Session 1

Prediction and Performance

General Report of Case History Volume, *Additional Volume to Vol. 1—3 of IX ICSMFE, Tokyo 1977*

Prévisions et Constatations

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This part of the General Report is to review the content of the Case History Volume, which was published by the Organizing Committee for the IX International Conference in Tokyo. This Volume is considered an additional volume to the three Proceedings of the Tokyo Conference held in 1977. The publication of this Case History Volume was requested by Professor Masami Fukuoka, the President of the International Society, form his strong belief that a substantial progress of our profession could not be achieved without an accumulation of well-documented case histories. It was pointed out that a fruitful discussion or exchange of information was difficult at the time of the International Conference from its limitation in time and in length of papers. Then each invited author was given something like 50 pages of space for presenting his well-documented case history. This Case History Volume contains twenty two papers and its total length is little more than 1000 pages.

It will be unusual for the General Report to review papers which are submitted to other publication than the Proceedings of the current Conference. Content of the Case History Volume, however, is closely related with the theme of this session, prediction and performance. This is the reason why this part of the General Report is doing this unusual sort of review. Twenty two papers may be classified into three groups, i.e. papers of introductory nature, papers on single project or topic, and papers edited by an ad hoc committee for particular problem. Fourteen papers out of twenty two are from Japan.

PAPERS OF INTRODUCTORY NATURE

<u>Case Histories in Soil Mechanics</u> (R.B.Peck)

This is a fine introductory paper to the case histories. In this paper the author describes an importance of well-documented case histories with a lot of interesting examples, in particular, correspondence between Terzaghi and the author on the Chicago Subway construction. Four kinds of aspects of the case histories are explained ; case histories for verifying principles, case histories under complex conditions, case histories to establish fundamentals and case histories in education.

<u>Coping with Uncertainty in Geotechnical Engineering</u> (E.D. D'Appolonia)

Scope of this paper is a little wider than that ordinarily considered an introduction to this kinds of subject. This paper describes how to cope with difficulties in engineering practice derived from various sorts of uncertainties. Under this title, the author explains three factors of the project uncertainty, i.e. geologic hazards, organizational relationship and societal issue. The author's consideration, therefore, is not confined in physical aspects of geotechnical engineering.

PAPERS ON SINGLE PROJECT OR TOPIC

<u>A Case Study of Seismic Soil-Structure Interaction</u> (H.B. Seed, J. Lysmer, J.E. Valera and C.F. Tsai)

This is a clear presentation of process for obtaining reasonably good agreement between predicted and observed motion of a embedded structure in an earthquake. The case is the Humbolt Bay Nuclear Power Station during the Ferndale earthquake in 1975. Based on the motion recorded at the ground surface in the free-field and soil properties, the type of seismic wave field is investigated. Then the characteristic motions likely to develop at the base of the structure at a depth of 85 ft below the ground surface level is computed by the use of an idealized complete interaction analyses. Considering the current absence of any other opportunity to check analytical methods for computing seismic responses of prototype structure, the result shown in this paper will be a valuable contribution.

<u>Foundation Engineering, Installation, and Performance Observations for a North Sea Gravity Platform</u> (K. Høeg)

The author states that this is a digest and compilation of published papers on this subject. The most impressive thing in this paper is the scale and severe condition of this work which have not been tackled before. Story of the North Sea Project is well known, however, this paper is an excellent introductory paper for geotechnical engineers. Various aspects of behaviours of foundation which rests on the deep sea bottom and also its interaction with the sea bed are beautifully explained item by item. Also this paper contains a list of fifty three references on this topic, which will be of great help to those who are not familiar enough to this work.

<u>Geotechnical Aspects of Trans-Alaska Pipeline System</u> (U. Lusher)

This paper presents how to have constructed the foundation of pipeline running over long distance on permafrost, the total length of the pipeline is 1280 km. This is an introduction to the Arctic Geotechnology and tells us how to prepare appropriate countermeasures against unusual behaviours of permafrost

Investigation of Bearing Capacity of Foundation Ground of Honshu-Shikoku Bridge (K. Takahashi, K. Miyajima, S. Kashima, N. Yamamoto, M. Yamagata and R. Aizawa)

Large scaled bridges are to be built across the Seto Inland Sea in Japan. The longest of them is the Akashi Kaikyo Bridge, which is a suspension bridge of total length of 3560 m and center span of 1780 m. This paper describes how the authors assessed the bearing capacity of foundation on sandstone and mudstone with complex stratifications. They carried out a series of loading tests using loading plates of diameter of up to 2 m. Test results were analyses by the use of seven different methods. This examination resulted in the use of Kötter's bearing capacity equation and De Beer's formula for the squeezing failure in the actual designing. They also carried out a series of loading tests on bored piles, diameter of which was 3 m at maximum and length of 70 m. This paper should be classified to the case history under complex conditions, however, this paper will give us some idea for extrapolating our present knowledge of design method to greater scale.

