GENERAL REPORT

TECHNICAL SESSION I

STRESS - DEFORMATION AND STRENGTH CHARACTERISTICS INCLUDING SOIL DYNAMICS

Z.C. MOH

Principal, Moh and Associates

R.O.C.

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Technical Session 1
Stress-Deformation and Strength Characteristics Including Soil Dynamics

Z.C. MOH Principal, Moh and Associates, Taipei, Taiwan

The purpose of a general report is to provide a review and critique of the papers presented in a session. The 24 papers assigned to session 1 of this Conference on Stress Deformation and Strength Characteristics (including Soil Dynamics) covers a wide range of topics. Most of the papers are not related to each other and they do not fall into groups. To give a critical review of all the papers in this Session will almost require to write comprehensive treatises on several subjects. The State-of-the-Art of Stress Deformation and Strength Characteristics up to the year of 1976 has been critically reviewed and excellently presented by LADD et al in their paper presented at the Ninth International Conference on Soil Mechanics and Foundation Engineering in Tokyo in 1977. Although there are a large number of papers which have been published on this subject during the last two years, there appears to be no new break through.

This report will attempt to assemble the 24 papers in this session in an orderly manner and to summarize the salient contents so that the participants can select papers of interest for more detailed study. The 24 papers can be broadly divided into the following four groups:

(1) Behavior of cohesionless soils - 9 papers

(2) Behavior of cohesive soils - 6 papers

(3) Evaluation of soil parameters - 5 papers

(4) Soil dynamics - 4 papers

BEHAVIOR OF COHESIONLESS SOILS

Granitic soil, a residual soil derived from in situ weathering of granite and/or gneiss, is commonly found in many countries in the Asia region. This type of soil is particularly abundant in tropical or semi-tropical areas. The physical and engineering characteristics of granitic soils depend greatly on the degree of weathering or decomposition of the parent material which is controlled by the climatical conditions, primarily temperature and humidity. Most of the granitic soils is relatively well graded containing a large amount of coarse particles, e.g. sand and gravel size. Because of its abundance and the generally well-graded characteristics, granitic soils are of interest to geotechnical engineers as good road building material.

According to the current design specifications for highway materials used in many countries, a lot of granitic soil deposits cannot be used for soil-aggregate subbase or base for highway construction due to the upper limitation on the amount of fines (those finer than 74μ or passing through U.S. No. 200 sieve) present in the soil. This limitation has prompted the Thai Highways Department to look into the possibility of more effective use of the extensively available granitic soil deposits in Thailand for highway construction. In their paper, *RANANAND and RUENKRAIRERGSA* present the results of a comprehensive study on the compaction and strength characteristics of 41 granitic soils selected from various parts of Thailand. Previous results (RUENKRAIRERGSA and CHANGSUWARN, 1978) indicate that granitic soils in Thailand, irrespective of their original parent rock being granite or gneiss, derived from the same geological mode of occurrence and have similar constituents. Most of the samples obtained have excessive amount of fines, but the fines are mostly non-plastic. Modified Proctor compaction method was used to determine the compaction curves and the strength of the compacted soil was evaluated by the CBR test. The compaction curves of most of the samples have conventional single peak whilst samples with combined gravel and sand content of more than 75% tend to have one and one-half peak. Unfortunately, data at low moisture contents were lacking to give a more definite conclusion. A linear relationship with a high coefficient of correlation of 0.926 was obtained between the maximum dry density and the optimum moisture content of laboratory tests. This type of relationship was further demonstrated by field compaction control tests employing both standard and modified proctor compaction energy.

One of the findings which have important practical implication is that both amount and range of sizes of gravel in granitic soil play an important role in developing strength. For the granitic soils from Thailand, the optimum gravel content for highest stability as evaluated by CBR tests, was found to be about 25%. For the same amount of gravel content, soil with larger gravel size has higher CBR values. From this extensive study, the authors suggest that some modifications on specifications especially on grain size distribution limit should be made. In this paper, CBR test was used for evaluating strength characteristics of granitic soil. However, the authors did not state clearly whether the compacted samples were subjected to soaking or not prior to the CBR test.

