of clays as follows:

$$\beta = 0.01(PI)+\gamma \tag{2}$$

where γ is a constant depending on the kind of clay, which takes the value of -0.01 for a bentonite clay and of 0.3 for an oven-dried clay. Although a fine-grained clay of high consistency seems to have a smaller value (the negative value in some cases) for γ, while a coarse grained clay of low consistency has a larger value for γ, this conclusion about γ is not definitive and further studies should be made. Figure 3 shows the relationship for clays of much higher consistency, including those data in figure 2. It can be found in figure 3 that the average value of γ in equation [2] is 0.05.

If the void ratio of a clay and its plasticity index are known, the coefficient of permeability of the clay, k, in cm per sec, can be approximated by equation [1] and equation [2] using the values of 0.01 for β, 0.05 for γ and $\alpha \doteqdot 10$, because these values obtained from test results on many kinds of clays, covering an extensive range of plasticity index – from 2 percent to 350 percent.

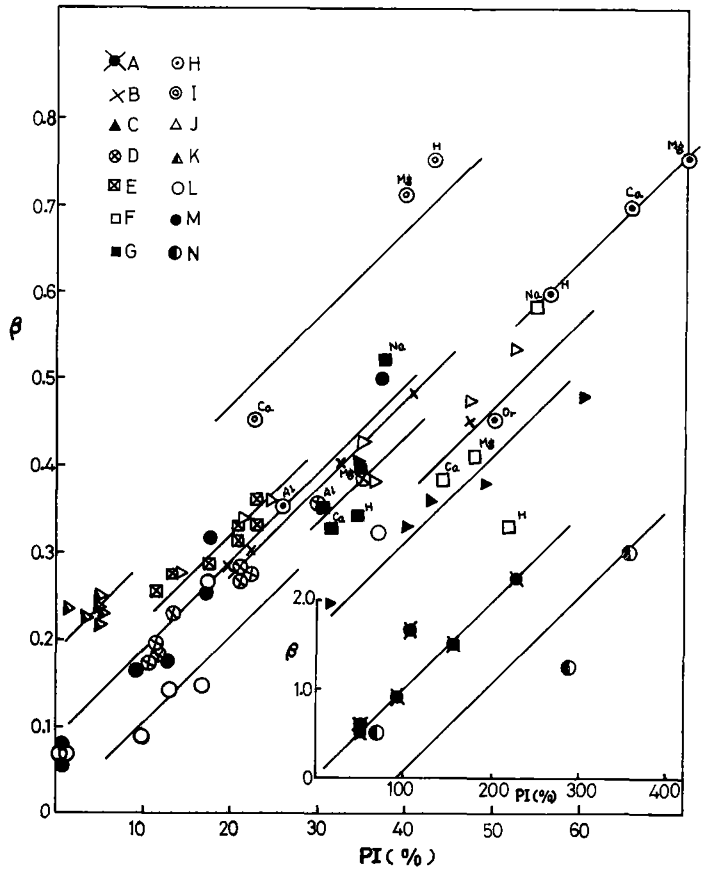


FIGURE 2. *β and PI in detail*

CONCLUSIONS

The following relationship is useful for prediction of the coefficient of permeability of water in clay.

$$e = \{0.01(PI)+0.05\}(10+\log_{10}k) \tag{3}$$

where

- e void ratio of clay;
- PI plasticity index of clay, in percent;
- k coefficient of permeability of clay, in cm per sec.

ACKNOWLEDGEMENT

The writers thank Mr. Hiroshi Koike who helped to carry out the experimental works.

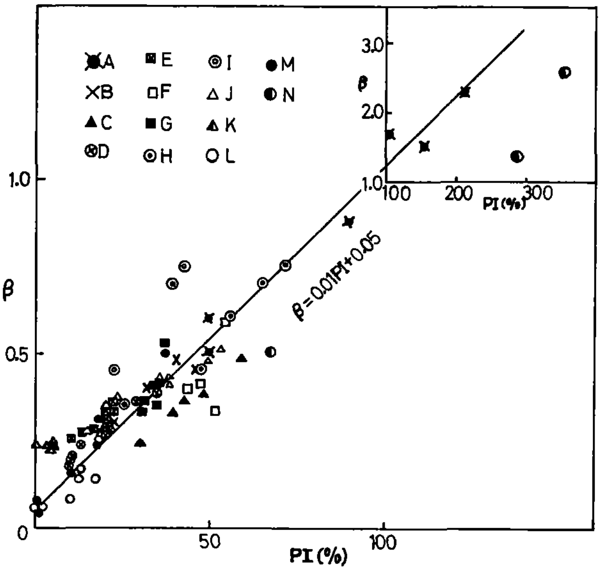


FIGURE 3. *β and PI*

DISCUSSION

Intervention by Mr. Arnold I. JOHNSON (USA):

Question:

I wondered if you ran tests of permeability on undisturbed cores of natural materials from the field or only on remolded or disturbed samples? If not, perhaps this is the reason you obtained as good a relationship as you indicated in your paper.

Answer of Prof. NIAHIDA:

We ran consolidation tests on undisturbed cores as well as on some remolded ones. Gathering up all data available, we are now carrying out a theoretical study: What I presented today are the experimental data. But I will say that en approximate relationship, linear relationship may be found between the plastic index by a very simple test.

RELATIONSHIP OF CONSOLIDATION CHARACTERISTICS AND ATTERBERG LIMITS FOR SUBSIDING SEDIMENTS IN CENTRAL CALIFORNIA, U.S.A.

A. I. JOHNSON and R. P. MOSTON[1]

ABSTRACT

As one phase of research on land subsidence, laboratory analyses were made on many undisturbed cores obtained from sediments in subsiding areas of Central California. In 1948 Terzaghi and Peck had presented equations relating compression index or coefficient of consolidation to liquid limit, but present research indicates that the same relationships do not hold for any of the sediments tested from Central California.

The compression index could be estimated from liquid-limit data, but the relationship was different for each area of subsidence. Comparison of compression indices obtained from consolidation curves with indices computed from liquid limits showed better correlation for sediments of alluvial and lacustrine origin than for sediments of marine origin. Equations for the relationships were obtained by computer solution of data trends.

In all three areas of subsidence, the coefficient of consolidation showed a general decrease for increasing values of liquid limit. However, the relationship could not be estimated with reasonable accuracy because the coefficient for any particular load range could vary through more than one order of magnitude for any given liquid limit.

RÉSUMÉ

Au cours d'une des phases de la recherche sur les affaissements, des essais de laboratoire ont été faits sur des échantillons non perturbés de sédiments dans la zone d'affaissement de la Californie Centrale. En 1948, Terzaghi et Peck ont présenté des équations reliant l'indice de compression ou coefficient de consolidation à la limite de liquidité mais les recherches actuelles montrent que la relation en question n'est applicable à aucun des échantillons de sédiments essayés, en provenance de la Californie Centrale.

L'indice de compression peut être estimé à partir de données sur la limite de liquidité, mais la relation diffère pour chaque zone d'affaissement. La comparaison des indices de compression obtenus par les courbes de consolidation avec les indices évalués d'après les limites de liquidité a donné une meilleure corrélation pour des sédiments d'origine alluviale ou lacustre que pour les sédiments d'origine marine. Les équations pour les relations ont été obtenues en utilisant des ordinateurs.

Dans les trois zones d'affaissements, le coefficient de consolidation présente une diminution générale pour des valeurs décroissantes de la limite de liquidité. Toutefois, la relation ne peut pas être estimée avec une précision raisonnable car le coefficient varie pour chaque charge particulière de plus d'un ordre de grandeur pour chaque limite de liquidité.

INTRODUCTION

This paper illustrates some of the relationships between consolidation characteristics and liquid limits for fine-textured sediments from areas of land subsidence in central California As one phase of research on land subsidence (Inter-Agency Committee on Land Subsidence in the San Joaquin Valley, 1958; Poland, 1960; Poland and Green, 1962), the sediments were cored to depths as great as 2,180 feet and selected core samples were tested in the US Geological Survey's Hydrologic Laboratory and the US Bureau of Reclamation's Earth Laboratory—both at Denver, Colo. The physical, hydrologic, and engineering properties of these samples were presented and discussed in a report by Johnson, Moston, and Morris (1968).

1. Staff Hydrologist, Water Resources Division, US Geological Survey, Denver, Colo.
2. Formerly with US Geological Survey, now Field Engineer, State Highway Department, Grand Junction, Colo.

Consolidation tests and the liquid limits were made by the Earth Laboratory on 70 representative samples from 6 core holes. Figure 1 shows the three major areas of subsidence in which these core holes are located—the Los Banos-Kettleman City and the Tulare-Wasco areas of the San Joaquin Valley, and the Santa Clara Valley. Papers by Poland (1969), Lofgren (1969), and Riley (1969), present details on the geology, hydrology, subsidence characteristics of these areas.

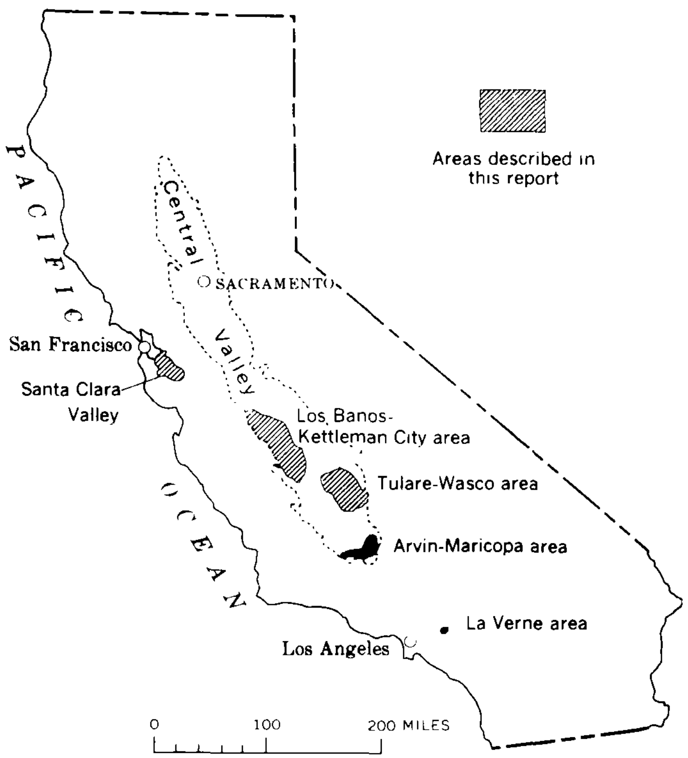


FIGURE 1. *Principal areas of land subsidence in California, due to ground-water withdrawal*

The liquid limit (w_l) represents the moisture content at which a fine-textured sediment ceases to behave as viscous liquid and begins to behave plastically. For the samples discussed in this paper, liquid limits were obtained using the standard methods of the American Society for Testing and Materials (1964, pp. 109-113).

The coefficient of consolidation (c_v) represents the rate of consolidation under a load increment and is computed from consolidation test data. The compression index (C_c) is a measure of the volume decrease due to increase in load; it is defined as the slope of the straight-line portion of the semi-logarithmic void ratio-load plot, when load is plotted on the logarithmic scale. Consolidation data for this study were obtained by means of a laterally confined compression test using a l-dimensional consolidometer (US Bur. Reclamation, 1960, pp. 57-59, 492-507). Loads were applied to the specimen in increments—the number of increments ranging from six to eight—with increments selected so that each succeeding load was double the previous load. The results of the tests were plotted as a semi-logarithmic graph, an example of which is shown in figure 2. Similar graphs for the other core holes may be found in the 1968 report by Johnson, Moston, and Morris.

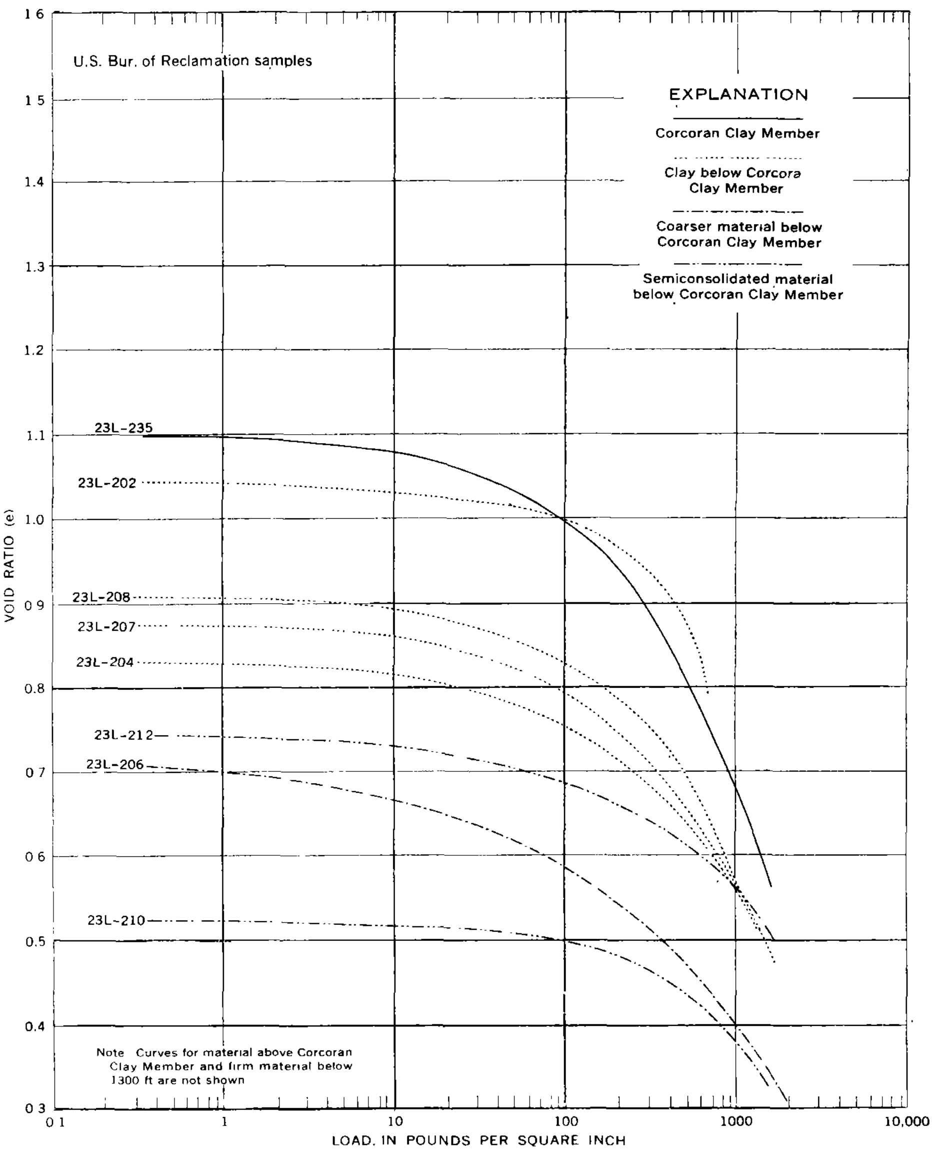


FIGURE 2. *Void ratio-load curves for selected samples from core hole 16/15-34 N1 in the Los Banos-Kettleman City area, California*

Because the amount of subsidence is considered to be related to the compressibility of the sediments, for a given stress increase, the compression index and coefficient of consolidation are important to subsidence studies. Several researchers have tried to relate these compressibility characteristics to the liquid-limit test because the latter does not require an undisturbed sample and requires much less time and equipment than the consolidation test. The study reported in this paper was initiated because use of the liquid-limit test would be less expensive than the consolidation test when used to obtain a reasonably close approximation of the compression index and the coefficient of consolidation.

ESTIMATING COMPRESSION INDEX

Terzaghi and Peck (1948, p. 66), in a continuation of work begun by Skempton (1944, p. 126), state that the compression indices for clays in a remolded state (C_c') increase consistently with increasing liquid limit limit (w_L). Using data from approximately 30 samples selected at random from different parts of the world and representing both ordinary and extra-sensitive clays, Terzaghi and Peck (1948) state that the data on compression indices and liquid limits for these clays plot on a graph within ± 30 percent of a line representing the equation

$$C_c' = 0.007\,(w_L - 10 \text{ percent}). \tag{1}$$

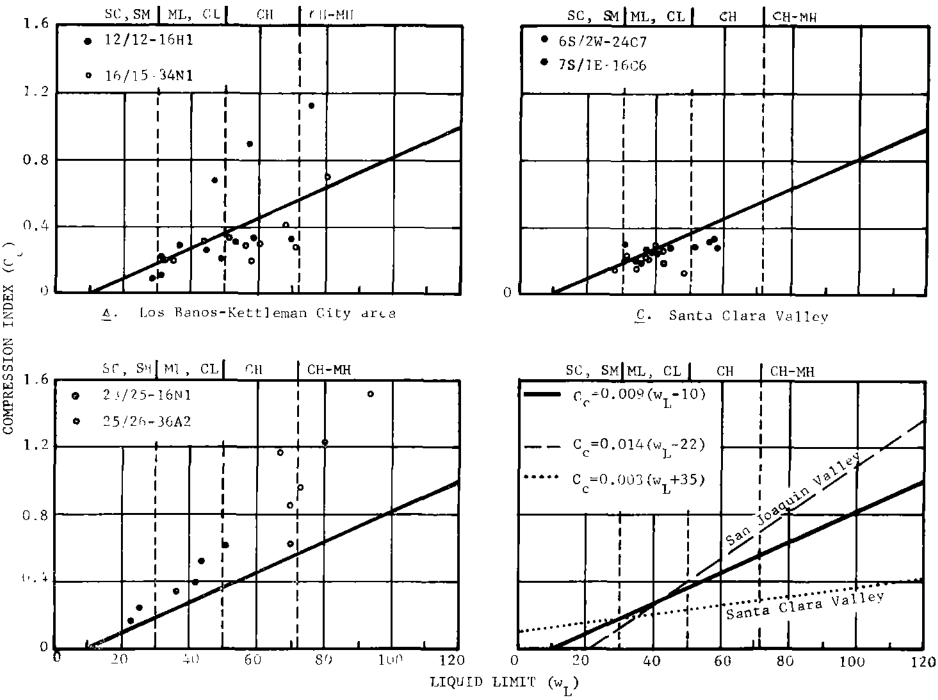
They state further that for an ordinary clay of medium or low sensitivity tested in the undisturbed state, the value of C_c corresponding to field consolidation is approximately equal to 1.30 C_c'; thus,

$$C_c = 0.009\,(w_L - 10 \text{ percent}). \tag{2}$$

Hence, these authors conclude that for normally loaded clays with low or moderate sensitivity, the compression index,.C_c, can be estimated approximately from knowledge of the liquid limit and use of equation (2). However, Terzaghi and Peck do caution that this approximate method of computation may furnish merely a lower limiting value for the compression index of an extrasensitive clay. Later papers by Nishida (1956), and Roberts and Darragh (1963), showed exceptions to the compression index-liquid limit relationships described by Terzaghi and Peck (1948) and indicated that there was a wide scattering of data. Furthermore, they found no simple correlation between these factors for the sample data they studied.

TABLE 1. Equations for egression lines for various groups of data from subsiding areas in central California

Data used	Equation
Los Banos-Kettleman City area	
Core hole 12/12-16H1, exclusive of the 3 samples with exceptionally high compression indices	$C_c = 0.005\,(w_L + 6)$
Core hole 16/15-34N1	$C_c = 0.007\,(w_L - 12)$
All amples in area, exclusive of 3 samples with exceptionally high compression indices	$C_c = 0.006\,(w_L - 3)$
Tulare-Wasco area	
Core hole 23/25-16N1	$C_c = 0.015\,(w_L - 11)$
Core hole 24/26-36A2	$C_c = 0.024\,(w_L - 32)$
All samples in area	$C_c = 0.018\,(w_L - 16)$
San Joaquin Valley, exclusive of 3 samples with exceptionally high compression indices	$C_c = 0.014\,(w_L - 22)$
Santa Clara Valley	
Core hole 6S/2W-24C7	$C_c = 0.003\,(w_L - 47)$
Core hole 7S/1E-16C6	$C_c = 0.0005\,(w_L + 370)$
All samples in area	$C_c = 0.003\,(w_L + 35)$



B. Tulare-Wasca area

D. Comparison of Terzaghi and Peck equation with equations calculated bycomputer for all samples shown in A. B and C

FIGURE 3. *Relation between liquid limit and compression index forselected sampels from core holes in San Joaquin and Santa Clara Valleys, California*

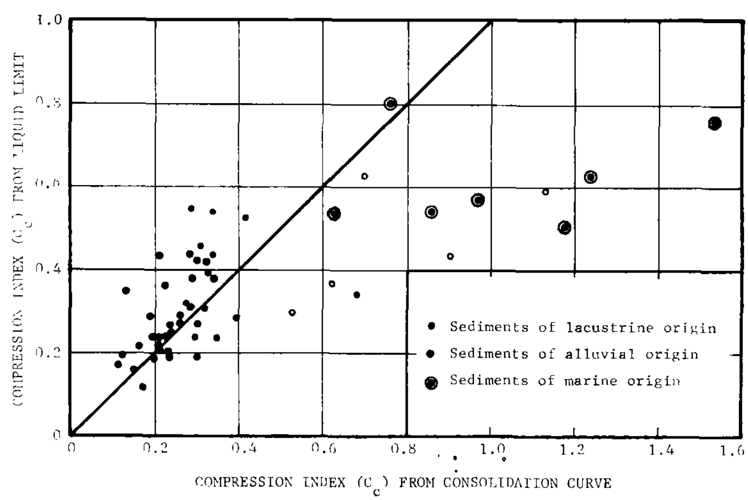
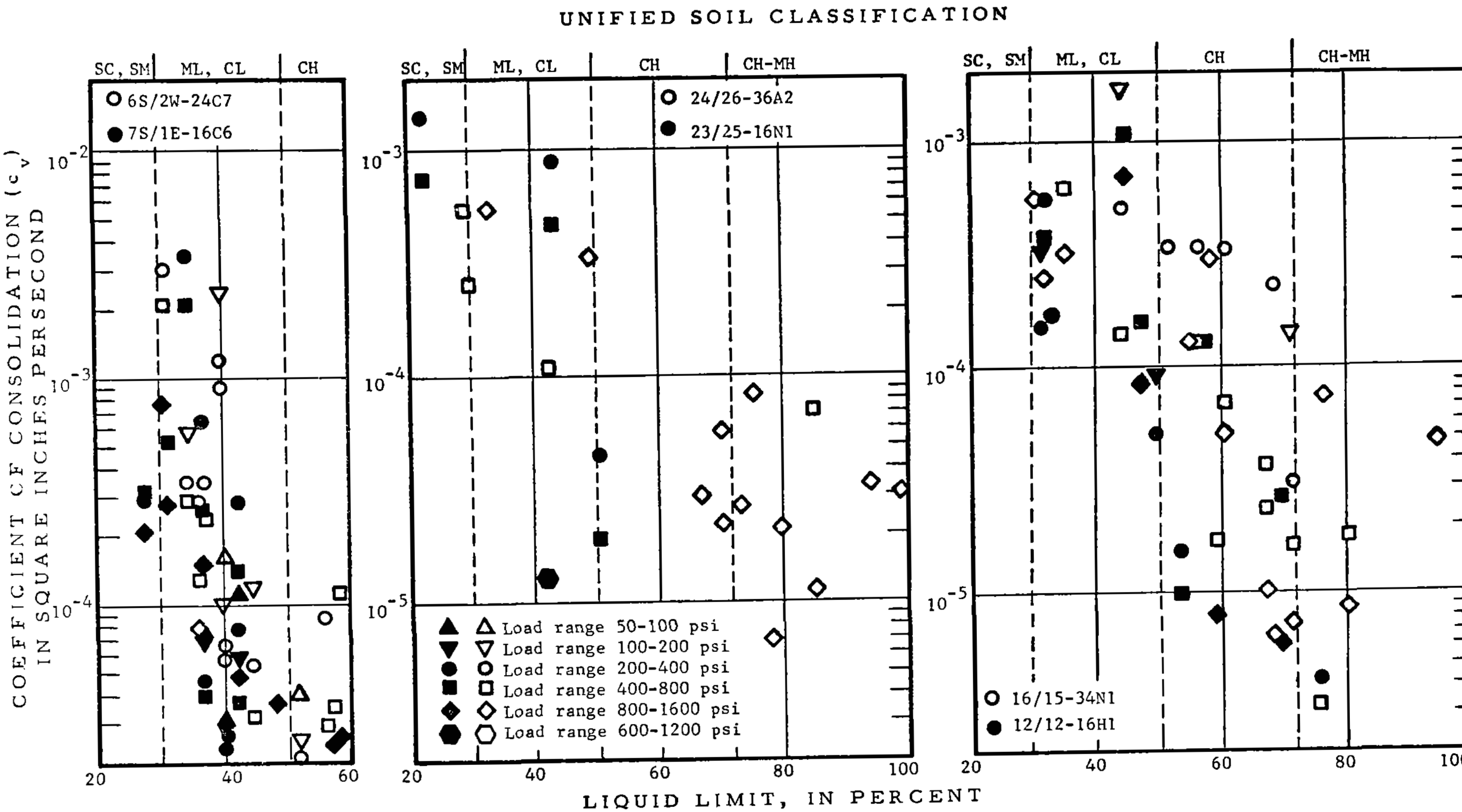


FIGURE 4. *Comparison of two methods for determination of compression index for all samples from subsidence areas in San Joaquin and Santa Clara Valleys, California*



A. Santa Clara Valley

B. Tulare-Wasco Area

C. Los Banos-Kettleman City Area

FIGURE 5. *Relation of coefficient of consolidation to liquid limit for samples from core holes in the San Joaquin and Santa Clara Valleys, California*

Figure 3 shows the relationship between liquid limit and compression index for core samples from the six test holes in areas of subsidence in the San Joaquin and Santa Clara Valleys of California. Although the liquid limit is calculated as moisture content in percent of dry weight, values usually are reported as numbers only. Thus, the values are reported only as numbers in figure 3 and henceforth in this paper. The compression indices used in these graphs were obtained by the Bureau of Reclamation's Earth Laboratory from consolidation tests, not by calculation from the Terzaghi and Peck equation. The solid line in each of the parts of figure 3 represents the regression line for the Terzaghi and Peck equation, $C_c = 0.009\,(w_L - 10)$.

The data in figure 3 show that 10 of the 22 samples from the Los Banos-Kettleman City area 11 of the 12 samples from the Tulare-Wasco area, and 4 of the 21 samples from the Santa Clara Valley, lie outside the $\pm$ 30-percent limits of scatter about the regression line for the Terzaghi and Peck equation, $C_c = 0.009\,(w_L - 10)$. Three samples of clay in the Los Banos-Kettleman City area have compression indices approximately twice as large as would be predicted from the Terzaghi-Peck equation. The void ratio-load curves for these three samples suggest that they are extrasensitive clays and, if so, they would be expected to plot well above the equation line. However, even if these samples are excluded, the data of figure 3 show that the relationship between liquid limit and compression index for fine-textured sediments on the west side of the San Joachim Valley does not fit the Terzaghi-Peck equation as closely as might be expected from the discussion by those authors (Terzaghi and Peck, 1948, p. 66).

Regression lines were determined by computer for the liquid limit-compression index relationship for samples from core holes in the San Joaquin and Santa Clara Valleys. Table 1 presents the equations of the regression lines for data from the San Joaquin and Santa Clara Valleys so they can be compared with the regression line for the Terzaghi and Peck equations, $C_c = 0.007\,(w_L - 10)$ and $C_c = 0.009\,(w_L - 10)$. The table shows that only the equation for core hole 16/15-34N1 is approximately equivalent to either equation discussed by Terzaghi and Peck. Figure 3, part *D*, shows that the equation of the regression line for all samples from the San Joaquin Valley (except the three samples with the exceptionally high compression indices) is $C_c = 0.014\,(w_L - 22)$
and the equation for the Santa Clara Valley is $C_c = 0.003\,(w_L + 35)$

CORRELATION OF COMPRESSION INDICES

Figure 4 demonstrates the correlation between compression indices estimated from liquid-limit tests and those determined from consolidation curves such as are shown in figure 2. In figure 4, the heavy line passing through the origin at an angle of 45 degrees to the x and y axes represents absolute correlation between the values represented by the two axes. The compression indices estimated from liquid limits for the Los Banos-Kettleman City area and Santa Clara Valley generally are higher than those determined from consolidation curves and lower for the Tulare-Wasco area.

The data in figure 4 also show that the sediments of marine origin have much higher compression indices when determined from consolidation curves than when estimated from liquid limits. Furthermore, sediments of lacustrine origin have somewhat higher compression indices, whereas sediments of alluvial origin have somewhat lower compression indices when determined from consolidation curves. Again, the explanation may be due to the difference in load conditions, the marine sediments being the deepest and the alluvial sediments being the shallowest.

ESTIMATING COEFFICIENTS OF CONSOLIDATION

Figure 5 shows the computed coefficient of consolidation for 1 to 4 different load ranges plotted against liquid limit for samples from the three subsidence areas. Although the coefficient of consolidation shows a general decrease for increasing values of liquid limit, figure 5 indicates that the coefficient of consolidation for any particular load range can vary through more than one order of magnitude for any given liquid limit. Terzaghi and Peck (1948, pp. 76-77) described a similar relationship for data from about 30 samples and noted that the relationship varied within a wide range. The data in figure 5 also show that the relationship is different for each core hole as well as for each area.

EFFECT OF SOIL CLASSIFICATION

Information on figures 3 and 5 indicates the effect of particle size and texture on the consolidation characteristics and the liquid limit. The Unified Soil Classification (Am. Soc. Testing Materials, 1964, pp. 208-220) designation, which is based on texture, is indicated at the top of the two figures.

In general, those samples with a classification of *CH-MH* have the largest liquid limits and compression indices, and the smallest coefficients of consolidation. Samples with a classification of *SC* and *SM* have the smallest liquid limits and compression indices, and the largest coefficient of consolidation. Samples with a classification of *ML*, *CL* and *CH* have values somewhere between these two extremes. Samples of sediments of alluvial origin tended to be classified as *SC*, *SM*, *ML*, or *CL*; those sediments of lacustrine origin tended to be classified as *CL* and *CH*; and those of marine origin were classified primarily as *CH-MH*.

SUMMARY

Data presented in this paper show that the equations presented by Terzaghi and Peck (1948) (equations 1 and 2) do not apply to the relationship between compression index and liquid limit for sedimentary deposits tested from subsidence areas in the San Joaquin or Santa Clara Valleys. Furthermore, the data show that no single equation applies to the relationship for all areas studied, with the following equations being obtained for the two valleys:

San Joaquin Valley—$C_c = 0.014\ (w_L - 22)$
Santa Clara Valley—$C_c = 0.003\ (w_L + 35)$.