Earth Pressure Measurements on Retaining Walls (M. Fukuoka, T. Akatsu, S. Katagiri, T. Iseda, A. Shimazu and M. Nakagaki)

In this paper authors present complete set of results of measurements of earth pressure acting on two types of large model retaining walls and five cases of measurements on actual retaining walls. The cantilever type model retaining wall is 3.5 m in height and 2 m in width. The gravity type model retaining wall is 6 m in height and 8 m in width. In conclusion of the measurement and analyses of the result, the authors pointed out the following items : actual earth pressure are largely influenced by the wall friction, the degree of compaction of backfill and condition of foundation soil, and the measured values of earth pressures were found to differ markedly from those estimated by the conventional earth pressure formulae by Coulomb and others.

Studies of Cut-Off Slopes in a Pumice-Flow Soil Deposit and Their Applications to the Design Standards for an Expressway (T. Yamanouchi, R. Mochinaga, K. Gotoh and H. Murata)

Shirasu is a pumice-flow deposit soil and counted to be a problematic soil in Japan for causing damage owing to rainfall and earthquake. This paper is a case history of construction of cut-off slopes for expressway through Shirasu distributed area. The authors present a detailed description of background information of Shirasu, mechanical properties of unidsturbed samples, stability analyses of cut-off slopes and field test results. These studies resulted in a compilation of design standard for the cut-off Shirasu slopes with special reference to the slope angle and arrangements for drainage system.

Case Study of Teton Dam and Its Failure (W.L. Chadwick)

Teton dam failed on June 5, 1976 while the reservoir level was 1 m below the spilway sill. This paper is a concice compilation of the two main reports on the failure of the dam by the Independent Panel to Review Cause of Teton Dam Failure and the U.S. Department of the Interior, Teton Dam Failure Review Group. Outline of the Teton dam, failure sequence and the results of various investigations are described. This paper indicates that prime factors contributed to the failure are erodibility and brittleness of the central core material.

Behavior of Madin Dam during Construction and First Filling (J. Flores and G. Auvinet)

Results of Soil Tests and Measurements during and after Construction of the Takase Dam (R. Takai, T. Iwakata and Y. Miyata)

Madin dam in Mexico is of the earth and rockfill type and is 77 m high, the crest is 250 m long. The Takase dam is the largest embankment dam in Japan, with height of 176 m and crest length of 362 m. Both of the papers contain detailed description of testing and controlling of construction material, design procedures and performance. Various sorts of measurements were carried out from the beginning of the construction through the end of the first filling of water. These two papers are to be added to a file of well-documented case histories of embankment dam construction.

Enclosing Embankment on Soft Ground - Hachirogata Tidal Reclamation Project (Y. Ohtsuki, K. Tanaka and M. Sato)

Construction work for the Hachirogata Reclamation Project started in 1957 as a national project and completed in 1977. Total length of the dike is 52 km and most of it was constructed on sand, however, 4 km portion had to be constructed on very soft clay stratum of 20 m thick. This paper describes geotechnical problem on this 4 km portion of the dike. Settlement analysis was made by the Terzaghi's one-dimensional consolidation theory. The clay stratum was classified into four layers with respect of difference in c_v value, which differed by 10^2 times at maximum. Average degree of consolidation was worked out by converting the actual multilayer to a single layer with particular value of c_v. Consolidation load was corrected for submergence of the dike. Accuracy in predicting the settlement was very high. The settlement observed from 1960 through 1977 was a little less than 3 m.

Geotechnical Problems during the Construction of the Madrid Subway Extension (V. Escario, J.M. Garcia Gonzalez, J.F. Moya, C.S. Oteo and C. Sagaseta)

Ground Movements due to Construction of Shields-Driven Tunnel (T. Hanya)

These two papers deal with the tunnelling in relatively soft soils. The paper by Excario et al contains detailed description of subsoil in Madrid and great effort devoted for measurement of the stress in the lining and deformation of the surrounding ground. This deformation problem is the main subject of the summary paper of shield driven tunnels in Japan by Hanya.

In the VII International Conference on SMFE, Peck presented the state-of-the-art report on " Deep Excavation and Tunnelling is Soft Ground". This report has been considered a sort of guideline of this particular subject. The present two papers deal with the same topic of " settlement trough ", which is a shape of distribution of settlement along the line perpendicular to the tunnel axis. Escario et al propose a method for obtaining the maximum settlement based on a theoretical solution, as a function of the depth of the tunnel, its diameter, the modulus of deformation and Poisson's ratio of the soil. Hanya collects 58 cases of settlement observations in 25 tunnels in Japan, and proposes four kinds of grouping of soils with respect of the characteristics of the settlement trough, which is a little different from that proposed by Peck. It is sure that these two papers made substantial contribution to this particular topic in tunnelling in soft soils.