Soaking or submerging of granitic soils often decreases their shear strength. *OHARA and MIURA* discuss the mechanism of the decrease in shear strength of decomposed granitic soils by submerging, especially from the view point of particle crushing phenomenon. Two types of decomposed granitic soils were investigated by means of three types of triaxial test. They included standard triaxial tests, special triaxial compression test with combination of stress-control and strain control, and dynamic repeated triaxial compression tests. All test results indicate that both in static and dynamic loading condition, the shear strength of granitic soils decreases due to submerging. The strength decrease is mainly due to particle crushing accelerated by the action of water. The authors use the terms particle surface area ΔS and plastic work done W to evaluate particle crushing and particle crushing rate. Unfortunately, no explanations were given on how these terms ΔS and W are determined. All the references given are in Japanese, thus the General Reporter was not able to fully appreciate the significance of the work. Testing conditions such as sample preparation by compaction or using undisturbed samples, submerging the sample, etc., are not given in the paper. It is difficult to compare the findings of this paper with those reported by other investigators.

THURAIRAJAH and WIJEYAKULASURIYA present the results of a series of one-dimensional consolidation tests on a lateritic soil by using different pressure-increment ratio and pressure-increment duration of one week. The results show that up to 95% primary consolidation, the relation between the degree of consolidation U and time factor T_v follows Terzaghi's one-dimensional consolidation theory very well. For large values of time in the secondary compression range, Gibson and Lo's theory defines the shape of the settlement-time curve satisfactorily. As reported by many other investigators on different types of soils, pressure-increment ratio was also found to have pronounced effect on the ratio of secondary compression to primary compression of the soil tested.

Strength anisotropy, which has been found in many types of soils, can be generally considered to be contributed by one or both of two factors. They are stress-induced anisotropy and anisotropy of the soil structure or fabric. Three papers in this session deals with the anisotropic behavior of granular soils. *RAMAMURTHY and SHANKARIAH* carried out a series of drained shear tests on laboratory prepared specimens of Ottawa sand. Cubical specimens were prepared by depositing saturated sand freely under water at seven different angles of deposition. All the specimens were consolidated isotropically and then sheared following three different stress paths, i.e., axisymmetric compression, general compression and plane strain compression. According to the testing conditions, the anisotropy introduced into the soil specimens is essentially the soil-structure anisotropy. A strength anisotropy of about 1.1 to 1.7 degrees in the angle of shearing resistance was observed in axisymmetric compression but not in plane strain compression. In fact, the degree of soil-structure anisotropy is small as compare to the effect of imposed stress path. The difference in the ϕ' value between plane strain compression and axisymmetric compression varies between 3.9^o to 5.5^o. The relatively small effect of the soil-structure anisotropy in this study is probably due to the uniform size and shape of the Ottawa sand although no specific data on these properties are given in the paper.

KIMURA et al present the results of a study on effect of anisotropy on the bearing capacity and settlement behavior of footing on dense sand by means of centrifugal model experiments and theortical computations based on Kötter's equation. Drained plane strain compression test was carried out on specimens of a sand with mean grain size diameter D_{50} equal to 0.20 mm and uniformity coefficient C_u of 1.38. Specimens were prepared by depositing the sand through water into a tilting mold with varying angle of inclination and then compacted to a dense state at void ratio between 0.70 to 0.75 by horizontal vibration. Compression tests were carried out until residual state. Test results indicate strength anisotropy due to difference in the deposition angle. The peak ϕ' value differ by 4^o between vertical specimen and horizontal specimen at low confining pressure and the difference decreases with increasing in confining pressure. The residual angle of shearing resistance did not show effect of anisotropy. The authors attribute the strength anisotropy to difference in dilatancy. Based on the test results, bearing capacity values were obtained by the Kötter's equation in terms of anisotropic rigid plastic theory. The results of analysis are presented in terms of ratio of bearing capacity factors. In the case of vertical specimens, i.e. direction of deposition being normal to the bedding plane which is similar to the natural condition, it was found that the conventional method by using isotropic rigid plasticity theory may overestimate the bearing capacity by about 30% for condition of K = 0.9 and ϕ_{max} of 38° to 42°, where K is the ratio of angle of shearing resistance of horizontal specimen to that of vertical specimen. Since conventional method tends to overestimate the bearing capacity to a significant amount, it is on the unsafe side, the effect of soil anisotropy should be carefully examined.