In essentially every case, the equations of the regression lines represent only general trends because there is considerable scatter of data for all core holes. The trend line for data from the Santa Clara Valley is so nearly horizontal that a rather narrow range of compression indices could be obtained over a wide range of liquid limits. Compression indices estimated from liquid limits, however, showed better correlation with indices determined from consolidation curves when the sediments were of alluvial or lacustrine origin than when they were of marine origin.

All coefficients of consolidation showed a general decrease for increasing values of liquid limit. However, because the coefficients for any particular load tange could vary through more than one order of magnitude for any given liquid limit, the relationship could not be estimated with reasonable accuracy. In fact, the general trend for the relationship even varies for each subsidence area and for each core hole.

For the areas studied in central California, the consolidation characteristics of the undisturbed sediments in the field cannot be closely approximated by liquid limits, which are made on disturbed samples of those sediments. The studies also indicate that the equations reported by Terzaghi and Peck (1948) must be used with extreme caution to

estimate the consolidation characteristics of sediments in areas of subsidence—especially if the compaction sediments are at considerable depth.

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DISCUSSION

Intervention of Dr. Seishi NAKAGAWA (Japan)

Question:

You said the coefficient in the equation representing Cc from the liquid limit is a variable depending on the kinds of clays.

I have information that the compression index is represented by void ratio with a simple linear equation available for any clay with the same coefficient. I think we had better express compression index by void ratio rather than liquid limit, as Dr. Y Nishida reported in 1957.

We have the relationship $Cc = 0.58e$ theoretically now.

Answer of Mr. JOHNSON:

I agree that void ratio may be a better property by which to estimate the compression index because it takes into account the natural structure of the soils. However, in my paper I am not advocating the use of liquid limit in preference to void ratio for estimating compression index. I am merely showing that the relationships originally proposed by Terzaghi and Peck did not hold for the sediments we studied in California.

ANALYTICAL METHODS FOR PREDICTING SUBSIDENCE

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ABSTRACT

The paper discusses the need for predictive methods in subsidence problems. The importance of correctly formulating the problem is emphasized. The various steps in formulating a problem are discussed. The finite element technique is considered to be the most useful technique for application to subsidence problems. The results of the analysis of an example problem using this technique are presented.

RÉSUMÉ

L'auteur discute la nécessité de méthodes de prédiction pour des problèmes de l'affaissement des sols. L'accent est mis sur l'importance d'une formulation correcte du problème. Les diverses étapes dans la formulation des problèmes sont discutées. L'auteur estime que la méthode des " éléments finis " est la plus utile pour analyser les problèmes de subsidence. Il présente des résultats de l'analyse d'un problème type utilisant cette méthode.

INTRODUCTION

Subsidence of the ground surface can occur for a variety of reasons. Removal of fluids (e.g. oil, water) from the ground and the creation of undergound openings can cause the ground surface to subside. This paper deals solely with the problem of subsidence which is a consequence of the creation of underground openings.

These underground openings are constructed during the course of mining operations using conventional or solution techniques. In the U. S. A., until recently, the great majority of the mined areas were located away from the population centers. However, the expansion in the populated areas, together with the limited number of suitable mining sites, has resulted in the location of underground openings under buildings and other structures. Consequently, a great deal more attention is being given to the prediction and control of subsidence above underground openings.

There are two general approaches to the problem of predicting subsidence, (*a*) experimental (*b*) analytical. The major disadvantage of the purely observational (experimental) approach is that it is not applicable to conditions outside the range of experimentation. A typical example of this approach has been developed in the U.K. [1]. The major advantage of the analytical methods is that they are predictive. However, it should be recognized that all analytical methods are based on a number of assumptions. These assumptions must be evaluated within the context of the real problem. Analytical methods based on the principles of solid mechanics and placed within the framework of a good understanding of the physical nature of the problem have proved to be the most useful means for developing predictive techniques for engineering problems. The prediction of subsidence using such methods requires the formulation and solution of appropriate boundary value problems. Solution techniques are developed in general terms by workers in solid and structural mechanics. The formulation of the problem and the application of these solution techniques fall within the scope of the engineers directly concerned with the subsidence problem.

FORMULATION OF PROBLEM AND METHODS OF SOLUTION

In order to formulate a boundary value problem for analysis it is necessary to go through the following steps:
(1) define the geometry of the problem;
(2) define the materials by suitable constitutive equations;
(3) define the loads, and
(4) define the boundary conditions.

The major variables that have to be included in formulating a problem for investigating subsidence are:
(I) location of the underground opening;
(II) shape of the opening;
(III) the rock profile and geologic conditions;
(IV) the properties of the rock mass surrounding the opening, and
(V) the initial (insitu) state of stress in the rock.

In general it is necessary to use various idealizing assumptions to reduce the real problem to an idealized boundary value problem which is amenable to solution. The more powerful the solution techniques the fewer the assumptions that have to be made in the formulation of the problem. There are no solution techniques of sufficient versatility which can solve boundary value problems formulated to include all the above listed variables in complete generality. There are two general techniques for obtaining solutions to pertinent boundary value problems for investigating subsidence: (1) classical methods which often result in closed form solutions and (2) numerical methods. The majority of the previous work on subsidence [2, 3, 4, 5] has utilized results obtained by using classical mathematical methods for obtaining solutions to boundary value problems in linear elasticity [6, 7, 8, 9]. The inability of these existing solutions to predict subsidence has been pointed out by Voight and Pariseau [10].

The most powerful of the numerical techniques is the finite element method which is a modern computer oriented approach to the analysis of continuous structures. The theoretical basis and the utilization of the method for a wide class of boundary value problems has been discussed extensively in the literature [11, 12, 13]. Because of its versatility, the method is ideally suited for problems in rock mechanics [14, 15, 16]. However, the application of the technique to the solution of subsidence problems has been very limited. This section discusses the steps in formulating the problem, the major variables that have to be included in the formulation, and the capability of the finite element method to account for these variables.

Problem Geometry

The location of the cavity, its shape, and the rock profile are factors which have to be considere in defining the geometry of the problem. While there is generally little ambiguity with regard to the location of the opening it is often necessary make various assumptions with regard to shape. All real shapes are three dimensional; existing methods of analysis can only account for plane, and axisymmetric three dimensional shapes. Many underground openings are in bedded deposits. From the standpoint of analysis, this implies that the material around the opening is non-homogeneous. Another factor to be considered is the continuity betweenlayers in bedded deposits. In addition to bedding planes the occurrence of faults and joints can be very influential in causing subsidence. The mapping of these and their representation is an important aspect in formulating the problem. With the finite element method, any location of the cavity can be included in the problem. Arbitary plane or axisymmetric shapes can be investigated. With regard to rock profile, in the plane analyses any system of bedding can be analyzed, however, in the axisymmetric analyses only horizontal bedding can be investigated. The ability to analyse layered deposits is a very significant advance. In addition, there are various techniques which can be used to analyse joints and conditions of continuity between layers of rock [16, 17].

Material Properties

In subsidence problems the properties of the rock mass surrounding the opening have to be defined. It is necessary to select constitutive equations to represent the various materials. The numerical magnitude of various parameters which are necessary to quantify the selected constitutive equations have to be determined. It is well known that rocks in general are not isotropic and because of their different behavior in tension and compression are non linear. In addition, yielding (e.g. elasto-plastic) and time dependent (creep) behavior may also be present. The present knowledge on constitutive equations and the ability to solve boundary value problems does not permit us to completely model the complex behavior of rock. Therefore, it becomes necessary to select more than one constitutive equation to represent the materials in order to obtain bounds on the subsidence that might occur. It is important to recognize that the results of laboratory tests cannot be assumed to represent the properties of the rock mass. Various correlative techniques to relate laboratory values with rock mass properties are available [18]. Approximate methods for including nonlinear, bilinear, elasto plastic, and time dependent material properties are currently available [13]. Furthermore, methods to include progressive failure in the rock mass which could lead to additional subsidence have also been developed [19].

Definition of Loads

The existing equilibrium state of stress often termed the "initial" or "insitu" stress is disturbed by the creation of an opening. In addition, the weight of the material (i.e., gravity load) is acting to close the opening. The initial state of stress may be entirely due to the weight of the material or in some cases stresses may exist in the rock due to tectonic history. The initial state of stress is usually defined by the vertical stress and the ratio (k) of the horizontal to vertical stress; for subsurface problems these are the principal stresses. In order to compute the gravity loads it is necessary to know the unit weight of the overlying materials. The inclusion of gravity loads in the analysis is a routine matter with the finite element method technique. Arbitary initial stresses can be easily introduced by varying the pressure applied to the external boundaries. In addition, any pressure applied to the internal face of the cavity can be readily included in the analysis.

Boundary Conditions

There are two boundaries in the problem, the ground surface and the interior face of the opening. Both these surface are stress free and have no restrictions on their displacement. In some cases it might be required to determine the surface subsidence due to a certain percentage of closure in the opening. Under these circumstances displacement boundary conditions have to be imposed on the interior surface of the cavity. In the real physical problem (i.e., a half space) the vertical boundaries and the lower horizontal boundary are at an infinite distance in horizontal direction and vertical direction respectively. For purposes of making the problem amenable to solution, it is often necessary to locate these boundaries at a finite distance from the opening. These boundaries should be located at a sufficient distance away from the opening so that their influence on the conditions at the opening is negligible and vice versa. Therefore, the conditions at these boundaries can be assumed to be those that existed prior to the creation of the opening. All the boundary conditions required by the problem can be easily accommodated. Conditions of no horizontal displacement or any practical form of pressure distribution for representing the initial stress state can be applied to the vertical boundaries. Displacement boundary conditions at the lower horizontal boundary and at the interior face of the cavity can also be incorporated into the analyses. Since the finite element technique is developed for use with a digital computer, a range of conditions e.g. material properties and initial stress states can be easily investigated. This is of considerable significance in subsidence problems where because of the uncertainty about geologic factors it is prudent to investigate various possible conditions.

It can also be seen from the above discussion that the finite element technique has capabilities of solving realistic boundary value problems as they apply to the prediction of subsidence, and provides greater capacilities for analyses than were heretofore available. Its use in analysing subsidence problems should increase greatly in the next few years.

EXAMPLE PROBLEM

The engineer evaluating the possibility of subsidence is cast in the role of a user of the finite element method. Digital computer programs based on the finite element method are available for solving appropriate boundary value problems. It is the use of these programs that the practicing engineer is concerned with. Using an existing operational program can be accomplished by following certain set instructions and requires a minimum understanding of the method. However, new and improved programs are always being developed and the engineer should try to keep informed of these developments.

The application of the method is best illustrated through the use of an example. The example chosen is relatively simple but it does serve to illustrate some of the capabilities of the method. An axisymmetric analysis is chosen for this example, as a plane analysis has been published in the literature [20].

The steps followed in formulating the problem were those discussed earlier in this paper. As applied to this specific case they are as follows.

Problem Geometry

The opening under consideration was 25 feet high and was assumed of circular cross section with a diameter 1272 feet located at a depth of 4225 feet below the surface. The rock profile consisted of surface sand and gravel overlying approximately 2000 feet of interspersed layers of sandstone, shale and siltstone. Underlying this there is limestone, dolomite, anhydrite and salt, below the salt is limestone. The opening is in the salt. Material properties had to be obtained from three dimensional velocity logs. The rock profile is idealized into seven layers as shown in figure 1.

Material Properties

It was assumed for purposes of this analysis that the materials surrounding the cavity would be treated as linear, isotropic and elastic. Material properties were obtained from three dimensional velocity logs. Conservative estimates of the modulus of elasticity (E) were utilized and are shown in figure 1. For illustrative purposes it may be assumed that the geophysical data indicated that there might be a weak zone in the rock in layers II and IV. The worst condition could be that layers II and IV were composed entirely of this weak material.

Loads

An average unit weight of 144 pcf was used in the analysis. The initial state of stress was assumed to be hydrostatic.

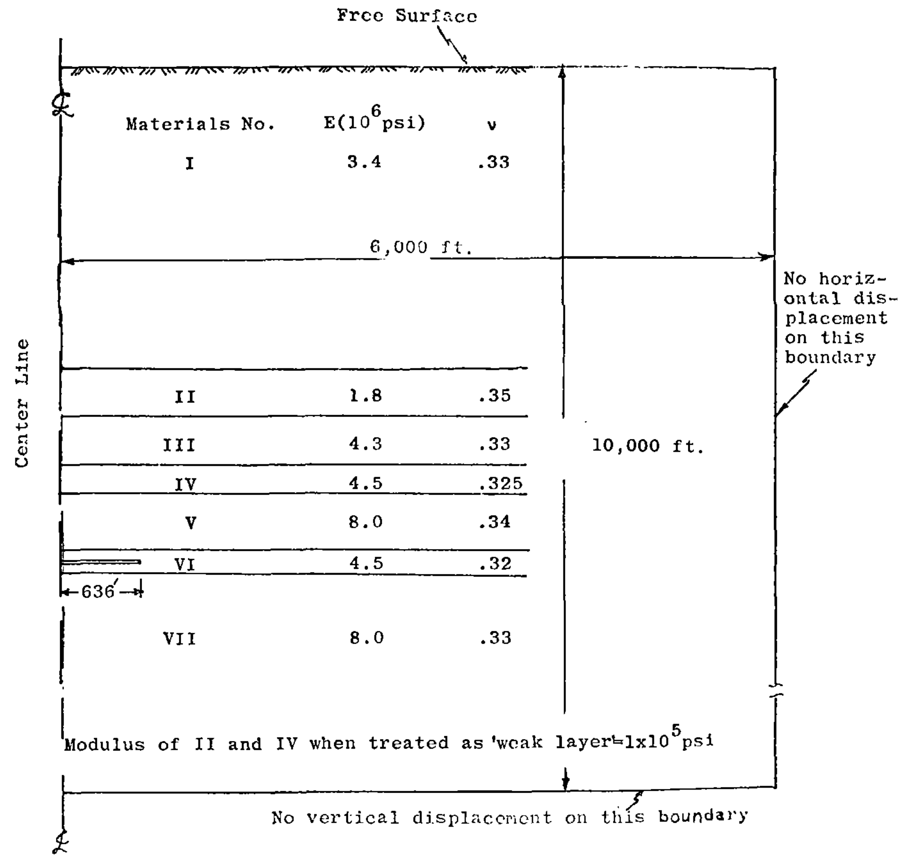

FIGURE 1.

BOUNDARY CONDITIONS

The surface and interior of the cavity was assumed stress free. The displacement boundary conditions on the other boundaries are shown on figure 1. The boundaries are at a sufficient distance from the cavity so that their influence on the results of the analysis are not significant.

The problem having been formulated it is necessary to construct a finite element mesh. The mesh should be constructed so that the boundaries between different materials are also boundaries of various elements.

The data which includes the problem geometry material properties, loads, boundary conditions and the finite elements mesh are input fot the computer program. The output consists of the stress and displacement field in the rock mass. The surface displacement is of course the subsidence. In determining the subsidence due to the cration of a cavity it is important to exclude the displacement of the rock mass due to gravity loads and initial stress prior to the creation of cavity.

The results of these analyses in the form of surface subsidence profiles is shown in figure 2. These results indicate the ability to analyse a bedded deposit using the finite element technique. The results indicate that the weak layers as located in this problem have little effect on the surface subsidence but do cause an increase in the surface displacement of the openings. Discussion of this behavior and the possibility of rock failure resulting in greater subsidence is outside the scope of this paper. It also shows how uncertainties in the rock profile can be investigated by analysing a number of cases. It is difficult to establish trends as the results are greatly influenced by the particular geologic conditions. However, the ability of the finite element technique to account for these conditions is a significant advantage.

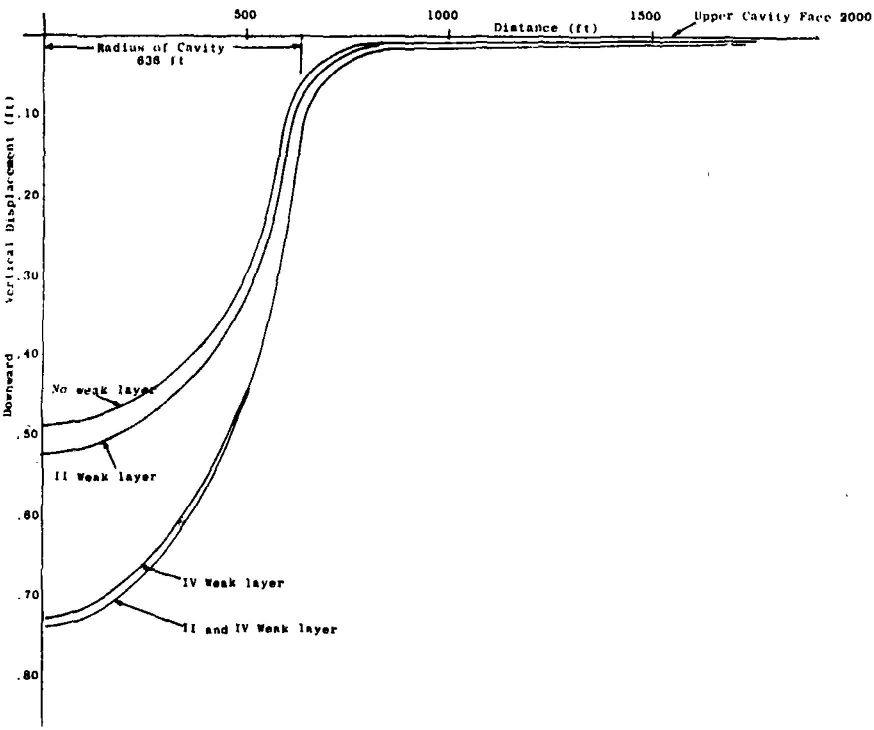

FIGURE 2a.

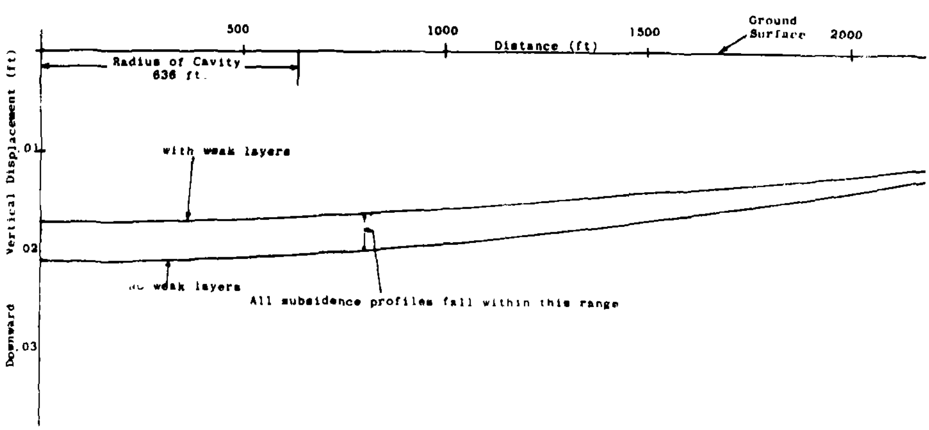

FIGURE 2b.

FINAL REMARKS

It is evident that using the finite element technique permits the solution of boundary value problems which are realistically representative conditions under which subsidence is likely to occur. It is also possible to investigate a range of conditions with relatively little effort. Care must be exercised in formulating the problem. Comparison of predicted and observed phenomena is the only way in which confidence can be developed in the formulation of the problem and in the methods of solution. Analysing phenomena which has already occurred and obtaining good agreement is always suspect.

ACKNOWLEDGEMENTS

Dr. C.Y. Chang of Woodward-Clyde & Associates assisted in performing the numerical calculations. The basic computer program was developed by Dr. E.L. Wilson of the University of California. His generosity in making the program available is gratefully ackowledged.

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AN EXAMPLE OF GROUND SUBSIDENCE ESTIMATION

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ABSTRACT

The Kôtô delta area of the Tokyo Metropolis is a low alluvial land of triangle shape, extending for about 5 km from east to west, about 10 km from south to north, with an area of approximately 40 km^2, bordered by the River Sumida on the west, and by the Arakawa canal on the east. The external embankment was planned for the purpose of defending this area from the calamity of floodtide. Before carrying out this undertaking, the estimation of future sinking has been found necessary.

The group involved in the investigation including the author, having been engaged in the undertaking, found that the data serviceable for the estimation were not sufficient. Acoordingly, the sinking amount for abuot 20 years after 1955 was estimated by the calculation on a very bold assumption, based upon technical decision. About 15 years having elapsed thereafter, it became possible to compare facts with estimation, which was proved to be satisfactory on the whole. In the present paper, explanation has been made of the method of estimating the ground subsidence, taking the Minami-Sunamachi, of the Kôtô district, as an example.

RÉSUMÉ

La zone detaïque de Kôtô à Tokyo est une région basse de forme triangulaire d'alluvions, dont la surface couvre environ 40 kilomètres carrés, s'étendant sur environ 5 kilomètres de l'est à l'ouest, et environ 10 kilomètres du sud au nord. Cette surface se situe entre le Fleuve Sumida à l'ouest et le Canal de fuite d'Arakawa à l'est.

Une digue d'enceinte a été prévue pour que la zone puisse être préservée du désastre des grandes marées. Avant que cette entreprise ait démarré, il fallait prévoir le volume futur de tassement. Le groupe de recherche dont faisait partie l'auteur a constaté que les données qui servaient à l'établissement des pronostics n'étaient pas suffisantes.

Par conséquent, des pronostics ont été faits en se basant sur une hypothèse ancienne basée elle-même sur un jugement technologique, et avec une hypothèse bien audacieuse, le volume d'affaissement de 20 ans après 1955 a été calculé. Environ 15 ans ayant passés depuis, nous avons pu comparer le pronostic avec la réalité, et nous avons trouvé que la comparaison est à peu près satisfaisante.

1. Report on the Study of Counter-measure Works Against Flood Tide and Ground Sinking, March 1957, by the Sabo Section, Public Works Research Institute, Ministry of Construction.

Dans cet exposé nous avons expliqué notre méthode de pronostics d'affaissements du sol, en citant comme exemple le quartier de Minami-Sunamachi dans la zone du delta de Kôtô.

Although it is not known when ground subsidence began in the Kôtô area, indications thereof were observed at the end of the 19th century. About 1918, the rate of sinking began to increase, and about 1923, the rate accelerated, reaching a maximum about 1941. From 1942 to 1947, the rate of sinking began to slacken. But since 1947, the rate again began to accelerate. The solid line in figure 1 shows the trend of sinking in Minami-sunamachi.

In figure 2, is an example of a boring log for the Minami-sunamachi area.

The geological features of this area are alluvial and tertiary layers. (Although it is not certain the tertiary layer may belong to the Alluvium.) The alluvial layer may be subdivide into upper, middle, and lower sublayers. The upper sublayer consists of alternate sandy loam and sand layers, at depths of 5-10 m. The upper portion of the middle sublayer has less clayey material than the lower portion of the middle sublayer. The consolidation of the middle sublayer which has both a big void ratio and compression index, is considered to account for the major part of the ground subsidence. The lower sublayer of the Alluvium is composed of silty sand.

The tertiary layer is also divisible into upper, middle, and lower parts of sublayers. The upper sublayer thereof is almost entirely eroded and the sublayer directly under the Alluvium is the middle one. The middle sublayer is composed principally of alternate strata of sand and gravel, sandy loam and loam; the sand and gravel stratum is thickest. This sand and gravel stratum is found to exist chiefly at depths of 60-150 m, comprising an extremely favorable permeable layer. The pressure level of this middle permeable stratum is changing on an extensive scale. It is inferred that the consolidation of this stratum greatly contributes to the sinking of the ground.

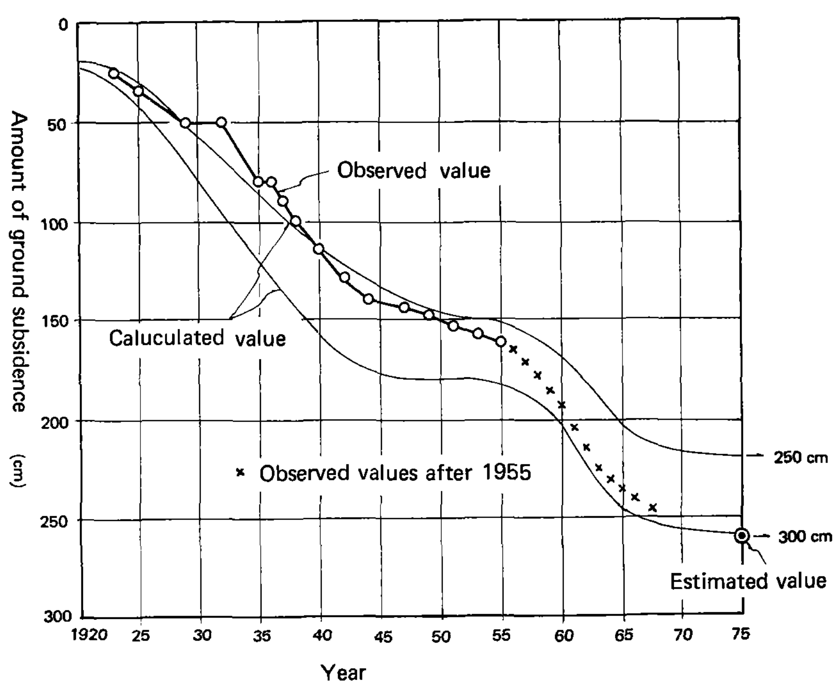

FIGURE 1. *Amount of ground subsidence*

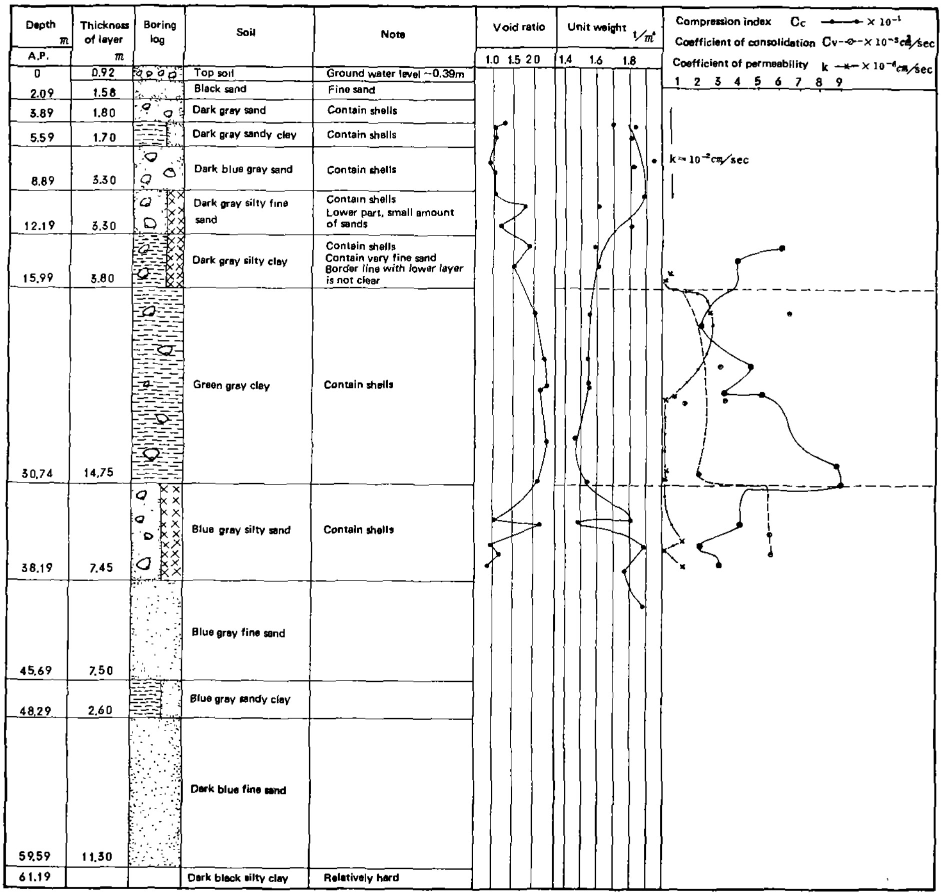

FIGURE 2. *Characteristics of soil*

In 1956, the depths of underground water observation wells were 70-130 m. The depths of strainers in the observations wells in Minami-sunamachi were 66.5-71.5 m, which were positioned in the sand and gravel stratum of the tertiary layer. The changes in water levels are shown in figure 3. In the same figure, the estimated position of the water level in the future is also indicated. Incidentally, this estimation was based upon the predicted pumping of underground water in the entire district of Kōtō area.

The estimated amount of sinking was based on the following assumptions:

1. As an initial condition, the water level in the permeable sublayer in the alluvial layer and that in the middle permeable sublayer in the tertiary layer coincide;
2. All of the material in the compacted sublayer in the Alluvium is consolidated clay;
3. As the result of excessive pumping of underground water from the middle permeable stratum in the tertiary layer, the ground sank because of consolidation in the middle clayey sublayer in the Alluvium as well as in the middle permeable stratum of the tertiary layer located at depths of up to 80 m;
4. Where the water level in the middle permeable stratum fell below the lower face of the alluvial layer, atmospheric pressure was effective between the lower face of alluvial layer and the water table.

In order to calculate the total amount of settlement, the following formula was used.

$$S_{\mathrm{I+II}} = H_0 \frac{C_c}{1+e_0} \log_{10} \frac{p_0+\Delta p}{p_0} \tag{1}$$

Where:

$S_{\mathrm{I+II}}$ Total amount of settlement including secondary compression settlement;
H_0 Thickness of consolidation layer;
e_0 Void ratio of soil before consolidation;
C_c Compression index of soil;
p_0 Effective vertical pressure before the commencement of consolidation;
Δp Increment of the effective vertical pressure that worked until the completion of consolidation.

Because the void ratio, unit weight, compression index and coefficient of consolidation before the ground subsidence were not available, the soil constants aqcuired by testing of samples obtained from cores were used without adjustment. However, it was assume that there were two kinds of clay according to depth in the middle clayey sublayer of the Alluvium, and that the soil constant of each kind of clay had a certain range. Of the consolidation sublayer of the tertiary layer, one kind of clay was assumed to occur, but as regards the soil constant thereof, a certain range was allowed. As the result of calculations, it was determined that the minimum amount of settlement was 141 cm, the maximum 342 cm, when the water level reached -42 m.

