Behaviour of soil improved by sand compaction piles

Comportement des sols améliorés par des colonnes de sable compacté

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SYNOPSIS Centrifuge tests were conducted to investigate the mechanical behavior of clay improved by sand compaction piles under inclined load. Kawasaki clay was normally consolidated in a centrifuge and model sand piles were installed. Loading tests were carried out by applying lateral force to a model caisson placed on improved soil for four different combinations of lateral load to vertical load. The improvement by sand compaction piles increased the bearing capacity by about 200 to 700% and it was extremely effective in reducing the lateral displacement of caisson. The postmortem studies of improved soil revealed that sand compaction piles were sheared when they were subjected to inclined load. It was found that for the improvement area ratio of 70% the caisson tended to move horizontally, while for the ratio of 33 and 55% the caisson tilted even at the early stages.

INTRODUCTION

Major part of industrial activities in Japan has been being conducted in the coastal areas where very soft alluvial clay deposits prevail. Various techniques for soil improvement have been devised in order to enable structures to be constructed on these soft foundation soils. Recent Japanese developments in this area are deep mixing either by lime or cement, dry jet mixing and sand compaction piles. Among them the sand compaction pile method seems to be drawing wide attention because of simplicity in application. Several investigations have been carried out on the stability of cohesive soil improved by sand conpaction piles(Murayama, 1962; Matsuo, 1967; Mogami et al., 1968; Kimura et al., 1983b). Most of these research works in the past considered only with the mechanical behavior of improved soil under vertical load. Improved soil, however, is also subjected very often to inclined load as the resultant of the weight of a caisson and earth pressure. In this paper authors attempted to clarify the mechanical behavior of improved soil subjected to inclined load by centrifuge model tests.

CENTRIFUGE TESTS

Soil used for the centrifuge model tests was Kawasaki clay sampled from Tokyo Bay. It is one of the typical marine clays in Japan and the plasticity index was 27(Kimura et al., 1983b). The clay was thoroughly remoulded at a water content of 75% and was deaired at a negative pressure of about 98kN/m^2. A drainage layer of soaked quartz sand with 50mm in thickness was placed at the bottom of a centrifuge strong box with 500mm in length, 390mm in depth and 145mm in width. Main parts of the strong box were made of steel and the front face was made of a highly shockresistant plastic plate with the thickness of 50.8mm so that surface markers placed on a lateral side of clay can be viewed through during centrifuge rotation. A sheet of filter paper was laid on the surface of sand and slurry was poured into the box taking care not to trap air bubbles in slurry. Pouring of slurry was carried out in six layers. Preliminary consolidation was conducted for each layer under the vertical pressure of 9.8kN/m^2. After preliminary consolidation of each layer, consolidation pressure was once removed and surface markers with 8mm in diameter were placed to form grids of 20mm by 20mm. The surface marker consisted of a rubber cupule cut out of a soap holder and a thin needle glued to the back of the cupule which supports the cupule as a stand in clay. The cupule was filled with silicone oil to reduce the friction between the cupule and the plastic plate.

Having completed consolidation of sixth layer the consolidation pressure was again removed and the strong box was tilted. Horizontal holes were drilled through by a small hand auger. Each hole was filled with deaired slurry and a Druck pore pressure transducer was inserted into the hole. The final consolidation on the lab floor was then conducted under the pressure of 19.6kN/m^2. On completion of consolidation on the lab floor a layer of lead shot was placed on the surface of clay so that the surcharge pressure of 19.6 kN/m^2 could be applied to clay during subsequent centrifuge consolidation. This procedure was necessary for preparing normally consolidated clay by centrifuge consolidation. Subsequently the strong box was mounted on the centrifuge and centrifuge consolidation was taken place under the centrifugal acceleration of 80g. The detail of the centrifuge used for the current tests was given elsewhere(Kimura et al., 1982). The progress of centrifuge consolidation was monitored by a L.V.D.T. installed on the surface of lead shot and by the pore pressure transducers embedded in clay. In order to maintain the water table on clay at a constant level during centrifuge consolidation, a solenoid valve coupled with a water level sensor was made use of. When the water table rose, the water level sensor settling with clay triggered off the solenoid valve to drain some amount of water to waste through holes provided at the side wall of the strong box. It took approximately 800 minutes to attain the consolidation degree of 90%. The centrifuge was then stopped and model sand compaction piles were installed in clay after removing the lead shot. The detail of preparing the model sand piles and of the installation technique was introduced in a previous paper by the authors(Kimura et al., 1983b). Clay which swelled up because of installation of sand piles and subsequent thawing was scraped off and the surface was levelled off. The thickness of clay in a completed model was 110mm. In the current tests three different improvement area ratio(A_s) were employed. They were 33, 55 and 70%. A layer of Toyoura sand was then laid on the surface of clay to form a sand mat with 50mm in thickness and a duralmin model caisson with 50mm in breadth was placed. The caisson had two chambers for filling lead shot so that the dead weight could be varied in the range of 19.6 to 215.6kN. Quartz sand was glued to the base of the caisson to create rough contact with the sand mat underneath. A bellofram cylinder for applying lateral force to the model caisson was attached to the flanges of the strong box. The whole setup is illustrated in Fig.1. In this test series the ratio of breadth of improved area to that of model caisson was taken as 2 based on the previous finding by the authors(Kimura et al., 1983a). The strong box was once again taken to the centrifuge and reconsolidation was conducted under 80g in an attempt to eliminate various disturbances which the clay suffered. Reconsolidation was continued until pore pressures monitored registered the magnitudes nearly equal to those recorded at the final stage of centrifuge consolidation.

