SOILS AND FOUNDATIONS Vol. 15, No. 3, Sept. 1975
Japanese Society of Soil Mechanics and Foundation Engineering

ONE-DIMENSIONAL VOLUME CHANGE CHARACTERISTICS OF SANDS UNDER VERY LOW CONFINING STRESSES

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ABSTRACT

One-dimensional compression and swelling tests were conducted on loose sands under very low confining stresses including liquefied condition. The vertical effective stresses from zero to 0.3 kg/cm² were applied by means of vertical seepage through a column of sand 28 cm in diameter. Liquefaction of the specimens was caused by upward seepage or impact. The relationship between the void ratio and the logarithm of vertical effective stress during post-liquefaction compression was nearly linear over the stress range from 0.002 kg/cm² to 0.2 kg/cm². Even under a very low confining stress, the sand that had not undergone liquefaction was considerably less compressible than the sand that had been liquefied. The coefficient of volume change during unloading was around one-fifth of that during post-liquefaction compression for loose or very loose sand from Niigata. On the basis of the test results, the average vertical strain of a horizontal layer of sand during consolidation under its own weight following liquefaction was estimated to be on the order of a few per cent.

Key words: compression, consolidation test, earthquake, laboratory test, liquefaction, pore pressure, rebound, sand, seepage, settlement, stress-strain curve, void ratio

IGC: D5/E2/E7

INTRODUCTION

It has been shown that boundary value problems involving zones of completely liquefied sand could be analyzed as transient flow problems in which the coefficient of permeability and the coefficient of volume change are the key variables (Ambraseys and Sarma, 1969; Yoshimi and Kuwabara, 1973; Yoshimi, 1973). While the coefficient of permeability can be estimated without much difficulty, there has been little quantitative information on the volume change characteristics of liquefied sand in one-dimensional compression under low confining stresses.

The object of this paper is to determine experimentally the one-dimensional stress-strain relationship of sand under very low confining stresses including a completely liquefied condition, and to apply the test results to boundary value problems involving a horizontal layer of liquefied sand.

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Written discussions on this paper should be submitted before July 1, 1976.

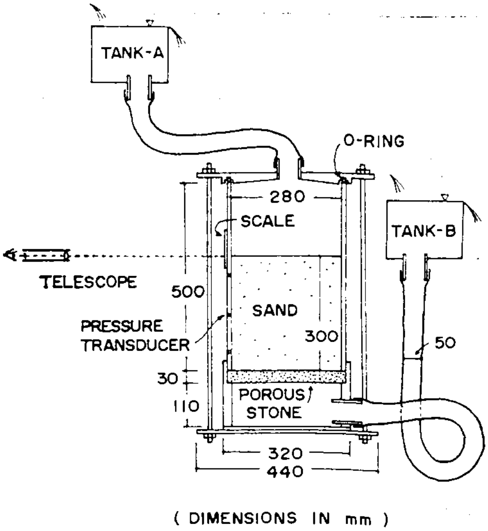

Fig. 1. Apparatus for seepage tests

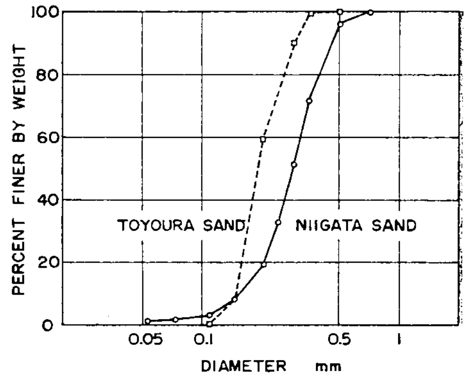

Fig. 2. Grain size distribution of sands tested

TEST APPARATUS

For conducting one-dimensional compression tests on saturated sand, vertical seepage force was utilized to apply the vertical effective stress in a lower stress range (0 to 0.2 kg/cm^2), and a conventional oedometer was used in a higher stress range (above 0.1 kg/cm^2).