In view of the fact that ground-water storage changes with time, the method of graphical solution proposed by Terzaghi and Fröhlich was used in order to determine the

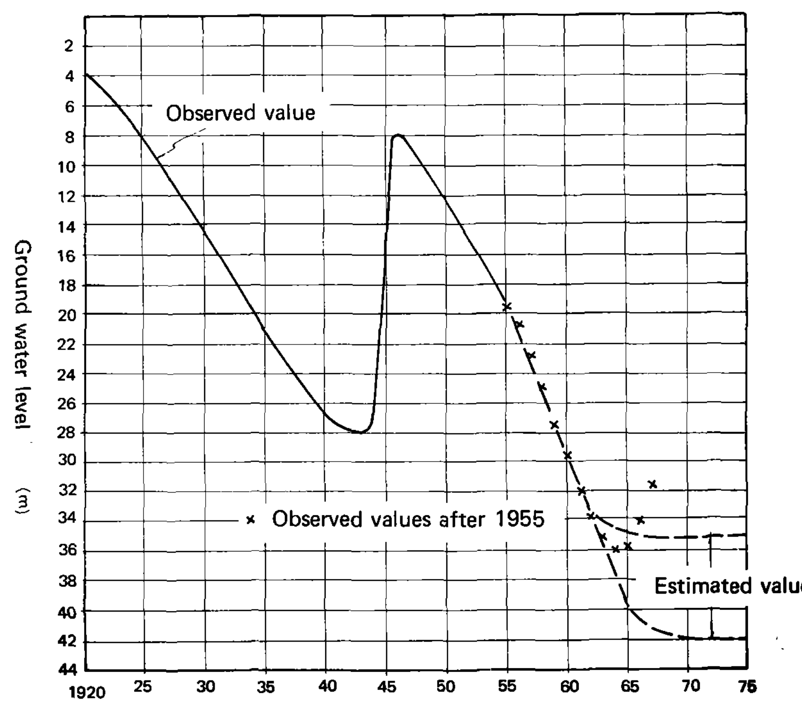

FIGURE 3. *Ground water level*

TABLE 1. Result of Settlement Measurment of Ground Surface by Iron Pipe Head Driven into Ground

Station No.	Depth of iron pipe *m*		1964	1965	1966	1967	Total	B:A %
4	45	A	115	138	98	127	478	41.4
		B	46	53	27	72	198	
15	70	A	118	121	65	120	424	45.7
		B	53	63	8	70	194	
23	130	A	119	124	68	121	432	46.8
		B	55	64	11	72	202	

A: Surface settlement amount (mm)
B: Settlement amount of iron pipe head (mm)

relation between time and the degree of consolidation. However, secondary consolidation was also taken into consideration. From the relation of total settlement and time-consolidation, time-settlement curves were drawn. By contrasting the actual measurements of settlement and the results of calculations, the total settlement was decided to be 250 cm and 300 cm. Figure 1 shows the estimated time-settlement curve. This curve that the calculations agree fairly well with the observed values. Accordingly, it was estimated in 1955 that the settlement in 1975 would be about 100 cm. In the figure, the position was marked by the symbol O. In the figure, the estimated trend of the sinking of the ground is also indicated. In addition, the actual measurements of water levels and the estimated level in 1975 are shown in figure 3. Although 20 years has not yet elapsed after the estimation in 1955, it is now estimated that the amount of settlement by the end of 1975 will reach 120 cm – about 20 cm greater than the 1955 estimated value. Of the several reasons for the reevaluation of the estimated settlement, the greatest one is the added effect of deep layer settlement caused by pumping of underground water from deeper levels than before. Table 1 shows the amount of settlement determined by measurements of an iron pipe driven into the ground in order to measure the soil surface as well as settlement. From this table, the following conclusion may be introduced. If only the upper layer to a depth of 80 m had sank, as originally assumed, the estimated value would have predicted precisely the amount of ground settlement during the 20 years covered.

PREDICTION OF FUTURE SUBSIDENCE ALONG DELTA-MENDOTA AND SAN LUIS CANALS, WESTERN SAN JOAQUIN VALLEY, CALIFORNIA

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ABSTRACT

About 120 miles of the Delta-Mendota and San Luis Canals, constructed by the Bureau of Reclamation, have been affected by subsidence caused by overdraft of ground water. When the Delta-Mendota Canal was completed in 1951, the subsidence was not recognized. From 1905 to 1951 piezometric ground-water levels along the alignment declined up to 200 feet. Canal deliveries stopped the overdraft, but continuing readjustment of pore pressures has caused some "residual subsidence" to continue at gradually diminishing rates. The subsidence has affected the downstream 35-40 miles of the canal, locally exceeding 6 feet from 1953 to 1966. Estimates of ultimate future subsidence, varying from traces to 3.6 feet, were derived from bench mark data treated as an exponential decay function.

About 80 miles of the San Luis Canal, constructed in 1963-67, are subsiding due to ground-water overdraft. From 1905 to 1962, piezometric levels along the alignment declined up to 470 feet, causing up to 18 feet of subsidence. Ultimate subsidence after 1963 was estimated using subsidence rates or piezometric decline-subsidence ratios and varied from traces to 15.2 feet (based on subsidence rates) or 10.4 feet (based on the ratios).

Hydrocompaction of low-density sediments above the water table occurs in two portions of the San Luis Canal. The phenomena may be related to freezing swelling and following freeze-drying of Pleistocene mudflows. Ultimate amounts of hydrocompaction ranging from traces to 20 feet were calculated from average dry densities of sediments and "ultimate" density values obtained in a few long-flooded areas.

RÉSUMÉ

Les canaux Delta-Mendota et de San Luis construits par le Bureau of Reclamation ont été affectés sur environ 120 milles par des affaissements provoqués par des pompages exagérés de l'eau souterraine. Quand le canal de Delta-Mendota fut achevé en 1951, on ignorait l'existence d'affaissements. De 1905 à 1951, les niveaux piézométriques le long de cet alignement tombèrent de 200 pieds. L'eau du canal arrêta l'action des pompages, mais le réajustement continu des pressions des pores a causé une continuation de l'action d'" affaissements résiduels " à des taux graduellement décroissants. Les affaissements ont affecté les 35-40 milles de la partie aval du canal, dépassant localement 6 pieds de 1953 à 1966. Une estimation des affaissements futurs, variant de traces à 3,6 pieds, a été déduite de l'examen de la descente de repères, traitée comme une fonction exponentielle décroissante.

Environ 80 milles du canal de San Luis, construit en 1963-67, s'affaissent sous l'action du retrait de l'eau souterraine. De 1905 à 1962, le niveau piézométrique le long de son alignement a décru de 470 pieds, causant un affaissement de 18 pieds. L'affaissement futur après 1963 a été estimé en utilisant les taux d'affaissement ou les rapports entre la descente du niveau piézométrique et l'affaissement. Il varie de traces à 15,2 pieds (en utilisant les taux d'affaissements) ou 10,4 pieds (d'après les rapports dont question ci-dessus).

La compaction hydrique de sédiments de faible densité au-dessus du niveau phréatique se produit dans deux portions du Canal San Luis. Le phénomène peut être mis en relation avec le gonflement par congélation et le séchage des flots de boue congelés du Pleistocène. Pour l'avenir, la valeur de la compaction variant de traces à 15 pieds a été calculée des densités moyennes des sédiments secs et des densités "finales" obtenues dans une zone assez longuement inondée.

INTRODUCTION

Land subsidence due to irrigation overdraft of ground water and hydrocompaction related to wetting of porous, low-density sediments are serious geologic problems in several areas in the United States. Estimates of ultimate amounts of subsidence are important for design, construction, rehabilitation and operation of water conveyance projects in such areas. This paper describes basic data, ideas, and methods of predictions which were recently used by the US Bureau of Reclamation in the San Joaquin Valley, California (fig. 1).

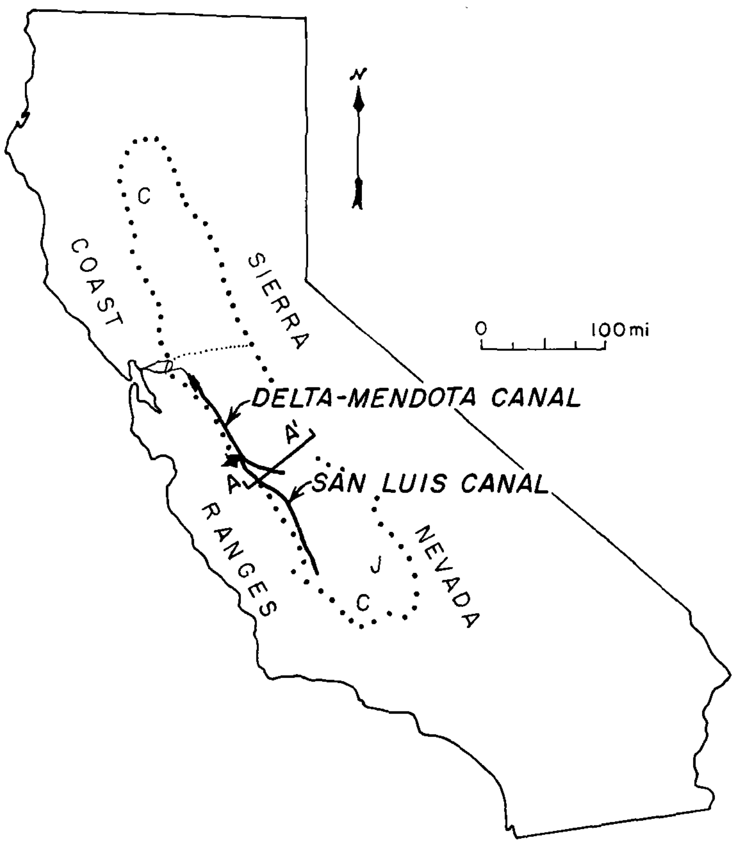

FIGURE 1. *State of California-location map*
C *Central Valley*
J *San Joaquin Valley*
A-A′ *Geologic section shown on figure 2*

The valley is the southern portion of California's Central Valley and is one of the world's most productive agricultural areas. The climate is semiarid of Mediterranean type; modern farming requires intense irrigation. Artesian ground water has been used for irrigation since around 1900. Irrigated acreage and the overdraft of ground water expanded notably in the late 1940's causing an acceleration of land subsidence.

Importation of surface waters was pioneered by USBR in 1951 with the completion of the Delta-Mendota Canal (USBR, 1961). Additional water imports to the south were accomplished in 1967 by the completion of the San Luis Unit with a 101.5-mile-long canal (USBR, 1966). Some reaches of these canals are affected by subsidence. Methods of prediction of ultimate future subsidence along the canals are described below.

REGIONAL GEOLOGY AND GROUND WATER

The 250-mile-long San Joaquin Valley is a large, northwesterly trending syncline between the Coast Ranges and the Sierra Nevada (fig. 1, Bailey, 1966). The valley is 30 to 50 miles wide, and is filled with thick predominantly alluvial, Cenozoic clays, sands, and gravels, overlying more or less consolidated older, mostly marine sedimentary beds (fig. 2).

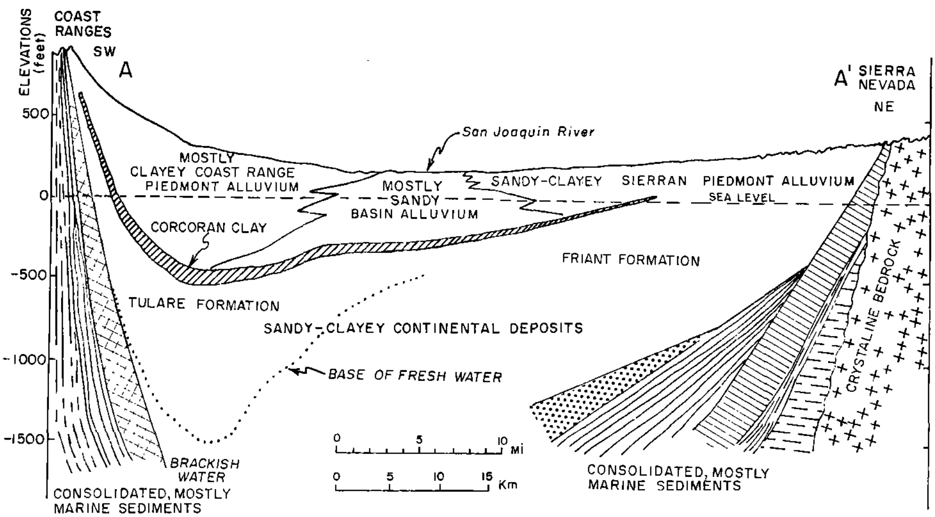

FIGURE 2. *Diagrammatic geologic section actoss San Joaquin Valley looking northwesterly (for location see fig. 1)*

The near surface deposits consist of eastern and western belts of piedmont alluvium separated by the central zone of river and lake deposits. The Coast Range piedmont consists of sandy-gravelly deposits in main fans and more clayey, coalescing interfan bodies deposited essentially by mudflows.

The most widespread bed in the upper continental portion of the valley fill is the Pleistocene Corcoran clay, a buried lacustrine bed (fig. 2, Frink and Kues, 1954). The clay creates confined ground-water conditions in the underlying fluvial beds which are the principal aquifers. A semiconfined ground-water body occurs in river sands above the Corcoran clay (fig. 2).

SUBSIDENCE—GENERAL DATA

Increases of irrigation pumping in the area and associated decreases of sub-Corcoran piezometric water levels resulted in increased effective load and corresponding compaction —"deep subsidence." The phenomenon was first recognized by the Bureau in 1951-52 and an "Interagency Committee" was initiated by the Bureau to study subsidence (Poland, 1958). Particularly comprehensive studies were made by U.S. Geological Survey.

The basic data for present studies were: (*a*) periodic levelings of numerous bench marks, (*b*) maps showing piezometric ground-water levels, and (*c*) a few observation holes with compaction recorders. The recorder data (fig. 3) indicate that most subsidence occurs in and below the Corcoran clay.

A comparison of topographic quadrangles surveyed in 1923-28 and 1926-33 failed to indicate any elevation changes caused by subsidence. However, subsidence was noted by releveling of a bench mark near Mendota in 1935. The early 1930's are therefore a probable time of the beginning of subsidence. Total subsidence in the area has locally exceeded 25 feet.

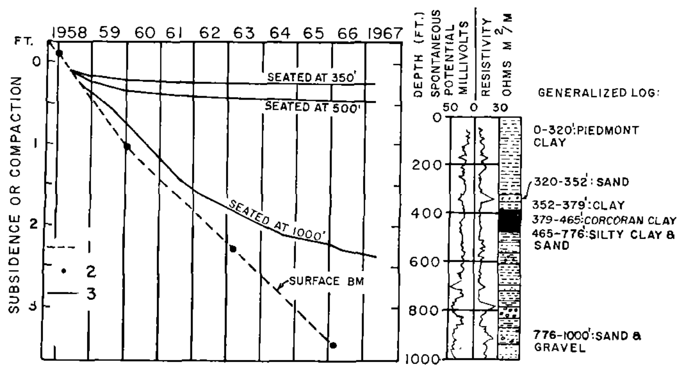

FIGURE 3. *Subsidence of a surface bench mark and compaction of deposits to depths indicated, from compaction recorders, Dalta-Mendota Canal near Ora Loma*

Hydrocompaction is restricted mostly to the Little Panoche-Panoche and Panoche-Cantua interfans (fig. 4). Moisture contents and dry densities of clayey sediments prior to the irrigation were locally as low as 5 to 8 percent and 75 to 80 pounds/ft^3. When wetted, these deposits compact as much as 12-15 feet, and produce tension cracking and sinkholes which damage canals, ditches, roads, buildings, etc. (fig. 5A, B, C).

The origin of the low-density sediments is uncertain: Present-day mudflows and near-surface (0 to 3 or 4 ft) piedmont deposits have high dry densities (about 95 pounds/ft^3). Most of the piedmont alluvium was deposited during Pleistocene time. Hummocky soil surfaces and intra-formational, severely contorted bedding are common in the area and are very similar to typical permafrost features (Prokopovich 1969). Perhaps the origin of the low densities may be related to paleo-climates as follows: Pleistocene mudflows from the Coast Ranges were frozen in the valley. Expansion of water by freezing expanded the saturated sediments which were subsequently not melted but "freeze dried" in the semi-arid environment. Consequently, the low densities of the sediments were preserved. Sandy well-drained fan deposits were not expanded by freezing and became nonsubsiding terrains.

DELTA-MENDOTA CANAL—ESTIMATED SUBSIDENCE

The existence of subsidence in the area was not known during preconstruction and construction studies, but discrepancies between old and new levelings were consistently encountered in the downstream 35 to 40 miles. Numerous bench marks were established after completion of the canal in 1952-1953, when the subsidence was recognized by the Bureau. From 1953 to 1966 subsidence along 35-40 miles of the canal locally exceeded 6 feet (fig. 6) and canal lining, bridges, pipe crossings and other structures became more or less flooded (fig. 5E, F).

From 1905 (the earliest record) to 1953, sub-Corcoran piezometric levels along the canal declined as much as 200 feet causing up to 6 feet of subsidence. Canal deliveries ended the overdraft and, since 1953, piezometric levels below and above the Corcoran clay

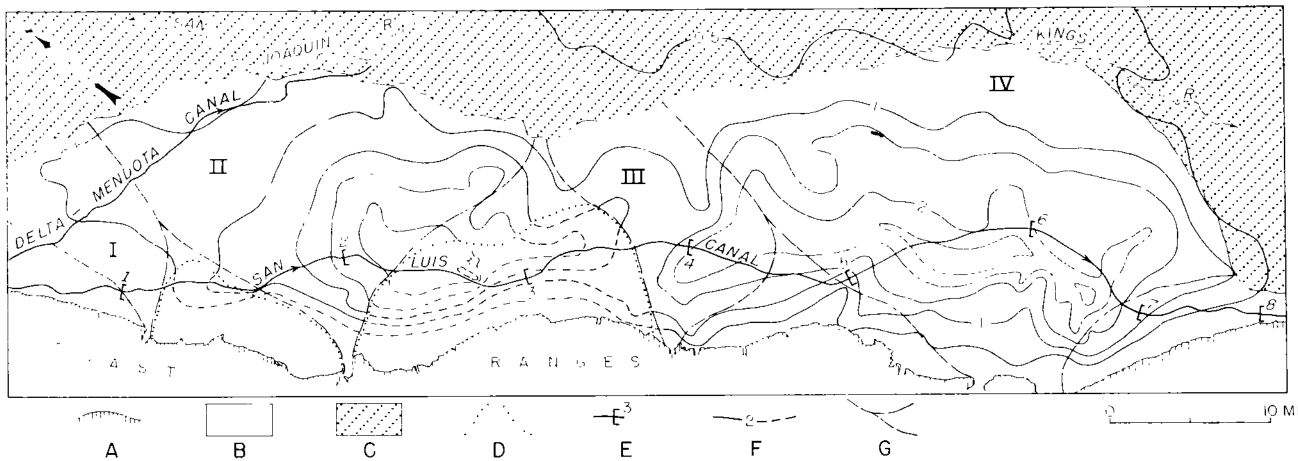

Figure 4. *Geologic map of west-central San Joaquin Valley and land subsidence between 1963 and 1966.* A. *Edge of Coast Ranges;* B. *Piedmont Coast Range alluvium;* C. *River and lake alluvium;* D. *Boundary of areas affected by hydrocompaction;* E. *Canal check with number;* F. *Subsidence contour line, contour interval 1/2 ft; dashed where affected by hydrocompaction;* G. *Boundary of piedmont fans.*
Major fans: I *Little Panoche*, II *Panoche*, III *Cantua*, IV *Los Gatos-Zapato*

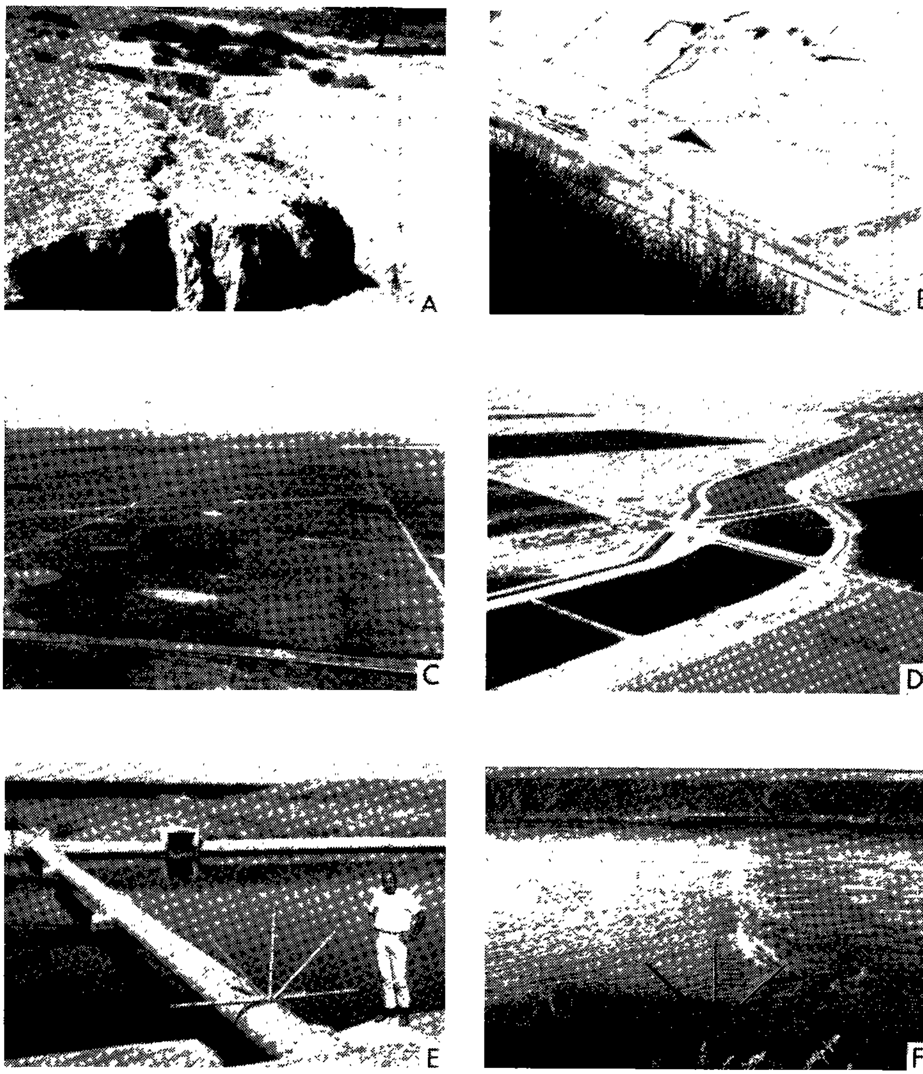

FIGURE 5A, B, C, D. *Hydrocompaction on Panoche-Cantua Interfan:*

A. *Soil cracks caused by a nearby irrigation ditch;*

B. *Damage to a test section of a canal at the State test site;*

C. *Sinkholes created by irrigation in originally flat fields;*

D. *Preconstruction flooding of San Luis Canal alignment.*

FIGURE 5E,F. *Land subsidence along Delta-Mendota Canal:*

E. *Normal pipe crossing and concrete lining in stable area;*

F. *Similar crossing in subsiding area.*

showed some fluctuation but have not progressively declined. Hence the postconstruction subsidence is a progressively diminishing residual action (fig. 7), caused by the gradual readjustment of pore pressures.

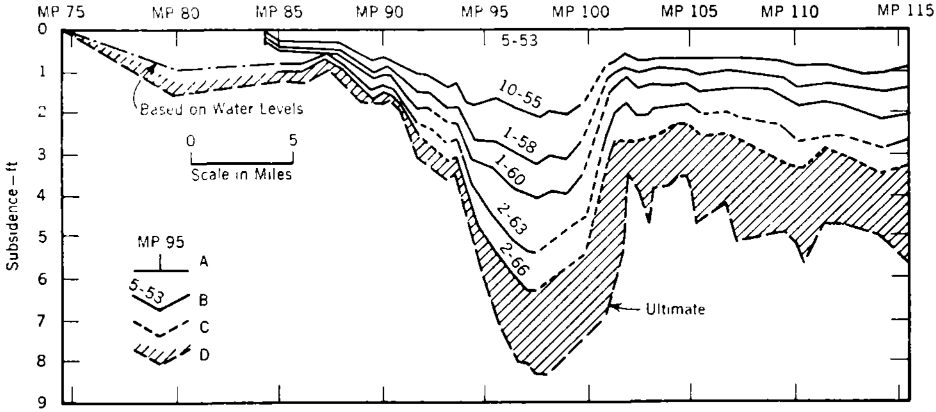

FIGURE 6. *1953-1966 and future land subsidence in downstream reaches of the Delta-Mendota Canal.*
A. *Mile post distance from intake;*
B. *Date of survey;*
C. *Interpolations to bridge data gaps;*
D. *Predicted ultimate subsidence*

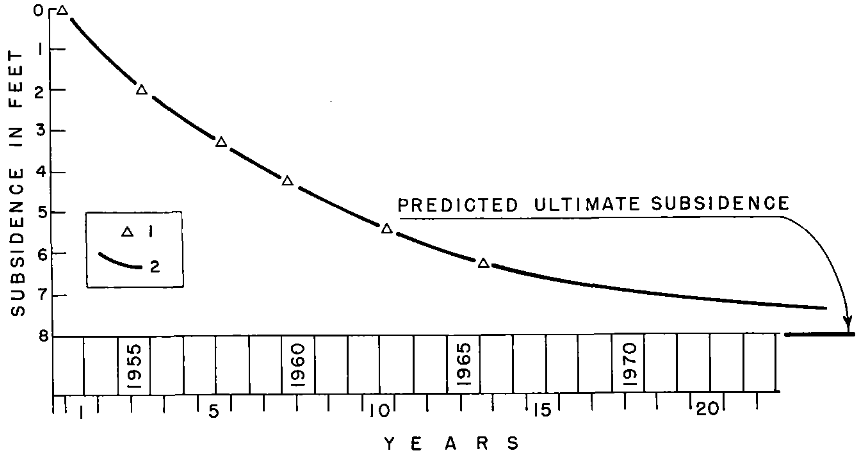

FIGURE 7. *Typical graph of residual and calculated subsidence of the bench mark 96.61.* 1. *Measured values;* 2. *Calculated curve*

Attempts were made to calculate ultimate subsidence using laboratory consolidometer tests of samples from a core hole at the canal (USBR 1956, 1959. Miller 1961). For the entire canal this method would require numerous holes and laboratory tests. It is also questionable how well 20-30 samples would represent a heterogeneous column of sediments several hundred feet thick.

Time-subsidence graphs of bench marks, influenced by residual subsidence, could be considered as "consolidation graphs" under a constant load. Rates of change in such graphs appear to be proportional to the amount of uncompleted consolidation. This is characteristic, and suggests the use, of an exponential decay function with the following form (fig. 8): $Z-y = Ae^{Bt}$ in which $Z-y$ is uncompleted subsidence, y is recorded subsidence after 1953, t is time after 1953, A and B are unknown constants and Z is ultimate subsidence after 1953. With 3 data points from a bench mark graph the three unknowns including ultimate susidence (Z) can be calculated (fig. 7).

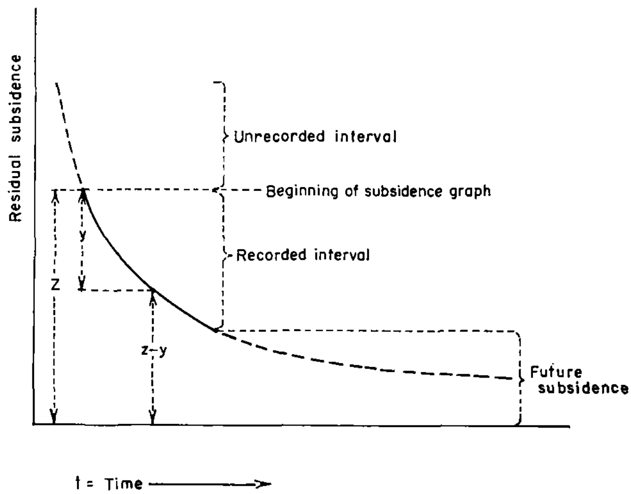

FIGURE 8. *Exponential decay curve of residual subsidence*

The calculated values of subsidence after January 1966 varied from traces to 3.6 feet (fig. 6). The validity of the approach was confirmed indirectly by correlation with recorded subsidence data (fig. 7). (Prokopovich and Herbert 1968). It was estimated that the residual subsidence (lag) will be virtually completed during the next 20-25 years.

SAN LUIS CANAL—ESTIMATED SUBSIDENCE

Land subsidence in the area was well known and studied prior to the canal construction. It affects about 80 miles of the alignment. Fom 1905 to 1962, piezometric levels along the alignment declined 300-470 feet (fig. 9) causing as much as 18 feet of subsidence (fig. 10). Historic subsidence piezometric decline ratios along the canal varied from 0.01 to 0.06.