By using TV camera and X-ray, the authors observed the settlement pattern and the state of slip lines inside model ground of footing test in a centrifuge apparantus. Outstanding effect of anisotropy was observed in the settlement characteristics. At various load intensities, settlement values in the horizontal case are higher than that in the vertical case.

NAGARAJ and SOMASHEKAR compare the stress-deformation and strength behaviors of two soils in plane strain compression with that in isotropic triaxial compression. The two soils used in the laboratory experiments are a remolded Kaolin and an Indian standard medium coarse sand. Test results show that for both soils, plane strain tests yield higher effective strength parameter ϕ' in terms of normal Mohr-Coulomb failure criterion. When the effect of intermediate principal stress is taken into account, unique relationships between the maximum deviator stress and the mean effective principal stress are found.

Sampling of undisturbed sand has always been a difficult problem to geotechnical engineer. Sampling of dense sands are known to loosen the soil due to dilatation of sand. Loose sand generally becomes denser by consolidation. The disturbance is usually not only confined to change in density but also to the soil fabric. Large diameter sampler (ISHIHARA and SILVER, 1977) and Osterberg sampler have been used with reasonable success to obtain high quality samples of loose sand. But their use in dense sand deposit is questionable. *MORI and ISHIHARA* describe the technique of block sampling for both loose and dense sand. The sand sampled was located in Niigata where extensive damges occurred during the earthquake in 1964 due to soil liquefaction. Cyclic triaxial compression tests were performed on the block samples and the results were compared with that obtained by using laboratory reconstituted samples. Test results show that for loose sand, no significant difference exists between the cyclic behavior of block samples and reconstituted samples. However, for dense sand, the block samples had about 20 to 50% higher cyclic strength than the reconstituted samples. Since liquefaction is generally not a problem in dense sand deposit, it will be interesting if static test (i.e. normal strength tests) results could also be compared.

There are many soil deposits around the world which cannot be classified properly by its grain size distribution or physical properties because of their peculiar inherent characteristics. In many cases, local or regional geology dominates the soil behavior. Pumice soil is one of those special soil which has mechanical behavior like a material intermediate between granular soil and rock. *YAMANOUCHI and MURATA* present a paper discussing the failure mechanism of undisturbed samples of a pumice soil in southern Kyushu of Japan. In cut slopes, this type of soil was found to fail only in tensile or brittle type of failure mode, rotational type of slip has never occurred. The authors performed a series of consolidated-drained triaxial compression tests on "reformed" samples taken from the field with a specially devised sampler. Based on laboratory test results, the authors concluded that this type of soil possess some tensile strength due to a "welding" effect and this component of the strength is almost all consumed by resistance to the plastic deformation. On the basis of stress-volumatric strain relationship, the failure was divided into four stress stages, they are: elastic deformation, two stages of combination of elastic and plastic deformation, and purely plastic deformation. The authors used Griffith criterion to describe failure. In the paper there are a number of items which are not very clear to the Reporter perhaps due to limitation on the length of the paper. The Reporter is not clear about the meaning of "reformed" sample. Does it mean sample trimmed from the field sample or reconstituted sample? Several terms used in the text and figures have not been clearly defined.

