Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Protection of buried structures from soil liquefaction hazard by means of cutoff walls

Yoshiaki Yoshimi, Katsuyuki Tamaoki, Nobuo Mori & Tsukasa Kondo
Institute of Technology, Shimizu Corporation, Tokyo, Japan

ABSTRACT: During past earthquakes buried structures such as sewage treatment tanks and fuel tanks floated to surface as a result of soil liquefaction. Shaking table tests were run on a structure buried in loose, saturated sand, with or without cutoff walls around the structure. The behavior of the models were compared under strong shaking that caused complete liquefaction of the sand both inside and outside the cutoff walls. The upward displacement of the structure when the cutoff walls were present was related to inward deflection of the walls, and was much less than that without the cutoff walls. It is concluded that enclosing a buried structure with reasonably stiff cutoff walls is a viable means to mitigate liquefaction hazard to buried structures, and can compete with other methods such as vibratory compaction, cement stabilization, and gravel drains in terms of reliability and economy where noise and vibration are not allowed.

1 INTRODUCTION

During several earthquakes which shook the northern parts of Japan since 1964, soil liquefaction caused upward displacements of buried structures such as sewage treatment tanks and fuel tanks as shown in Figure 1. To mitigate such damage to utility tunnels and underpasses at liquefaction-prone sites, sand compaction piles, deep cement mixing, and vertical gravel drains have been used or proposed in Japan. Vibratory densification methods cannot be used in a populated area because of the noise and vibration they generate. The deep cement mixing method is less noisy but more costly. Reliability of gravel drains in loose sand is somewhat questionable (Yoshimi, 1991). Thus, there is a need for a reliable, quiet and inexpensive method for mitigating damage to buried structures in liquefaction-prone soil. The object of this paper is to propose such a method.

2 PRINCIPLE OF THE PROPOSED METHOD

A large upward displacement of a buried structure involves significant movement of liquefied sand from around the structure to the space that has been vacated by the structure. Conversely, the displacement of the structure may be minimized by preventing such movement of liquefied sand, e.g., by surrounding the structure with cutoff walls that extend into a nonliquefiable stratum.

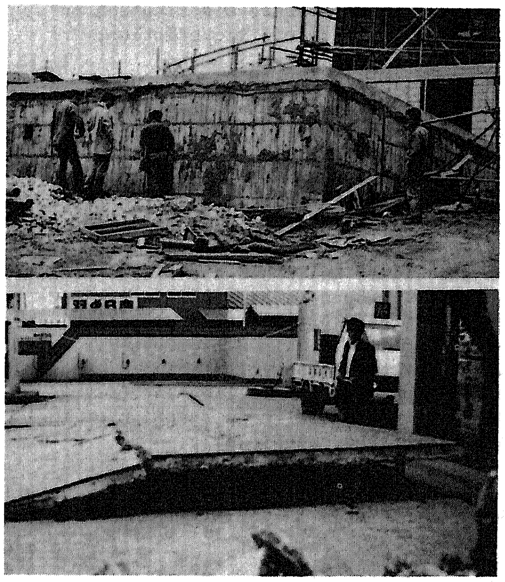

Figure 1. Examples of flotation of buried structures due to soil liquefaction: empty sewage treatment tank and partly full fuel tank.

Consider an ideal case in which the cutoff walls are rigid and frictionless, and the sand undergoes complete liquefaction both inside and outside the cutoff walls. Then, the earth pressure on the cutoff walls become equal to the total vertical stress as shown in Figure 2, in which γ=unit weight of sand, d=depth to the bottom of the structure, and q=contact stress at the bottom of the structure. Because the structure is lighter than the sand, the pressure from outside exceeds that from inside by a differential pressure p which is given by:

$$p=\gamma d-q \qquad (1)$$

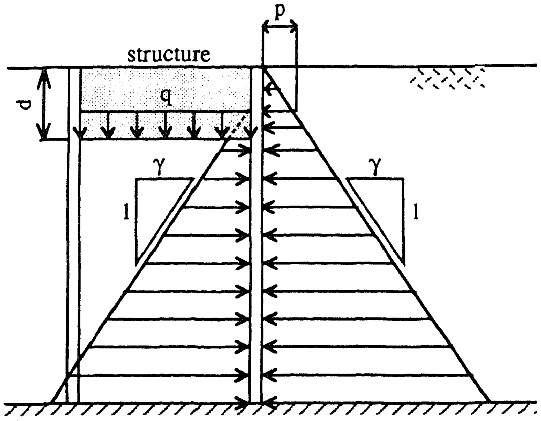

Figure 2. Earth pressure distribution on cutoff wall during soil liquefaction.