Estimates of ultimate future subsidence were required for design of the canal. The first estimate was made in April 1961, using 1956-1958 and 1958-1960 subsidence maps and published 1:24,000 topographic quadrangles. It was assumed that subsidence would continue at the same rates until the end of ground-water overdraft which would occur in January 1968 with the completion of the canal and expected completion of the distribution system and that residual subsidence would continue with diminishing rates and would eventually total an amount equal to 5 years of subsidence at the same rates. Strip maps of the canal area were drawn showing estimated topography at the beginning and the end of construction (January 1963, January 1968) and under the assumed ultimate conditions. The next estimate was made in 1963 and was based on the same assumptions but

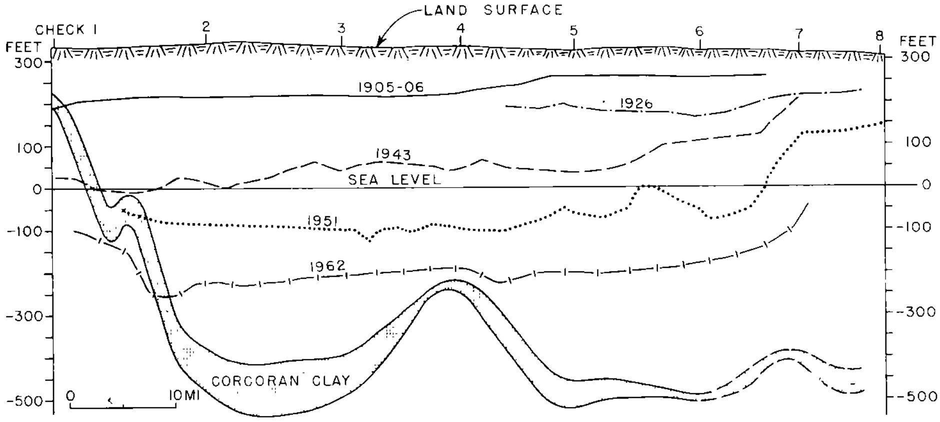

FIGURE 9. *Historic decline of sub-Corcoran piezometric water levels along the San Luis Canal*

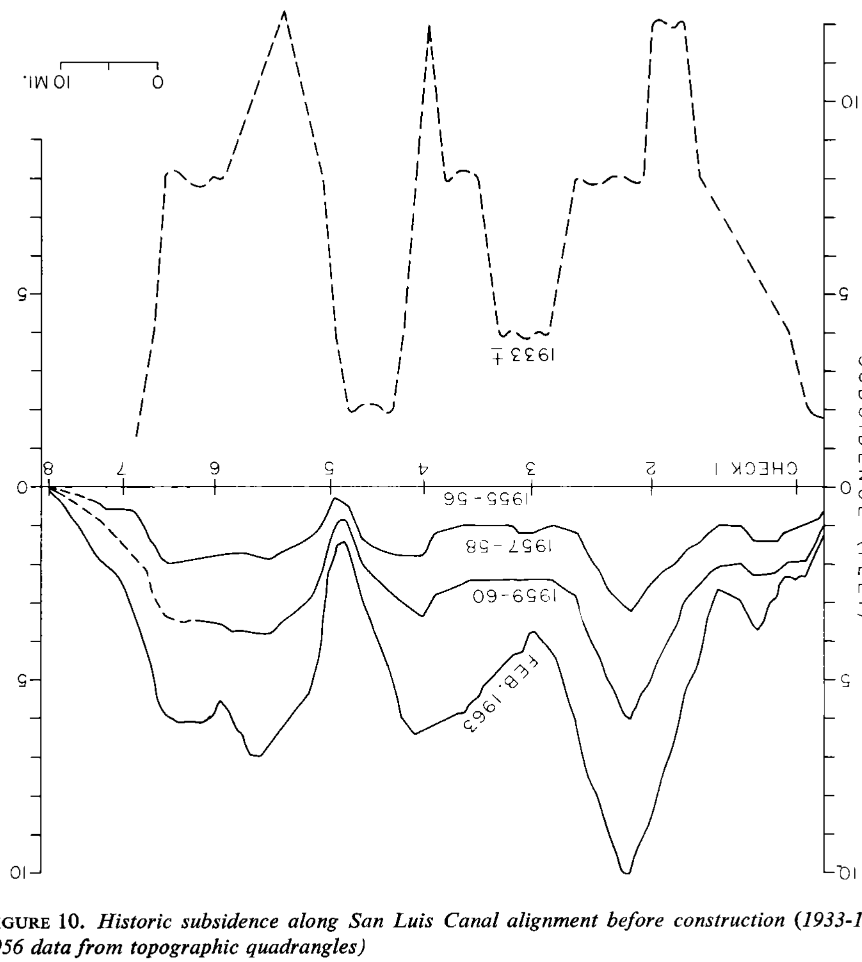

FIGURE 10. *Historic subsidence along San Luis Canal alignment before construction (1933-1955/1956 data from topographic quadrangles)*

using 1960-1963 subsidence rates. In areas affected by hydrocompaction, its magnitude was estimated graphically and was subtracted from the total subsidence rates. Residual subsidence was assumed to be equal to 10 percent of the total and was added to the estimates. Estimated ultimate values of subsidence after January 1963 ranged from traces to 15.2 feet and averaged 8 feet.

Somewhat smaller values, ranging from traces to 10.4 feet and averaging 5 feet,were derived from the piezometric decline data and piezometric decline-subsidence ratios. Use of the ratios, however, ignores the fact that essentially complete compaction of thick sedimentary sequence requires at least several decades. Subsidence-piezometric decline plottings are therefore HISTORIC rather than FUNCTIONAL graphs. An attempt is now being made to adjust San Luis subsidence-piezometric decline graphs using data obtained along Delta-Mendota Canal.

SAN LUIS CANAL—HYDROCOMPACTION

Hydrocompaction affected two reaches of the San Luis Canal, totaling about 20 miles, and 90 miles of the alignments of distribution pipelines. The area has received variable amounts of irrigation water and has been subjected to different amounts of hydrocompaction.

Estimates of remaining hydrocompaction along the canal were made using more than 2,000 dry density values from 120 test holes. Average densities from the holes were compared with "ultimate densities" from 14 holes in local areas which had been flooded for a long time and were believed to be completely hydrocompacted (Prokopovich 1962):

$$\Delta h = H - \frac{H \cdot d}{D}$$

where:

Δh amount of compaction;
H depth to ground water;
d present average density, and
D "ultimate density" for the same depth interval.

Estimates of future hydrocompaction ranged from traces to 20 feet. It is possible, however, that this method yielded too large values because of variable compaction during sampling with 3-inch diameter Shelby samplers.

To minimize the effects of hydrocompaction, 20 miles of the canal alignment were flooded prior to construction by a system of 128 preconsolidation ponds (Hall and Carlson 1965). Gravel-packed infiltration wells up to 125 feet deep were used in some areas to speed wetting at depth. The flooding was conducted for 12 to 18 months and resulted in up to 8 feet of settling. The total cost of the treatment (fig. 5D) was about $4 million.

Various types of preconstruction treatment, including flooding of trenches, are under consideration for the distribution pipe-lines to be constructed in soils susceptible to hydrocompaction.

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DISCUSSION

Intervention by Mr. Herbert H. SCHUMANN (USA):

Question:

What are cost estimates for repairs caused by unpredicted additional subsidence along the San Luis Canal? (Note: The question deals with final remarks of the speaker about unpredicted excessive subsidence, which is not discussed in the published text.)

Answer of Mr. PROKOPONICH:

The estimates are not completed, but are in an order of several millions. In original predictions we faced two problems:

1. Bznch marks along the alinement were very sparse and subsidence rates were obtained by an interpretation of subsidence contour maps.
2. Construction of distribution sytem was assumed to be completed in 1967. The system is still under construction.

COUNTERMEASURES AGAINST LAND SUBSIDENCE WITH RESPECT TO DISPLACEMENT OF SURFACE FLOODING

UKENA, KANNO, and TERANAKA
Bureau of Port and Harbour, Tokyo Metropolitan Government

ABSTRACT

The increase of land subsidence has brought about some change of the techniques for the countermeasure in the low land. In the Kôtô Delta, various countermeasures, such as protection against storm-surge, the measures for control of land subsidence, drainage of inner water by rainfall together with the countermeasures against earthquake shock, are or will be put to practice.

This report mainly describes theoretical calculation processes in determining the conditions of the plans for the countermeasures against storm-surge and inner water to be investigated hydrotechnologically. The description contains the data of the past storm-surges, the numerically analysed storm-surges, the distributions of rainfall at storm-surge, the estimated amounts of its runoff from rainfall and land subsidence and so on. Some earthquake engineering problems are also dealt with.

RÉSUMÉ

L'accroissement des affaissements a amené une modification de la technique des contremesures prises dans le pays bas. Dans le delta de Kôtô des mesures variées ont été ou seront mises en pratique : mesures contre les marées-tempêtes, mesures pour le contrôle de l'affaissement, drainage de l'eau intérieure due à la pluie, mesures contre le choc des tremblements de terre.

Ce rapport s'étend surtout sur la description des procédés de calcul théorique dans la détermination des mesures contre les marées-tempêtes et pour l'évaluation des eaux intérieures. La description comprend les données sur les marées-tempêtes passées, leur analyse numérique, la distribution des pluies lors des marées-tempêtes, l'estimation des valeurs d'écoulements dus à la pluie, des affaissements, etc. Quelques problèmes posés par les tremblements de terre sont aussi examinés.

THE INEVITABILITY OF THE MEASURES FOR REMOVAL OF THE INTERNAL WATER

The Koto delta area primarily made its emergence at the mouth of the Arakawa River (about 44 sq km in acreage, an expanse of alluvium 30 meters deep on the average) mostly as a farmland about 100 years ago. Its damage in the past from the rampage of a typhoon was an exceptionally limited one, when estimated in cost. However, because of the intensified concentration of industries in recent years there has been a marked degree of subsidence in this area, of which the excessive pumping of ground water for the industrial purpose is found to be a major cause. Earlier than approximately 1920 ,this area had a ground level higher than that of the daily high tide, and rain water was permitted to run-off freely through the well developed waterways. However, the drop in the ground level below the level of daily high tide has gradually made it difficult for natural runoff of rain water. In conjunction therewith, a forced drainage operation by installation of pumping plants began around 1928 in this area. Meanwhile, the subsidence became increasingly intensified, so that around 1940 about 50 percent of the Koto delta area became lower than high tide level and around 1953 the ground level became lower than the daily low tide level. At this point the area which permitted natural runoff had became almost non existant and a forced drainage operation for rainfall had to be put into action on a massive scale.

Currently, the drainage areas within the delta are distributed as shown in the figure 1. The planned drainage of runoff for the total drainage area is 278 cu m per sec.

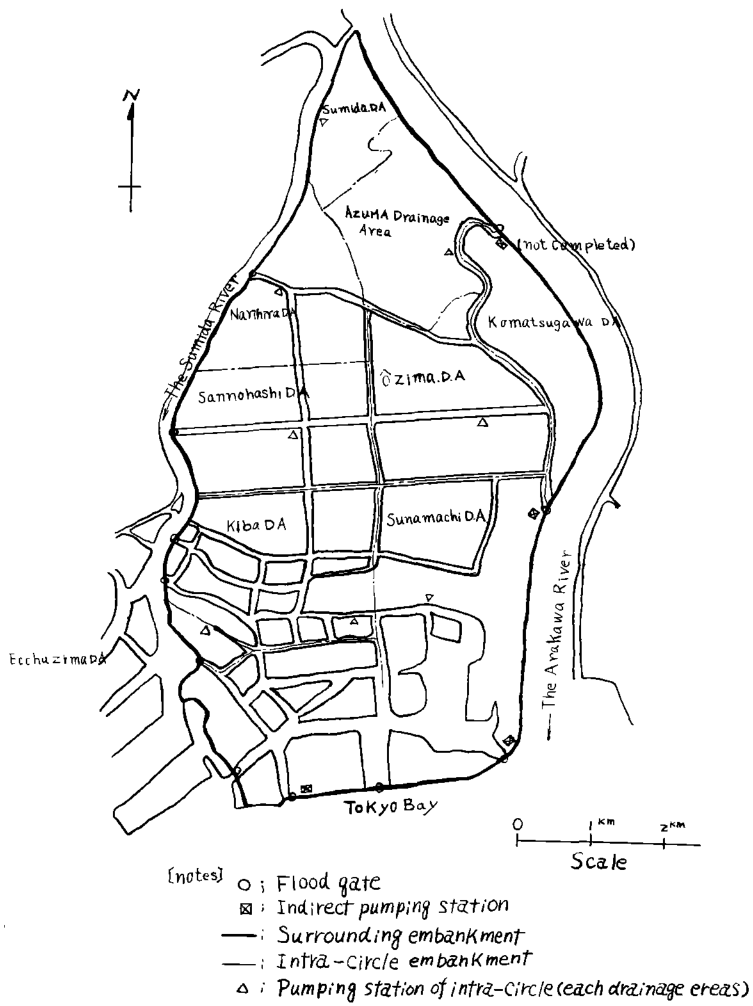

FIGURE 1. *Positions of pumping station concerned with flood tides*

HIGH TIDE MEASURES AND INTERNAL WATER DISPLACEMENT MEASURES

The Koto delta area is located at the deepest head of Tokyo Bay. For this reason it has been habitual for this area to suffer high tide damage due to typhoons since early days. In addition, the rapid progress in subsidence is an effect that aggravates such disastrous effect even more. The high tides of 1917 (H.W.L.A.P. of + 4m12, the maximum velocity of wind 39.6 m per sec SSE, the minimum atmospheric pressure 950 mb), and the high tides of the 1949 Kitty typhoon (H.W.L.A.P. of +3.25m, the maximum wind velocity 24.9 m per sec, SE, the minimum atmospheric pressure 950 mb) are typical examples.

On the other hand, the typhoons that bring high tides to Tokyo are of a type that, generally speaking, accompanies heavy rainfall. In the case of the 1917 typhoon course, the overlapping of the high tide peak and the rainfall peak with each other was rather lesser in degree, whereas in the case of the 1949 Kitty typhoon this sort of superposition was remarkably heavy. (fig. 2). The typhoons that follow the Kitty typhoon course are the ones that cause the greatest high tide mark at Tokyo. This gives rise to problems of how to eliminate flood water at the moment of invasion of a high tide in the Koto Delta area.

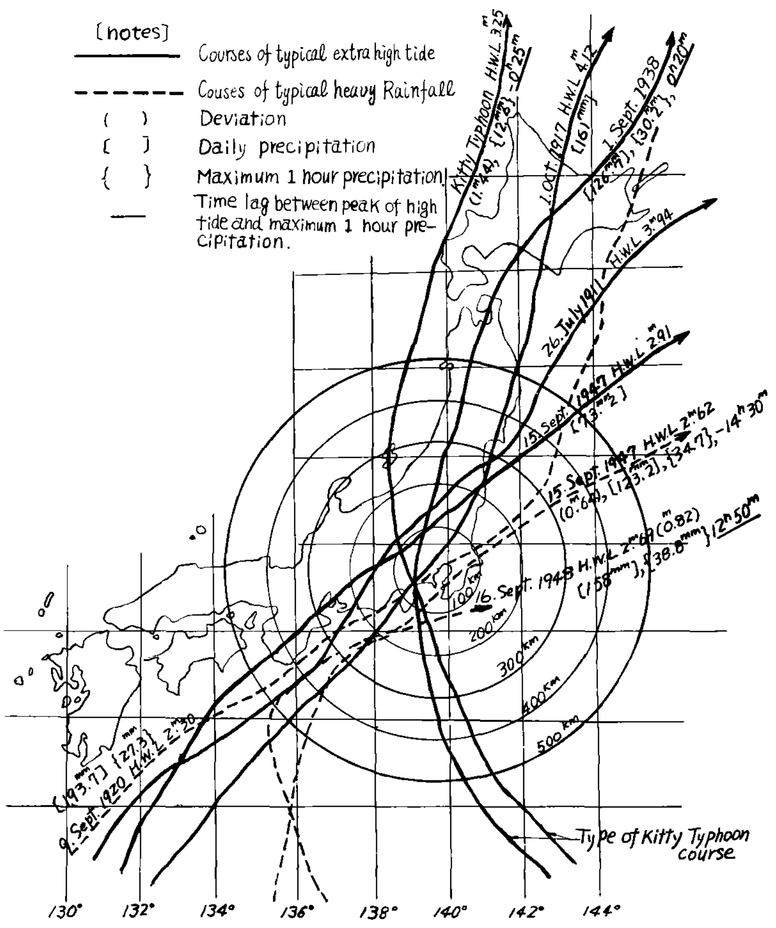

FIGURE 2. *Course diagrams of typical heavy rainfall and typical extra high tide by tymhoon*

A 1959 typhoon, attacked Ise bay, which is located in the middle of Japan's mainland —locally known as Honshu. This typhoon's center was reported at 940 mb, the maximum velocity of wind as 40 m per sec and the storm circle measured upward of 20 m per sec in wind velocity extending over about 700 km to record itself as the biggest of typhoons in history, as far as our country is concerned. This typhoon brought forth a H.W.L.N.P. of +5.31m, and thus left a really critical scar of damages. Because of these bitter experiences, a series of measures were initiated in 1960 to prevent the low land spreading over the

Koto delta area from having devastation such as that wrought by an Ise Bay class typhoon. One segment of these countermeasures was a flood water disposal project designed to meet a possible emergency upon arrival of high tides.

A variety of problems were studied and discussed from both technical and economic points of view. As a result, a flood water removal plan was developed, consisting of the following sequence of precautionary measures: (1) erect an external embankment that embraces each of an inner circle of embankments, (2) displace the flood water first to the internal waterway from a pumping plant in each of the inner circle of embankments, and (3) then move the flood water to the open sea through the internal waterways. Following is a summary of the basic items of the proposed flood water elimination program

A COMPUTATION OF PLANNED PRECIPITATION

The rainfalls that overlap the high tides when a typhoon visits Tokyo form a very important condition. For this reason, we emphasized an estimation of the maximum amount of precipitation. Thus, we have made calculations following these three methods:

(*a*) Method of computation based on an equation of meteorological dynamics.

We sought a vorticity by solving a kinetic equation of the atmosphere with reference to a circular isobar such as a typhoon, and from such vorticity, then have calculated a converged volume and a vertical air current velocity integrated from the lower to the upper layers of atmosphere. From such results and the volume of condensed air, we then calculated the volume of precipitation due to the swirls arising from the typhoon.

(*b*) Method of calculation based on a multiplex correlation with the application of a variety of parameters relative to the precipitation.

The precipitation due to the swirls of a typhoon is a function of parameters of manifold meteorological phases conducive to the precipitation. As a result of the so-called estimated computation, we have tried to find the 6-hour volume of rainfall due to typhoon swirls based on the coaxial diagram of graphical statistics, employing these 3 parameters: (1) a difference between the central atmospheric pressure of typhoon and the minimum atmospheric pressure in Tokyo, (2) a relative vorticity, and (3) the type of typhoon course.

(*c*) Method of computation for the excess probability precipitation.

We shall explain about this method later in a little more detail as we have used it in the determination of a planned volume of rainfall.

In addition, we have investigated these points: relationship between the hours of rainfall and the volume of rainfall and the intensity of precipitation; distribution of rainfall in Tokyo, the correlation between the weather station and the rainfall in the Koto delta area, and so on.

The rains due to a typhoon can be classified into these types: (1) rains due to the front and the converging line, (2) rains due to the influence of terrain condition, and (3) rains due to the swirls in the vicinity of the typhoon center, in particular.

Generally speaking, the rainfall due to a typhoon varies with the course of the typhoon, the atmospheric pressure at the center, and whether the front does or does not exist. A typical example is illustrated in figure 3.

As to the planned volume of rainfall, we have determined it by method c, mentioned above. This method of computation concerns excess probability precipitation, and is described below.

Because the hourly volume of rainfall at a place threatened by an imminent approach of typhoon has a correlation with the position of a typhoon and its course, we have made a probability computation with reference to the forty cases of typhoons that had approached Tokyo in the period from 1917 to 1960. The typhoon position (longitude and latitude) and the hourly volume of rainfall in Tokyo in relation to the position of typhoon was considered. For this reason, we have plotted on the typhoon course chart (classified by

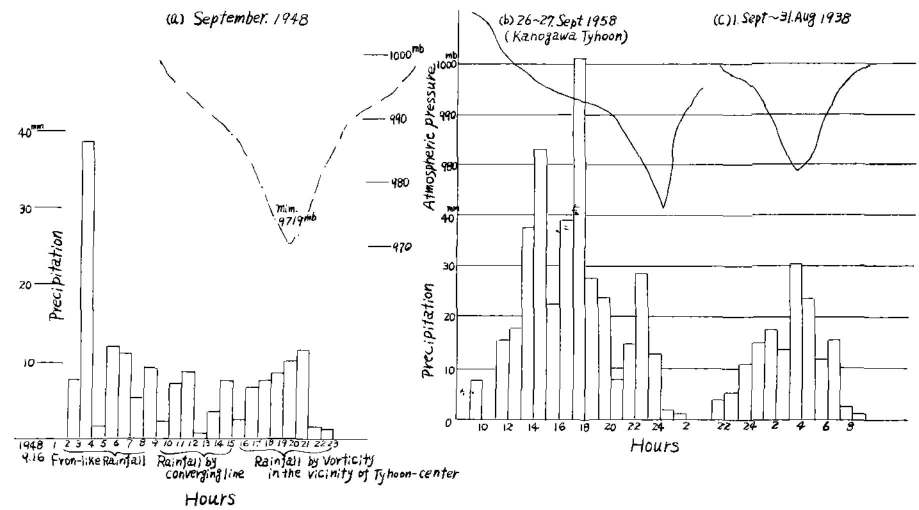

FIGURE 3. *Typical actual rainfall by typhoon*

the year) the hourly volume of rainfall in Tokyo at every hour on the typhoon course. Thus, we have sought to find the annual maximum value within each degree of longitude and latitude, and made it data of probability calculation for each year. The segment taken up by our computation was one ranging from 30° to 38° north longitude relative to the volume of rainfalls in Tokyo. Further, we have computed the hourly volume of rainfall for each of the individual sectors for different "return periods" of 30, 50, 100 and 200 years, respectively. We have thus tried to find an hourly probability volume of rainfall that may occur in Tokyo at each of the hours based on the model course of the 1949 Kitty typhoon and that of the 1917 typhoon. However, because the precipitation for each of these sectors is the maximum hourly volume of rainfalls that might occur in Tokyo, all the hourly distributions of precipitation in Tokyo do not necessarily indicate the maximum value. In order to determine the volume of rainfall for each hour, with reference to more than one typhoon showing the same course, we have divided the rainfall into two types: one of which is due to a typhoon itself and another mainly due to the action of the front. We have thus worked out an average distribution model from the hourly volumes of rainfall for each degree of longitude and latitude. In addition, we have had to ascertain the distribution of precipitation from the aspect of the typhoon's traveling speed. The distribution of probability precipitation classified by the course of typhoons can be noted in table 1. For planning purposes, we have adopted the distribution of rainfall as was observed in the course of the Kitty typhoon, representibg a 50-year probability.

The rainfall, largely influenced by the terrain condition, grows fairly complicated in its distribution. The variation is particularly conspicuous in a region like Tokyo where its eastern sector forms a flat land to face the sea shore, and its western portion constitutes a hilly stretch of land. In addition, the terrain condition and its relative precipitation ought to possess their own characteristic features according to the types of atmospheric turbulence, the low atmospheric pressure, front, etc. The distance, as the crow flies, between

TABLE 1. Course of Kitty Typhoon　　　　Course of 1917 Typhoon

Course of Kitty Typhoon

Date hour \ Return Period	30 years	50	100	200
22	4 mm	5 mm	7 mm	9 mm
23	5	7	10	12
24	4	6	8	10
31. 1	4	6	8	10
2	5	7	9	12
3	6	8	11	14
4	7	9	12	16
5	7	9	13	16
6	7	9	13	16
7	6	8	11	4
8	4	5	7	9
9	0	0	1	1
10	2	2	3	4
11	2	2	3	4
12	2	2	3	4
13	3	4	5	7
14	4	6	8	10
15	4	6	8	10
16	7	10	18	17
17	4	5	2	9
18	1	0	1	2
19	5	7	9	12
20	24	31	42	53
21	36	47	63	80
22	26	31	38	49
23	19	25	34	43
24	10	14	18	24
1. 1	5	7	10	12
Total amount of rain-fall	213	278	370	479
Total amount of rain-fall (except the front like rain-fall)	154	199	261	341

Course of 1917 Typhoon

Date hour \ Return Period	30 years	50	100	200
30. 2	2 mm	3 mm	5 mm	6 mm
3	1	1	2	3
4	0	0	0	0
5	1	1	2	3
6	2	3	4	5
7	10	13	17	22
8	11	15	20	26
9	11	15	20	26
10	8	11	15	19
11	15	20	27	35
12	27	32	41	60
13	22	29	39	49
14	9	12	16	20
15	27	32	41	60
16	21	28	38	48
17	3	5	6	8
18	1	1	2	3
19	4	5	7	9
20	8	11	15	19
21	19	24	33	42
22	17	22	30	38
23	22	29	39	49
24	21	28	37	48
1. 1	25	35	45	60
2	30	40	54	68
3	36	47	63	80
4	27	31	39	49
5	21	24	28	34
6	17	22	26	35
7	3	5	6	8
Total amount of rain-fall	421	548	717	932
Total amount of rain-fall (except the front like rain-fall)	251	324	424	542

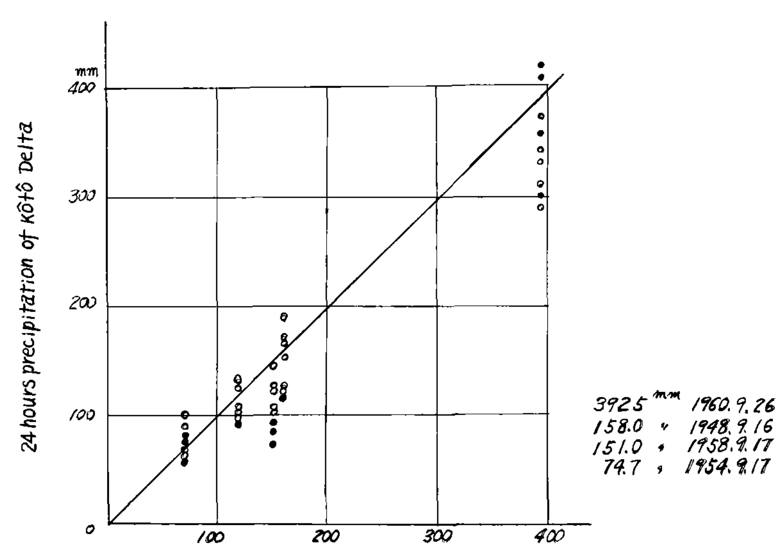

FIGURE 4. *Correlation between 24 hours precipition of Koto Delta and meteorological observatory*

Tokyo (Meteorological Observatory) and the heart of the Koto delta area is about 5 km. But it may be noted that the correlation degree of the daily volume of rainfalls between the point selected on the plan and Tokyo (M.O.) is very close to 1. In addition, because it is reported that the higher the typhoon's meteorological turbulence the closer to 1 is the degree of correlation, we have chosen 1 as the basis for our planning (fig. 4).

SUPERPOSITION OF HIGH TIDES AND RAINFALLS

The type of rainfall at the moment of a visiting typhoon can be classified approximately into that of the 1917 typhoon course and that of the 1949 Kitty typhoon.The possibility of superposition between the neighborhood of the high-tide peak, of the Kitty typhoon's course in particular, and the peak of rainfall is fairly significant viewed from the past examples and the meteorological standpoint. In the case of the Kitty typhoon, we find that the peak of high tide and that of rainfall precisely coincide with each-other. Therefore, as far as the course of a Kitty-type typhoon is concerned, it may be said that the possibility of superposition generally is great. In looking at past data, we find that the maximum 30.2 mm per hour recorded at the time of the 1938 typhoon was the heaviest of rainfalls, but this is a value predictable in the field of meteorology. When a typhoon arrives along the course of the Kitty typhoon, the front-like rainfall lies, mainly in the offing of Boso penninsula and it is unnecessary to consider a front-like rainfall for the Tokyo district. However, we do have to consider a front-like rainfall when a typhoon approaches first along the course of the 1917 typhoon, and then changes in Tokyo Bay to follow the course of the Kitty typhoon. It would be no exaggeration to say that typhoons arriving along the 1917 course all bring front-like rainfall to Tokyo. The peak of this sort of front-like rainfall is, generally speaking, larger than that of the rainfall when the center of the typhoon is in the vicinity of Tokyo. In our analysis we have regarded high tides and rainfalls as phenomena that superpose with each other, taking into consideration (table 2) the nationwide results and also the fact that massive typhoons of the Ise Bay class are taken on assumption.

TABLE 2. Rainfall amount of continuous hours with the different Typhoon Couresand the other Realationship

(1) Kitty Typhoon Course

	Excess probability Rainfall				actual			Remarks
	Retur Period 30 years	50	100	200	31.9.1949 (Kitty)	23~21.7.1941	30~1.8.1938	
amount of daily Rainfall	196	254	337	438	65 (Extra high tide)	142	134 (Extra high tide)	(notes) 1 unit : mm 2. (Extra high tide) : actual example, Extra high tide coincide with the peak of heavy Rain-fall. 3. preceding Rain-fall : Total amount Rainfall before Continuous 8 hours Rainfall. 4. Daily Rainfall : amount of 24 hours during Extra high tide generated.
Continuous Rainff amount for 3 hours	36	104	143	182	25	12	50	
〃 〃 5 〃	110	141	186	237	37	35	82	
〃 〃 6 〃	120	155	204	261	39	44	95	
〃 〃 7 〃	121	155	205	263	39	48	96	
〃 〃 8 〃	126	161	215	275	40	64	112	
amount of preceding Rainfall	91	116	155	204	26	104	50	

(2) 1917 Typhoon Course

	Excess Probability				actual			Remarks
	Return period 30 years	50	100	200	1~30.9 1917	22~23.7.1958	18~17.9.1958	
amount of daily Rainfall	412	535	698	907	83 (Extra high tide)	121 (Extra high tide)	131	The same as the above.
continuous Rainfall amount for 3 hours	93	118	156	197	18	26	11	
〃 5 〃	139	180	229	191	20	48	18	
〃 6 〃	160	208	266	239	20	50	25	
〃 7 〃	177	230	292	374	21	51	25	
〃 8 〃	199	259	331	423	21	59	35	
amount of Preceding Rainfall 〃	209	296	397	523	73	68	135	

THE RAIN WATER RUNOFF INSIDE THE INTRACIRCLE EMBANKMENTS

The removal of flood water in the Koto delta area is arranged on the basis of an indirect drain formula. For this reason, the volume of water drained by the pumping plant (indirect drain) inevitably becomes qualified by the water drainage capacity of the pumping plants located within the intra-circle embankments. Therefore, as far as the drainage at the time of high tide is concerned, the rain runoff modulus (volume of drain/volume of rainfalls) within the intra-circle embankments should be taken into consideration rather than the rain-water runoff mechanism of the urban area within those embankments. In low-lying urban area, there are lots of aspects of the mechanism of rain-water runoff that remain unclear . In addition, no data in conformity with that in the Koto delta area were available. Therefore, we have had to carry out investigations through the aid of some actual drainage results and urban aerial photographs centered on the model drainage sector (Sannohashi), with the eye set mainly on the runoff modulus at the time of heavy torrential rains. We have chosen to approach the study by seeking data from the pump's own characteristic curve and the consumption of electric power. Although the investigations left something to be desired in respect to the degree of precision, it was possible to acquire a tentative solution in reference to the observations effected in some other sectors of Tokyo. The methods of investigation followed were these three:

(*a*) A method of computation utilizing the pump's own characteristic curve, based on the pumping of a water volume at the model drainage sector (Sannohashi) plus investigations done about the pump's operational condition and the state of rain-water runoff at the relatively massive scale rainfalls during the period ranging from October, 1958 to February, 1959.