To determine the load-deformation characteristics of rock formation or soil formation which are difficult to sample, such as sand-gravel layers, in situ plate loading tests are often utilized. However, the reliability of the test results is sometimes restricted by the size of the loading plate used. This is particularly critical in soil or rock formation with fissures or other irregularities. *MORI et al* describe a large scale plate load testing carried out on a Tertiary sandstone and mudstone formation. The results obtained from the plate loading tests are compared with those interpreted from in situ pressuremeter test by using Menard Pressuremeter. Test results indicate that deformation of the soft rock tested under a rigid loading plate of 2 m in diameter was approximately elastic for range of pressure less than yield value. Settlements predicted from pressuremeter results are found in general in good agreement with the plate loading test. The authors suggest that for heavily loaded foundations particularly for offshore structures where large scale loading test is economically unfeasible, pressuremeter test can be considered as a suitable means for the prediction of the load-deformation behavior of foundations.

BEHAVIOR OF COHESIVE SOILS

Six papers in this session deal with the stress-deformation behavior of fine-grained cohesive soils. *FRYDMAN et al* present a review of the shear strength characteristics of 46 saturated clays from 7 different locations in Israel. The clays are primarily late quaternary clays of the coastal area. These include both marine and lagoon deposits of generally highly plastic, predominantly calcium montmorillonitic clays. Many of the clays are stiff, fissured clays. For the saturated Israel clays, it appears that the in situ vane test leads to a significantly higher value of shear strength whilst laboratory triaxial tests appear to lead to a more reasonable estimate. The ratio of undrained shear strength c_u obtained from vane test to the present overburden pressure p_o does not appear to be related to the plasticity indices of the soils as suggested by LEONARDS (1962). The authors attribute the difference between in situ vane strength and laboratory determined strength to the presence of fissures. They further suggest that the residual vane strength is probably more close to the strength along fissures. The authors also conclude that in situ vane test is not suitable to determine undrained shear strength of overconsolidated clays. This limitation of vane test has long been recognized. In the paper, a series of relationships are presented between parameters related to shear strength, plasticity, in situ void ratio, moisture content, and the effective overburden pressure. These relationships could be useful in predicting the in situ strength value of other Israli clays of similar geological origin.

There are two papers dealing with overconsolidated clays. *OHMAKI* presents a study on artificially prepared overconsolidated samples of a silty clay. Overconsolidation of the specimens was achieved by consolidating the specimens isotropically in triaxial cell. Four series of tests were carried out, they are: recompression and swelling under constant stress ratios {the stress ratio is defined as equal to q/p where $q = \sigma_1 - \sigma_3$ and $p = 1/3\ (\sigma_1 + 2\sigma_3)$}, drained triaxial tests under constant mean effective principal stress, tests along a series of more general stress paths, and a series of consolidated-undrained triaxial tests. Analyses of results were made in regard to stress-strain behavior, soil parameters, and Hvorslev's failure criterion. In the paper, the author suggests that the void ratio at residual state of the overconsolidated clay can be expressed by two critical void ratio lines with different slopes, one at the wet side and the other at the dry side. It is not clear what are the definition or meaning of the so called wet side and dry side. The soil used in the tests contains a large amount of sand (clay 17.5%, silt 50.8% and sand 31.7%), whether its behavior is similar to other overconsolidated clays needs further study.

SADIQ and DANISH report results of a soil investigation for the construction of a port in Pakistan, with special emphasis on the strength characteristics of an overconsolidated clay with overconsolidation ratio, OCR, between 2 to 5. Triaxial compression tests were carried out on core drilled samples of the overconsolidated clay. Variation of strength with soil depth is presented and the values adapted for pile design are shown. However, the paper did not discuss the degree of disturbance of the core-drilled samples which usually involve considerable torsional strain. This is particularly important for clays with relatively low degree of overconsolidation. The authors did not present any discussion on the rational in the selection of soil strength for pile design. No performance data of the foundation are included in the paper.