Note that the structure cannot move upwards for lack of unbalanced upward force, in the same manner in which the upper part of level ground cannot move upwards except at sand boils. On the contrary, the structure will settle if the sand becomes denser and excess pore water is allowed to escape. When the cutoff walls are not rigid, however, they will bend inwards due to the differential pressure p, and the deformation of the sand will result in an increase in its height if there is no volume change. Thus, the amount of upward displacement of the buried structure will be a function of the stiffness of the cutoff walls.

3 LABORATORY TESTS

Horizontal shaking table tests were conducted on a model consisting of saturated sand, a buried structure and impermeable cutoff walls surrounding the structure, with or without drainage around the structure, as shown in Figure 3.

A laminar container 1.2 m long, 0.8 m wide, and 1.0 m deep, with rubber lining was used for all the tests. To simulate a plane strain problem, both the structure and the cutoff walls extended across the full width of the container. As shown in the upper part of Figure 4, the structure consisted of three parts of which only the center section was observed with the end ones serving as dummies to eliminate possible interference with the side walls of the container. The structure was made of aluminum plates, measured 45 cm wide and 20 cm high, and had a density of 1.2 g/cm^3.

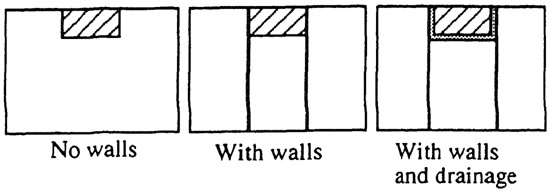

Figure 3. Model types tested.

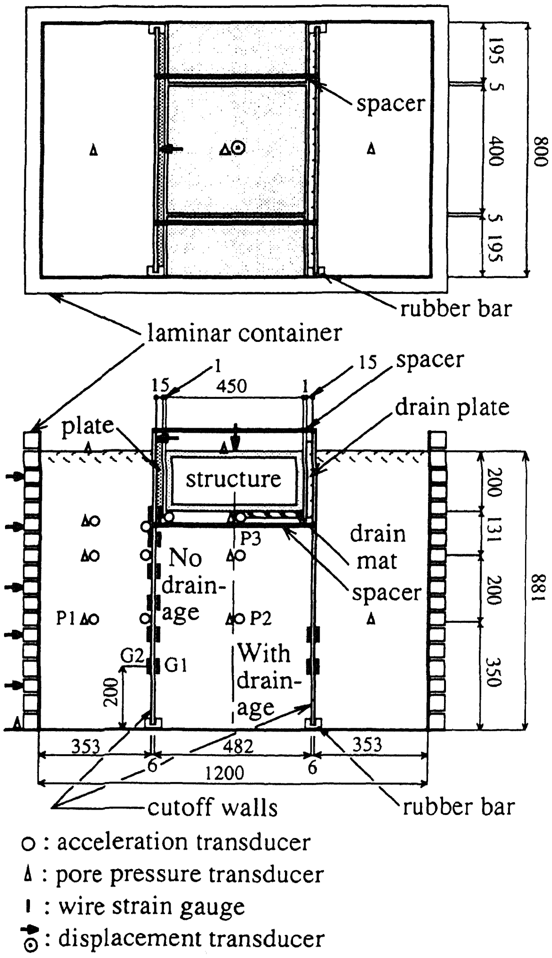

Figure 4. Plan view and longitudinal cross-section of model with cutoff walls (dimensions in mm).

Because the model was considered about 1/20 scale, the prototype structure would be

9 m wide and 4 m high. The cutoff walls of 6 mm thick acrylic plates were fixed on the bottom of the container via grooved rubber bars. The stiffness of the model cutoff walls was selected to represent a popular type of earth retaining wall consisting of steel wide flanges encased in soil cement (called SMW in Japan). The length of the cutoff walls below the bottom of the structure, however, was made larger than normal to exaggerate their deflection.