(*b*) A method of seeking data from the consumption of electric power of the pumping stations at several different intra-circle embankments. The volume of water drain is

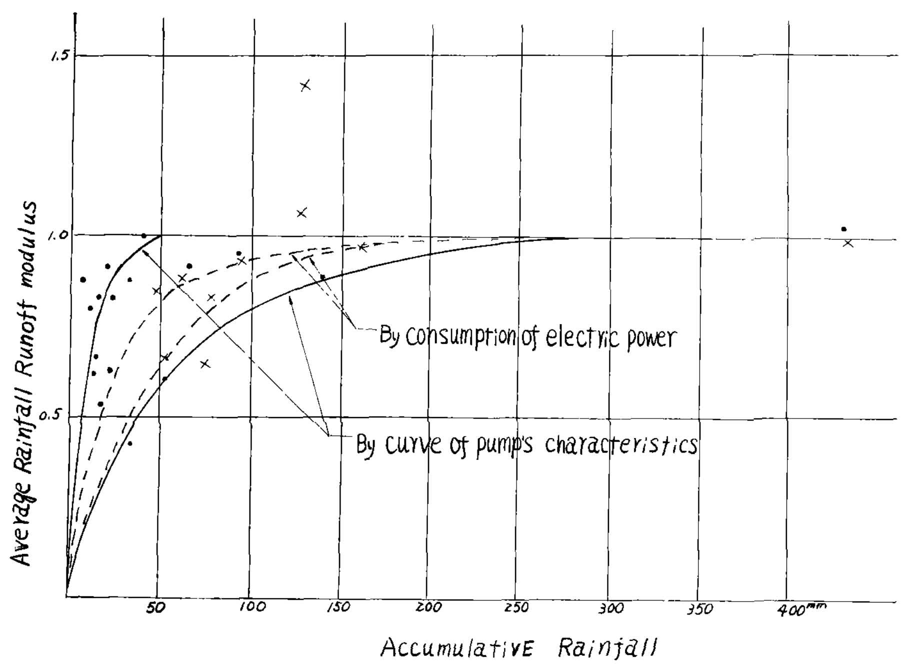

FIGURE 5. *Relationship of average rainfall runoffmodulus and accumulative rainfall*

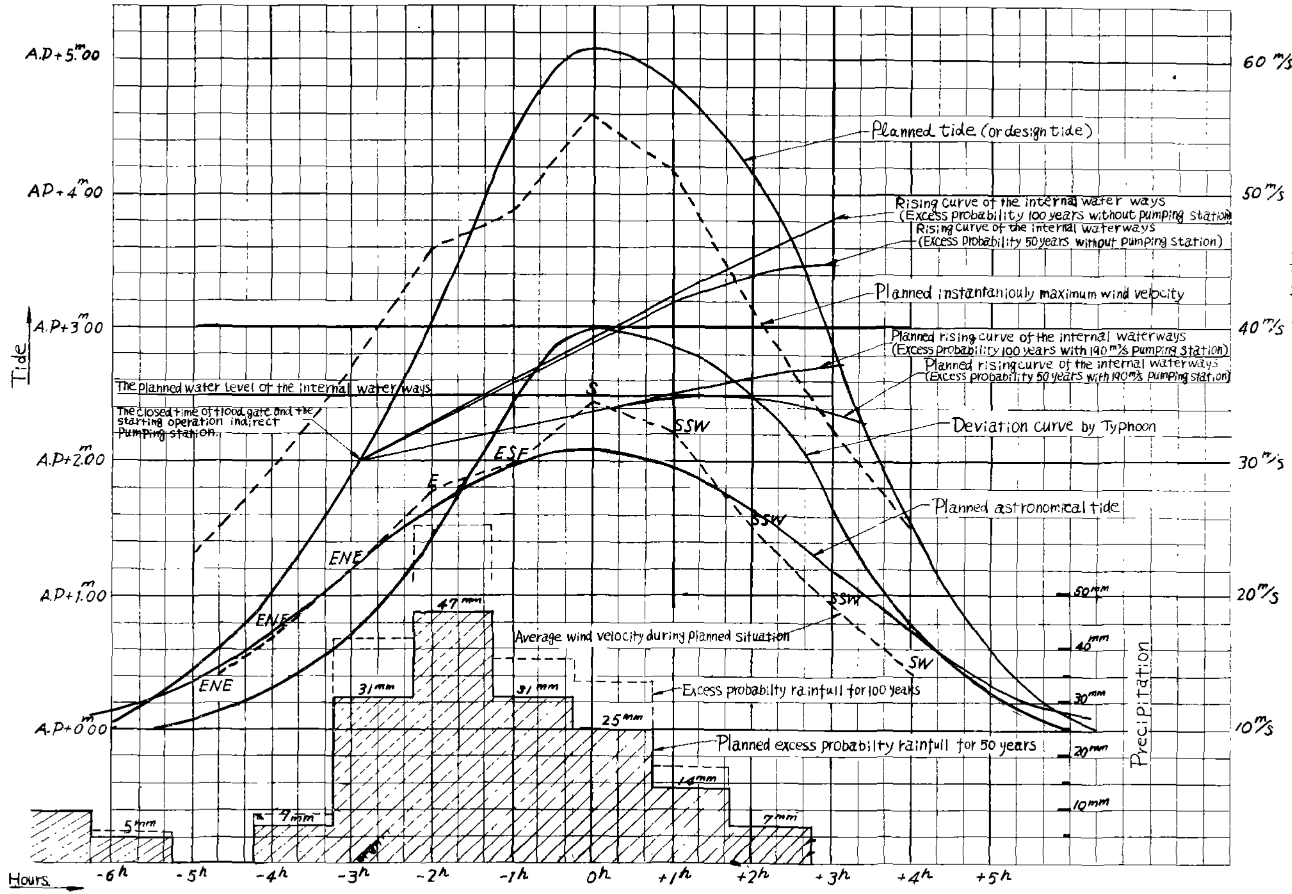

FIGURE 6. *Curves for a planned oceanographi and meteorologic situation*

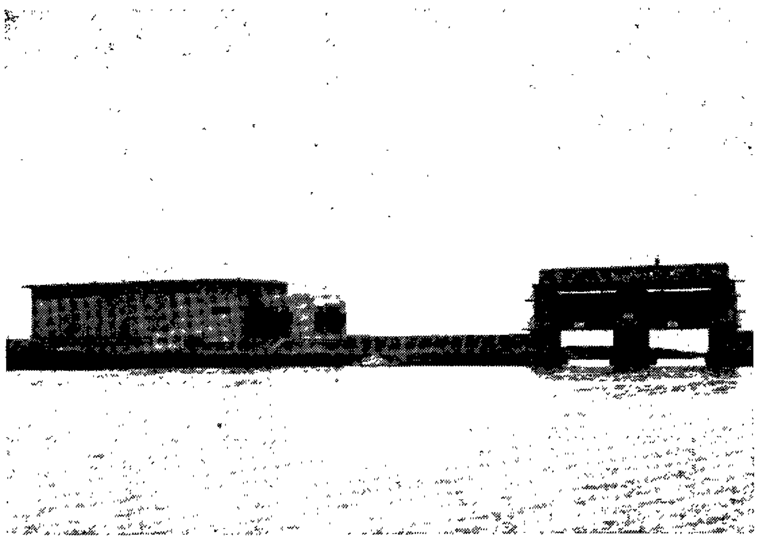

FIGURE 7. *Tasumi flood gate*

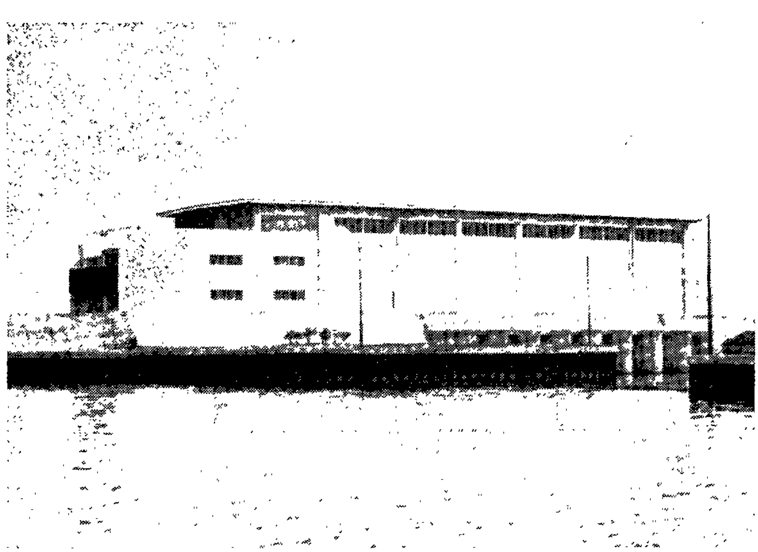

FIGURE 8. *Tasumi indirect pumping station*

usually computed at the pumping stations and is empirically based on the power supply they get. The data which we have used herein are those relating to the period from 1952 to 1955.

(*c*) A method of computation based on a road coverage and a building coverage (through utilization of aerial photography).

The results obtained from the methods (*a*) and (*b*) are illustrated in figure 5. According to figure 5, the runoff modulus is close to 1 when the cumulative volume of rainfall is upward of 150 mm, as found in either of the different methods of computation. We have, therefore, adopted the runoff modulus 1 in our planning, taking into consideration the possible future progress in urbanization and making reference to the data for other sectors of Tokyo. No definite results have been obtained for the hours consumed for the runoff. At the time of rainstorms, this was estimated at around one hour but this lately has had a tendency to grow critically shortened in conjunction with the accentuated modernized buildup and maintenance of the urban areas.

THE FUNDAMENTAL CONDITIONS FOR THE FLOOD WATER DRAINAGE PROGRAM

The basic conditions for the removal of flood water in the Koto delta area have been decided as a result of numerous studies and discussions:

(1) The planned water level of the internal waterways has been set at AP + 2.50 m (H.W.L. 2.10 m), and water beyond this level is prescribed to be drained off by means of pumping operations.

(2) The height for permanent maintenance of the embankments of the internal waterways is set at A.P + 3.00 m, taking into consideration the possible ground-level subsidence and it is ruled that this height should be maintained.

(3) The hours of sluice gate and lock closedown will be set so as to match the tide level of A.P + 2.00 m., in consideration of the surface craft navigation and other allied factors.

(4) High tides and rainfalls are assumed to superpose each other in a most critical condition.

(5) For the volume of rainfall, the one-hour rainfall for the 50-year excess probability was taken as basis and 254 mm of rainfall for 24 hours is set as the planned volume of rainfall. However, the 337 mm rainfalls for 24 hours based on the one-hour volume of rainfall for the 100-year excess probability is taken into consideration for our planning.

(6) The rainfall following the closedown of sluice gates and locks are assumed to be drained off to the internal waterways without any loss.

(7) The planned high tides level of the Koto delta area based on the assumption of Ise Bay-class typhoons is set at A.P + 5.10 m.

What have been computed based on the aforementioned basic conditions are those given in figure 6.

We regret that we are not in a position to reveal the details connected with the actions taken for deciding the basic conditions referred to above because of the limited space available for this paper. Finally, we might state that from September, 1964 to date the sluice gates have been closed on 35 different occasions due to the visits of typhoons, that the indirect water pumping plants (fig. 8) have been put into operation in four cases during the same period, and further that the tide embankments at the coastal sectors are sinking approximately at a rate of 17 to 18 cm annually. Further, we acknowledge that we have been favoured with valuable information from Mr. Ishihara, Engineer of the Government Meteorological Bureau, with regard to the rainfall at the time of the visits of various typhoons.

HIGH TIDES COUNTERMEASURES IN LAND SUBSIDENCE AREA

Hazime TAGAMI, Tatsuo KANNO, Keiichiro TERANAKA and Kôryu KÔNÔ

ABSTRACT

In the low land of Tokyo, where land subsidence is still going on, an area covering 100 km² is already lower than the height of the high tide.

It is therefore of immediate importance to construct dikes and water gates to keep out sea water. The description of the present report is devoted to the procedure of estimating the height of high water taking into account the astronomical and meteorological tides and height of waves. The report also contains explanations of the characteristics of disaster, the practical plan of the countermeasures against storm-surges and that of drainage pumps.

RÉSUMÉ

Dans la région basse de Tokyo où les affaissements continuent, une surface de 100 km² est déjà sous le niveau des marées hautes.

Il est par conséquent urgent de construire des digues et des vannes pour empêcher l'introduction de l'eau de mer lors des hautes eaux. La présente étude expose la méthode d'estimation des hautes eaux en tenant compte des marées astronomiques et météorologiques et de la hauteur des vagues. Le rapport donne aussi certaines indications sur les caractéristiques des désastres, le plan pratique des contre-mesures contre les marées-tempêtes et les pompes de drainage.

1. INTRODUCTION

Occurrences of land subsidence are in industrial districts in Osaka, Hagoya, Niigata and other cities, give rise to various hazards.

Land subsidence in Tokyo is more conspicuous in eastern Tokyo (fig. 1) particularly in Hirai-machi and Kōtō-ku, where cumulative subsidence to 1968 amounted to 4.2 m. Consewuently, the area has become a lowland, with as much as 115 km_z in and around Kōtō-ku under sea level. The area faces Tokyo Bay and is cut across by many rivers which are influenced by tides in the Bay.

In spite of the levees constructed to control high tides, the area has suffered flood damage, from high tides in the Bay, many times in the past. On the other hand, the high tide control project to construct tide embankments and sluices as countermeasures against immediate damage caused by high tides has undergone changes as natural conditions and requirements altered with time. The project target shifted from Typhoon Kitty Class (tide level A.P. + 3.15 m) in 1949, to 1917 Typhoon class (tide level A.P. + 4.12 m), and again to Isewan Typhoon class (tide level A.P. + 5.10 m).

The typhoon of 1917 brought with it a record high tide in Tokyo. Recently however, Nagoya was hit by a typhoon which brought a high tide of greater magnitude. This was the Isewan Typhoon which struck the Kii Peninsula in September 1959. This rare extra-large typhoon caused an unprecedently high tide (A.P. + 5.02 m) in Nagoya Port, causing tremendous damage to Nagoya City and other coastal areas. In Nagoya alone, casualties amounted to 42,400 people, of which 1,900 were dead or missing.

Considering the importance of the metropolis, future urban development, the kinds of damage incurred, and the expanding areas under land subsidence pressure, the Tokyo Metropolitan Authorities decided to construct permanent tide embankments and sluices to resist high tides caused by typhoons, on the assumption that a typhoon of the Isewan class

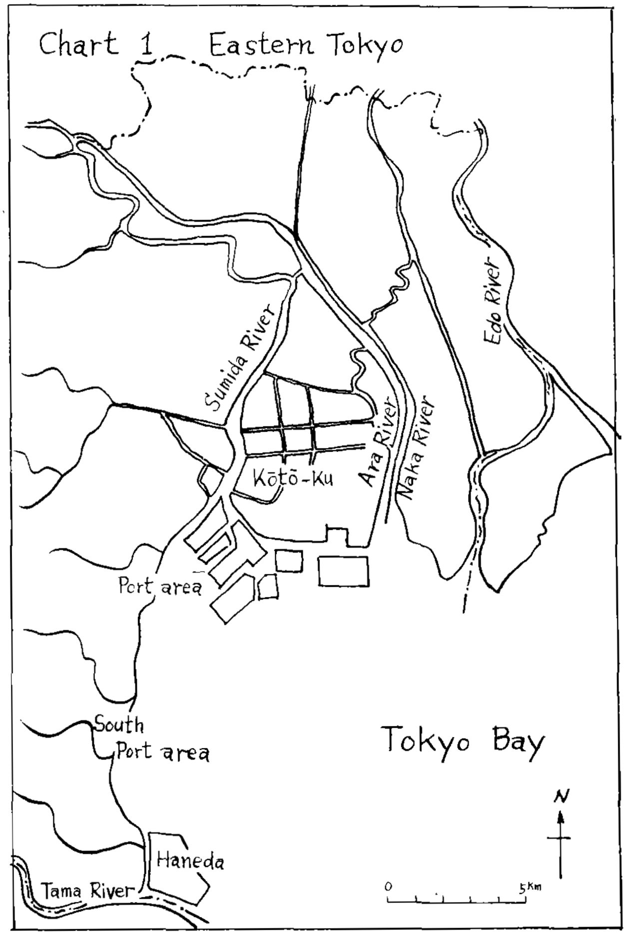

FIGURE 1. *Eastern Tokyo*

might come to Tokyo Bay. Thus the object of the project was switched to the Isewan typhoon class and the project started in 1963 on a full-fledged scale.

In order to determine the estimated height for tide embankments as described above, one must consider astronomical tide levels, meteorological tides caused by typhoons, the retrogressive flow of high tides in rivers, wave range, and subsidence.

In the following, an account is given as to the method of determining the estimated height of tide embankments. A brief summary of the high tide countermeasures project as implemented in Tokyo is given.

2. ESTIMATED HIGH TIDE LEVEL

Estimated high tide level, which is the basis for the determination of the estimated height of tide embankments, was assumed to be the sum of the celestial tide level and that of the meteorological tide caused by the typhoon. Further, in relation to the upper stream of the rivers, retrogressive flow height of the high tide was considered.

Table 1 Parameters of the Tide Embankments

		Astronomical Tide Level	Meteorological Tide Level (Deviation)	Retrogressive Flow Height	Wave Height	Height of Tide Embankments	
						Maintained	Constructed
Rivers	Sumida	A.P. + 2.10	3.00	0 -- 0.30	0.90 -- 1.20	A.P. + 6.30	A.P.+6.40--6.90
	Old Edo			-	0.90 -- 2.90	A.P.+6.00--8.00	A.P.+6.50--8.40
	Sumida (brnach)			-	0.50	A.P. + 5.50	-
	Ara			1.70 -- 2.90		A.P.+6.80--8.00	
	Naka			0.30 -- 0.80	1.30 -- 1.90	A.P.+7.10--7.30	A P.+7.90--8.30
		Brimming water level	A.P. + 4.00	-	1.00*	A.P. + 5.00	-
	Ayase		A.P. + 4.00	-	1.00*	A.P. + 5.00	-
	Internal Rivs in Koto Dists.		A.P. + 2.50	-	0.50*	A.P. + 3.00	A.P. + 3.60
	Shin		A.P. + 3.00	-	0.50*	A.P. + 3.50	A.P. + 4.00
	Furu	A.P. + 2.10	2.50	-	0.50	A.P. + 5.10	-
	Meguro		2.00	-	0.50	A.P. + 4.60	-
	Tachiai			-	0.50	A.P. + 4.60	-
	Nomi			-	0.50	A.P. + 4.60	-
	Ebitori			-	0.50	A.P. + 4.60	-
	Uchi	High water level 3.00 -- 3.50		-	0.60*	A.P.+3.60--4.10	-
Coasts	Kasai	A.P. + 2.10	3.00	-	0.50 -- 1.50	A.P.+5.60--6.60	A.P.+6.40--7.50
	Koto Dist.			-	0.50 -- 2.90	A.P.+5.60--8.00	
	Tsukishima Harumi Dist.			-	0.50 -- 1.20	A.P.+5.60--6.30	
	Minato		2.50 --3.00	-	0.50 --1.20	A.P.+5.10--6.30	
	Konan		2.00	-	0.50	A.P. + 4.60	

unit: meter * Allowance Height

2.1 Astronomical tide level

The tide level differential between flow and ebb tides in Tokyo Bay is in the vicinity of 2 m in the spring tides. As for high tides due to typhoons, it was assumed that this high tide coincides with the flow tide, for safety, and the flow tide levels at new moon and full moon during typhoon season (July-October) were taken as the astronomical tide level to be used in the project.

That is to say, new and full moon flow tide levels during typhoon seasons for 9 years beginning from 1951 through 1959 were obtained, first, then the cumulative annual mean tide level for mean tide levels at both new moon and full moon during each typhoon season were calculated. The mean values for the above two values were also computed.

As a result of the above computation, new and full moon mean flow tide levels during the 9 years from 1951 through 1959 were found to be A.P. +2.10 m, which was adopted as the final figure (table 1).

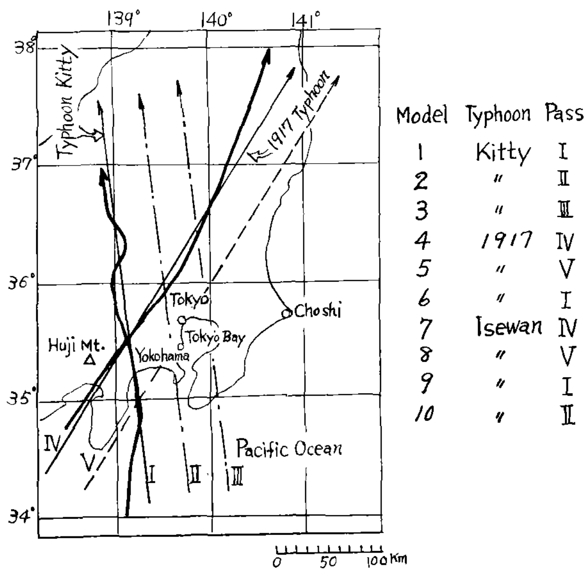

FIGURE 2. *Typhoon pass used for calculations*

Model

1. *Typhoon Kitty's Pass*
2. *Typhoon Kitty deviating its pass by 40 km east*
3. *Typhoon Kitty further deviating its pass by another 40 km east*
4. *1917 Typhoon Pass*
5. *1917 Typhoon deviating its pass by 40 km east*
6. *1917 Typhoon following Typhoon Kitty's Pass*
7. *Isewan Typhoon following 1917 Typhoon Pass*
8. *Isewan Typhoon following the same pass on Model 5*
9. *Isewan Typhoon following Typhoon Kitty's Pass*
10. *Isewan Typhoon following the same pass as Model 2*

2.2. METEOROLOGICAL TIDE LEVEL

Meteorological tide level is an increase in tide level due to meteorological phenomena such as typhoons. Some of the major meteorological tide levels recorded in Tokyo Bay in the past were 190 cm (Komatsu gawa) in December 1917, 220 cm (Komatsu gawa) in September 1938, 141 cm. (Reiganjima) in August, 1949.

Therefore, the Meteorological agency and Tokyo Metropolitan Government jointly conducted research on forecasting the meteorological tide which might be created at various spots in the Bay if a typhoon of similar magnitude to the Isewan Typhoon occurred.

The research method involved selection of the 1917 typhoon, Typhoon Kitty, and the Isewan Typhoon of 1959 as model typhoons, and computation of high tides as these typhoons go through 10 model passes (fig. 2).

Prior to computation, theoretical analysis included movement equations followed by examination of conditions at the time of typhoon attacks in the past, development of empirical formulas, and subsequent determination of elements necessary for high tide computation. Elements thus identified were fed into an electronic computer for computation.

The sea area under consideration covered all of Tokyo Bay north of Kurihama-Kanaya. The area was broken up into a grid with a fixed interval (1.5 km) between points, and the increase in sea level from mean sea surface at each grid point and the accompanying current were obtained.

The above computation gave us the maximum meteorological tide within the Bay, with an Isewan-class typhoon following a 1917 typhoon pass (model 7), corresponding to a high of 302 cm at Inage. The maximum value near Tokyo was obtained when an Isewan-class typhoon followed a Typhoon Kitty pass (model 9), corresponding to 233 cm recorded at Yumenoshima.

In the above computation, however, 3 kinds of typhoons and 5 typhoon passes which would contribute to substantial meteorological tides were selected and combined to give 10 model typhoons. Moreover, typhoon velocity, atmospheric pressure and wind direction were adjusted to facilitate computation. The computation is extremely limited in range and includes the following errors.

(1) Shifting the direction of Typhoon Kitty pass slightly allows a meteorological tide increase of some ten cm.

(2) Errors in various assumptions used in computation.

(3) Errors between observed results at known high tides and computed results.

Known typhoons	Observed results	Computed results
October 1, 1917	+2.12 m (Reiganjima) +1.90 m (Komatsu gawa)	+1.50 m (Tsukiji) +1.98 m (Yumenoshima)
August 31, 1949	+1.41 m (Reiganjima)	+1.69 m (Yumenoshima) +1.65 m (Tsukiji)

(4) According to on-site observation at Kurihama, a tide level differential in the vicinity of 0.8 m occurred due to mass transportation by waves between the near-store area and the area a bit further offshore. Consequently, the increase in tide level along a shallow coast should be considered with respect to Tokyo Port as well.

(5) A maximum value of 3.02 m for a meteorological tide recorded in the Bay when an Isewan-class typhoon followed a 1917 Typhoon pass implies that such a meteorological tide is possible in Tokyo which lies at a relatively short distance from the site.

Considering the above, the meteorological tide was determined as 3.00 m. As for the port area and south port area, the meteorological tide actually decreased from the inner end of the Bay toward the bay entrance.

2.3. Retrogressive flow height

The Public Works Research Institute of the Ministry of Construction calculated retrogressive flow at high tide in the Tama River, Ara River, Naka River, Edo River and Tsurumi River, considering the calculation interval, mean river bed height, wind, natural flow, and river mouth conditions.

These rivers, which are under different conditions, showed differences in retrogression. To take the case of the Ara River, the river has a whole runs southeast and the river mouth opens southward. An Isewan Typhoon is characterized by long wind duration, particularly in a southeast-south direction. Because of this, the peak area of the wave profile grows toward the upper stream, resulting in differential as much as 50 cm between the river mouth and Iwabuchi upstream, where the tide is higher.

Since a maximum wind velocity for typhoons in Tokyo is often recorded when the wind is southerly—if we consider the above computed results in this connection, an estimated retrogressive flow height was given for the Ara, Naka and Sumida Rivers (table 1). On the Naka River a sluice was to be constructed 7.5 km from the river mouth, retrogressive flow at the time of sluice closure was computed.

3. WAVE RANGE AND LAND SUBSIDENCE

At the Public Works Research Institute, Ministry of Construction experiments using wave models were conducted in relation with the upper Ara River and Naka river. The experiment was made to show the state of waves when a high tide strikes. Wave range was measured on a 1/60 model as wave injection direction was altered.

On a coastel line which faces and receives them, waves were computed, using the maximum wind velocity of the Isewan Typhoon (40 m per sec) and 40 km fetch (Cape Futsu, Chiba Prefecture), giving a maximum value of 2.9 m. From these experiments and calculations, wave range was calculated (table 1).

3.1. Land subsidence

In constructing tide embankments in areas with conspicuous subsidence, the amount of anticipated subsidence is vital. However, there are many uncertain factors involved: the subsidence area covers a wide area and tide embankments were constructed over a number of years; estimation of, and allowance for, subsidence cannot be generally assumed.

Therefore, tide embankments were designed to allow addition of greater height. In implementing the design, embankments were raised by a height corresponding to subsidence during 5 years. The amounts are varied over the areas, from a minimum of 10 cm to a maximum of 90 cm (table 1).

4. ON MAINTAINED HEIGHT AND CONSTRUCTED HEIHGT

We have discussed astronomical tide level, meteorological tide level, retrogressive flow height, wave range and land subsidence. The maintained height (estimated height) of tide embankment is the height exceeding astronomical tide level, meteorological tide level, retrogressive height and wave range. In order to combat high tides on the order of those of the Isewan Typhoon, this height should be maintained.

In areas subject to land subsidence, the maintenance height is increased by land subsidence as well as compaction due to the weight of embankment construction.

When allowance is not made for land subsidence and compaction, the maintenance height is equal to constructed height. Maintenance height and constructed height are as shown in table 1.

5. SUMMARY OF THE TOKYO HIGH TIDE COUNTERMEASURES PROJECT

Rugged tide embankments are constructed along the coastline from Haneda to the Old Edo River as well as the Sumida, Ara, Naka and other rivers that flow into Tokyo Bay, and sluices are provided at the entrance of each river or waterway. Pumping stations are constructed in order to drain the river water inside the tide embankments and sluices over the embankments during high tides.

In implementing the project, the government agencies concerned have their own respective area of jurisdiction: The Ministry of Construction is in charge of the Ara River and the Tama River, while the Construction Bureau of the Tokyo Metropolitan Government is responsible for the Sumida River, Naka River, Old Edo River and other rivers and Kasai Beach. The coastline from Haneda to the Ara River is under the jurisdiction of the Port and Harbor Bureau of the Tokyo Metropolitan Government.

As for the overall scale of the project, it includes 253 km of tide embankments, 41 sluice units, and 9 pumping stations. So far 70 km of tide embankments, sluice units, and 2 pumping stations have been completed. In Kōtō ward with a high incidence of land subsidence, a 3-year Emergency Plan (1963-1965) was planned by the Ministry of Construction and the Tokyo Metropolitan Government to promote the project. Due to these efforts, the area is now completely safeguarded against high tides on the order of the Isewan Typhoon tides.

However, the project has not been completed along the coastline and upper streams of the rivers in the port area and south port area. In the future, the project in the remaining areas will concentrate on building tide embankments in the outer areas which are more exposed to the effects of high tides. The project is also currently being promoted as an integral part of the Tokyo Medium Range Plan of 1968.

6. CONCLUSION

Subsidence is measured every year for tide embankments completed so far. The results show that subsidence has been continuous. In some places it is forecasted what the maintained height would not be kept in the future.

Therefore, to maintain the required tide embankment height, obtaining accurate prediction of the final amount of subsidence and countermeasures to it are vital problems to be solved in the future.

However difficult the problems may be, good results are expected from various subsidence control measures developed recently as well as from the many studies on subsidence.

EXPERIMENTS ON WATER INJECTION IN THE NIIGATA GAS FIELD

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Geological Survey of Japan

ABSTRACT

Experiments of water injection against the natural gas reservoirs of "dissolved-in-water type" (confined aquifers with dissolved methane gas) were carried out in Niigata City from 1960 to 1963. The purpose was to study field practice of water injection which aimed at the maintenance of the reservoir pressures. Both degassed formation water drained from a gas-water separator and surface water taken from a river were used for injection fluids. The results are summarized as follows: (*a*) permeability of the main reservoirs ranges from 10 to 50 darcys, (*b*) injectivity index is, in general, less than a quarter of productivity index, and (*c*) back-washing at adequate time intervals is necessary to continue long-term injection.

RÉSUMÉ

Des expériences d'injection d'eau dans les réserves de gaz naturel du " type dissous dans l'eau " (couches artésiennes avec gaz méthane dissous) ont été réalisées à Niigata de 1960 à 1963. Le but en était d'étudier l'utilisation d'injections d'eau pour essayer de rétablir la pression dans la couche aquifère. On utilisa à cet effet tant de l'eau provenant de la couche et dont le gaz savait été séparé que de l'eau de rivière. Les résultats peuvent être résumés comme suit : *a*) la perméabilité du réservoir souterrain variait de 10 à 50 darcys, *b*) l'indice d'injection était, en général, moins du quart de l'indice de productivité et *c*) des lavages en direction inverse à certains intervalles de temps sont nécessaires pour continuer la recharge à long terme.