<u>Construction of Submerged Tunnels on Soft Ground</u> (T. Monji)

This paper describes a detailed records of designing and construction of submerged tunnel in port of Kawasaki, Japan. The tunnel consists of eight elements, size of which is 31 m x 8.8 m in cross section and 110 m long. Subsoil in the site is soft cohesive soil. Change in soil strength and deformation of surrounding ground due to large scaled excavation is investigated, where the result of observation is reasonably close to the predicted one. Various types of analyses are carried out for assessing behavior of the tunnel in earthquakes. Settlement of the tunnel by backfilling is predicted by the use of elastic beam method and the result is comparatively coincides to the observed settlement.

<u>The Seikan Undersea Railway Tunnel</u> (Y. Mochida)

The Seikan Undersea Railway Tunnel is 53.85 km long, 22.3 km of which passes below the sea bottom, deepest elevation is 140 m below the sea level. This paper describes details of geotechnical survey and construction of this unpreceded scale of undersea tunnel. Results of measurements of earth pressure and deformation in the experimental tunnel are shown. This paper should be classified to the case history under complex conditions. The author of the paper states that the main purpose of this paper is to hand over the experiences and technologies acquired through this project to geotechnical engineers of the next generation.

<u>Rock Mechanical Approach for the Construction of the Underground Power Station</u> (Y. Mimaki, S. Katano, M. Kamijo and T. Tonegawa)

Cavern excavation of unpreceded scale was made in hard granite for construction of underground Shin-Takasegawa Power Station in Japan. Scale of the cavern is 27 m in width, 54.5 m in height and 165 m in length, the maximum depth of excavation was 250 m below the ground surface. Substation cavern was 41.5 m apart from the main one, size of which was 20 m in width, 35.3 m in height and 109 m long. Excavation of these two caverns were carried out simultaneously without any local collapses. This paper describes the details of investigation of rock properties, stress - strain analyses of surrounding rock during excavation and results of observation. Visco-plastic analysis was made on progressive relaxation due to excavation. Decrease in the inner span at the time of completion of excavation was predicted to be in the range of 30 to 65 mm, however, 40 to 100 mm of the decrease was observed.

<u>Mechanism of Neocene Mudstone Landslide - Sarukuyoji Landslide</u> (M. Watari, H. Fujita, H. Nakamura, A. Sakai and M. Kondo)

Landslides in Japan are concentrated in the Neocene strata. The Sarukuyoji Landslide is located in Niigata, Japan. An area of major slide is 24 ha, sliding mass is 4 to 15 m thick and 50 to 100 m in width and 1.8 km in length. Annual movement as of 1960 was 3 m. This area has been assigned the experiment site since 1960. This paper describes the detail of site investigation, correlationship between the movement and rain- and snow-fall, distribution of movement speed with depth. In particular, the use of pipe strain gauge and creep well contributed for clarifying the feature of the slide.

PAPERS EDITED BY AN AD HOC COMMITTEE

<u>Blast Furnace Foundations in Japan</u> (K. Ishihara, A. Saito, Y. Shimmi, Y. Miura and M. Tominaga)

Most of steel works in Japan locate in the coastal area, where typical soil profile is reclaimed sand in the surface and then alluvial clay and diluvial clay continue. Typical types of blast furnace foundation, concrete caisson, pile group and interlocked pipe piling, are briefly explained together with their design concepts. This paper consists of three parts. All of them are well-documented with detailed quantitative data.

Part 1 is a construction of blast furnace at Kimitsu Works of the Nippon Steel Co. Foundation is steel pipe group. Load distribution among group piles was measured on actual foundation piles in the course of construction of balst furnace through actual operation. 50 piles out of 168 were selected for the measurement, area of which covered approximately one quarter of the total foundation area. In spite of the fact that actual load by superstructure was concentrated to the central part, the distribution of pile load indicated an opposite pattern. This pattern became more marked with time, and at eleven months after the start of measurement, the load in the vicinity of foundation edge increased to about twice that in the central part.

Part 2 deals with two types of foundation of blast furnace at Ohgishima Works of the Nippon Kokan K.K., caisson and pile group. Dynamic response of the blast furnace on caisson foundation was analysed by using the three-dimensional dynamic model. A good agreement between observed and calculated response was obtained. Large difference, however, in the values of the maximum acceleration of the upper structure was observed between the result of analysis and actual record. Negative skin friction was a serious problem in the site, because the Ohgishima Island was a man-made island and consolidation settlement of 20 to 30 cm per year was expected to continue for about fifteen years time. Then, the SL pile, piles coated with asphalt slip layer, was used for reducing the negative skin **friction**. Me**asurement** of test piles proved remrkable effect of the SL pile. Measured negative skin friction of the SL pile was about 200 kN, while it was 5600 kN in non-coated pile.