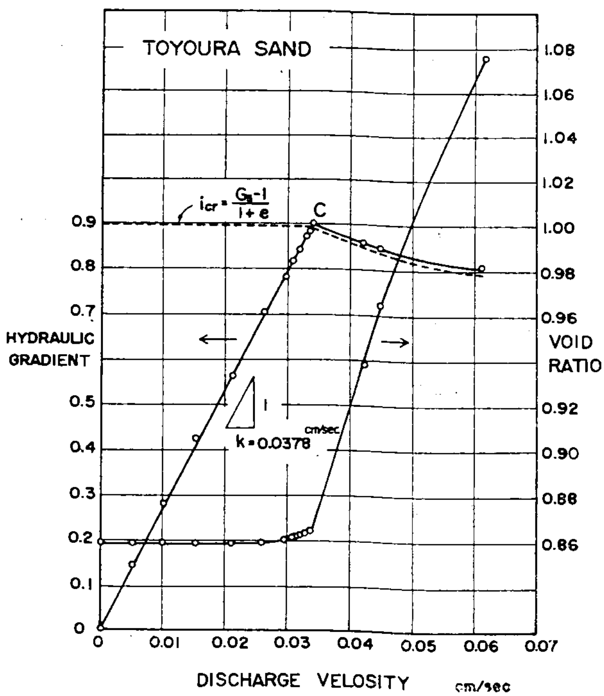

Fig. 3. Typical results of upward seepage test

Fig. 1 shows the apparatus for the seepage test. The sample container is a transparent plastic cylinder having a porous disk at the bottom. The top and bottom of the container are connected to Tanks A and B, respectively, with flexible vinyl tubes 50 mm in diameter. Three pressure transducers of the wire strain gage type are installed in the wall of the container to measure the pore water pressure. The transducers have a full range of 2.0 kg/cm^2 with diaphragms 8.0 mm in diameter which are separated from the sand by 0.074 mm mesh screen. A telescope and a glass scale with 0.1 mm graduation are used to measure the vertical displacement of the surface of the sand.

The physical properties of Toyoura sand and Niigata sand used in this study are given in Fig. 2 and Table 1. The maximum and minimum dry densities were determined

Table 1. Sands tested in this study

Origin of sand	Toyoura	Niigata
Specific gravity of solids, G_s	2.65	2.72
Maximum void ratio, e_{max}	0.978	1.000
Minimum void ratio, e_{min}	0.612	0.628
Effective grain size, D_{10} (mm)	0.15	0.16
Uniformity coefficient, U_c	1.4	2.0
Coef. of permeability at $D_r=32\%$, k(cm/sec)	0.0378	0.0766

using a stainless steel container having the inside dimensions of 59.9-mm diameter and 40.0-mm depth and the outside dimensions of 79.6-mm diameter and 44.5-mm height. For the maximum density, oven dry sand was placed in the container in 10 to 12 lifts. After each lift the container was tapped horizontally 40 times with one hand using a wooden rod 20 mm in diameter. For the minimum density, oven dry sand was poured gently in the container from a negligible height through a paper funnel. For both the maximum and minimum densities the excess sand was removed by quickly sliding a straightedge, taking extreme care not to jar the container. Fig. 3 shows the result of a typical upward seepage test on the Toyoura sand. It can be seen in the figure that departure from the Darcy flow is accompanied by a sharp increase in the void ratio, and that the hydraulic gradient beyond the critical point (marked C) agrees with the computed critical hydraulic gradient which is shown in dashed line.

TEST PROCEDURE

Three types of seepage tests were conducted by varying the elevations of Tank A and Tank B and conventional oedometer tests were run on dry specimens of the sands in an oedometer 60 mm in diameter and 20 mm high. The procedure of the seepage tests are described below.