A pair of aluminum plates 15 mm thick were fixed on the cutoff walls on the same level as the structure for the tests without drainage. For the tests with drainage the plates were of perforated, acrylic plastic, and were in contact with a horizontal drainage mat of nonwoven fabric 12 to 18 mm thick.

In order not to interfere with free vertical movement of the structure, the inside clearance between the cutoff walls which was 2 mm greater than the width of the structure was maintained with spacers of 8 mm steel rods and grease was applied on the vertical surfaces of the structure.

Pore pressure transducers of wire strain gage type, displacement transducers of LVDT type, and acceleration transducers of wire strain gage type were installed in the sand, on the structure, or on the container, and wire strain gages were mounted on the cutoff walls as shown in Figure 4.

An aqueous solution of glycerin (concentration = 70%, specific gravity = 1.17) which was about 20 times as viscous as water at room temperature was used in place of water to simulate the time required for drainage in the prototype, so that beneficial effects of pore pressure dissipation would not be exaggerated in the model experiment. Note that the structure was heavier than the pore liquid by a very narrow margin: 1.2 vs. 1.17.

Fine sand (50% diameter = 0.38 mm, uniformity coefficient = 2.7, specific gravity of solids = 2.698, maximum void ratio = 0.979, minimum void ratio = 0.643) was used for the model ground. A loose mass of sand with relative density of approximately 58 percent was obtained by pouring the sand soaked in the liquid into the container, with a depth of the liquid maintained at about 5 cm above the surface of the sand. For each test the model ground was prepared the same way after removing the sand from the container. The average dry density of the sand for seven tests was 1.511 g/cm^3, and the coefficient of variation was 0.55 percent.

The EW component of the ground motions at Hachinohe, Aomori Prefecture, Japan, during the Tokachioki earthquake of 1968 was used as the table motion. The maximum table acceleration of 142 cm/s^2 was selected in such a way that the sand within the cutoff walls would liquefy completely and that the structure would float up significantly when no cutoff walls were used.

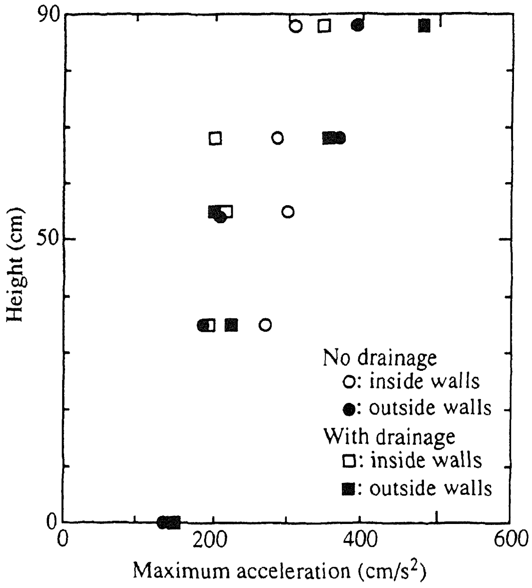

Figure 5. Distribution of maximum horizontal acceleration for tests with cutoff walls.

4 TEST RESULTS

4.1 Behavior of sand outside the structure

Figure 5 shows the distribution of maximum horizontal acceleration. The open symbols at the surface were recorded on the structure, and all the rest were in the sand. There was considerable amplification towards the surface. The resulting surface accelerations of 350 to 480 cm/s^2 were more than enough to cause liquefaction in the loose sand.

Although the distance between the edge of the structure and the container was limited, the sand outside the structure showed typical liquefaction behavior of loose sand in the free field, in that it underwent large shear deformation and subsequent settlement as pore water emerged. The final surface settlement was roughly the same for all the tests with or without the cutoff walls, reaching about 0.8 percent of the thickness of the sand.