Part 3 describes blast furnace foundations constructed by the Kawasaki Steel Co. The foundation is the double circular interlocked steel pipe piling method, in which two concentric circular piled walls are installed down to the depth of 30 m, and area between the two walls is excavated and then filled with cast-in-place concrete. The realtime construction control system was employed in this case, which compared predicted values to observed values in order to provide a correct and accurate picture of the current state and also this system allowed prediction of any future stages based on the current data. Various control charts were prepared by an autoplotter in a form readily understandable to field engineers. Some 2000 sensors were installed for recording stress and deformation of the structures during construction. Details of the observation during construction is given. When the observed values differed from the predicted ones, appropriate countermeasures were performed by processing a vast amount of data in short time.

<u>Excavation Works in Japan</u> (Ad hoc Committee of Case Studies of Excavation)

This paper contains eleven case records, each of them are well documented with detailed information. In the preface, a new design concept for earth retaining structures is explained, which is referred to the "elasto-plastic method-2 ", recommended by the Architectural Institute of Japan (AIJ). The assumption in this analysis is ; i) elastic compressive displacement of strut supports is considered, ii) lateral pressure, in terms of total stress, at the back of the retaining wall is assumed to be equal to the one actually measured, and lateral resistance of penetration does not exceed the Rankine's passive pressure, and iii) retaining wall has a finite length.

Case 1 is an excavation work in central area of Tokyo. Excavation covered an area of 115 m x 53 m and extended through soft alluvial clay of 20 m thick down to 26 m below the ground surface. The excavation area was first surrounded by diaphragm wall, then continuous mixed-in-place piles were installed in the shape of buttress. In the course of excavation, 400 sensors were installed in the diaphragm wall, struts and other parts for automatically monitoring the state of stress and deformation of the whole structure continuously. During the mixed-in-place concrete pile installation, the diaphragm wall was observed to move outward by 20 mm at maximum, then reduced to 15 mm. This deflection of the diaphragm wall was found to be in between the values predicted by the elasto-plastic method-2 and the FEM analysis.

Case 2 is an excavation for subway in down town Tokyo. Subsoil is a quick and soft clay silt of 20 m thick. The excavation was 18 m in width and 20.8 m in depth. The earth retaining wall consisted of a staggered arrangement of steel pipe piles driven to a depth of 30 m below the ground level and plain mortar piles were driven at the reverse side of the steel piles. Since stability against heaving was not satisfactory, quicklime piles of 45 cm in diameter were driven. 170 sensors were installed for manual measurement of stress and deformation of the structure. The maximum wall displacement was 68 mm on one side and 73 mm on the other side. The maximum settlement of the ground surface of 20 cm at the rear of the wall took place at the distance of 10 m. Heaving of the excavated bottom was 57 mm at maximum. Bending moment and deformation of the wall calculated by the elast-plastic method-2 were found to agree with the measured values all through the construction process. It was found, however, the passive earth pressure was underestimated by this design method to large extent.

Case 3 is an excavation for expressway in Yokohama, the total length of which was 1 km. Subsoil is soft marine clay of 20 m to 40 m in thickness. This soft cohesive soils is underlain by hard silty layer. Width of excavation was 25 m to 33 m and the depth was 20 m at maximum. In consideration of influence to the adjacent structures, the excavation was made separately in 33 blocks. The stability against heaving was found to be not satisfactory, then the cohesive soil was improved by the use of quicklime piles. Rigid steel pipe wall was used for the retaining wall. Earth pressure distribution was assumed following the Peck's recommendation in 1969. Measured value of the coefficient of active earth pressure was 0.15 at lowest and 1.1 at highest. The higher values were observed in the blocks where struts were installed prior to excavation and it was interpreted that a compressive force due to the quicklime pile installation concentratedly acted on the upper stage struts. The lower values were encountered in the blocks where struts were installed after excavation progressed to 2.5 to 3.5 m. As the excavation progressed, the higher values tended to decrease and vice versa. The maximum settlement of the surrounding ground was in the order of 0.8 % of the excavation depth and appreciable settlement occurred within the distance of about 4 times of the excavation depth. It was concluded that the measured values greatly differed from those predicted in the design pahse in the distribution of bending moment and deflection of the steel pipe wall and strut loads.