(1) *Consolidation following liquefaction by impact*: While upward seepage at a hydraulic gradient approximately 80% of the critical gradient was maintained, liquefaction was induced by hitting the container with a wooden mallet. As shown in Fig. 4 (a) the peak excess pore water pressure after the impact resulted in complete liquefaction with the upward hydraulic gradient reaching the critical value. The oscillograph records of Fig. 4(b)

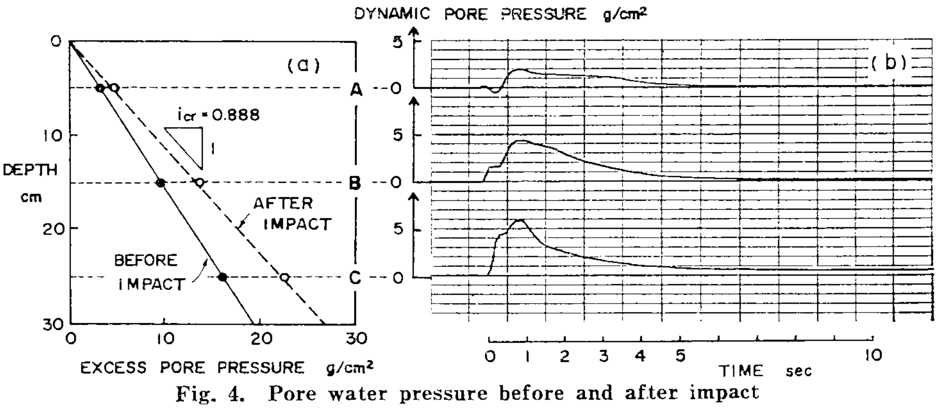

Fig. 4. Pore water pressure before and after impact

show that the pore water pressure at the three depths reached the peaks simultaneously. Following the impact the upward flow was gradually reduced by lowering Tank B. The direction of flow was then reversed by raising Tank A until the vertical effective stress at the bottom of the specimen reached approximately 0.3 kg/cm^2.

(2) *Consolidation following liquefaction by upward seepage*: After the specimen was caused to "boil" by upward flow at a discharge velocity about three times the critical value at Point C in Fig. 3, the specimen was gradually compressed in the same manner as in the case of the consolidation following liquefaction by impact.

(3) *Swelling-Recompression*: After the vertical effective stress was reduced by upward seepage to a very small value, approximately 0.002 kg/cm^2, the specimen was recompressed without applying a shock.

TEST RESULTS

During the seepage test, both the vertical stress and the density vary with depth. But because only the displacement of the surface of the sand is measured, the density of the sand below the surface is estimated indirectly in the following manner. The oblique lines in Fig. 5 show the vertical effective stress at various loading steps. At Step I where the effective stress is very small, the density γ_1' is assumed constant throughout the specimen. At Step II, the density of that portion of the specimen where the stress exceeds the maximum stress at Step I is assumed to reach a higher value γ_2'. Accordingly, at Step n, the specimen is divided into n layers whose density increases from the surface down starting with γ_1'. For example the portion of the specimen where the density is equal to γ_4' moves upwards with the loading step as shown by the shaded area in Fig. 5.

The above assumption enables one to compute the density values γ_2', γ_3', ..., from the initial density γ_1' and the surface settlement. The vertical effective stresses can be computed from the density and the hydraulic gradient, and the void ratios from the density and the specific gravity of solids.

The results of the compression tests are plotted in Figs. 6 and 7. The e-p curves in Figs. 6 (a) and 7 (a) show that most of the compression takes place under small stresses. As shown in Figs. 6(b) and 7(b), the e-log p curves in the lower stress range are nearly linear and their slopes are nearly independent of the method of producing liquefaction, whether by supercritical upward seepage or by impact. For the Toyoura sand, particularly, the slope of the e-log p curves is nearly independent of the initial void ratio, probably because the liquefaction has destroyed the effect of the previous stress history.

