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A STUDY ON EFFECT OF COUNTERMEASURES FOR PILE FOUNDATION UNDER LATERAL FLOW CAUSED BY GROUND LIQUEFACTION

Kiyoshi SATO[1], Shunichi HIGUCHI[1], Takashi MATSUDA[1]

SUMMARY

Three countermeasures for lateral flow caused by ground liquefaction during and after large earthquakes were developed by the authors. In this development, the dynamic centrifuge test, the shaking table test and the numerical investigation were carried out. The countermeasures consist of drain system, stream line shape of footings and front shield system. A dynamic centrifuge test under centrifugal acceleration field of 50g was carried out to investigate effect of these countermeasures. In the centrifuge test, three countermeasure models and original model were simultaneously installed in a soil container. The dimension of each section is 1.0m length by 0.4m width and 0.6m depth. Four models are shaken with a sine wave at once. The results of the test, the effect of countermeasures were obviously indicated. Especially, the models of front shield system and stream line shape of footings indicated that the residual deformation of footings decreased by 30% and less, and strains of the pile head decreased by 60% and less for that of the original model.

1. INTRODUCTION

Damage of the piles caused by ground lateral flow has been observed along the backfill of quay walls during and after strong earthquakes. It is thought that the acting forces to the footing and piles in the unliquefiable surface layers are larger than those in the liquefiable lower layers, which means that countermeasures against the lateral ground flows to piles are more effective in the case of implementation to the upper unliquefiable layers. The authors have developed three countermeasures from that viewpoint. These countermeasures are earth retaining walls in front of the footing, drain piles from liquefiable layers to unliquefiable layers and streamlined shield implemented at the footing.
The dynamic centrifuge test for the countermeasures was performed to verify the effect of these methods. This paper describes the result of the test and the mechanism of the prevention for damage on piles.

[1] Technical Research Institute of Obaysahi Corporation, Tokyo, Japan.
Email:sato.kiyo@obayashi.co.jp

2. COUNTERMEASURES

Three countermeasures are shown in the figure 1.