Case 4 is an excavation work for building in Osaka. Area of 8700 m^2 was excavated to a depth of 17 m at one side and 23 m at the other side. This work is characterised from two points. In order to support very heavy building, total cross sectional area of Benoto piles was 14.74% of the excavated area. The other was the use of " sakauchi " method, in which the construction of super-structure and underground structure are simultaneously executed, and earth retaining wall being successively supported by the basement structure during construction. Top soil is alluvial sand which is underlain by soft alluvial clay, then alternating layers of diluvial sand and clay continue In the first stage of excavation, sudden increase in displacement of the head of the earth retaining wall took place, which was 10 cm against predicted value of 5 cm, when the Benoto piling was executed in parallel with the grout cut-off. This was interpreted to be caused by reduction in soil strength due to the Benoto piling. After completion of Benoto piling, therefore, quicklime piles were driven over an area of about 15 m in circumference for recovering passive resistance of alluvial clay. As an additional effect of this quicklime piles, the earth retaining wall was found to be pushed back by 3 cm at maximum. In the fifth stage of excavation, however, the deformation of the earth retaining wall again was found to increase beyond the control value, which compelld the lowering of external groundwater level. Stress and deformation were predicted by the elasto-plastic method-2. The predicted values compared reasonably with the measured ones, except for bending moment.

Case 5 is an excavation for a pumping station over an area of 76.4 m x 23.4 m. Braced excavation using diaphragm wall was performed. Subsoil down to a depth of 13 to 15 m is an alluvial soft silt, with the stability number of N_b = 10. Lower portion of this alluvial layer tends to sandy, then diluvial sand layer underlies. Excavation was made in six steps down to the depth of 20.5 m. This work is featured by the fact that at one corner of the excavation, a concrete caisson is to be sunk to the depth of 20 m and the edge of the caisson is only 7.6 m apart from the outer face of the diaphragm wall. In addition, a number of PC piles are to be driven in outside vicinity of the diaphragm wall at other part. Influence of sheetpile driving for the caisson work and sinking of caisson was seen in the lateral earth pressure on the wall in front of the caisson. The coefficient of lateral earth pressure was observed to be in the range of 0.7 to 0.85, whereas the value in other part was in the range of 0.6 to 0.7. Direct influence of caisson sinking appeared in the increase in lateral earth pressure at the depth of 10 m by 20 kN/m^2 . Influence of the PC pile driving to a depth of 25 m was observed also. For the range from 5 m to 25 m in depth, earth pressure and water pressure increased by 70 kN/m^2 and 60 kN/m^2 respectively. These increase was observed to dissipate in four to five days time. Lateral deformation of head of the earth retaining wall in front of the caisson was 30 mm, while the deformation on the other side was 10 mm which coincided with the predicted value.

Case 6 is an excavation for building in Osaka, its area being 90 m x 43 m and 14 m in depth. Top layer is a reclaimed sandy fill of 5 m thick, which is underlain by soft silty soil of 30 m thick. The silty soil was soft and various formulae showed that the critical depth against heaving was about 10 m. Ordinary excavation, therefore, was carried out down to this depth, and then the island method was employed for deeper depths. Since a zone of occurrence of heaving was confined by an existence of island in central part of excavation bottom, quicklime piles were installed in between the earth retaining wall and island and with a depth of 11 m below the bottom. Main purpose of instrumentation was to catch a sign of occurrence of heaving during excavation. When the fourth stage of excavation was performed to a depth of 14 m, axial force in the third strut at a dpeth of 9.8 m showed a sudden increase. If a sliding was assumed to be located outside of the quicklime improved zone, the factor of safety was 1.02. After installation of the fourth strut, the force in the third strut stopped increasing, and no large force was observed in the fourth strut. Bending stresses in the sheetpile wall were larger in the embedded portion than in the excavated portion. Deformation of the sheetpile showed that it was of a simple beam with one of support being at the first strut and the other being at the bottom of the soft soil layer. The maximum displacement of the wall was found at a part deeper than the excavated bottom. This was considered a sign of heaving too.

Case 7 is a braced excavation in Tokyo. Subsoil is composed of top earth fill of 2 m thick and very soft silt layer of 22 m thick. Area of excavation is 55 m x 22 m with a dpeth of 13 m. The sheet piles were driven down to a depth of 19 m. The base stability factor by Peck was estimated to be N_b = 8. Steel sheet piles used for the work were left in the ground after the completion of the work, and the measurement of lateral pressure, pore water pressure and ground water level have been continued for five years. As a result of check of durability of sensors, it was found that approximately 90 % of the earth pressure gauges and 60 % of the pore water pressure gauges were still functioning normally. Assuming the triangular distribution of lateral pressure, the coefficient of pressure before the excavation was 0.58. This value decreased to about 0.4 during excavation, then recovered to the initial value in six years time after completion of the excavation.