According to Figs. 6 (b) and 7 (b), the slopes of the e-log$p$ curves for the range of

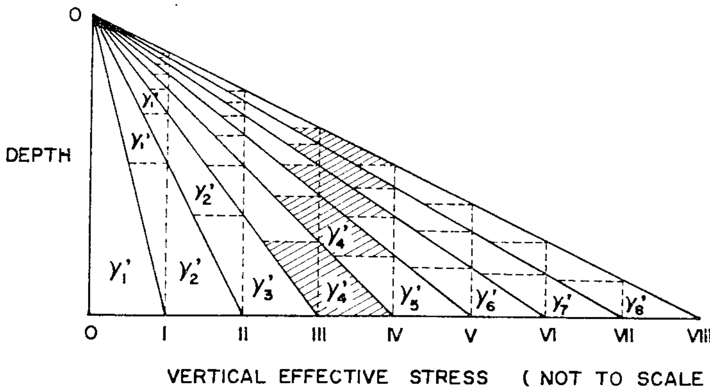

Fig. 5. Procedure for estimating soil density

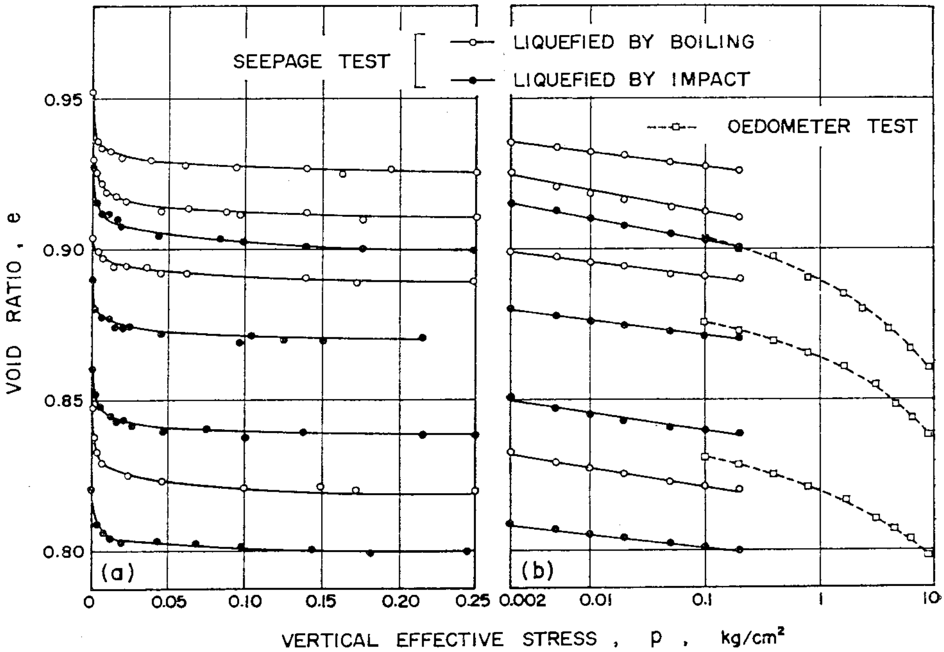

Fig. 6. One-dimensional compression test on Toyoura sand

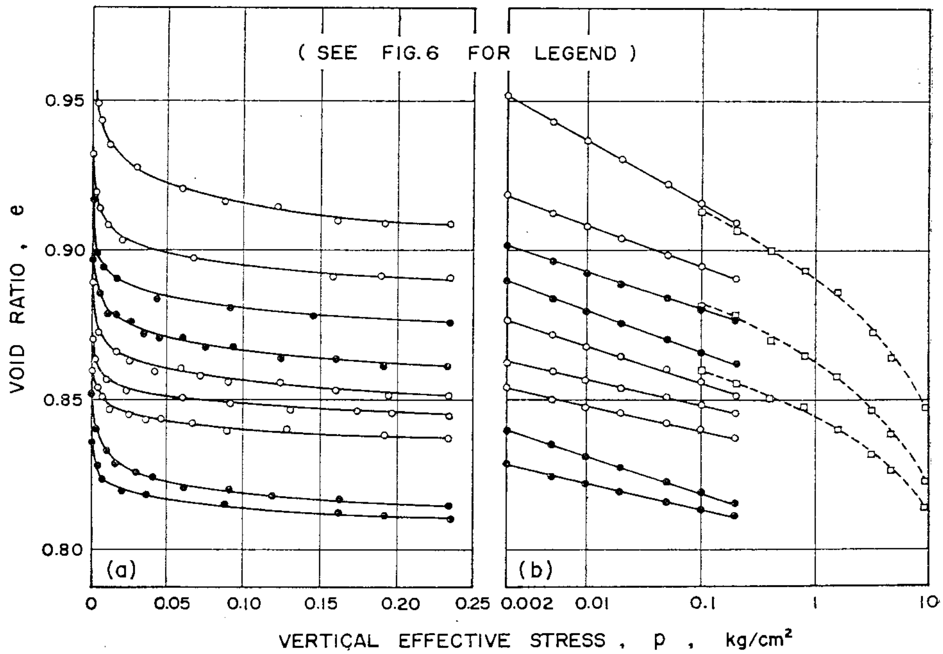

Fig. 7. One-dimensional compression test on Niigata sand