Loading tests were carried out by applying lateral force to the model caisson with the bellofram cylinder which was driven by compressed air fed through hydraulic slip rings built in the centrifuge shaft. Four different combinations of lateral load to vertical load were employed to obtain a design curve. During the loading tests photographing of surface markers was conducted, the lateral force was measured by a loadcell attached to the tip of bellofram cylinder and the displacement of the caisson was measured by a L.V.D.T.. In some series of the test attempts were made to measure the contact stress at the base of the caisson by installing three small loadcells at the base of caisson.

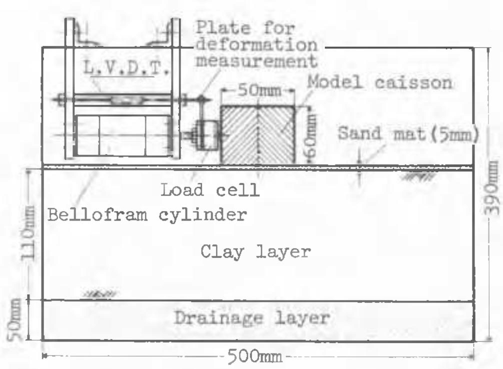


Fig.1 Test Setup and Loading System

TEST RESULTS AND DISCUSSIONS

Observed distributions of contact stress at the base of model caisson are shown in Fig.2. At the initial stage when the caisson was subjected only to vertical load the distribution is fairly uniform. As lataral load increases the contact stress near the edge of caisson at loading side decreases and that at the opposite side increases, while the contact stress at the center does not vary very much. Around the yield load the contact stress at loading side becomes nearly equal to zero, which implies that the whole load is taken by the right hand side of the caisson. Observations by photographing showed that tilting of the caisson becomes more marked as lateral load and lateral displacement of caisson increase. This is considered to support the well-known procedure proposed by Meyerhof for inclined load that the load bearing area should be reduced with the increase of inclination of load.

Relationships between the lateral load intensity and lateral displacement of model caisson are