Case 8 is an excavation in Osaka, size of which is 73 m x 40 m and depth of 28 m. Subsoil is divided into six different layers, most of which are sandy layers. The layer down to a depth of 32 m is the first water bearing layer, and the second water bearing layer is sandwiched between clay layers and has an artesian force of 210 to 230 kN/m^2. Result of pumping tests predicted that, during excavation down to a depth of 28 m, considerable drainage would be required for preventing boiling and heaving. The " sakauchi" method by the use of diaphragm wall was employed. The diaphragm wall was penetrated into the clay layer by more than 50 cm so that the ground water in the first water bearing layer would be cut off during excavation. For drainage purpose, sixteen deep wells were installed inside of the earth retaining wall, and the ground water in the first and second water bearing layer was pumped up simultaneously. Total quantity of drainage was predicted by the Thiem's equation as 14.3 m^3/min. The maximum quantity of pumped water recorded during excavation was 16.3 m^3/min and excavation work was performed safely.

Case 9 is an open excavation for transit subway system which crosses a river. Cross section of the excavation is 14.6 m x 19.1 m and the excavation level is at 20.6 m below the river bottom. Subsoil is alluvial loose sandy soil of 10 m thick, below which is a diluvial stiff clay of 20 to 25 m thick, then medium to dense sand and gravel layers continue. At a depth of 35 m below the river bottom, a hard shale layer is found as base rock. Underground water is separated into two formations, free surface flow in the top sandy soil and confined artesian flow in the diluvial sand and gravel. Since the boiling was a main problem at the final excavation level where the water flow in deep sand and gravel was under an artesian condition, a series of tests was made to study boiling mechanism under confined flow state and to find factors controlling the boiling failure during excavation. In the tests, the ground near sheet pile was found to move with progressive manner from near the bottom of the pile upwards to the surface where heaving was observed at first and subsequently sudden boiling failure was recognized. The first movement of ground was found to correspond to the critical gradient by Terzaghi and the final sudden boiling was found to correspond to the critical state by Kochina. Based on the test results and theoretical analysis, the factor of safety against boiling was expressed in terms of the excess pore water pressure at the bottom of the sheet pile, and this excess pore water pressure was to be measured for safety control. Automatic measuring system of pore water pressure was provided and linked with alarming device. State of safety during excavation was represented as the " water head path " in a chart, in which the safety factor against boiling is indicated in water head vs excavation level space. At final stage of excavation, the computer simulation of dewatering indicated the state of 'caution' or 'danger'. The excavation procedures were, then, modified for increasing safety against boiling. Small scaled piping at a few points and seepage near the wall were found during the final stage of excavation, however, the excavation was completed safely.

Case 10 is featured by the fact that a subway shield construction had been planned to start in the vicinity of this work at a similar depth. Subsoil at the site consists of sand, silt and gravel of the Tokyo diluvial formation with high N values. In order to execute both the underground works safely, the control values were determined to predict the possible underground phenomena due to progress of works, under cooperation of two bodies concerned. In the excavation side, continuous basement diaphragm wall was built to make use of its air and water tightness and its rigidity, and also the toe of the diaphragm wall was penetrated into the lower silt layer and chemical grouting was made at the back of the wall, and preload was introduced into the struts. In the shield work side, larger size of bolts were used in the lining and the chemical grount was injected into the ground to prevent loosening of the surrounding soil. In designing of earth retaining wall for the excavation, 1.96 N/cm^2 of pneumatic pressure was added. During construction of the excavation and the adjacent shield work, it was observed that the axial force in the struts fluctuated more by the temperature change than by the passing of the shield work. In practical point of view, the displacement of the wall and the axial force of the struts were found not to be affected by the passing of the shield work. It was found, however, in the fourth stage bracing that the axial force at the part in the vicinity of the shield side showed larger value by 12.7 kN/cm^2 than those in other parts. This was considered to be due to the influence of the pneumatic pressure, however, the increment in the lateral pressure due to pneumatic pressure was 45 to 65 % of the predicted one. In the shield work side, it was observed that the lateral displacement of the lining was larger at the portion in the vicinity of the excavation side, however, the maximum value of displacement was 6 mm against the control value of 12 mm.

Case 11 is an excavation for highway in Tokyo at a site where other street underpass construction, subway construction and utility culvert construction were being made. Depth of excavation was 19.5 m below the ground level. Subsoil at the site is 6 m of hard load, 4 m of silty clay, 6 m of fine sand, then clay and sandy silt continue. H piles were driven down to a depth of 24 m and wood plank wall was inserted between the H piles. The lateral earth pressure was found to be of triangular distribution with the coefficient of pressure of about 0.5. As the excavation progressed, the pressure decreased and changed its distribution from triangular to trapezoidal shape. During the fourth stage of excavation down to a depth of 13 m, the measured lateral earth pressure was about 65 % of the design value, however, in the fifth stage down to a depth of 18 m, the measured value was about 20 % larger than the design value. The maximum settlement of the surrounding ground was about 10 mm and its ratio to the excavation depth was 0.5 % and its extent of occurrence was 80 % of the excavation depth.