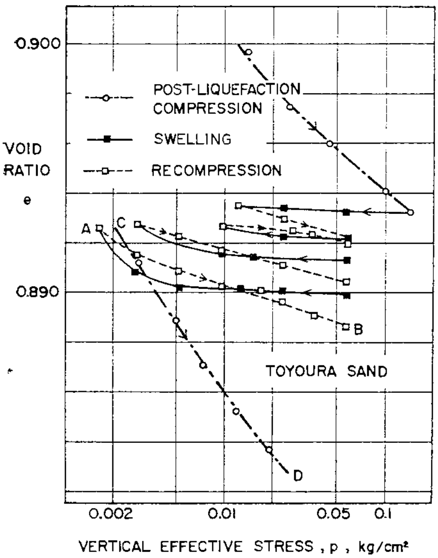

Fig. 8. Compression and swelling tests under low stresses

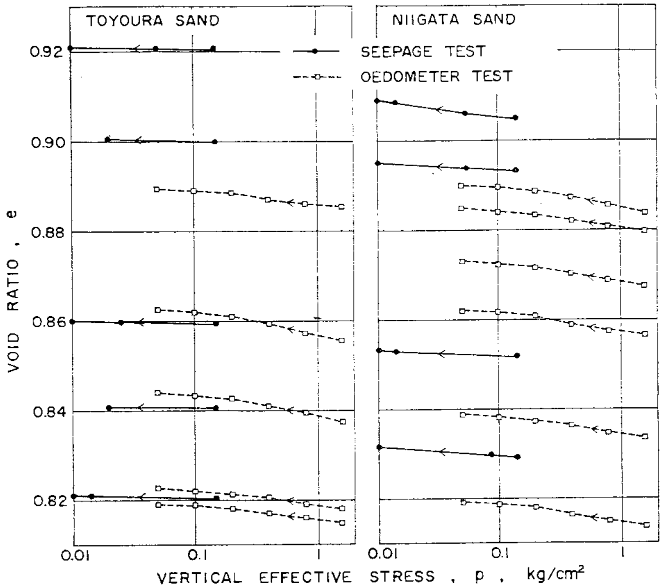

Fig. 9. One-dimensional swelling tests on Niigata sand

stresses from 0.002kg/cm² to 0.20kg/cm² are from 0.005 to 0.007 for the Toyoura sand, and from 0.009 to 0.021 for the Niigata sand.