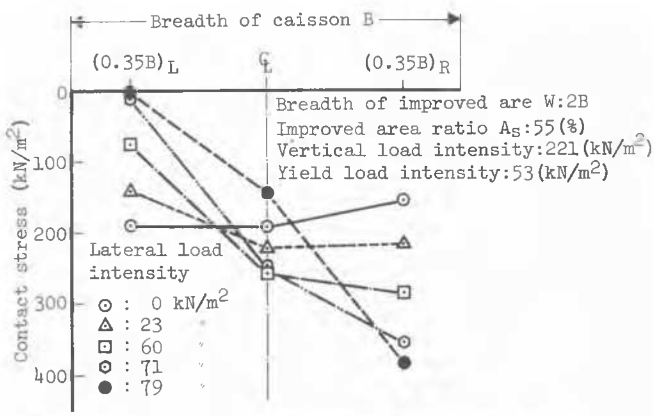


Fig.2 Contact Stress at Caisson Base

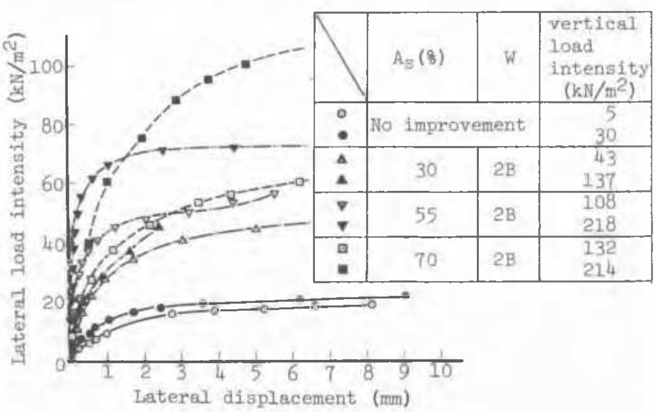
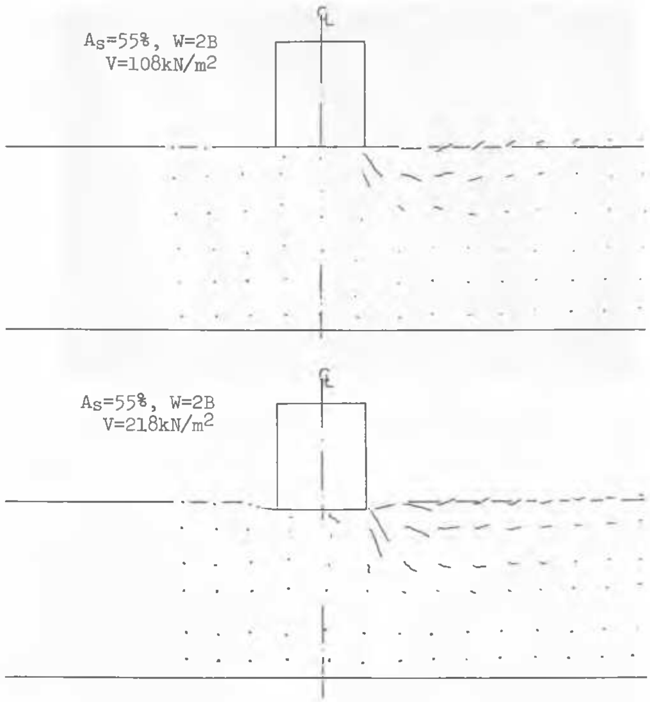
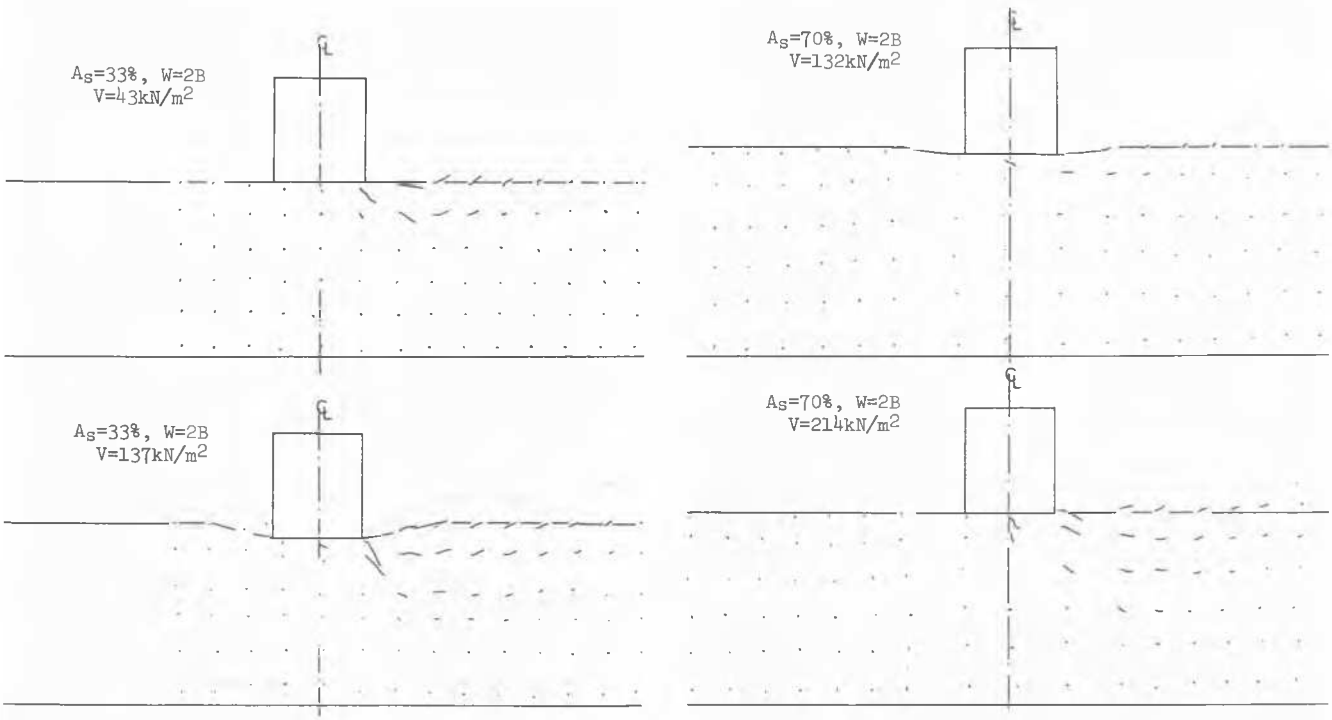