TOH and DONALD present a simple but adequate review of strength anisotropy in soft clays. The authors discuss the various methods currently being used for measuring strength anisotropy including in situ vane test, triaxial or plane strain compression and extension tests, direct simple shear tests, and true triaxial tests. A comprehensive set of data on a highly plastic Launceston Clay in Australia is presented to illustrate the principles discussed. It is interesting to note that the range of ultimate bearing capacities calculated for a flexible strip footing on a finite layer of homogeneous Launceston Clay is approximately 2:1 by using different type of strength test results. For the analysis, the authors concluded that the best current approach to determine strength anisotropy appears to be a combination of triaxial compression, direct simple shear and triaxial extension testing. True triaxial testing would give the best results but testing is expensive and interpretation is difficult. It should be pointed out that at the present, direct simple shear testing equipment is not commercially available yet. The use of this type of test in practice would therefore be limited. The comparison of using various strength results presented by the authors would be even more valuable if an actual loading test of the footing, or to a less extent a model footing, could be carried out.

TING presents the results of a series of consolidation tests on a poorly compacted partially saturated residual soil from Malaysia. Date indicate that the soil would experience further consolidation when inundated under constant load and the terminal points of the further consolidation fall close to the consolidation curve of a compacted fully saturated soil with the same initial void ratio. This type of behavior is very similar to many naturally occurred, partially saturated "collapsible" soils reported in the literature. For this type of soil, one of the method to take care of additional consolidation due to inundation is by modifying the standard procedure for consolidation tests. The specimen of the partially saturated soil is first consolidated under load without flooding the consolidation cell. Special care should be taken to prevent loss of moisture due to evaporation. When the applied load reaches the estimated design load, the specimen is allowed to adsorb water by inundating the cell. Consolidation due to saturation under constant load can then be measured. *SINGH and AL-LAYLA* utilized this technique to measure the collapsibility of a brackish water sediment in Iraq. The authors attribute the collapse behavior due to water inundation to the collapse of clay buttresses and peals of the flocculated clay particles. These phenomenon brings about a relatively dispersed soil structure from the original flocculated structure. Tests were also carried out to determine the consolidation behavior of natural undisturbed soil and samples after leaching out the water-soluble gypsum content in the soil. Both the compression index and coefficient of consolidation of the soil increased due to leaching. On the basis of microscopic study on thin-sections, the authors state that the leaching process changes the original flocculated structure to a less flocculated one. Theoretically if leaching caused the soil structure changing from a flocculated one to a less flocculated, the coefficient of consolidation of the leached soil would be lower than that of unleached soil as reported by WOO et al (1977) on Bangkok Clay. The opposite effect obtained by the authors could be complicated by the calcium and sulfate ion concentrations of gypsum.

EVALUATION OF SOIL PARAMETERS

Five papers are classified under the group of evaluation of soil parameters. In recognition for the need of a simple method to predict decrease in undrained shear strength of soft clay due to rebound, *NAKASE et al* carried out a study to examine this problem. Artificially prepared mixtures of clay and sand were used in the study to give a wide range of soil textures from clay to sandy clay. Isotropically consolidated-undrained triaxial compression tests and isotropically consolidated-rebound-undrained triaxial compression tests were performed. A wide range of overconsolidation ratio from 2 to 40 was included in the testing program. Test results show that the rate of decrease in strength due to rebound increases with increase in plasticity index of the soil. Change in strength was analyzed in terms of effective stresses at failure and Hvorslev's parameters. Based on the analysis, a simplified method for predicting strength change due to rebound is proposed. This simplified method requires only two undrained shear tests, one on a normally consolidated soil and one on an overconsolidated soil. The authors also state that although isotropic consolidation-rebound was used in the study, the basic expression proposed would not change for anisotropic consolidation and rebound. The study was carried out on artificially prepared mixtures of clay and sand. These soil specimens most likely would not acquire any structure or fabric as the soils in natural condition. If the validity of the proposed method could be extended to natural clays, the value of the proposed method will be greatly increased.