The results of the oedometer tests in Figs. 6 and 7 show that the slope of the *e*-log *p* curves between 0.1kg/cm² and 0.2kg/cm² is nearly identical with that of the seepage tests, and that the linearity of the *e*-log *p* curves is limited to stresses below a few tenths of one kilogram per square centimeter.

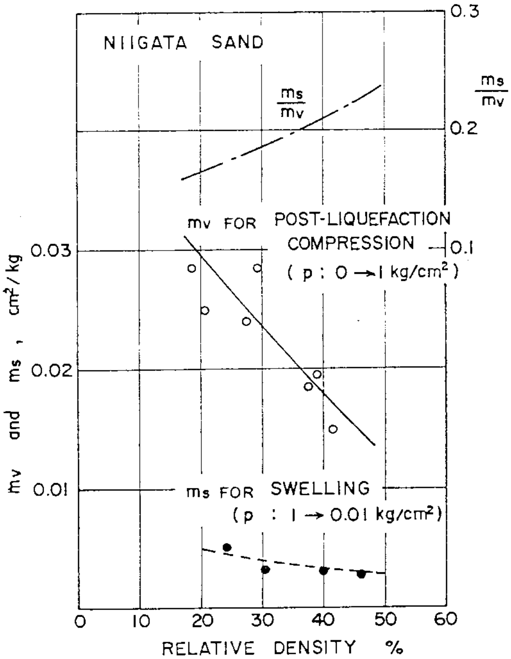

Fig. 10. Coefficients of volume change for Niigata sand

The result of a sequence of swelling and recompression tests following a post-liquefaction compression test is shown in Fig. 8, together with the result of another post-liquefaction compression test, CD, started at a lower void ratio. It can be seen in the figure that the *e*-log *p* curves during recompression are nearly linear while those during swelling are curved in lower stress range. It is interesting to note that curves AB and CD show markedly different slopes (compression indexes) despite the proximity of their points of origin, A and C. Obviously, there is an important difference between the state of very low effective stress as represented by Point A and the liquefied state involving collapse of soil structure as represented by Point C.

Fig. 9 shows e-log p relationship during unloading. The oedometer test results for the Toyoura sand and the Niigata sand are quite similar in shape. As to the seepage test results, on the other hand, the Niigata sand appears slightly more expansible than the Toyoura sand.

The coefficients of volume change of the Niigata sand during post-liquefaction compression and swelling, m_v and m_s respectively, are determined from the results of Figs. 7 and 9, and plotted in Fig. 10 against the relative density. Both m_v and m_s are average values over the range of stress indicated in the figure. Also shown in the figure is the ratio of the coefficient of volume change for swelling to that for post-liquefaction compression, m_s/m_v, which is smaller than 0.24 for the relative density less than 50 per cent. The significance of this figure will be discussed in the following sections.

SETTLEMENT OF LIQUEFIED SAND

The results of the one-dimensional consolidation tests on liquefied sand can be applied to estimating the surface settlement of a horizontal layer of liquefied sand. For example, Fig. 11 shows the vertical strain and the surface settlement of horizontal ground composed of the Niigata sand, estimated from Fig. 7 (a) on the assumption that the void ratio of the sand is constant at 0.849 (in solid lines) or 0.931 (in dashed lines). As shown in Fig. 11(b), the rate of increase of the strain becomes small when the depth exceeds about two meters, and that the strain falls in the range of 1 per cent to 3 per cent. These results are in good agreement with the findings by Lee and Albaisa (1974) which are based on their volume change measurements following cyclic loading triaxial tests.

Fig. 11 (c) shows that the surface settlement of a layer of liquefied sand 10m to 15m thick will be from 10cm to 40cm, which appears to be a reasonable estimate of the settlement of the ground surface in Niigata during the Niigata earthquake of 1964.