4.2 Structure without cutoff walls

Figure 6 shows the time histories of table acceleration, excess pore pressures, and the vertical displacement of the structure. Note that the time scale after the shaking is logarithmic. The excess pore pressure at

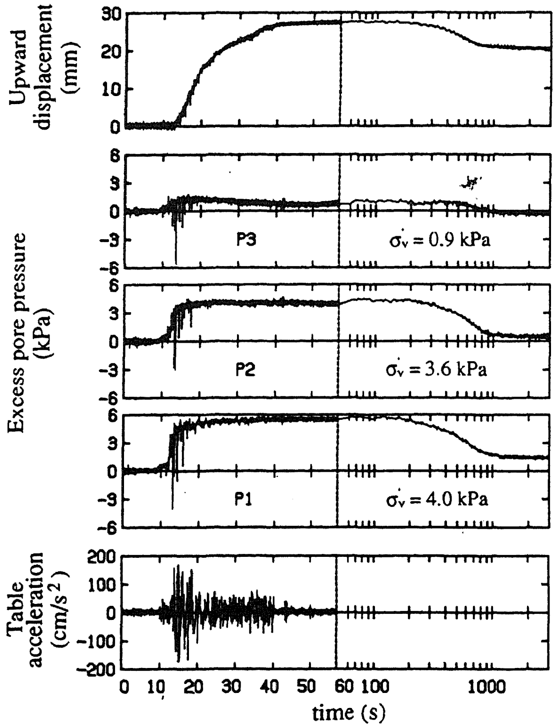

Figure 6. Time histories for test without cutoff walls.

each location increased rapidly until it reached a plateau, and began to decrease some time after the shaking had stopped. The peak values of the excess pore pressure below the center of the structure were higher than the initial vertical effective stress, σ_v', presumably under the influence of the excess pore pressures away from the structure.

The structure began to rise immediately after the pore pressures had increased, kept its ascent until about 40 s, and descended when the pore pressures did so some time after the shaking had stopped. Unlike the pore pressures, however, there remained a considerable amount of residual displacement.

4.3 Structure with cutoff walls

Figures 7 and 8 show the time histories of flexural strains in the cutoff walls in addition to those shown in Figure 6. Except immediately below the structure (P3, see Figure 4), the excess pore pressures increased rapidly and reached a plateau which is approximately equal to the initial vertical effective stress, σ_v'.

Figure 9 shows the distribution of the excess head at 20, 30 and 40 s. As predicted, the head of the pore liquid measured outside the cutoff walls is higher than that inside, and the hydraulic gradient given by the slope of the dotted lines for both outside and inside is 0.73 which corresponds to the critical hydraulic gradient, i_{cr}, determined from Equation (2):

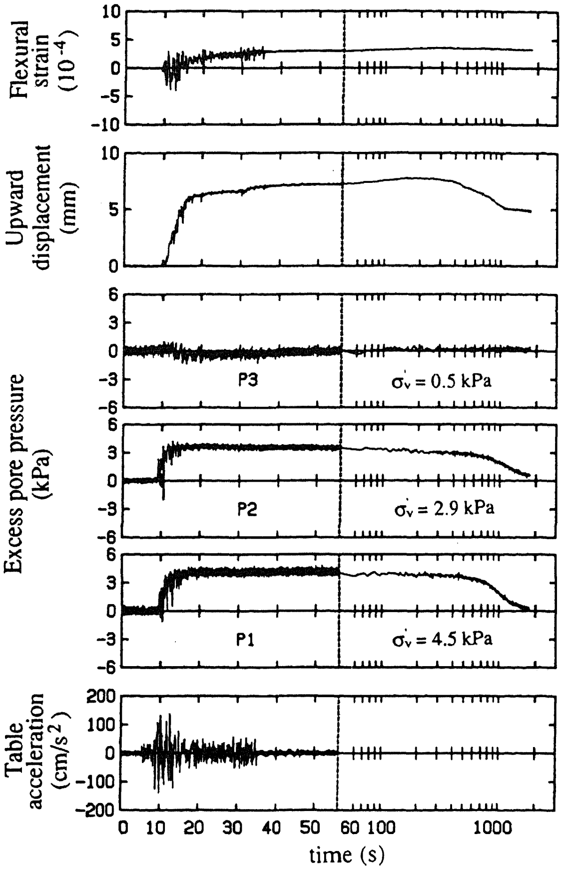

Figure 7. Time histories for test with cutoff walls.

$$i_{cr}=(G'-1)/(1+e) \qquad (2)$$

in which $G' = 2.70/1.17 = 2.31$: specific gravity with respect to the pore liquid, and $e = 0.793$: void ratio. Thus, the sand was in completely liquefied state in which upward seepage at critical gradient was maintained.