Fig.3 Relationship between Lateral Load Intensity and Lateral Displacement

given in Fig.3. In the figure results for soil not subjected to improvement are also added. It can be clearly seen that the improvement by sand compaction piles is extremely effective in suppressing lateral displacements of caisson. The improvement increases the bearing capacity of foundation soil by about 200 to 700% of that of non-improved soil. One marked trend is that the bearing capacity is greater for higher vertical load intensity. When the magnitude of vertical load intensity is greater, the stress in the upper parts of sand compaction piles becomes higher giving rise to higher shearing resistance of improved soil. This is considered to be the main reason for greater bearing capacity for higher vertical load intensity. Observed deformation vectors illustrated in Fig.4 may justify this reasoning. Generally speaking the bearing capacity increases with the increase in A_s value. The trend is more marked for higher vertical load intensity. Detailed examinations of the relationships between the load intensity and displacement shown in Fig.3 reveal that they are classified into two groups. The relationships for A_s equal to 33 and 55% show a similar trend. The bearing capacity increases with the increase in A_s value and as the vertical load intensity increases the bearing capacity becomes greater and the rigidity of improved soil or the initial tangent of the curves increases. Two curves for A_s value equal to 70% are considerably different from the former group. There is practically no difference in bearing capacity between A_s value of 70% with



(b) A_s = 55%



(a) A_s = 33%

(c) A_s = 70%

Fig.4 Deformations in Improved Soil

Fig.5 Sand Compaction Piles after Loading Test

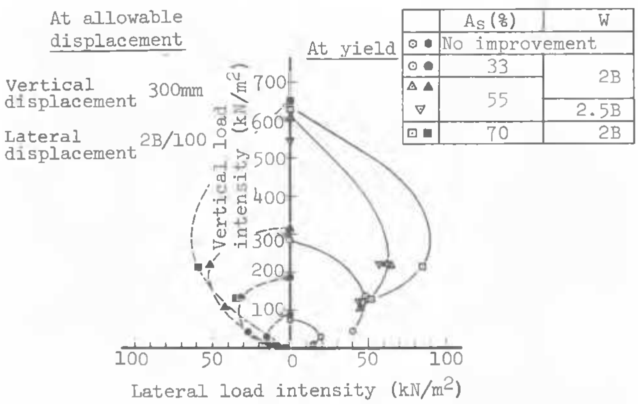


Fig.6 Design Curves for Sand Compaction Pile Improvement

the vertical load intensity of 132kN/m^2 and A_s value of 55% with the vertical load intensity of 108kN/m^2 and the rigidity of improved soil of the former is even smaller than that of the latter. Moreover the two curves for A_s value of 70 % have a similar characteristic showing rather low rigidity. This difference in behaviors between two groups of the load intensity versus displacement relationships suggests that deformation mechanism for A_s of 70% is different from that for A_s equal to 33 and 55%. In this test series the vertical displacements of the model caisson was also measured. By plotting the vertical displacements against the lateral displacements it was found that for A_s equal to 70% the vertical displacement takes place after the lateral displacement reaches some considerable value, while for A_s equal to 33 and 55% the two components of displacement appear at the same time. The difference in displacement behaviors is clearly seen in deformation vectors shown in Fig.4. This explains the difference in rigidity between the two groups. The rigidity is higher for A_s value of 33 and 55% because the lateral force applied to the caisson is taken partly by soil in front of the caisson even at early stages.

There has been a controversy about how sand compaction piles behave when they are subjected to lateral force. The sand compaction pile method has received a criticism that the piles may collapse like domino pieces. A postmortem photograph shown in Fig.5 clearly demonstrates that it is not the case. It can be concluded that the sand compaction pile method is effective even when improved soil is exposed to lateral force.

In Fig.6 design curves deduced from the current tests are shown. The curve at the right hand side is for allowable displacements of caisson and the one at the left hand side is for yield load. The current design criteria used in Japan allow a caisson to move laterally by 2 to 3% of the breadth of caisson. This gives about 1mm(=50 mmx2/100) as the allowable lateral displacement. According to the Japanese building code the allowable settlement of a mat foundation is 300mm. Converting this into the centrifugal field of 80 g gives the allowable settlement as 4mm(=300mmx 1/80). These design curves imply that high values of A_s do not necessarily give rise to greater bearing capacity. It should be borne in mind that the bearing capacity of soil improved by sand compaction piles is dependent upon both A_s value and the vertical load intensity given by caissons.

CONCLUSIONS

Following conclusions are drawn from the current studies on clay improved by sand compaction piles.
(1) The improvement by sand compaction piles increases the bearing capacity by about 200 to 700% and it is effective in reducing the lateral diplacement of caissons.
(2) The bearing capacity of soil improved by sand compaction piles is dependent upon both the improvement area ratio and applied vertical load intensity.
(3) Deformation mechanisms for high improvement area ratio are different from those for low area ratio.
(4) Contact stress at the base near loading side becomes very small at yield load.

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