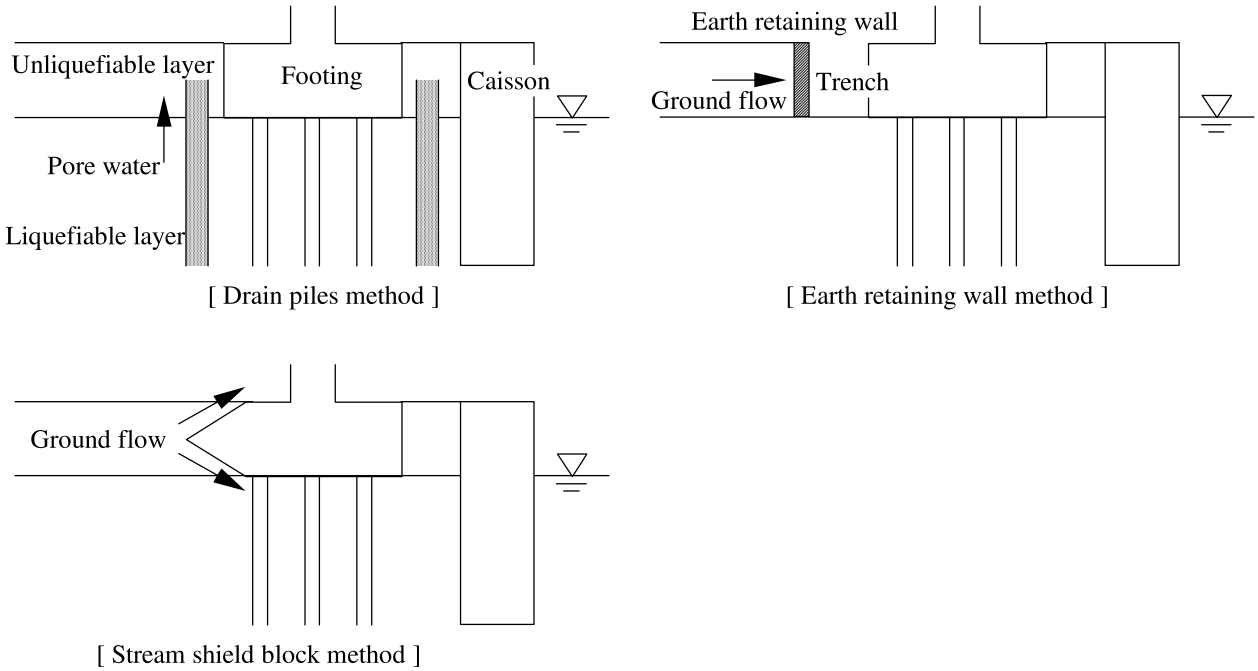

Figure 1 Countermeasures for ground lateral flow

2.1 Drain piles method

The permeability of drain piles around the footing should be smaller than that of surrounding liquefiable ground. These drain piles are installed from the bottom of liquefiable layers to the midway of upper unliquefiable layers. The drain piles have effects not only on the prevention to liquefaction of lower layers but also decreasing the stiffness of the upper layers associated with the footing deformation. Namely, the excess pore water pressures built up in the lower liquefiable layers are transmitted to the upper unliquefiable layer by the drain piles, the stiffness of the upper unliquefiable layer is deteriorated by the excess water pressures propagated from the lower layers.

2.2 Earth retaining wall method

In this method, an earth retaining wall is installed in front of the footing foundation. The direction of ground lateral flow should be determined, so that the retaining wall can directly block lateral ground flow from upper side.

2.3 Streamlined shield block method

In this method, an acutely angle facing of the footing foundation is casted. The acute angle face can disperse soils from lateral direction to slant direction or ups and downs direction.

3. CENTRIFUGE DYNAMIC TEST

3.1 Models

The cross and plan section drawings are shown in Figure 2. The centrifugal acceleration was 50g, and four models including without any countermeasures were shaken simultaneously. Each soil container has 47.5m long by 20m wide in prototype scale. In the test, four models were subjected same input motion simultaneously, so that the effect of the countermeasures was directly compared with each other.