Figures 7 and 8 show that both the upward displacement and flexural strain increased with a slight time lag behind the pore pressures. Some time after the shaking had stopped, the structure began to settle slightly ahead of the reduction in the pore pressure.

Figure 10 shows the peak and residual displacements of the structure. The upward displacement of the structure without the cutoff walls exceeded 30 mm which is equivalent to 60 cm in the prototype. It can be seen that the presence of the cutoff walls

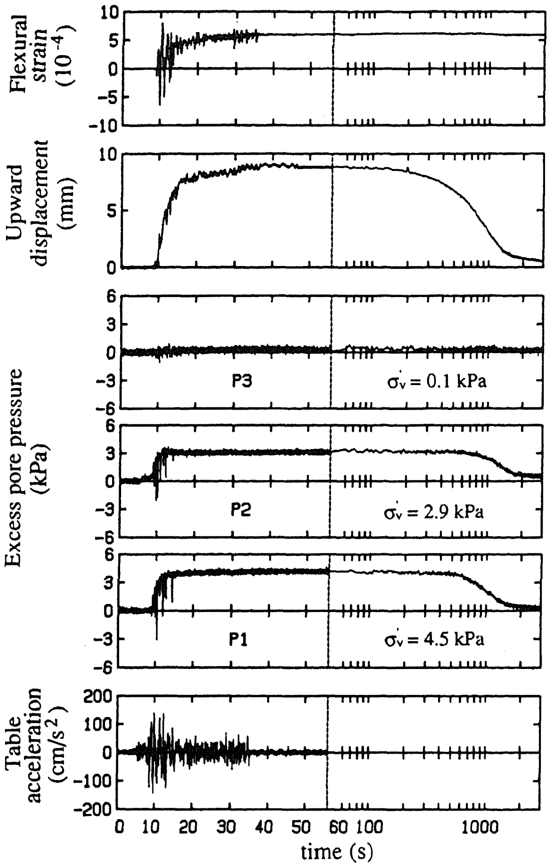

Figure 8. Time histories for test with cutoff walls and drainage.

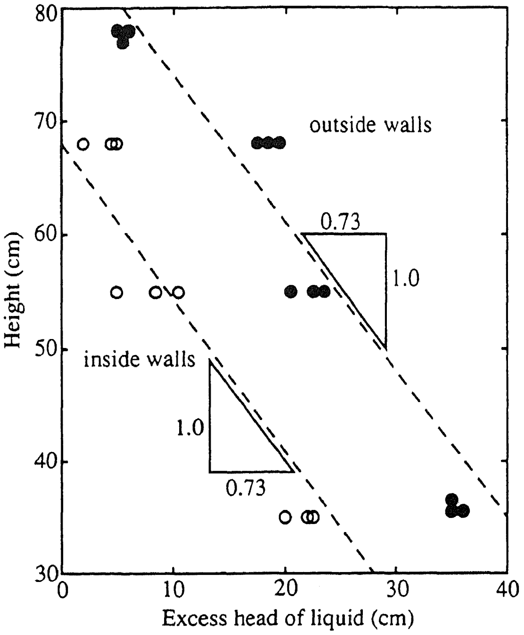

Figure 9. Excess head of pore liquid in liquefied sand.

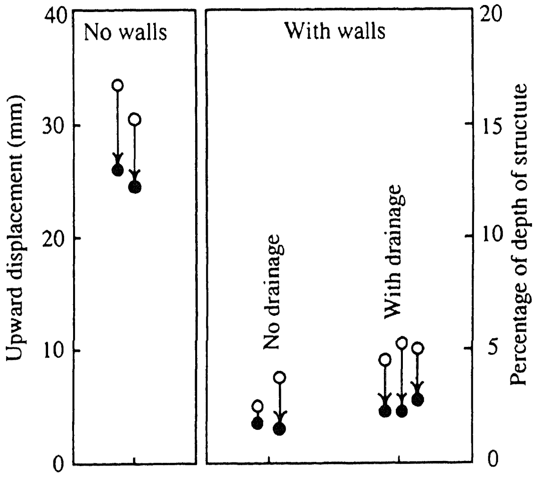

Figure 10. Peak displacement (open symbols) and residual displacement (solid symbols) of the structure.

caused considerable reduction in the upward displacement of the structure, although the addition of drainage did not appear to make a significant difference.