Geotechnical Aspects of Coastal Reclamation Projects in Japan (Ad hoc Committee of Case Studies of Coastal Reclamation)

Japan is composed of a group of island, 80 % of the area is mountaineous, so coastal region has extensively been reclaimed for development of industrial, residential and harbour areas. Total area of reclamation for the period from 1953 to 1980 is 60,000 ha, most of which located on alluvial plains where soft marine clay prevails. This paper contains five cases of reclamation projects. In the following, a little more detailed review will be given. The reason is that quite a lot of works have been done in Japan in soft cohesive soils, however, almost all of them have been published only in Japanese language.

Ohgishima Island is a man-made island located on soft alluvial marine clay of 15 m to 40 m thick with water depth of 10 m on an average. 85 million m^3 of sand was used for filling an area of 515 ha. This part of paper describes measurements of density and N value by SPT test of filled sand, which is important control item against possibility of occurrence of liquefaction during earthquakes. It was found that the density of filled sand was mainly dependent on whether the sand was deposited below or above the sea level, irrespective of filling method. A relationship between N value, relative density and effective overburden pressure is proposed for filled sand deposited below the sea level, which is a little different from well known relationship proposed by Gibbs and Holtz. Vibratory compaction resulted in great increase in N value, which could not be explained by increase in relative density alone. As the result of in-situ pressuremeter tests, this great increase in N value by vibratory compaction was found to depend on increase both in relative density and coefficient of earth pressure at rest K_o .

Kobe Port Island is a man-made island constructed on soft alluvial clay of about 15 m thick, with water depth of 10 m. 80 million m^3 of Masa-soil (weathered granite) was used as the fill over an area of 436 ha. In the early phase of designing, a geotechnical interest was confined to the surface alluvial layer, because underlying alternating layers of hard sand-clay interlayered stratum and stiff diluvial clay layers at great depth were considered to have very high value of consolidation yield stress p_c. Main topic in this part is observation of consolidation phenomena by preloading work for a hospital to be built in the centre of the island. The duration of preloading was two years and ground behaviours were observed. At the time of start of preloading, settlement of the sea bed was 3 m on an average over an entire area of the island, thickness of the fill being about 20 m. Total consolidation load under the preloading embankment was about twice of that of other area, so it was expected that an increase in vertical stress in deeper stiff layers might approach the consolidation yield stress, and compression of each soil layer was measured by the multi-layer settlement gauge. It is noted that the diluvial clay layers at depths exceeding 70 m below the sea bottom caused almost the same amount of settlement to that of the top alluvial clay layer. Back analysis of these compression of deep soil layers has not been made thoroughly, however, this result may indicate an importance of settlement analysis in reclamation works in deep sea areas.

West Hiroshima Reclamation is a multi-purpose project, located at the west coast of Hiroshima city. Water depth at the site increases towards the offshore ranging from 0 to 8 m. A uniform clay layer of 16 m to 26 m thick is underlain by sand and gravel layer. Liquid limit of the clay is 70 to 150 % with plasticity index of 40 to 110, which are typical of soft marine clays in Japan. The clay is in normally consolidated state. Detailed description of construction is given. For construction of revetment on soft clay, different kinds of improvement are applied in step loading process. In one region, sand compaction piles, 80 cm in diameter with spacing of 1.8 m, were driven into the clay for length of 11 m. The strength of clay was found to decrease by 5 to 30 % immediately after the pile driving, however, it recovered to its original strength in one month time. In stability analysis, design strength of clay was that reduced by 30 %.

In the other region, vertical sand drains, diameter of 40 cm with spacing of 2.1 m, were installed for a length of 18 m. Predicted settlement rate was found to be much higher than the measured one. By fitting the observed settlement curve with Barron's theoretical curve, the coefficient of consolidation c_v was found to be 4.8×10^{-2} cm^2/min, whereas the design value was 7.2×10^{-2} cm^2/min based on conventional oedometer test results. Reason of this reduction in c_v value was attributed to smear effect in displacement type sand drains, head loss in a long drain well or way of interpreting the consolidation test results. Based on this finding, loading step was increased from four to five. By the use of this reduced c_v value, calculated values of settlement of 5.93 m for three years compared well to the measured settlement of 5.75 m.

Expansion of the Hiroshima Airport required a reclamation towards the sea. Water depth is 3 to 7 m and soft alluvial clay extends to a depth of 30 m and is underlain by a stiff sand and gravel layer. For the purpose of minimizing the unequal settlement of runway and also for increasing strength of clay below revetment, vertical sand drains of 50 cm in diameter with 2.5 m spacing were installed in these limited areas. Reclamation work covers an area of 520 m x250 m, so it presents situation suitable for proving the effectiveness of vertical sand drains under a one-dimensional condition. The clay layer showed a typical multi-layer condition, where c_v value of the top and bottom portion was much larger than that of the middle portion.