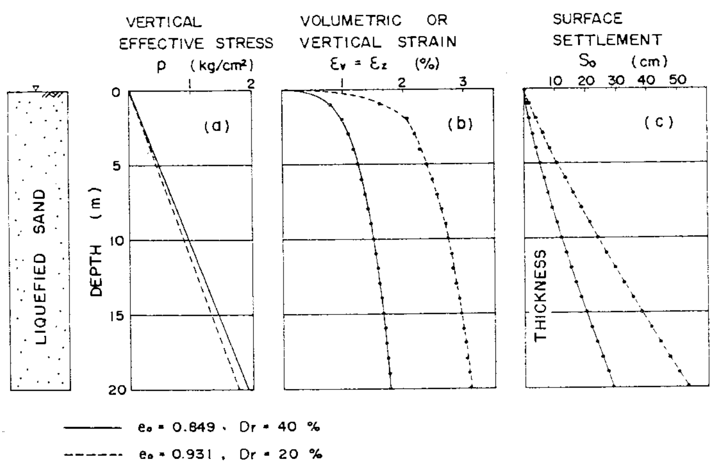

Fig. 11. Vertical strain and surface settlement of a liquefied layer of the Niigata sand

EFFECT OF SUBSURFACE LIQUEFACTION ON THE STRENGTH OF SURFACE SOIL

In this section, the results of the authors' analytical studies (Yoshimi and Kuwabara, 1973; Yoshimi, 1973) are reviewed in the light of the experimental data obtained in the present study. In the one-dimensional problem as shown in Fig. 12, the initial condition

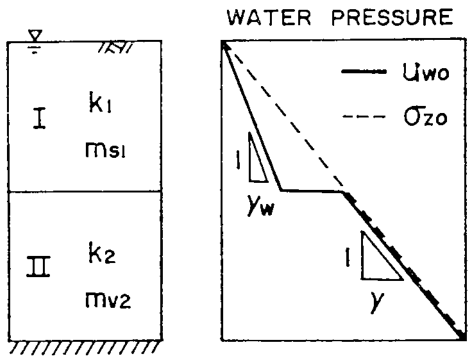

Fig. 12. Pore water pressure immediately after liquefaction of Stratum II

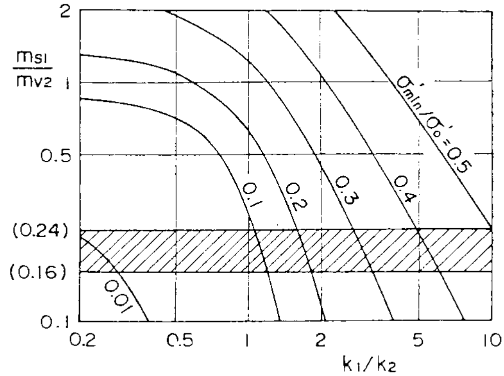

Fig. 13. Minimum effective stress ratio in Stratum I

is set in such a way that Stratum II is completely liquefied with its pore water pressure u_{wo} equal to the total vertical stress σ_{zo}, while Stratum I remains stable with its pore water pressure equal to hydrostatic pressure. The excess pore water pressure in Stratum II causes an upward flow of water through Stratum I. In the process, the pore water pressure in Stratum I will first rise causing rebound, and then fall causing recompression, whereas the pore water pressure in Stratum II will decrease nearly monotonically as in the case of the conventional consolidation problem.

On the basis of the authors' numerical analyses, the ratio of the minimum value of the vertical effective stress at the peak pore water pressure to the initial vertical effective stress prior to liquefaction at the mid-depth of Stratum I, σ'_{zmin}/σ'_o, can be expressed as shown in Fig. 13 in terms of the ratios of the coefficient of permeability and the coefficient of volume change between Strata I and II, k_1/k_2 and m_{s1}/m_{v2}. It can be seen in Fig. 13 that both the permeability and the coefficient of volume change have considerable influence on the development of pore water pressure in Stratum I.