I. INTRODUCTION

The Niigata gas field produces methane gas dissolved in water. At the beginning of 1960, the output of water from gas-water separators totalled more than 450,000 kl a day. The experiments on water injection were intended to reduce the water output from the gas reservoirs, i.e., aquifers containing gas in workable quantities.

The purpose of the experiments was to reveal

(i) the actual field practice;
(ii) the relation between reservoir properties and injection behaviour;
(iii) the relation between production and injection capacities;
(iv) channelling, and
(v) the influence on the water level.

The experiments were started in 1960 and completed in 1963. They were divided up into two stages: the first was undertaken in the Uchino area, west of Niigata City, and the second in the Niigata harbour area. The most important difference between them was the nature of the injected waters. In the former, degassed formation water drained from a gas-water separator and containing nearly no oxygen was used for injection, while in the latter river water saturated with oxygen was used.

The paper chiefly deals with the results obtained from the experimental water injection in the Nügata harbour area.

II. HYDROGEOLOGY

The majority of gas reservoirs in the Niigata gas field belong to the Uonuma group, assigned to Pleistocene in geological age by radiometric dating and mammalian fauna. Lithofacies of the group are characterized by the alternation of a small sedimentation cycle, which consists of clay, siltstone, sand, and gravel beds. The sedimentary environ-

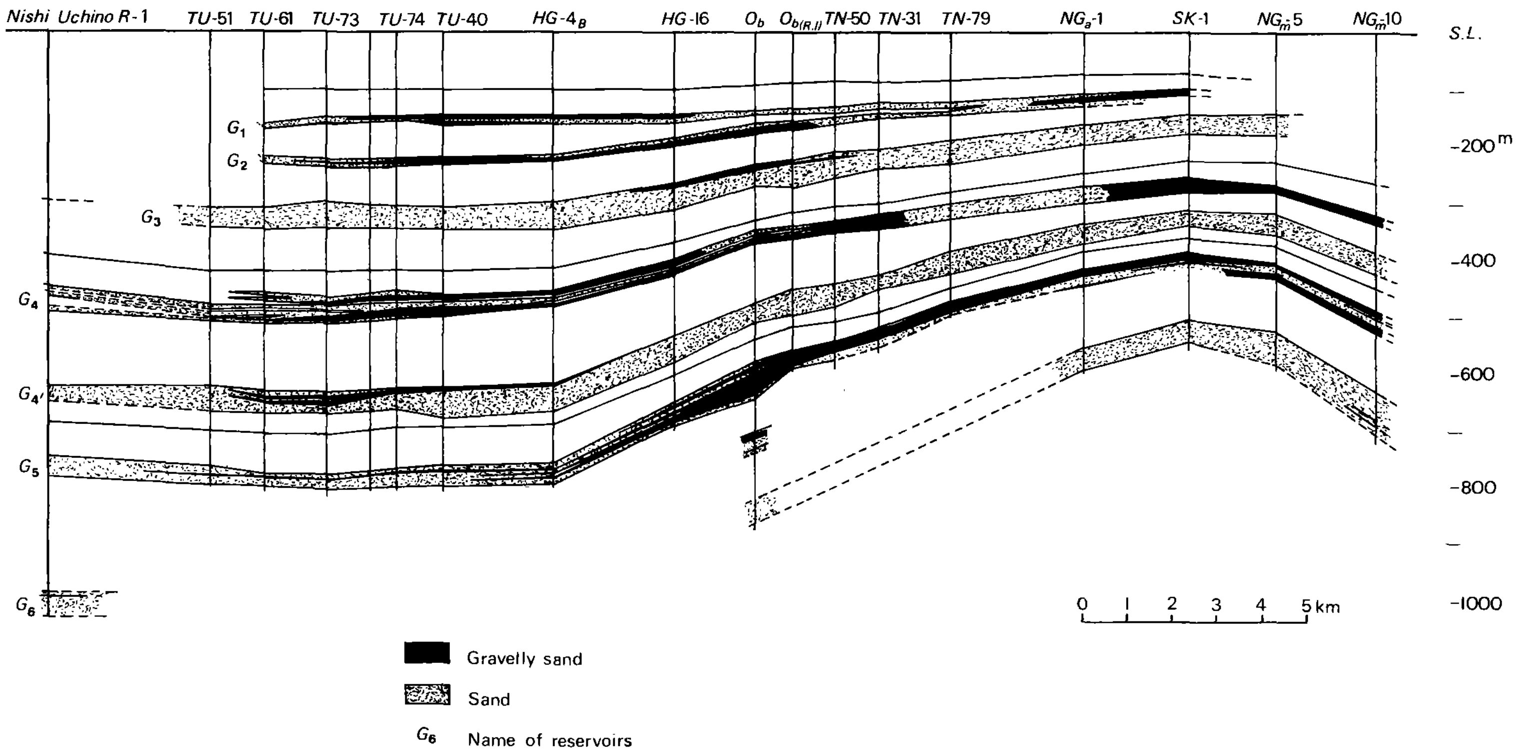

FIGURE 2. *Geological profiles at* (a) *Kameido and* (b) *Minami-Sunamati observation stations*

ment may be non-marine and shallow marine alternately. Gas reservoirs, i.e., confined aquifers consisting of sand and gravel, are filled with brackish to saline connate water characterized by strongly reduced condition. The surface gas-water ratio approximates roughly to the solubility of methane in water under the reservoir conditions. Although development of the field has been limited to the coastal zone, the reservoirs can be considered to extend widely over the Niigata Plain. Figure 1 shows the gentle structure of the Uonuma group under the Plain. Figure 2 shows the geological section along the sea coast.

III. EXPERIMENTAL METHODS

1. Installations

The injection system is made up of several units. These are used for pumping original river water, pretreating injected water, injection, measurement and back-washing.

2. Origin and properties of injected water

The water is taken from the harbour, i.e., the mouth of the Shinano river, by means of four submersible bore-hole pumps. The pump intakes are set at about three meters below the water surface. The properties of the water in the harbour vary considerably with the season and marine conditions outside the harbour. An example of analysis is as follows:

Chloride ion	105 ppm
$KMnO_4$ consumption	17 ppm
Alkalinity	24 ppm
pH	6.8
Turbidity	30 degrees.

The average concentration of chloride ion in the injected water was 800 ppm throughout the experiments. The most worrying suspended matter was fibres from a pulp factory.

3. Pretreatment of injected water

At first, chlorine gas, aluminum sulfate (10% solution) and sodium carbonate (10% solution) were added to the pumped river water. The flocks obtained were precipitated in a settling basin (30 × 30 × 4 m). Crude water was then purified by passing it through sand filters.

4. Injection wells

In order to detect the break point of wells, the spontaneous potential was measured in bore-holes. As a result, six pre-existing gas wells were selected as injection wells. (In the case of Uchino area, a spinner flow meter was used for break point detection.) In the course of the experiments, three wells were abandoned, and finally three wells were retained as injection wells until operation was completed. The characteristics of these three wells are as follows:

Well	Producing Zone	Total Depth (m)	Cement Hole (m)	Perforation (m)
J-5	G_5	627.86	—	545.89-584.98 597.16-615.14
J-7	$G_{4'}$	514.70	437.36	464.00-506.58
M-9	G_5	620.00	532.09	570.10-616.57

Diameter of casing pipe and depth:
J-5: 244.5 mm × 596.75 mm, 168.3 m × 37.38 m
J-7: 219.1 mm × 514.70 m
M-9: 244.5 mm × 616.57 m

A pipe 2 inches in diameter was installed in each well for air lift back-washing. An air compressor of 30 HP/7 kg/cm^2 was also prepared for back-washing.

5. Pumping devices

In the early five months of the operation the injection was carried out without pumping. During the eight following months, submersible bore-hole pumps were used to increase the injection rate.

Pump diameter	150 mm
Delivery capacity	4 cu.m/min.
Head	40 m
Power	41 kW
Rotation	2,900 rpm

Injection by pumping was carried out at 2 kg/cm^2 of well head pressure.

6. Observation

The pH, chlorinity and turbidity of the injected water were monitored continuously. For measurement of the injection rate, an orifice flow meter with electronic recorder and electromagnetic flow meter were used. Changes in water level due to injection or production (during back-washing) were recorded at adjacent observation wells by automatic water-level recording.

IV. RESULTS

1. Permeability and compressibility

Judging from several records, the permeability of reservoirs in the Niigata gas field may range from 10 to 50 darcys. An example of estimating the permeability and compressibility of the G_4 zone is as follows:

Mean injection rate	$q = 5.5 \times 10^4$ cc/sec at J-7
Viscosity	$\mu = 1.0$ cp
Porosity	$f = 0.3$ (assumed) tuffaceous sand
Well spacing	$r = 5.6 \times 10^4$ cm between J-7 and N-10
Effective thickness	$h = 4.6 \times 10^3$ cm from electric logging
Pressure rise observed at time t by N-10 well	ΔP atm
Permeability	k
Compressibility	c

k and c are easily calculated from a pressure build-up curve based on the following equation:

$$\Delta P = \frac{q\mu}{4\pi kh} Ei\left(-\frac{f\mu c r^2}{4kt}\right).$$

The calculated values are: k approximates 16.6 darcys and c 2.7×10^{-4} atm^{-1}.

2. Injection performance and clogging

In the case of J-7 well, the mean injection rate was 1,232 kl a day (well head pressure 0.2-0.3 kg/cm^2) or 2,700 kl a day (well head pressure 2 kg/cm^2). Figure 3 shows the

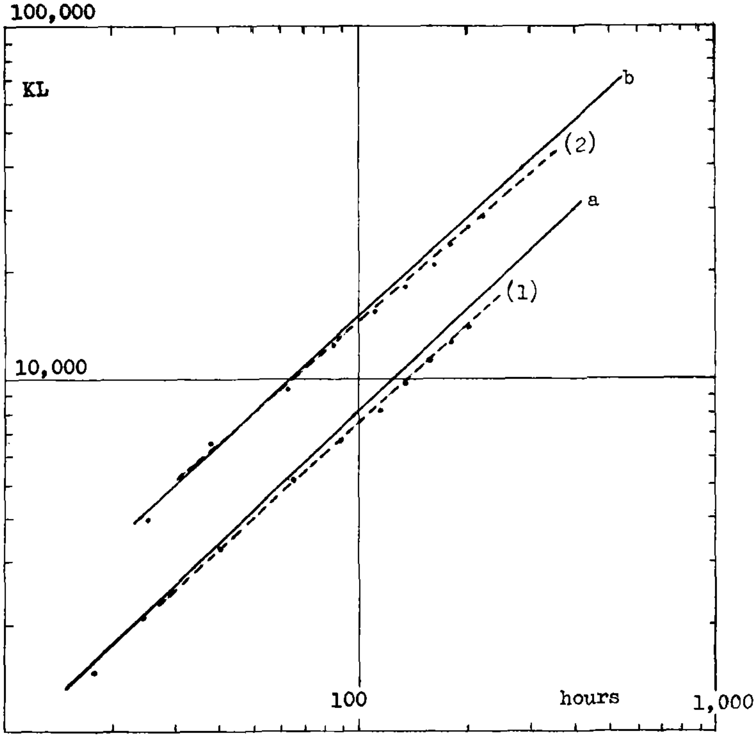

FIGURE 3. *Changes in yearly mean ground water level at Kameido and Minami-Sunamati observations stations*

performance curves of J-7 injection. On this figure the broken lines are actual field data. The apparent permeability calculated from the earlier portion of the curves is about 4.5 darcys.[1]

The fact that apparent permeability observed in injection wells is usually considerably below that in production wells seems to be characteristic of the field. In other words, the injectivity index (flow rate per unit pressure differential) is far less than the productivity index. The former is, in general, a quarter of the latter. This phenomenon may probably suggest the presence of a skin effect around a bore-hole.

In the case of the G_5 zone of the Uchino area, a high pressure injection test was tried.

Pressure differential	Injectivity index
15.3 kg/cm^2	77 kl/day/kg/cm^2
30.3 kg/cm^2	58 kl/day/kg/cm^2

A bottom-hole pressure survey of a well completed in the same zone of the same area showed an injectivity index of 680 kl/day/kg/cm^2 at a pressure differential of 3.7 kg/cm^2, while another well recorded a productivity index of 2,780 kl/day/kg/cm^2. Thus, the smaller the pressure differential, the larger the productivity index becomes.

The decrease in injection rate for using cumulative injected volume is fairly similar to the drop in flow rate calculated on the basis of apparent permeability derived from the injection well. On Figure 3, injection performance of J-7, permeability varies from 4.5

1. The computation was based on the transient radial flow equation of constant terminal pressure case of Van Everdingen & Hurst (1949).

darcys at the beginning of injection to 3.4 darcys after 210 hours. This difference may be imputed to the effect of clogging. If there is any clogging, it might be caused chiefly by the formation of precipitate of ferric hydroxide.

When injection rate decreases, it is easy to recover injection capacity by air lift back-washing. The interval back-washing generally ranges from 10 to 20 days. The time necessary for back-washing is normally less than 10 hours and exceptionally more than 20 hours in the case of loose sand reservoirs, such as J-7 well.

3. Channelling

Surveys for the channelling of injected water to wells in production were studied only in the Uchino area, because there was no producing well in the harbour area.

In order to detect the breakthrough of injected water at producing wells, changes in salinity and gas-water ratio were continually observed, it being found that no extremely marked horizontal channelling took place. But when a highly permeable layer is inserted in the reservoir (such as G_5 zone in the Uchino area), the front of injected water foregoes within this layer.

V. CONCLUSION

The Pleistocene reservoirs of the Niigata gas field have high permeability and fairly high compressibility.

The injectivity index is usually far less than the productivity index, and may be a proof of the presence of skin effect.

In the Niigata gas field long-range water injection is possible by back-washing at adequate time intervals, even if oxygen bearing water is injected into reservoirs under reduced conditions.

DISCUSSION

Intervention by Mr. Jose G. Mendez (Venezuela):

Question:

I would like to know how many wells do the Niigata Field had and what is the average daily production of Gas?

Answer of Dr. Ishiwada:

In Niigata, the number of producing wells is 116.

Intervention of Dr. Manuel N. Mayuga (USA):

Question:

We noted with interest your curve that shows, as you injected water you increased pressure and then your gas/water ratio went down. In Wilmington Oil Field, we noticed the same thing, where, as the pressure was raised, the gas/oil ratio has gone down considerably. Now, in your case what happens to the gas? Is that left there or do your produce remaining water to get the gas left there? And what is your course of action?

Answer of Dr. Ishiwada:

As you know in our case, gas is completely desolved in water.

PROBLEMS ON GROUNDWATER CONTROL IN TOKYO

Shigeru AIHARA, Hiroshi UGATA, Kanji MIYAZAWA and Yutaka TANAKA

ABSTRACT

As a result of long term scientific research on land subsidence, the excessive withdrawal of ground water is thought to be the main source of the development of land subsidence. In order to prevent disasters due to the development of the land subsidence, the Metropolitan Government has put into effect the control pact for the excessive withdrawal of the ground water in accordance with a certain standard. There is, however, some difficulty in the maintenance of proper height of the ground water level to make effective the control of land subsidence, under the ground water control pact.

RÉSUMÉ

Le résultat d'une recherche scientifique à long terme sur la question des affaissements est que les pompages excessifs d'eau souterraine sont considérés comme la cause principale de ces affaissements. Afin de prévenir les désastres à craindre du développement de ces affaissements, le Gouvernement Métropolitain a mis en œuvre un contrôle standardisé des pompages excessifs. Il n'en reste pas moins que certaines difficultés subsistent pour maintenir un niveau convenable et pour rendre effectif le contrôle des affaissements.

SUMMARY

With the advance in every field of activities in the urban area, centered around the activity in industrial production, land subsidence of Tokyo has gradually intensified. At present this trend of subsidence is in progress over a vast area.

To cope with this problem systematically, metropolitan Tokyo has been taking action since pre-War years on the flood tide measures and some other allied precautions. With the advent of the 1960's, the Metropolitan Government started some specific measures to restrict the extraction of the groundwater, which has been found to be a vital cause of land subsidence. However, the current restrictive measures dealing with groundwater leave some problems that await positive steps for seeking their solution in respect to the area to be covered by the restrictive measures, the restrictive limit on depths, and the *industrial* water works designed to supply substitute water.

In order to put an end to land subsidence, it is essential to further strengthen the study and research efforts on the geological aspect of the soil, the aspect of groundwater, etc. The study of comprehensive, effective measures based on the result of such research efforts, taking into consideration various social factors, also is necessary.

FOREWORD

In the early days when the largest of rivers in Japan—the Tonegawa river—ran across the Kanto plain, there lay a flood-bound field in the valley of this river, which presently contains a good part of Tokyo, with a downtown sector having a population of 3.3 millions. This low land is favoured with benefits that permit ease of surface navigation. Thus, the Koto area (a coastal sector), alongside the Johoku area (inner sector of Tokyo) forms an idustrialized zone that is continuing to grow.

However, the inordinate utilization of groundwater, spent in attempting to secure a adequate supply of water, had the effect of not only exhausting the groundwater but also of aggravating the progress of land subsidence. It was around 1955 that Tokyo succeeded in rehabilitating itself from the destruction and chaos wrought by World WarII, and Japan came to enjoy the situation that heralded the high, monumental economic growth that ensued. The aggravation of land subsidence began to call attention and metropolitan Tokyo thus began positive efforts in 1961 to control the withdrawal of underground water, which was believed to be the major cause of subsidence. Along with these controls messures to protect the subsiding area from the danger of flooding were indicated. However, the problem of the low area, thar continues subsiding lies in the fact that social factors rooted in the industrial development in Tokyo, the geological quality of this low-level area, and the structure of the groundwater are interlocking with one another in a highly sophisticated manner.

There are grave difficulties in the administrative efforts to analyse the above factors appropriately and to put into action the numerous measures designed to scientifically counter the negative factors in an efficient and adequate manner. This paper is a report on some discussions centered around some administrative cases oriented to such problems as described above, and also on some discussions made in relation to the possible future trend of such problems.

INDUSTRIAL ACTIVITIES IN THE LOW AREA OF TOKYO AND THE LAND SUBSIDENCE

The subsidence phenomena of the Koto delta area, where no fewer than 58 sq km are below the zero sea-level, are the ones discovered 45 years before in the history of flood disasters. Since then, the trend of subsidence has been continuing to show an exceptionally sharp relation to the rise and fall of the industries which are located there and to the changes in various conditions that encompass those industries. The subsidence phenomenon was particularly notable in the flowering of industrial production that marked the 1930's. As a matter of fact, the annual size of subsidence amounted to as much as 100 mm in some sectors. On the other hand, when production activities in Tokyo were paralyzed the subsidence was stopped completely.

Tokyo's industrial rehabilitation, associated with Japan's high economic growth, has made a rapid march since 1950, coupled with the centralization of management mechanism functions in Tokyo. In conjunction therewith, the growing demand for industrial water and the demand for water for the heating and air conditioning systems in many buildings have combined to require additional supplies of groundwater. As a result, in sectors of high density, ruthlessly excessive water withdrawal was allowed to persist. In consequence, the rate of subsidence went far beyond that of the pre-war years. The sectors whose annual subsidence amounted to upward of 100 mm grew to an area of 47 sq km (a value twice that of previous years), and this trend expanded from Koto to Johoku.

The whole of 290 km^2 of the low area is already counted as subsidence sectors, though one sector more or less differs from another in the degree of subsidence.

Both industrialized zones contain altogether 40,000 factories with 570,000 workers employed. Their shipments of manufactured products are valued at approximately 2,000 billion yen in money representing 40 per cent of the industrial activity in Tokyo. The total water comsumption is shared by fresh water to the extent of 1.25 million m$_3$/day by drained underground water withdrawal. Those industries of a type relying on a supply of industrail water occupy 80 percent of the city's requirement for water, forming its principal structure of industry.

As table 1 shows, though there are some difference in the types of major trade followed in both industrialized zones, they present no material difference in respect to the demand structure classified by purpose and also by water source. However, what is to be noted in this connection is the fact that the Johoku district requires water of a relatively superior quality.

TABLE 1. The main type of industry in the "Koto and Johoku" areas

Koto	Koto-ku Sumida-ku Arakawa-ku Egadowa-ku	Electricity Gas Iron and steel Fabricated metal products Machinery Chemical industries
Johoku	Itabashi-ku Kita-ku Katsushika-ku Adachi-ku	Pulp and paper Drugs Food and kindred products

In the present paper we have deliberately avoided a detailed explanation regarding the production activities of the industries involved, the volume of groundwater withdrawal,

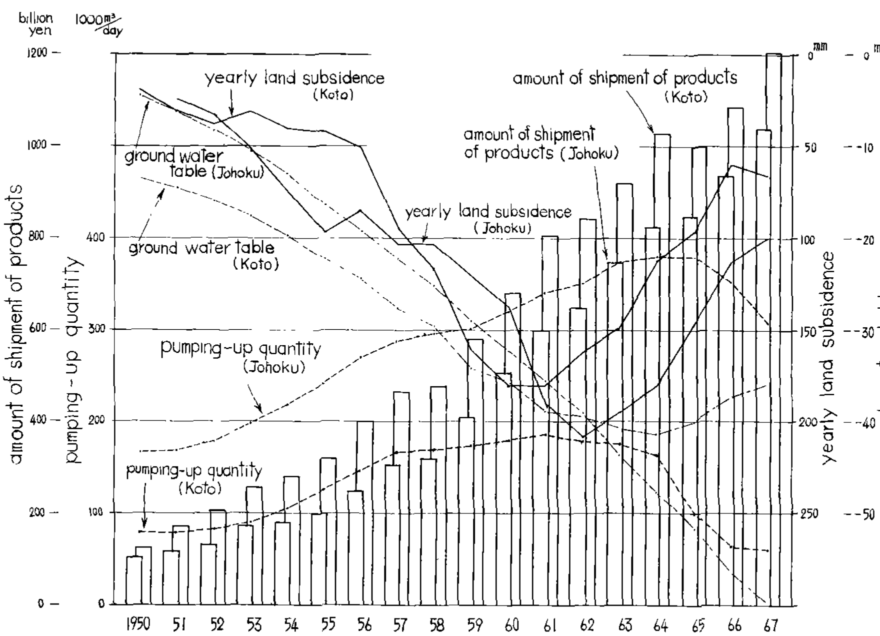

FIGURE 1. *The tendencies of Amount of Shipment of Products, Pumping-up Quantity, Ground Water Table, Yearly Land Subsidence*

(i) *The industrial rehabilitation in Tokyo has been in activity since 1950, and Japan's economy has been in the period of high growth since 1955*

(ii) *The control over the pumping-up of the ground water used for industries*
since 1961 Koto district
since 1963 Johoku district

(iii) *The control has been put in force in 14 designated districts over the pumping-up of the ground water for the use of building since 1963*

(iv*r* *The pumping-up of the ground water for the use of industries has been placed under strict control since 1966 in the supply area of the Koto Industrial Waterworks* changes in the level of water, and the amount of land subsidence. On the other ahnd, we have shown in figure 1 the trend of these four different factors. It may be noted that these factors work in association with one another.

In going into the trend after 1948, we find that the increase in the volume of water withdrawal that began from around 1953 is represented by the development of deep wells, whose depth is in the neighborhood of 100 m. Further, we find that the well depth had been growing deeper, ranging around 200 m around 1960. These details also are well indicated in the trend of water withdrawal and the trend in the amount of subsidence. The trend reflects the controls over the new well sinking enforced for the Koto area in 1961, controls for the Johoku district were put into effect in 1963, and controls on water withdrawal for the wells that were already existent as source of supply for industrial water were put into effect for the Koto district in 1966.

The Tokyo area has a series of groundwater-source systems, but it may be said that the Johoku district has a greater volume of groundwater. A total of 1,600 deep wells existed immediately before the enforcement of control, the water pumping capacity reaching 700,000 cu m per day (or 500,000 cu m per day in terms of the volume of water withdrawn). However, the wells which are currently being used total 1,000, their aggregate volume of water withdrawal amounting to 350,000 cu m per day. Thus, the water withdrawal is becoming smaller due to the controls now in effect. This decrease in pumping is being offset by a supply of industrial water as substitute water and the rationalization efforts on the part of the industries concerned. A complete control for the Johoku district remains yet to be carried out, and this partly accounts for the fact that the serious land subsidence is making its way northward from Koto along the Arakawa river, as it has in the past.

THE RESPONSIVE TREND OF THE ADMINISTRATION

1. Prior to the control of underground water withdrawal

"The main cause of ground subsidence lies in the fact that the lowering tendency of the underground water level due to the man-made excessive withdrawal of the underground water causes contraction of the alluvium, was proclaimed in an established scientific proposal and became a fact that was demonstrated in a series of programs that occurred before and after 1945. The responsive actions of administration began first with the observation and study of the underground phenomena, in the wake of which came the measures and projects for the prevention of low land disasters based on such observation and study. In particular, the massive flood damage caused by a series of typhoons that invaded the low area of Tokyo made clear the grave seriousness of the problem.

For these reasons, the Metropolitan Government of Tokyo made a series of comprehensive investigations on land subsidence for 5 years, starting in 1951. As a result of the fact-finding efforts, the dimensions of subsidence for two decades after 1955 was estimated at approximately 1 m. Based on this conclusion, measures were put into action on a permanent basis to combat the flood tides that frequently occur in the low ground-level of Koto district, where subsidence was found most critical. In addition, Metropolitan Tokyo's Deliberative Council of Inquiry into the causes of ground subsidence and countermeasures (Tokyoto Jiban Chinka Chosa Taisaku Kyogikai) set up in 1952 had some scientist members who, asserted the need of providing controls on groundwater and also the need of installing an industrial water system for the supply of substitute water.

2. Preparation of the water works project for the supply of industrial water

At the time when the problem of water for industrial purposes began to emerge, the regional development measures of Japan made a transition from the land preservation, maintenance, and development of natural resources toward development of industry. Thus, the Metropolitan Government began around 1955 a study of the policy regarding measures to be taken on industrial water for both industrialized districts, Koto and Johoku. The study was taken up as one of the problems for a comprehensive development program by the Tokyo Comprehensive Development Deliberative Council—or Tokyoto Sogo Kaihatsu Shingikai in local jargon. The abrupt demand for the industrial water as a result of the rapid development of industry caused an excessive withdrawal of groundwater, which forms the greatest potential source of industrial water. This in turn caused a shortage of groundwater and deterioration of water quality, and thus aggravated the land subsidence. However, the study mainly was directed toward securing a source of substitute water, conscious of the limited supply of groundwater and the need to protect and maintain the fundamental basis of industry.

At the same time, the population of Tokyo was having exceptional growth. Thus, the effort to secure a source of water for the maintenance of the population's life was extremely critical. Because of this, it was not possibile to seek a water source for industrial purposes from the rivers and streams. As a result, sewage-disposal effluent began to be the target of study and disicussion. The government instituted a law on industrial water in 1956, chiefly aimed at securing industrial water, but also aimed at the preservation of the groundwater supply. However, this law apparently made the prevention of land subsidence as its incidental secondary objective. As a result, the statutory control of groundwater withdrawal through the drilling of new wells came for the first time as a proposition possible for action, with the supply of substitute water sought as a precondition. With such background, metropolitan Tokyo launched action to investigate the utilization of sewage-disposal water as a water source for the industrial water works.

Since 1956, conferences have been held with the Comprehensive Development Deliberative Council on the above subject and as result some specialists were put on the comission to investigate the problem of water quality, in particular. At the same time investigations on industrial-water use were carried out, with the Koto industrialized zone chosen as the central target of the fact-finding efforts. In addition, an analitical study was launched on the underground hydrology of the area containing both industrialized zones of Koto and Johoku.

The following conclusions were reached from the above study: (1) Both industrial zones lie on the same underground water system, with the Johoku district forming the upper reach; (2) The major cause of subsidence in the southern sector, which represents about 2/3 of the Koto district, is this area's unique excessive withdrawal of water; (3) The excessive withdrawal of groundwater in the Johoku district is having a substantial effect not only on the subsidence of the district but also on that of the Koto district; (4) The reductive water, re-purified so its price will enable the industry to utilize such water on a paying basis, provides a possibility of use at a majority of factories in the Koto district in terms of its water quality. However, in the Johoku district, there are sizable numbers of factories that can scarcely afford to use such water.

3. The first phase of control on groundwater

Having arrived at the above described conclusions, the Metropolitan Government decided to install an industrial water system in the Koto district, which was then showing an extremely acute and critical water situation. In April 1960, an installation project was launched at the cost of 7.7 billion yen and with a capacity of 320,000 cu m per day of water supply. In addition, the drilling of new wells has been banned in the Koto district since January 1961. Although a law controlling withdrawal of groundwater by means of newly-drilled wells had been enforced, the subsidence showed no signs of decline thereafter. In addition, such major cities as Nagoya and Osaka came to suffer from subsidence one after another and in 1959 and 1961, respectively, great disasters by high tides and flood water hit the cities as a result of massive typhoons. In view of such developments, the government revised in 1962 the law concerning industrial water and added control over wells which already existed as well as newly drilled wells. Through these two measures, control became possible over the majority of groundwater withdrawal in major cities. Thus, land subsidence counter-measures turned from disaster-prevention measures toward measures designed to check land subsidence.