Figure 11 shows the observed peak displacement of the structure plotted against the displacement computed from the deflection of the cutoff walls, assuming that the deflection is symmetrical and the sand below the structure undergoes no volume change. That the observed displacement is generally smaller than the computed value seems to indicate that some volume reduction took place in the sand below the structure as a result of densification.

DISCUSSION

To prove that the cutoff walls and the side structures did not interfere with free vertical movement of the center structure, one needs only to show that the coefficient of friction was less than the ratio between the height and the width of the structure. That was indeed the case with an ample margin: the former was 0.16 and the latter 0.44 (=20/45).

In the field, however, no effort would ordinarily be made to allow free movement of a buried structure. If no drainage is pro-

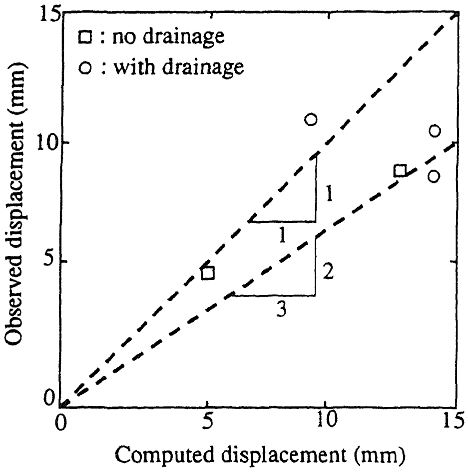

Figure 11. Observed upward displacement of structure compared with computed values based on cutoff wall deflection.

vided along the exterior surfaces of the structure, the concrete for the structure is probably poured directly in contact with the cutoff walls. In such a case, the buoyant weight of the structure (under hydrostatic conditions) will eventually be supported by the cutoff walls, leaving a gap below the bottom of the structure if the sand undergoes liquefaction and subsequent settlement. The situation is similar to a case in which a gap is formed below a pile cap after the soil around end-bearing piles settles.

The vertical displacements of the model structure with cutoff walls as shown in Figure 10 which are on the order of a few percent of the depth of the structure resulted from unusually flexible cutoff walls. The displacements would have been much less if the length of the walls below the structure had been smaller even by a moderate amount, because the volume of sand displaced by a uniformly distributed load acting on the walls should be proportional to the fifth power of the span of the wall. This is based on the assumption that the displacement was proportional to the volume as suggested by Figure 11.

Another factor that can contribute to further reduction of the upward displacement of the structure is a reduction in the liquefaction potential of the sand within the cutoff walls due to their structural constraint and overconsolidation of the sand resulting from dewatering for excavation. The latter effect can be enhanced by lowering the water table more than the level required for excavation. Thus, it seems reasonable to assume that most real structures surrounded by reasonably stiff cutoff walls will experience much less displacements than what these model tests indicated.

The results of this study do not show any noticeable effect of drainage around the structure. It must be pointed out, however, that even the "tests without drainage" permitted some drainage through the gaps between the structure and the cutoff walls. Thus, the behavior of the structure with complete absence of drainage is still unknown, although it is not expected to be significantly different from what was observed in the present test.

Unlike densification methods which may leave the periphery of the treated area open to possibly detrimental effects of the untreated soil, the proposed method protects the structure from external influence by clear-cut boundaries in the form of cutoff walls.

The cost of the cutoff walls is not necessarily totally additional because some form of earth retaining walls would be required for excavation.

Detrimental effects of lateral spreading of liquefied soil may be mitigated by adding transverse walls below the structure.

CONCLUSIONS

On the basis of the shaking table tests, it can be concluded that the upward displacement of a buried structure due to soil liquefaction can be mitigated by installing reasonably stiff cutoff walls around the structure. The method is believed to be able to compete with other remediation measures in terms of reliability and economy where noise and vibration are not allowed.

REFERENCES

Yoshimi, Y. & K. Tokimatsu 1991. Ductility criterion for evaluating remedial measures to increase liquefaction resistance of sands. Soils and Foundations. 31-1:162-168.