If the ratio m_s/m_v of Fig. 10 is independent of the coefficient of permeability, and if Strata I and II of Fig. 12 are similar in grain size and relative density, the range of m_{s1}/m_{v2} will be limited to the shaded area in Fig. 13. Because the contours of Fig. 13 are nearly vertical within the shaded area, the minimum effective stress ratio is primarily governed by the permeability ratio, as shown in Fig. 14 which has been replotted from Fig. 13. As shown in Fig. 14, the minimum effective stress ratio of Stratum I is quite small if the permeability ratio is less than unity. However, if the permeability ratio is greater than ten the minimum effective stress ratio exceeds 0.5. Thus, the permeability ratio has an important influence on the stability of a soil layer overlying a completely liquefied stratum, because the effective stress is closely related to the strength of soil.

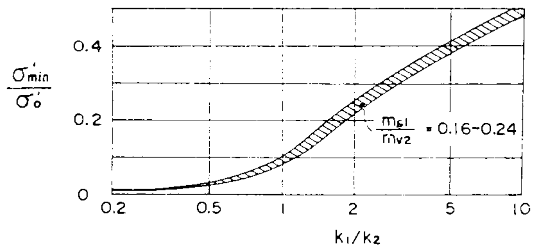

Fig. 14. Minimum effective stress ratio in Stratum I for a probable range of coefficient of volume change

Consider a deep deposit of loose saturated sand which is expected to liquefy during an earthquake. Densification of the loose sand over its entire depth will be a positive means to ensure the stability of the soil, but may not necessarily be mandatory if only a relatively light structure is to be constructed. It will be advantageous if one can limit the depth of densification, provided that the surface soil retains adequate stability when the unimproved soil underneath is liquefied during an earthquake. Such a problem is reduced to the case of Fig. 12, and the minimum effective stress ratio in Fig. 14 may give crude indication of the stability of the surface soil. Fig. 14 confirms the authors' previous views that such densification methods as vibroflotation and sand compaction piles using coarse backfill to increase permeability should be more effective than other densification methods in which permeability is not increased (Yoshimi and Kuwabara, 1973; Yoshimi, 1973).

CONCLUSIONS

The following conclusions may be made on the basis of the one-dimensional consolidation tests on saturated sands and the numerical analyses of boundary value problems involving completely liquefied sand:

1) One-dimensional stress-strain relationship of completely liquefied sand was independent of the manner in which the liquefaction was produced, whether by supercritical upward seepage or by impact.

2) The relationship between the void ratio and the logarithm of vertical effective stress during the post-liquefaction compression was nearly linear over the stress range from 0.002 kg/cm^2 to 0.2 kg/cm^2, the compression index being from 0.005 to 0.021.

3) Even under a very low confining stress of around 0.002 kg/cm^2, the sand that had not undergone liquefaction was considerably less compressible than the sand that had been liquefied.

4) The coefficient of volume change during unloading was around one-fifth of that during post-liquefaction compression for very loose or loose Niigata sand.

5) When a horizontal layer of loose saturated sand is liquefied to a depth of approximately 5 m to 20 m, the average vertical strain as a result of consolidation under the weight of the soil is estimated to be from 1 per cent to 3 per cent.

6) When a horizontal layer of soil overlying a liquefied layer of soil is about ten times as permeable as the liquefied soil, the surface soil may retain a considerable portion of the initial strength despite upward seepage resulting from the liquefaction.

NOTATION

a_s = coefficient of swelling
a_v = coefficient of compression
C_c = compression index
e = void ratio
G_s = specific gravity of solids
i_{cr} = critical hydraulic gradient
k = coefficient of permeability
m_s = coefficient of volume change in swelling
m_v = coefficient of volume change in compression
p = vertical effective stress in one-dimensional compression
U_c = coefficient of uniformity
u_w = pore water pressure
γ' = submerged unit weight of soil
σ_z = total vertical stress
σ'_z = effective vertical stress

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(Received April 7, 1975)