4. The second phase of control on groundwater—the present phase

Thanks to the government's buildup efforts on the water resources development administration in 1961, the industrial water project for the Johoku district was put in a position to secure water sources from the Tonegawa river water system. The district began its

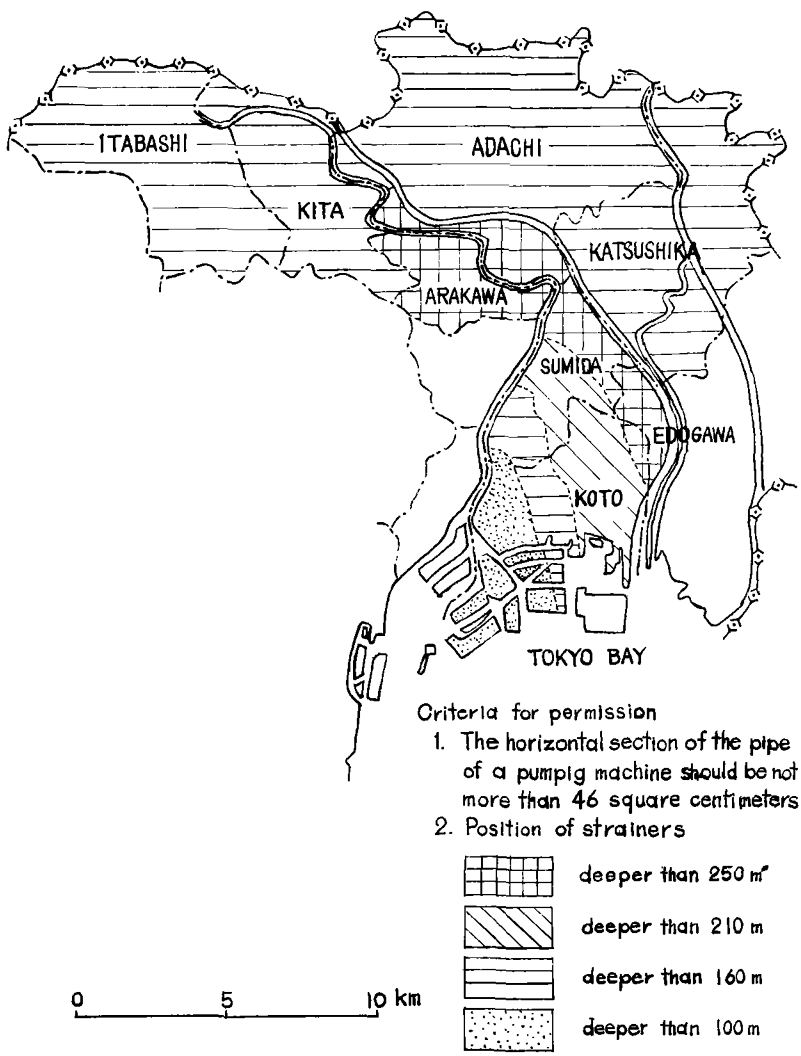

FIGURE 2. *The districts where control is laid over the pumping-up of the groundwater for the use of industries*

installation project in April 1963 at a cost of 22 billion yen and with 690,000 cu m per day as the water supply. Effective from July of the same year, withdrawal of groundwater by means of newly drilled wells was forbidden in the above described area.

Meanwhile, the Koto district industrial water project came to completion in 1965 and one year later the withdrawal of groundwater through old wells was put under control. Thus, the withdrawal of groundwater for industrial purposes had been placed under an overall restriction. In addition, control over the withdrawal of groundwater for buildings task effect in July, 1963. As a result, water needed for buildings has been provided almost wholly by the city water works.

The contents of control for both the Koto and Johoku sectors are as illustrated in figure 2. Withdrawal of groundwater from lower layers down to the diluvium was made the target of restriction for the Koto district because of the relationship between the water withdrawal and the amount of subsidence. In the Johoku district, withdrawal of groundwater from the upper layer of diluvium and alluvium was placed under restrictions because withdrawal of groundwater from the upper layer of diluvium, which contains a large volume of water, and also from the alluvium is considered as the major cause of subsidence. Further more control was made over those well bores greater than 46 sq cm because it was believed that water withdrawal would not be so extensive as to cause subsidence when the bore is below that value.

The contents of control over the use of groundwater for structures are as illustrated in figure 3. The restrictions over the depth and size of bore are the same as for the control of groundwater for industrial purposes.

SOME EFFECTS AND SOME CONTROVERSIAL POINTS OF THE CURRENT CONTROL MEASURES

1. The effects of control

In the Koto district, the effect of control imposed on the newly-drilled wells alone is not clear, as may be noted from figure 1. However, after the enforcement in 1966 of overall restrictions, the volume of water withdrawal began to show a marked decrease, the level of groundwater showed a rapid rise and a decline in the amount of subsidence was noted. In the following year a limit began to appear in the decrease in the volume of water withdrawal and the recovery in the water level. Thus, the dimensions of subsidence are showing a minor increase.

Because the existing wells in the Johoku district left outside the restriction and the numbers of the wells have increased, the volume of groundwater withdrawal showed only a slight decrease (fig. 1). Thus, the effect of control remains yet to be observed in the level of groundwater and the amount of subsidence is continuing to show only a slight decrease after the peak recorded in 1963.

For consumption of groundwater by buildings, the volume of water withdrawal following the enforcement of control began to show a substantial decrease, and it is believed that this is resulting in a satisfactory effect in terms of the subsidence in the area where such type of water alone is being pumped.

2. Some controversial points concerning the control imposed on the pumping of groundwater

Confining the land subsidence in Tokyo may not necessarily be sufficiently adequate, as indicated from the recent shift of subsidence in the Koto district. To adequately control the withdrawal of groundwater for industrial purposes, it is deemed essential to install an industrial water system that is capable of providing an ample supply of substitute

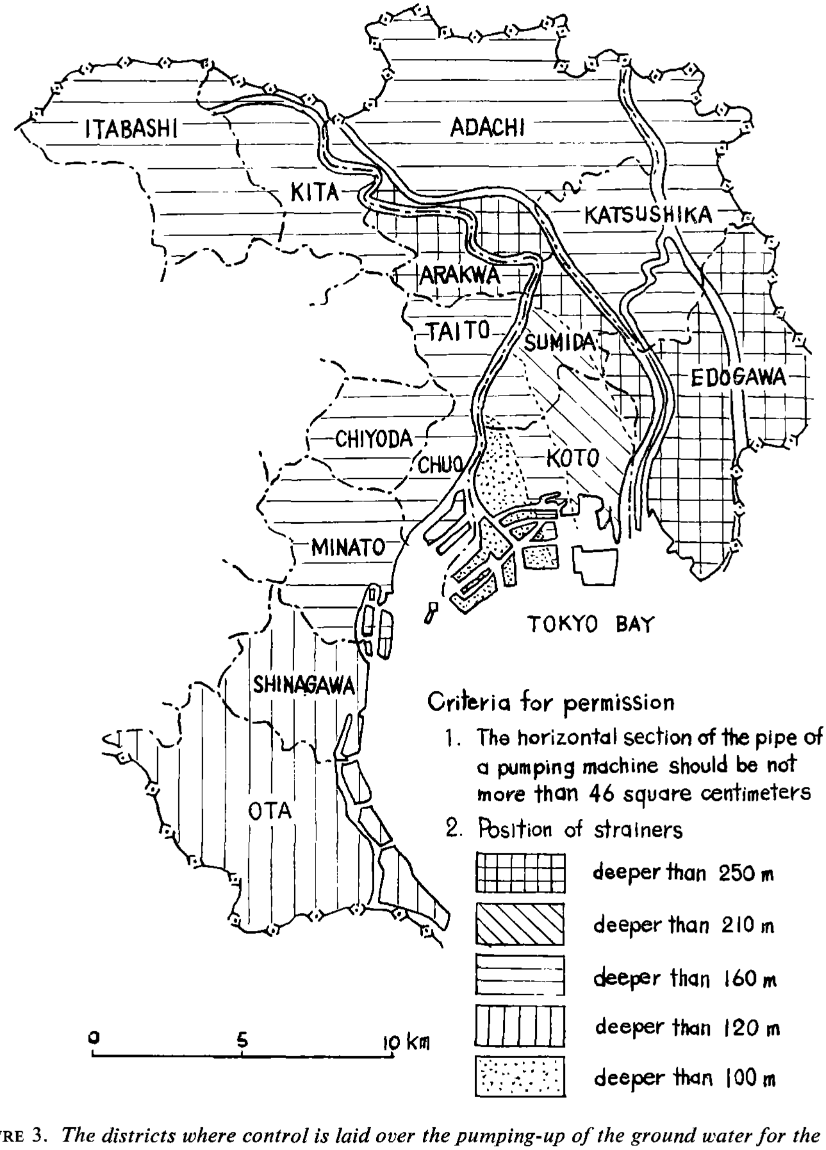

FIGURE 3. *The districts where control is laid over the pumping-up of the ground water for the use of buildings*

water of the same water quality, price, etc. An effective and efficient enforcement of the measures for restricting land subsidence are awaiting the installation of this water works and the clarification of the underground hydrology in the low land sector of Tokyo.

(*i*) *The underground water system and the area of control*

The shift of subsidence in the Koto district shows the necessity of providing restrictions on the excessive water withdrawal not only for the area which is affected by subsidence but also for other areas which are on the same groundwater system. Thus, it is all the more be necessary to acquire the correct understanding of the underground water system.

(*ii*) *The subsiding beds and the depths for restriction*

The depths for restriction in the Koto district may be considered as approximately corresponding to the condition of geological quality. On the other hand, for the Johoku district, the recent rapid drop in the water level and the growth of bed contraction in the deep-layer sections makes one feel some doubt about the effectiveness of the control as a countermeasure to deal with the land subsidence.

(*iii*) *The controls on the underground water for the industrial purpose and the industrial water works system*

Any attempt to seek a water source from rivers and streams is a really difficult proposition to accomplish because the shifts in the demand for living-purpose water requirements, like drinking water, continues to grow in volume. Even if such difficulty were overcome, there would still be a problem on its price that is feared to be costlier than the groundwater.

The second problem concerns the water quality of the reductive water as a substitute water, and its price. The reductive water is currently limited in the range of its application and in the purposes it permits because of its water quality. Some of its consumers have this water undergo a process of repurification, but, it is extremely difficult to improve its water quality in order to solve these problems without a rise in the cost.

The third of the problems is the fact that the demand for industrial water in Tokyo has now a tendency to decline. What are considered as responsible for such tendency are these factors: The moves of the intra-metropolitan industries toward dispersion, and progress toward rationalization of the use of industrial water in a variety of enterprises are con sidered the main factors.

THE FUTURE TREND OF THE MEASURES FOR RESTRICTION OF LAND SUBSIDENCE

In order to check subsidence in Tokyo's low-lying sector, to protect the lives and property of about 3.3 millions of inhabitants living there, and to maintain the fundamental basis of activities by about 150,000 businesses which are located there, some positive steps must be taken to impose restrictions on the withdrawal of underground water for industrial purposes. Such withdrawal forms a vital factor responsible for the subsidence. For this purpose it is essential to complete the installation of a water works for the supply of industrial water. In addition, the geology and the hydrology of the undergroundwater system in Tokyo's low sector as well as other regions that lie on the same underground water system should be broadly elucidated, taking into account the social activities which have influences on it. Further, in spite of restrictive measures on the pumping of groundwater in the Johoku industrialized zone, about 300,000 cu m per day of groundwater are present by being pumped and a substantial volume of groundwater is being pumped to cover the living requirements in Tokyo's perimeter regions which share the same underground water system.

With such a situation as background, the Tokyo Metropolitan Government in cooperation with some neighboring prefectures (Saitama, Chiba, and Kanagawa Prefectures) organized a broad regional cooperative structure and proceeded on preparations for development of new measures. At the same time the metropolitan government has strengthened its effort for observations and investigations within its jurisdictional area in conformity with the widening trend of the subsidence area. In addition, the metropolitan government began its preparation for the formulation of a re-development program, anticipating a change in the urban structure in Tokyo's lowland sector.

In the first place, efforts were made to strengthen the investigation and research for clarifying the hydrology of the regions lying on the same underground water system sprawling over the neighboring prefectures. For this purpose, the cooperative structure for investigations in cooperation with the neighboring prefectures must be strengthened and the observation and investigation structure must be built up to clarify the subsidence phenomena from a broader regional angle within the metropolitan government's own administrative area. Following the advance in investigation and research on these points, scientifically effective and appropriate measures should be put into action one by one, starting with those proposition that promise the easiest realization, and considering and studying various relative social factors. Needless to say, here again it would be imperative that steps be taken for fortifying the cooperative structure with the neighbor prefectures located on the same underground water system.

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DISCUSSION

Intervention of Mr. Herbert H. SCHUMANN (USA)

Question:

First I should like to congratulate you on a most interesting paper. I should like to ask if pumping of groundwater in other areas of land subsidence in Japan is controlled in the same way that it is in the Tokyo area?

Answer of Mr. TANAKA:

Yes, I would be happy to answer your question. At present pumping is restricted in more than twenty cities and villages and the manner of restriction or control depends on the local condition of subsidence and geological structure. Sometimes pumping from certain strata or up to a certain depth is restricted. Sometimes it is completely banned in a certain zone and restriction becomes less severe in proportion to the distance from the prohibited zone. Another case of restriction is the one that is laid on the capacity of pumps.

GROUND SINKING IN SHIROISHI PLAIN, SAGA PREFECTURE

Hisao KUMAI, Mitsuo SAYAMA, Tatsuo SIBASAKI, Kazuharu UNO

ABSTRACT

Shiroishi plain is a deltaic plain of about 8 km^2 on the Ariake sea. More than 150 deep wells which have been drilled since 1939 can supply about 1.0 × 10^7 m^3/year of water for irrigation in this plain.

Wells is this plain generally have a depth ranging from 100-200 m and lift confined ground water in the Deluvial deposits consisting of complex interfingered silt and gravel beds.

Deep wells have so increased that we produced only about 1.0 × 10^7 m^3/year of water, which was about half of demand except for surface water in 1964. This phenomenon has developed into a social problem, because the natural ground water level declined very fast, year by year and thus, intrusion of saline water and ground sinking occurred in this plain.

Ground sinking began in the spring in 1954. Zonal subsidence was found in this area in 1958 and occurred by the consolidation of alluvial clay. The sinking was found in the whole area of the plain in 1961.

The sinking in this plain is about 20 cm/year maximum. It is directly proportional to the thickness of alluvial clay and volume of water withdrawn for irrigation.

In order to solve this problem, a hydrologic balance in the confined ground water, artifical recharge and other technical research are studied.

RÉSUMÉ

La plaine de Shiroishi est une plaine deltaïque qui a environ 88 km^2 sur la mer Ariake. Plus de 150 puits profonds ont été forés depuis 1939 et ils sont capables de fournir 1 × 10^7 m^3/an d'eau pour l'irrigation de la plaine.

La profondeur de ces puits atteint en général de 100 à 200 m et ils relèvent l'eau artésienne dans les dépôts diluviaux consistant en couches entremêlées de vase et de gravier.

Les puits profonds se sont tellement multipliés qu'ils ne donnent que 1 × 10^7 m^3/an ce qui ne constitue que la moitié de la demande en exceptant l'eau de surface en 1964. Ce phénomène a donné naissance à un problème social, car le niveau de l'eau souterraine baisse d'année en année, permettant l'intrusion d'eau saline et la production d'affaissements dans cette plaine.

L'affaissement commença en 1954 avec l'arrêt du débit des sources. L'affaissement par zones fut observé dans cette région en 1958 et a été provoqué par la compaction de l'argile alluviale. L'affaissement s'étendait à toute l'étendue de la plaine en 1961.

Le maximum d'affaissement dans cette plaine est de 20 cm/an. Il est directement proportionnel à l'épaisseur de l'argile alluviale et au volume d'eau enlevé pour l'irrigation.

Pour résoudre ce problème, on envisage d'appliquer la recharge artificielle et d'autres méthodes techniques.

INTRODUCTION

The everdraft of groundwater in lowland areas constitutes the major ground water problem in the Kyushu district today. Since about 1960, the use of ground water for irrigation in this district has increased rapidly. When the pumping is excessive, management of ground water resources in the basin presents numerous problems of various degrees of complexity in each area.

In the Shiroishi Plain, Saga Prefecture, for instance, ground water uses have increased by so much that only about one-half of the total amount of irrigation water consumed is available locally. We can recognize many problems such as extensive ground water table drawdowns, increase in the chloride content of ground water, and land subsidence.

In this paper, a part of the writers' recent investigation will be discussed, particularly the relationship between land subsidence and hydrogeologic conditions from the macroscopic viewpoint. Since 1962, field investigation have been carried out by Kyushu Regional Agricultural Bureau, Ministry of Agriculture and Forestry, in co-operation with the Saga Prefecture Government.

In preparing this presentation, our cordial thanks are due to our many colleagues for assistance in various ways, and to Professor K. Kigoshi and Mr. K. Tanaka, of Gakushuin University, for their considerable assistance with the tritium dating of ground water samples.

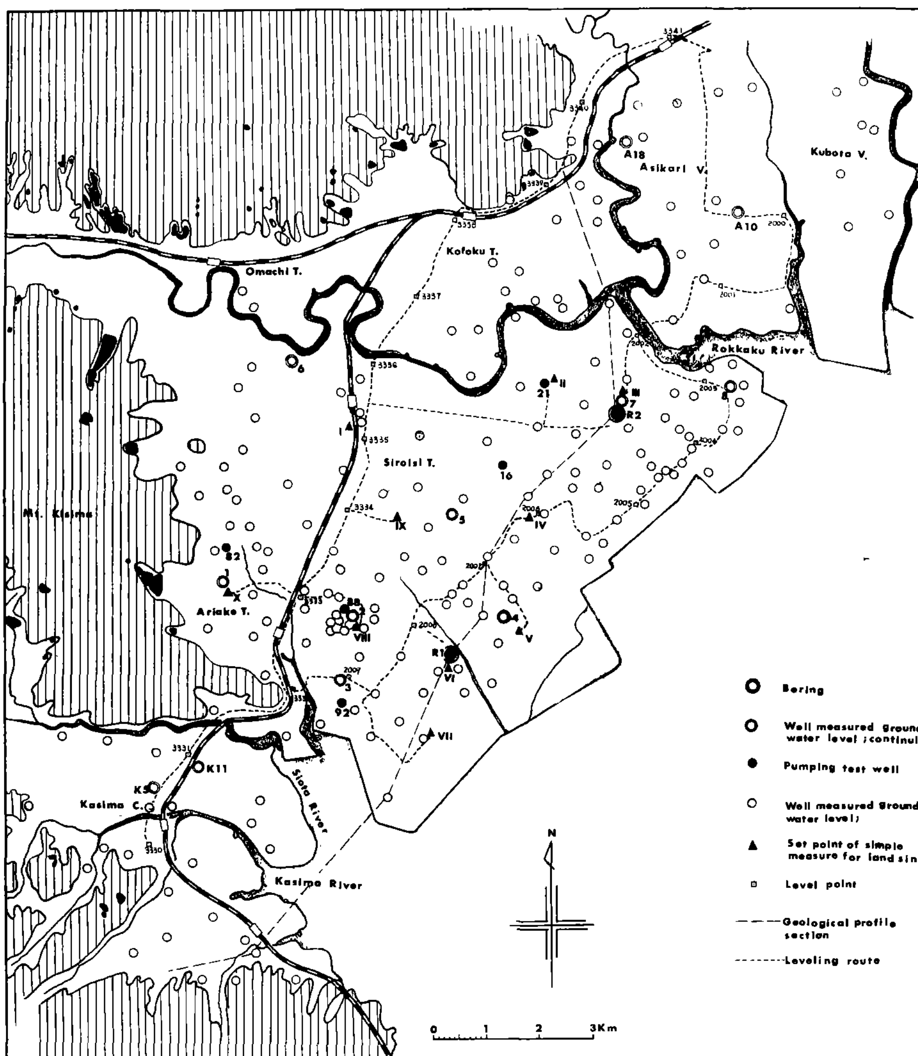

FIGURE 1. *General investigation map of the Shiroishi Plain*

GROUND WATER UTILIZATION

The Shiroishi Plain, about 8,800 ha in area, is situated on the north-west coast of the Ariake Bay (fig. 1). One-third of the area of the plain consists of land which was reclaimed 13 centuries ago. 80% of the area is used as paddy fields.

Without the levres about 90% of the area would be covered with sea water during the highest flood tides, because the height of land surface is only about 4 to 5 m, or even less, above sea level. Thus, land subsidence poses a serious problem for the inhabitants of this area.

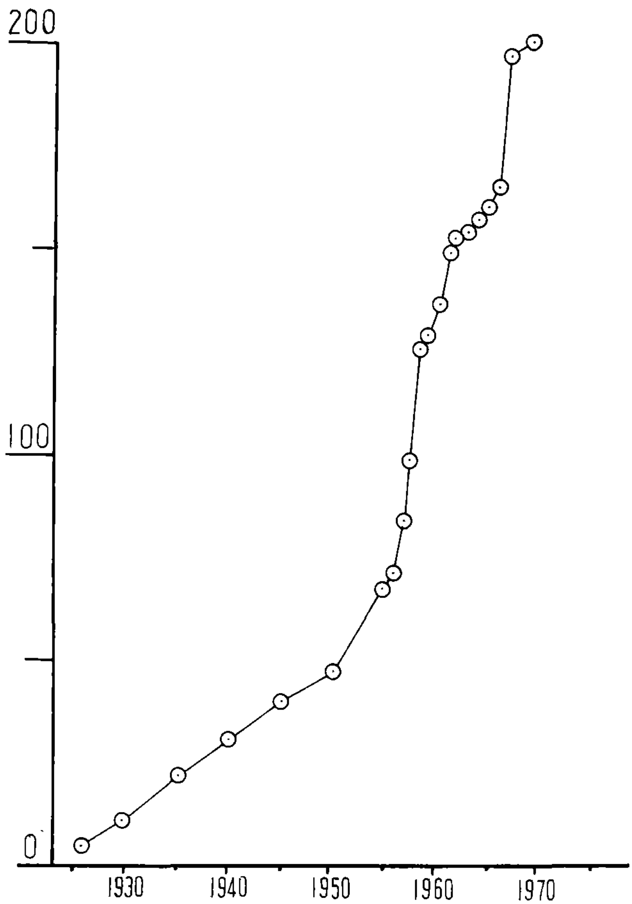

FIGURE 2. *Increasing the network of deep wells in the Siroishi Plain*

Since 1925 large amounts of ground water were used for irrigation in this area. The first boring of a deep well for irrigation was done in 1920. The number of deep wells was increased between 1939 and 1944, and accelerated with the drought in 1958 (fig. 2). About 200 deep wells are in operation in this area today. The annual consumption of ground water for municipalities and industries was about $2.88 \times 10^6 m^3$ in recent years, but annual

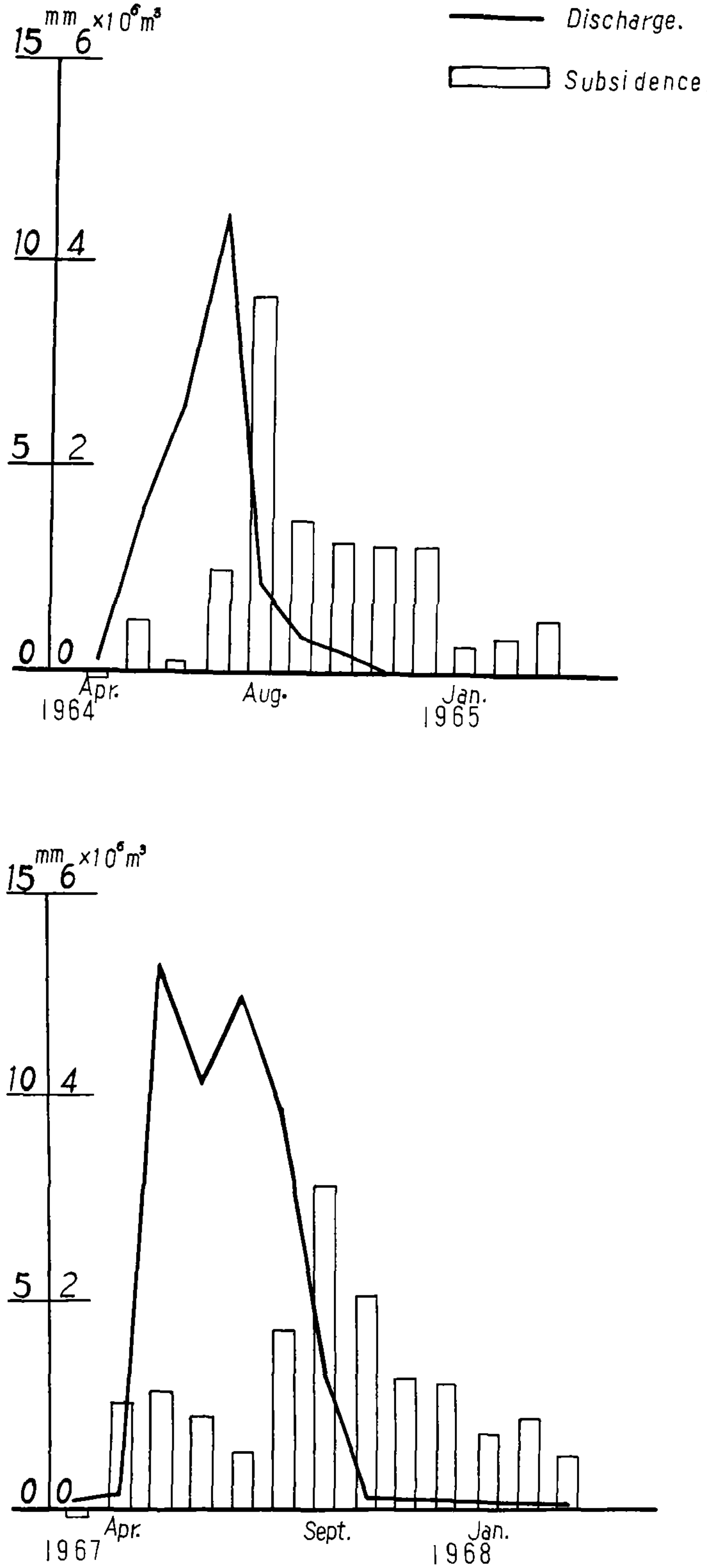

FIGURE 3. *Changes in the rates of discharge and average land subsidence*

consumption of ground water for irrigation naturally varies with the amount of the rainfall in the irrigation season. The total annual discharge of ground water is estimated to be about 2.5×10^6 to $2.1 \times 10^7 m^3$.

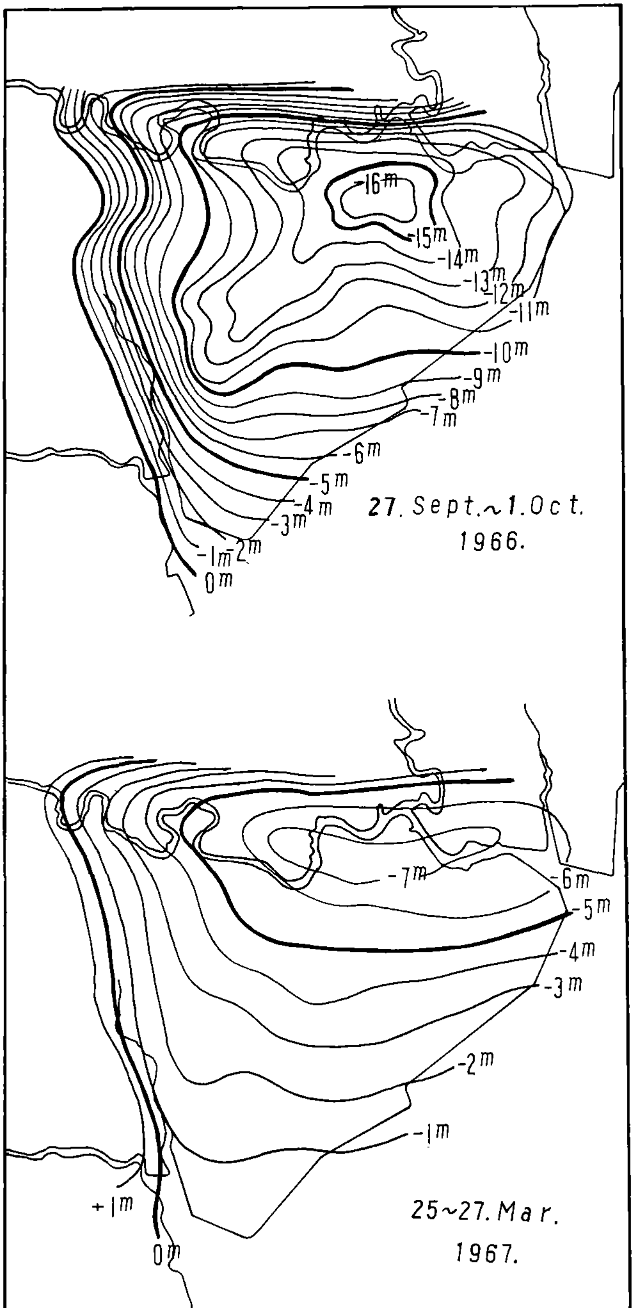

FIGURE 4. *Water table maps of the Shiroishi Plain*

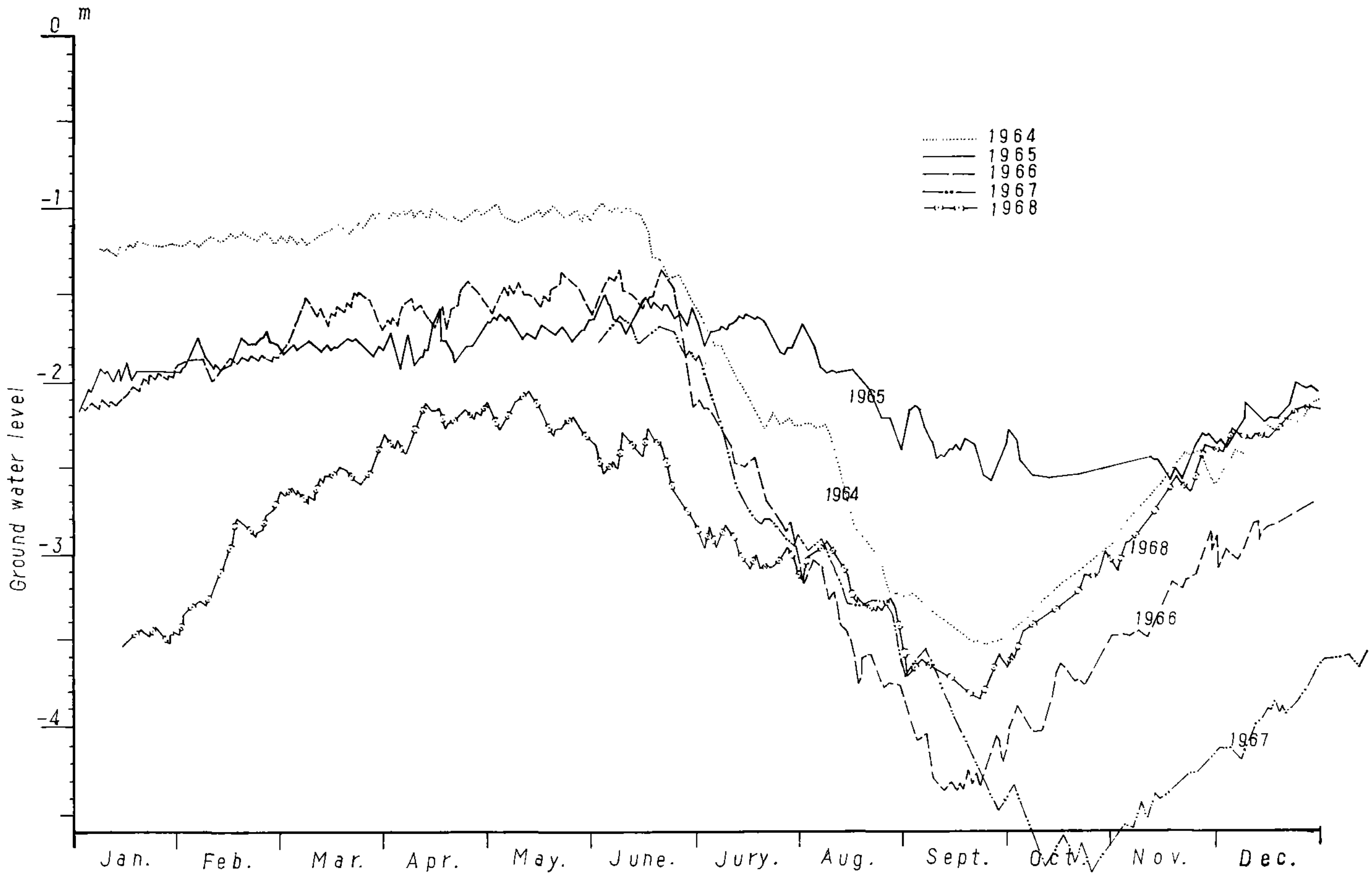

FIGURE 5. *Annual water level changes in No. 8 gaging station*

GROUND WATER PROBLEMS

1. Ground water level drawdown

Almost all of the wells lift the confined ground water through 2 or 4 strainers set in the Pleistocene sands and gravels. Ground water level data were collected since 1962. Figure 4 shows typical examples of water table maps constructed from data obtained during an irrigation season, (September 1966), and a non-irrigation season, (March 1967). Annual water level drawdowns are shown in figure 5, where it is seen that the water levels did not recover to their former natural level in non-irrigation seasons. The average annual rate of the decline or loss is 0.6 m/year.