Considering an accuracy in the time fitting method for clays of high c_v value, consolidation tests with longer drainage path were carried out. These tests gave larger c_v values than those obtained by conventional consolidation tests. Consolidation settlement in the un-improved area showed an excellent agreement with the calculated one when the larger value of c_v was used in each layer, together with the use of the finite difference approach for the multi-layer condition. As for the sand drain area, settlement analysis by the use of the larger c_v values seemed to overestimate the rate of settlement, though the amount of settlement of .4 m for five years was very close to the observed one. Vertical sand drains in this work proved their effectiveness in reducing the residual settlement, however, settlement in the un-improved area was going on with an appreciable rate. This differential settlement caused development of cracks in the runway pavement, and the whole pavement had to be reconstructed.

Osaka South Port Project started in 1958 for new port town and completed in 1980. Water depth is 3 to 9 m. Soft alluvial clay of 15 m to 22 m thick is underlain by alluvial sand layer. The clay was in normally consolidated state. The soil condition in this project is the worst among the five cases in this paper, because the clay dredged from a soft marine deposit, which was in a liquid state, was used as the fill material. Reclamation method adopted in this site was combination of vertical drains and dewatering. Sand mat was placed on the surface of original sea bottom. A number of drain wells were installed in the sand mat, then dredged clay was poured on the sand mat. Pumping-up operation was started during the pouring of dredged clay. This method was to utilize the weight of sea water as well as the atmospheric pressure as the consolidation pressure, without possibility of occurrence of failure of sea bottom clay. Prior to actual work, a field test was performed, where the observed rate of settlement was much higher than that predicted by the Terzaghi's consolidation theory. At this time, new consolidation theory was provided by Mikasa, which took into account the change in k, m_v , thickness of soil element and consolidation stress during consolidation process, also considering the effect of self weight of soil. This new theory was able to explain the result of observation of this field test.

The first soil improvement work by dewatering was carried out in 1963-1970 mainly for improving the alluvial sea bed clay. The sand mat was of 1.5 m thick and 108,000 sand drains of 13 m long and 111 set of drain wells were installed. This work proved effectiveness for improving the alluvial clay, however, an upper portion of the clay fill was not sufficiently improved as anticipated. The second work was carried out in 1973-1978. At first sand drains were installed in the sea bottom clay. After the sand mat was placed on the clay fill, packed drains of 12 cm in diameter or cardboard drains were penetrated through the clay fill to the sand mat. Then 5 to 8 m thick sandy fill was placed on the upper sand mat. In this case, therefore, the clay fill was sandwiched between two sand mats. Decrease in void ratio and the observed time-load-settlement relationship were excellently explained by the use of the new consolidation theory by Mikasa. Volume of the clay fill was reduced to 2/3 of the initial value and the clay fill was found to be brought to an over-consolidated state through this dewatering method.

In the Appendices to this paper, the followings are shown by Mikasa ; consolidation of clay with consideration of non-linear stress-strain relationship, summary of general one-dimensional consolidation theory for highly compressible clays, interpretation of Terzaghi model from the standpoint of new theory, method of calculation for vertical drains, determination of soil parameters from consolidation tests, significance of some new terms and symbols adopted in this paper.

Embankments Works on Soft Grounds (Ad hoc Committee of Case Studies of Soft Grounds)

This paper describes geotechnical problems encountered in construction of embankments for high way, railway and land development in Japan. Detailed information of soft ground in Japan is given together with typical range of values of index properties as well as mechanical ones. Case records of four large scaled projects and eight small scaled test embankments are given. One of major interests in these case records is the effectiveness of soil improvement method. It is seen that majority of cases indicate that the effectiveness of vertical sand drains can be seen in increasing the soil strength to some extent, however, it is not so marked in accelerating settlement as compared with the settlement record obtained in non-drained part.

As for accuracy of predicting consolidation settlement in soft soils, 57 cases are examined, in which 13 cases are with sand drains, 18 cases with sand compaction piles and 26 cases without any kinds of improvement. Frequency of the case, in which the observed settlement is in the range of $\pm$ 20 % of predicted settlement, is 91 % for the case of sand drains, 47 % for sand compaction piles and 52 % for non-treated. In this examination the 'observed' settlement is that predicted from the observed record by the use of hyperbolic fitting method. This finding seems to be favourable for the sand drains.

CONCLUDING REMARKS

Volume of the twenty two invited papers in the Case History Volume is equivalent to some hundred fifty papers for the Conference Proceedings. This would imply difficulty in presenting facts in practice as well as discussion on this theme of " prediction and performance ". In order to improve an accuracy in prediction, it is desired to compare the answers obtained by the use of unified procedures of obtaining soil parameters and unified desing method. In the case of excavation works reviewed herein, this seems to be accomplished to large extent. Finally it can be said that most of papers in this Case History Volume are good examples of well-documented case histories, and it is sure that this Volume will be a valuable reference material in geotechnical engineering.