The importance of adapting proper strain-rate for shear testing of soils has long been recognized for saturated soils, BISHOP and HENKEL (1957 and 1962) have proposed procedures for estimating strain rates for undrained triaxial tests in order to attain 95% pore pressure equalization. Problems involved in partially saturated soils are much more complicated due to the presence of both pore water pressure and pore air pressure. No theortical basis has so far been presented to enable determination of strain rates for tests on partially saturated soil. DONALD in 1961 suggested some guidelines for making such an estimate. In the paper by *SATIJA and GULHATI*, the authors present experimental results of constant water content tests and drained tests on one compacted soil. The results showed that Donald's approximate method yields strain rates which are much slower than necessary. Since only data on one soil have been presented and no information on the soil type is made available in the paper, it is not conclusive that Donald's method is not suitable for other types of soils.

BAKER presents a theortical analysis by using an inverse solution to Terzaghi's consolidation equation. Preliminary results of analysis on a limited number of cases indicate that variation of the coefficient of consolidation c_v with void ratio is much greater than commonly assumed. This implies that a faster rate of settlement than that predicted on the basis of the conventional approach. However the solution has a number of limitations - for example the author stated that in respect to the initial pore pressure distribution the present analysis is more restricted than the classical one. In the former case, the initial distribution is restricted to a well behaved functions whilst the classical solution allows initial distribution that are not continuous and do not conform to the boundary conditions. The study is being continued.

The use of penetration tests to evaluate soil properties has had long history. The most commonly used tests are standard penetration test (SPT) and static cone test. Much information has been published relating the number of blows N of SPT and cone penetration resistance q_c with soil parameters. Most of these correlations are nevertheless semi-empirical in nature, therefore, use of these correlations must be exercised with caution. *NATARAJAN and TOLIA* present a brief review of the various relationships proposed between N value of SPT and modulus of elasticity E of sandy soils and also between q_c of cone penetration test and E. Most of the correlations are found to be direct relationships without considering the relative density of the sands and the effect of overburden pressures on these parameters. The authors attempt to propose a new correlation between E & q_c, and E & N with the consideration of relative density and confining pressure. The relationships were obtained by combining the data of three completely unrelated investigations carried out by other researchers. The types of soils involved in the three previous studies were all different whether these data and empirical relations can be combined together remains to be proved. In the present paper, the authors did not offer any supporting evidence to the proposed correlations. Furthermore, the type of cone used for determining q_c is not clearly defined.

DESAI and DESAI present an empirical correlation between a dynamic cone penetration resistance with CBR values of a sandy soil. Since only one soil was tested, the usefulness of the proposed correlation appears to be very limited. In the conclusion, the authors stated that dynamic tests are quick and unskilled. Many researchers have pointed out that penetration test results are often affected by the manner in which the tests are carried out. In order to obtain any meaningful results, penetration tests should be carried out under careful supervision certainly not "unskilled".

SOIL DYNAMICS

Four papers are assigned to this session under the heading of soil dynamics. In fact, three of them deal with dynamic foundation design or soil-structure interaction and one paper is on soil stabilization. A brief review of these papers will be made below. It is suggested that discussion on these papers probably should be made in other sessions.

YEH et al present a dynamic response analysis of an embeded structure under a dynamic harmonic torque. In the analysis, a mathematical hybrid model is developed by dividing the structure and surrounding soil medium into two regions. The far region is considered as a half-space with a hemispherical pit and in the near region, the structure with a part of the soil medium is modeled by a conventional finite element mesh. By enforcing the continuity of displacements at the nodes of interface, equations of motions for the combined system were obtained. In the paper, the effect of embedment on the incremental stiffness and response spectra of a rigid circular embedded structure under a harmonic torque is presented. The results of analysis on the relationship between relative embedment and incremental stiffness are very close to that obtained by NOVAK and SACH's simplified method. For the response spectra, embedments cause an increase in the resonant frequencies and reduction of resonant amplitudes.