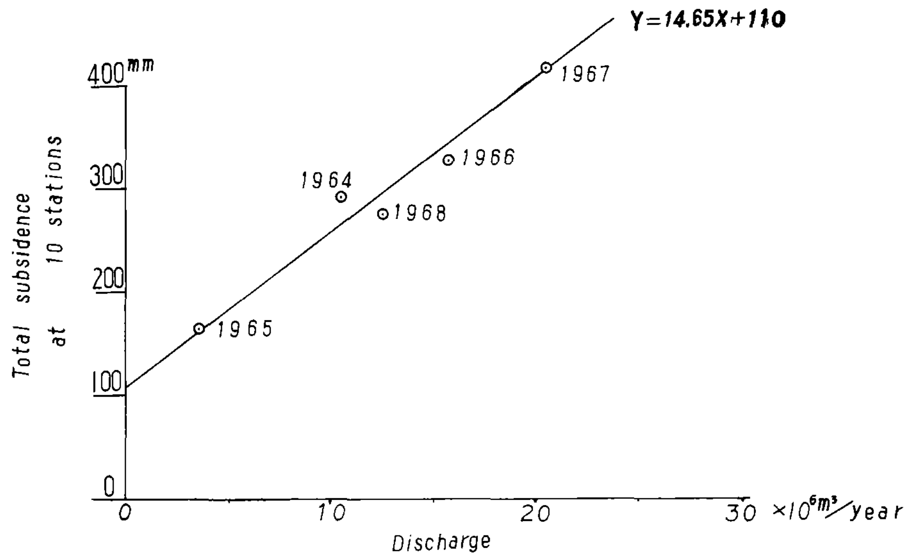

FIGURE 6. *Simple relationship between annual land subsidence and corresponding discharge*

2. Increasing of chloride content in ground water

Since 1962, the electric conductivity of the ground water was measured each September. The chloride content of the ground water was 0.025% and more, distributed about 15 km^2 in area on the shore side of the plain.

In the middle of September 1968, samples of ground water were taken for tritium and chloride analyses. The tritium content of samples ranged from0.2 to 3.0 T.U., and no relationship between tritium content and chloride content was observed. It is thought that the chloride content of the ground water was not directly derived from the sea, the main replenishment of water to the aquifers occurring by leakage through semiconfining strata or semipervious beds.

3. Land Subsidence

Land subsidence was firstly found out at the foot of Mt. Kishimadake, in the western part of the plain. During the three year period from 1958 to 1960, farm houses and farm lands were damaged by the marginal depressions which were from 0.2 to 0.3 m in depth and 30 m in width. These depressions with zonal sinkings and crackings broke out again in the summers 1966 and 1967, especially in 1967 which had its worst drought in 73 years. In 1967, about 2.1×10^7 m^3 of groundwater were pumped during the irrigation season.

After 1958, the slipping down of deep well tubes was observed here and there, and the land subsidence phenomenon expanded over the whole area.

GEOLOGY

Subsurface geologic succesion in this area is as follow;

1. Ariake clay formation

This formation, the Holocene deposits, consists of un-consolidated sticky clay, deposited in the valley's eroded underlaying strata. According to the penetration soundings, N-value of this formation is 5 or less. Average thickness is about 20 m.

2. Shimabara marine formation

This formation, deposited in the Late Pleistocene period consists of medium and coarse sands and silt. It was a remarkable contaminationfree aquifer for several years, but is now contaminated by saline water.

3. Aso welded tuff

This bed was provided from Mt. Aso, the famous volcano in central Kyushu. It consists of volcanic ash with pumice gravels, and the age of this Würm bed is a maximum of about 33,000 years. It is available as a key bed to correlated subsurface geology.

4. The Pleistocene formations

These deposits are the main aquifers in the basin with a thickness of more than 200 m consisting of sands and gravels.

MECHANISM OF LAND SUBSIDENCE

1. Ariake clay formation and land subsidence

10 simple measurement stations for land subsidence were set up on the plain in 1963 The core of the pipe was installed at the top of the Shimabara marine formation.

A weak relationship between total subsidence and the thickness of Ariake clay formation was observed. The average rate of land subsidence was 27 mm/year during a five-year period, the maximum was 59 mm/year, and the minimum was 2.7 mm/year.

The rate of land subsidence, measured by levelling, showed that the consolidation of the Ariake clay formation accounted for about 50% of the total land subsidence.

2. Discharge and land subsidence

Discharge for irrigation is concentrated in the four months from June to September, ranging from 1.5×10^6 to $2.0 \times 10^6 m^3$. The discharge for water supply, used for all

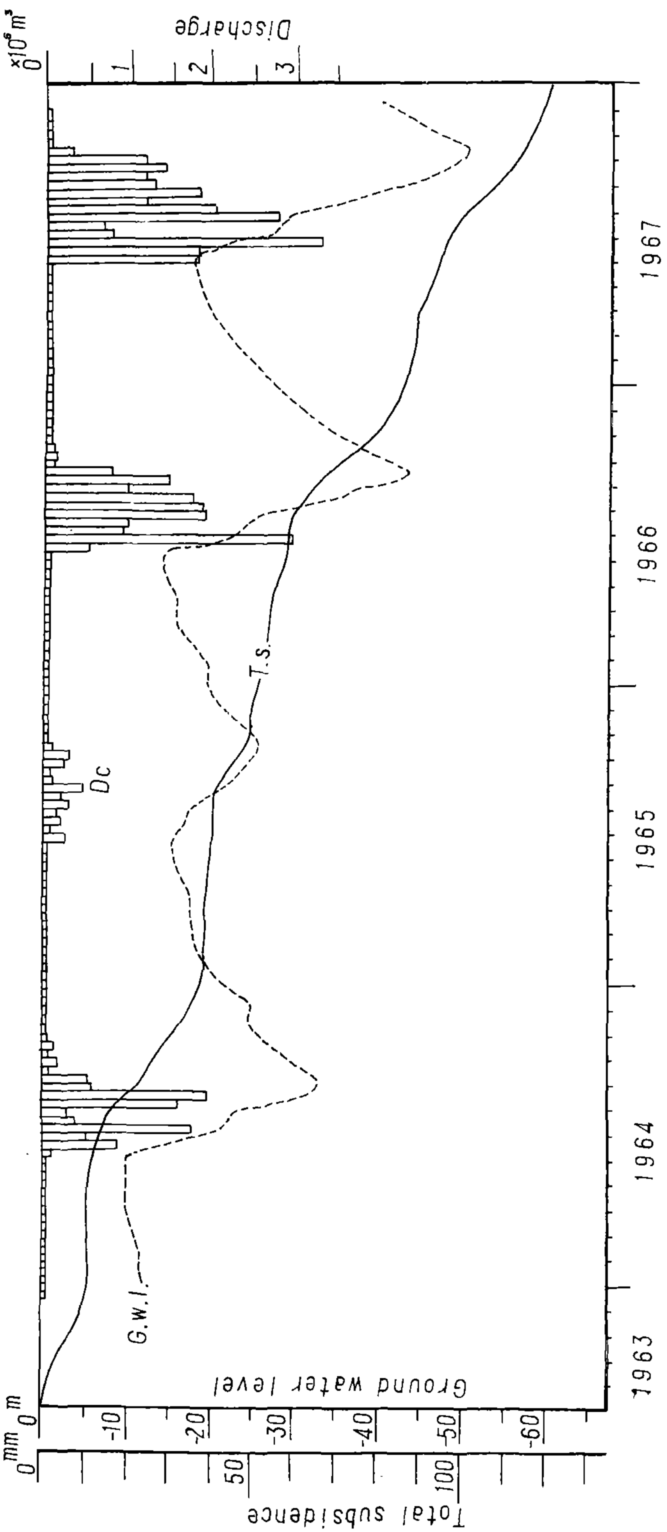

FIGURE 7. *Annual changes in total discharge (dc), water level in No. 8 gaging station (wl), and average land subsidence (gs)*

seasons, is about $1.2 \times 10^6 m^3$ through a year. The rate of land subsidence is affected by the rate of discharge. A time delay of two months was observed between them in the 1964 observation (fig. 5).

Figure 6 shows the relationship between the annual land subsidence and the corresponding discharge. The volumetric rate of subsidence was estimated as 18% of the discharge rate.

3. Water level and land subsidence

There is about one month's delay between land subsidence and water level change (fig. 7). The subsidence during a period of rising water level is less than that during a period of falling water level.

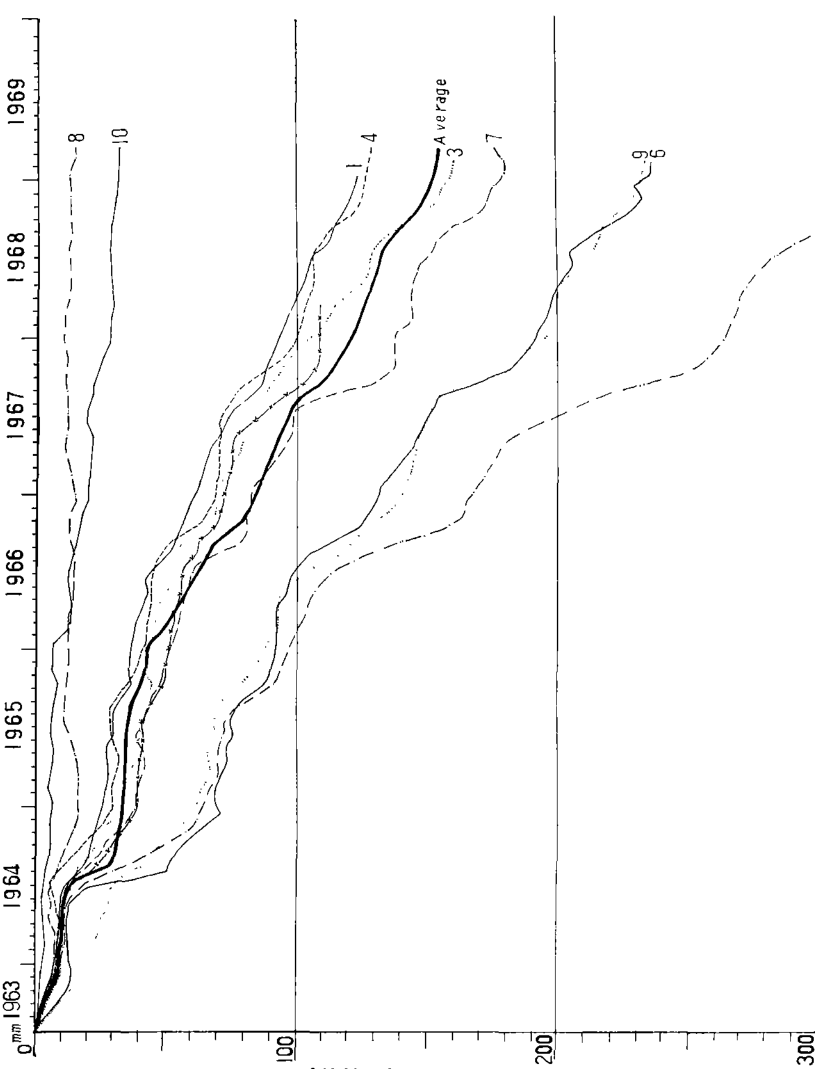

FIGURE 8. *Changes in total subsidence at measuring stations, in the Shiroishi Plain*

4. Process of land subsidence

It is possible to observe the process of land subsidence from an analysis of records of stations. The process can be devided as follows:

(*a*) The first stage: The land subsidence is continually progressing during the irrigation season, but recovers slightly in the off season. This stage is controlled by perfect elastic consolidation.
(*b*) The second stage: The plastic consolidation makes rapid progress with the lowering of the ground water level. When the discharge is stopped, secondary consolidation makes no further progress.
(*c*) The third stage: During off season, secondary consolidation progress at the rate of 1 or 2 mm/month.
(*d*) The fourth stage: During off season, secondary consolidation is progressing with the rate of 2 or 3 mm/month.
(*e*) The fifth stage: In spite of change in ground level, land subsidence makes no progress.

Various stages of the process of land subsidence can be seen from figure 8.

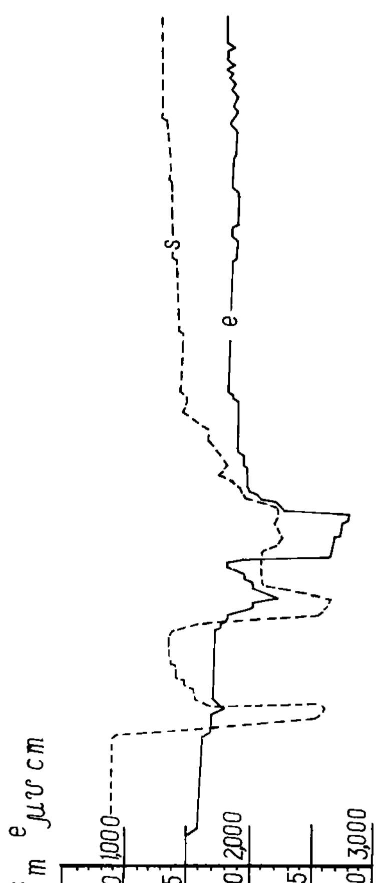

FIGURE 9. *Changes in chloride content of ground water and water level in well No. 31*

5. WATER BALANCE AND LAND SUBSIDENCE

The equation of the water balance in a confined ground water basin is given by

$$Q_r = R + L = AS\frac{dh}{dt} + Q_d$$

where

Q_r recharge to the basin per unit time;
R recharge through lateral seepage flow;
L recharge through leakage from semi-pervious beds;
A area of the basin;
S average storage coefficient of basin;
dh/dt average change in the height of ground water level in the basin per unit time;
Q_d discharge per unit time from the area of basin.

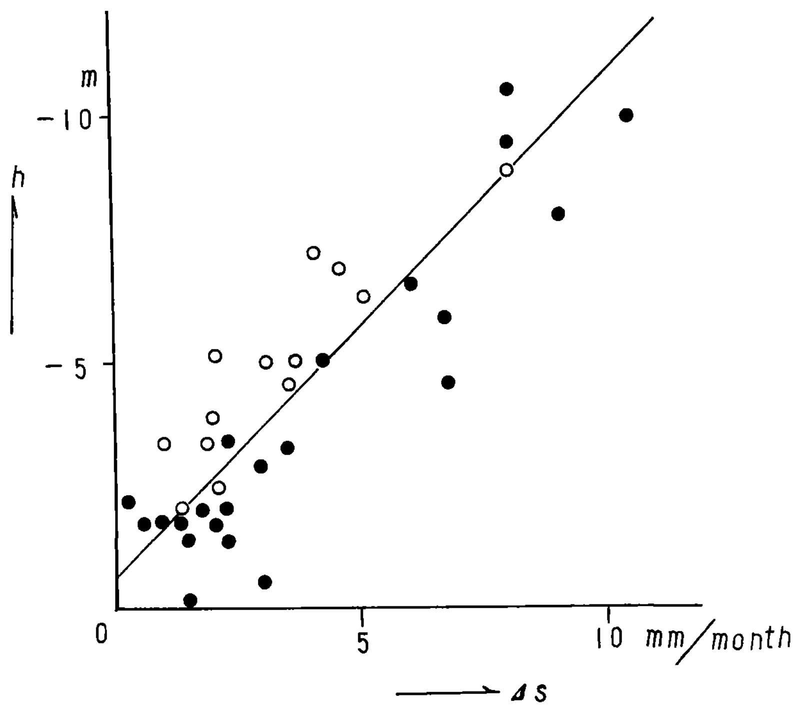

FIGURE 10. *Simple relationship between average height of water levels (h) and average rate of land subsidence*

It is easy to estimate the water balance by computer programs. The solution of the equation on the digital computer was reported by one of the authors (Shibasaki, *et al*, 1969), and for further details the reader should refer to that paper.

When actual field values from the Shiroishi basin are introduced in the equation, the following result was obtained.

It can be seen that the additional recharge is about 90% of the total discharge from the basin, and the leakage is estimated to be about 67% of the total recharge. The leakage water is squeezed from the aquicludes near the aquifers. This does not contradict the results of tritium dating of the ground water samples. From these facts described above, we may conclude that the increase of chloride content is caused by squeezing of the fossil saline water from the aquicludes rather than by the direct intrusion of sea water.

CONCLUSION

The conclusion is that the land subsidence in the Shiroishi Plain was caused by the overdraft of groundwater for irrigation, but it is necessary for us to concentrate more research into the following subjects;

1. Mechanism of the consolidation of the Pleistocene deposits.
2. Mechanism of the land subsidence with the repeated fluctuation of ground water levels.
3. Mechanism of the sea water intrusion.
4. The improvement of artificial recharge methods.
5. The simulation of future ground water basin management.

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Closing Session

Chairman Prof. Naomi Miyabe

Chairman: Gentlemen

On closing the working sessions of our symposium, I would like to express my heartfelt thanks to International Association of Scientific Hydrology, particularly to Prof. Tison, its Secretary-General and to more than twice as much participants as we have first expected who have presented so many interesting and useful contributions.

I would like to ask you for your permission to talk about my impression on the scientific works presented in this symposium. In my impression, the relation between the land subsidence and the groundwater pressure was one of the star topics in this symposium. This relation has been emphasized. Of course, it is a fundamental concept for explanation of most cases of land subsidence, by application of Terzaghi's theory of soil consolidation. This announced relation, however, contains several problems on its application to, in explanation of the land subsidence phenomenon. In this symposium, the results of a number of investigations have been presented to give complementary explanations from hydrological, geological, geotechnological, chemical, physical standpoints.

I was also impressed by the research on the injection of water into soil layers and the rebound of ground surface. The results of this research will contribute much, throw lights in putting into practice the countermeasure program, except for its high cost.

Discussion on the countermeasures to the disasters which may be caused by the land subsidence are consisting in prediction of future subsidence and in construction of flood preventing facilities based on fundamental studies of the nature of the land subsidence.

In connection with the problems of countermeasures, I would like to make a comment, if permitted, that some public nuisance which caused by the lack of groundwater should be taken up as a project, because the problem of groundwater is closely connected with the phenomenon of land subsidence.

In conclusion, I would like to express my deep appreciation to the participants who came over long distance to support and to make active the present symposium.

Lastly I would like to mention that several persons have told me of the next meeting, taking this opportunity, I would like to state my personal opinion on this problem, I think a number of participants have wished to have a next meeting to make possible the convening of next meeting, we think it maybe necessary to set up some organization, but there are some difficulty in organizing such a body and several experienced scientists in the fields of sciences concerned are in consultation how to conquer the difficulty.

By the way I would like to mention that Prof. Tison referred to an "enquete" on the information on land subsidence. I very much favor his suggestion. Until the time when scientific body is organized, I personaly am ready to take responsibility on mediation of exchanging information on the problem of land subsidence.

I thank you again for your kind and useful support of this symposium.

Exposé de Monsieur Léon J. Tison. Secrétaire général de l'Association internationale d'hydrologie scientifique.

Monsieur le Président, je vous remercie beaucoup de me donner l'occasion de vous adresser quelques mots à la clôture de ce que je peux appeler ce fameux symposium. Si vous me le permettez, je voudrais tout d'abord commencer par quelques commentaires sur ce que vous venez de dire. En fait, vous avez, d'une façon magnifique, résumé non seulement ce qui a été fait ici, mais vous avez présenté ce qui pouvait être fait pour l'avenir. Et, je dois vous en féliciter, parce que vous l'avez fait en un temps tellement réduit et d'une façon aussi excellente. Je voudrais ajouter, en reprenant d'ailleurs ce que vous avez dit à la fin de votre exposé, les quelques petites remarques suivantes. Vous avez bien voulu parler de ce que vous avez appelé mon idée de l'enquête. Je pense, en effet, que cette enquête est absolument nécessaire. Nous nous connaissons encore mal, nous les gens qui nous occupons de l'action des affaissements. Il y a beaucoup d'autres hommes de science qui s'occupent encore de ces affaissements, et nous n'avons pas pu toucher. Il faut que nous entrions en contact avec eux. Et, pour celà, il faut que nous sachions où ils sont. Et, celà, seule, l'enquête peut le donner.

Je vous dirai, donc, que notre Association a décidé, il ne s'agit plus ici d'un peut-être, mais d'un bien d'une certitude, notre Association a décidé de faire cette enquête. Mais, nous la ferions volontiers avec vous, avec votre appui, parce qu'il y a certainement des choses que notre Association connaît mal et que ceux qui s'occupent de mécanique du sol connaissent mieux. Donc, je crois que, pour qu'elle atteigne tout son objectif, que cette enquête doit être faite, en commun, par vous et par notre Association. Si je vous dis que cette enquête est certaine, c'est parce que nous sommes obligés de la faire. Le Conseil international des Unions scientifiques, comme je vous l'ai dit à la séance d'ouverture, a été frappé par le fait que l'action de l'homme, l'action dégradante de l'homme s'exerce, de plus en plus, non seulement sur la beauté de la nature, mais aussi sur les ressources de cette nature. Il faut donc, que l'on étudie cette action de l'homme et qu'on tâche de trouver les remèdes à cette action. Et, pour l'étudier, encore une fois, il faut connaître, et pour connaître il faut faire l'enquête.

Deuxième point, vous avez parlé aussi de grouper les gens qui s'occupent de cette action des affaissements. Je suis tout à fait de votre avis. Et, je crois que la chose la meilleure que je puisse vous dire, c'est qu'à l'Association d'hydrologie scientifique nous ouvrons toutes les portes, si vous jugez bon de vous joindre à nous.

Un petit point encore. Vous avez mentionné les sujets les plus importants traités à ce symposium. Et, effectivement, comme vous l'avez dit, c'est certainement ce qui est relatif à la mécanique des sols qui a primé. Je voudrais, cependant, qu'à l'avenir, puisque nous parlons surtout d'avenir aujourd'hui, je voudrais qu'à l'avenir on ne néglige pas l'aspect sur lequel quelques-uns de ces auteurs, dont je suis, ont bien voulu s'exposer à traiter quelques points: l'action des affaissements sur ce qui se passe à la surface, particulièrement au point de vue hydrologique. Peut-être ne ressentez vous pas aussi fortement que nous la nécessité d'étudier plus profondément cet aspect, mais dans les pays de l'Ouest qui ont de grosses exploitations minières, nous avons des problèmes difficiles et énormes, dus à cette exploitation minière. Et, je pense que là aussi, si vous vouliez bien accepter cette partie du programme, pour vos études futures, il y a quelque chose à faire.

Je crois avoir parlé suffisamment longtemps, peut-être déjà trop longtemps de ce que nous pourrions faire à l'avenir, mais il me reste une partie beaucoup plus agréable à développer. Cette partie très agréable, c'est d'adresser nos remerciements au Comité organisateur, au Comité national japonais organisateur de ce symposium. Je trouve, à la tête de ce comité d'organisation, tout d'abord, Monsieur Wadati; Monsieur Wadati est un vieil ami que j'ai rencontré maintes fois aux réunions de l'Union internationale de géodésie et de géophysique. J'étais très heureux de le retrouver ici à l'occasion de ce symposium. Il y a aussi dans votre comité un autre vieil ami, (quand je dis vieil ami, ce n'est pas parce qu'il est vieux, mais parce que notre amitié est vieille). Cette amitié vieille, c'est celle qui me lie à Monsieur Koiche Aki. Je suis très heureux de lui renouveller ici l'expression de cette amité, de lui dire tous mes remerciements d'avoir bien voulu s'occuper de ce symposium. Et puis, il y a vous, Monsieur le Président, vous qui avez été la cheville ouvrière de cette organisation au point de vue scientifique. C'est vous, au fond, qui avez créé ce symposium. Sans vous probablement qu'on n'en aurait jamais parlé. Et puis, il y a un autre ami, Monsieur Inokuti, que j'ai été aussi très heureux de retrouver parmi vous. Tout d'abord parce qu'il a bien voulu accepter de s'occuper de l'organisation, et aussi, permettez-moi de vous le dire, parce qu'il parle ma langue maternelle. C'est si agréable, d'entendre résonner à ses oreilles la langue de votre mère. Je voudrais étendre aussi mes remerciements aux autres membres de ce Comité d'organisation, mais he ne veux pas abuser de votre temps, et ils me pardonneront donc, si je ne les cite pas tous en détail, mais en tout cas, à tous vont nos remerciements

Il y a, alors, des remerciements qui doivent s'adresser à une deuxième catégorie de personnes qui ont réussi à donner à ce symposium sa grandeur. Ce sont les auteurs. Tous ces auteurs, je tiens à les remercier tout spécialement. Et, vous me pardonnerez, si je donne une mention spéciale aux auteurs japonais, car à eux seuls ils ont occupé plus de la moitié de notre programme. Nos amis américains, je les cite bien volontiers aussi, car, sous la conduite de Monsieur Johnson, ils ont fait un effort considérable. Les autres pays, je sais, ont été un peu paresseux. Le mien est parmi ceux-là. Mais, il n'importe. Je suis très heureux de mentionner ici la présence d'un certain nombre de délégués de l'URSS qui m'ont agréablement surpris et qui me font augurer très favorablement de l'avenir par suite de leur large présence ici. Les autres pays me pardonneront de ne pas les citer : je deviendrais absolument trop long.

Je vous ai parlé de l'organisation. Évidemment, les membres du Comité organisateur, étaient les grands messieurs qui donnaient les ordres et qui faisaient les grandes lignes de l'organisation. Mais, il y avait aussi un plus petit monsieur que j'ai rencontré, Monsieur Okamura. Monsieur Okamura a été réellement au point de vue exécutif, ce que vous avez été, me semble-t-il, au point de vue scientifique, Monsieur le Président. Monsieur Okamura était à la tête d'un bataillon de jeunes, aimables, agréables jeunes filles, qui nous recevaient le matin avec le sourire et nous donnaient beaucoup de courage pour travailler pendant toute la journée et qui, le soir, nous souriaient de la même façon pour nous détendre des fatigues de la journée. Il y avait aussi tout le personnel du secrétariat, tout le personnel de le réception. Je ne saurais tout citer parce que vous avez organisé tout tellement dans le détail qu'il y aurait des remerciements à adresser pendant plus d'une heure, si je devais les faire d'une façon complète. À tous, vont mon remerciement et les remerciements de nous tous.

Et puis, il y a une catégorie de personnes que l'on néglige parfois, et sans lesqu elles particulièrement ici, il autrait été impossible de faire la moindre chose. Ce sont nos charmantes interprètes. Je n'ai pas à vous dire la difficulté de leur travail. Elle a été énorme cette difficulté. Elle a été compliquée par le fait de la présence d'un vieux conservateur comme moi qui semble exiger encore que l'on parle le français aux réunions telles que les nôtres, et celà nécessitait la création d'une traduction, que j'appelerai à deux étages, mais nous, je le répète, avons compliquée d'une façon atroce le travail de nos interprètes. Je crois être votre interprète à tous, en leur disant tous nos remerciements, tous nos chaleureux remerciements.

Je deviens long, mais il faut que je tâche de n'oublier rien ni personne. Nous avons travaillé dur, très dur peut-être pour certains d'entre vous. Mais, vous aviez tout prévu. Vous aviez prévu qu'à côté de ce travail très dur, il fallait une détente très douce. Et, c'est pourquoi certains soirs vous avez prévu des réceptions qui étaient réellement admirables. Lors de ces réceptions, nous avons fait une entrée dans le monde du Japon. Nous avons une idée de ce que pouvait être la vie de votre pays. Nous nous remercions de tout coeur d'avoir bien voulu penser à celà.

Monsieur le Président, j'ai terminé. Je me répète, en vous disant encore une fois, j'ai peut-être été long, mais je crains d'avoir oublié encore, tellement vous aviez préparé bien les choses. Il y avait trop de personnes à remercier. Merci, de tout coeur, Monsieur le Président.

Mr. G. Owen Ingles (Australia):

Thank you Dr. Miyabe for this opportunity to say a few words at the closing session. (Japanese)

Watashi no nihongo wa jozu dewa arimasen. gomen nasai. Shimpojume wa taihen omoshiro katta. Nagaku hanase masen. Sekaiju doko nimo Nihon-jin no skinsetsusa, kirei na keshiki, johin na shukan wa nai to omoimasu. watashi tachi wa nihon no hito tachi to otomodachi ni narete ichiban ureshii desu. Domo arigato gozaimashita.

(My Japanese is not proficient enough to make a long speech, I am sorry. I do not think we can find in anywere else in the world, the kind people, beautiful scenery, elegant manners and customs. What I enjoyed most is that I can make friends with the Japanese hosts. Thank you very much).

It has been a most enjoyable symposium and I congratulate you on the arrangement that you have made.

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FIGURE 1. *Locations of compaction recorder stations with hydrographs, in subsiding area of Tokyo, as represented contours with numerals, which designate subsidence in cm.*