BAGCHI and RAMAN present a case study of dynamic response of frame foundations. The case reported involves five high speed centrifugal compressors supported on frame foundations. The foundations are supported on friction and bearing piles resting on a medium fine sand approximately 20 m below the ground surface. To evaluate the dynamic soil-pile stiffness required for the design, forced vibration tests in both horizontal and vertical modes of vibration at various excitation levels and free vibration tests were carried out on piles in situ. Effects of fixity and group action of piles were considered in the evaluation of vertical and horizontal stiffnesses of piles. Unfortunately due to limitation of space, these test results are not presented and discussed in the paper. Three well known simplified models were chosen for the dynamic analysis of foundations. Amplitudes of vibration have been measured on foundations of compressor units 1 and 2 since only two compressors are commissioned, so far. The measured values were found to be in fair agreement with the computed amplitudes. Although the authors did not state clearly in the paper, it appears that both the theoretical analysis and field measurements were made on single compressor. It will be of great value if the performance study could be extended to evaluate the dynamic response of foundations for the cases of simultaneous operation of more than one compressor.

OHASHI et al present a theoretical analysis of aseismic design of piles and caissons considering deformation of ground. Three types of soil layers were considered, including layer with a constant deformation modulus, layer with a deformation modulus increasing proportionally with depth and layer consisting of two strata with different deformation modulus. The authors stated that computer programmes and graphs have been worked out for determination of displacement, deformation, and bending moment of piles and caissons but these are not included in the paper for obvious reason of lack of space. Probably also due to limitation on the length of the paper, detailed discussion on the assumptions and derivation of the theoretical equations are not presented. These are probably given in the references written in Japanese. In the paper, the authors also present a comparison of the analytical results with the results obtained from laboratory model tests.

Liquefaction of loose sand deposit under earthquake or other dynamic loading has long been concern to geotechnical engineers. This problem has drawn greater attention since the well known Niigate earthquake in Japan in 1964 where considerable damages to structures had occurred due to liquefaction. *FUJINAMI et al* present a liquefaction safe design of an underground cable by stabilizing the backfill material. Along the route for a proposed electric power transmission cable for the northeastern part of Tokyo, the site is covered by a thick layer of alluvial sand with very low standard penetration resistance. Seismic analysis by using laboratory cyclic triaxial compression test results indicate that the alluvial deposit has liquefaction potential. Due to limitation of the presence of the cable pipes, the backfill material can only be compacted to a relatively low density, about 80 to 90% of the maximum dry density. It was found that backfills compacted at these densities have high liquefaction potential. From the analysis, the authors however conclude that the extent of liquefaction of the surrounding alluvial sand is not likely to cause extensive soil movement and therefore is not likely to affect the cable adversely. Studies were therefore concentrated on the possibility of stabilizing the backfill material. Various combinations of admixtures, including cement, bentonite and water glass mixtures. It was found that mixing the backfill sand with 1.5% by weight cement and compacting to a density of 90% of maximum dry sensity would substantially increase the liquefaction resistance. The authors further suggest that cone test may be used to check the in place strength variation of the treated backfill. In the liquefaction analysis, results of laboratory cyclic strength tests were used. It would be of interest to know how were the undisturbed sand samples obtained. Since sampling of undisturbed sand is always a problem to the geotechnical engineers, many liquefaction potential analyses utilized the empirical method of standard penetration test results such as that suggested by SEED (1976) and NISHIYAMA et al (1977). It would also be of interest if results of analyses by using these semi-empirical methods can be compared with those obtained from analytical method by using laboratory determined strength values.

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