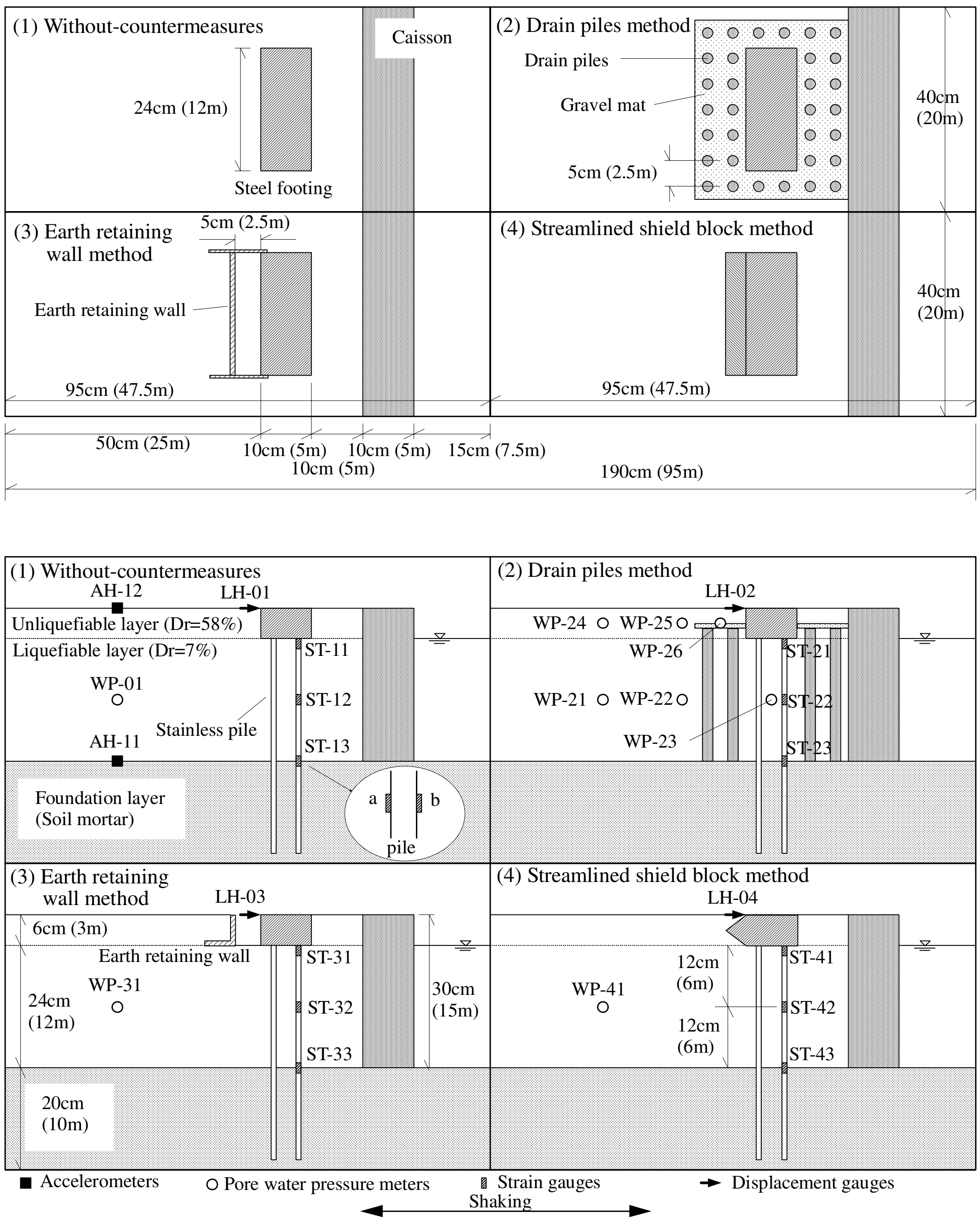

Figure 2 Dynamic centrifuge models

The centrifuge model consisted of steel caisson facing toward water such as the sea or river, backfills as liquefiable layers and group piles foundation with a steel footing. The back fill ground should move to the waterside derived by sliding and overturning of the concrete caisson.

The concrete caisson was designed to resist the earth pressure at rest by the friction between bottom of the caisson and the bearing soil layer. All dimensions of the centrifuge model are described by prototype scale and model scales are described in parentheses.
The backfill layer consisted of upper unsaturated sand layer of 3m thickness (6cm in model scale) and lower saturated sand layer of 12m (24cm). The material of sand layer was uniform silica sand of No.8 Japanese standard, and physical properties of used sand are tabulated in Table 1. The model ground was manufactured by air dispersion method, and the density of sand layers was tuned by dropping height and application amount. The de-aired normal water was used for the pore water, so that the coefficient of permeability of prototype ground should be 50 times of model ground. Before injections of degas water, carbon dioxide was filled in the pore. The physical properties of the model ground are tabulated in Table 2. The relative density of the upper unsaturated layer was 58% and that on the lower saturated layer was 7% at 1g fields. The relative density of the lower layer was consolidated by centrifugal acceleration, the final density reached about 20% at 50g. The coefficient of permeability of prototype ground was computed as 8.5x10-3m/sec in prototype ground. The bearing ground layer was installed under saturated sandy ground using soil mortar of 10m (20cm) thickness.

Table 1 Material Properties of sand

Grain density Gs (t/m^3)	2.650
Max. Grain size D_{max} (mm)	0.25
Max. void ratio e_{max}	1.403
Min. void ratio e_{min}	0.705

Table 2 Material Properties of sand

	Density (t/m^3)	Relative density (%)	Permeability (m/s)
Unsaturated	1.33(Dry)	58	——
Saturated	1.70	7	8.5×10^{-3}
Caisson	2.16	——	——

The caisson structure consisted of steel footing and 4 piles (SUS304) of 1m (2cm) in diameter and 25mm (0.5mm) in thickness. 4 piles were supported into the bearing layer in 9m (18cm) depth.
In the case of the drain piles method, gravel drain piles consisted of coarse-grained soils of 5cm (1mm) in diameter. 32 gravel drain piles were installed around the footing by 2.5m (5cm) spacing. Each gravel pile was connected to gravel flat mat in the upper unsaturated layer to dissipate the pore water pressure into those layers.
In the case of the earth retaining wall method, the deep trench of 2.5m (5cm) in width was installed in front of the footing at the upper flowing side. This trench was maintained by the steel angle of 3m by 3m (6cm by 6cm).
In the case of the streamlined shield block method, wedge shape of footing side face had an angle of 90 degree. The crest of wedge as shield block was located in the middle of the footing.

3.2 Measurements

Measuring instrumentations were for accelerations (AH), pore water pressures (WP), displacements of the footings and caissons (LH) and strains of the piles (ST). Two strain gages were installed on the upstream side and downstream side of ground lateral flow in the part of pile head, midst part and boundary part to the bearing layer.

3.3 Input motion

The maximum amplitude of the input motion at the shaking table was set up to 600Gal (30g). 20 waves of sinusoidal wave in 1.2Hz (60Hz) were subjected. The observed time history at the bottom of the saturated sand layer is shown in Figure 3. Under the influence of liquefaction of sandy layers, the input motion to the sandy layers was degraded and decentered.

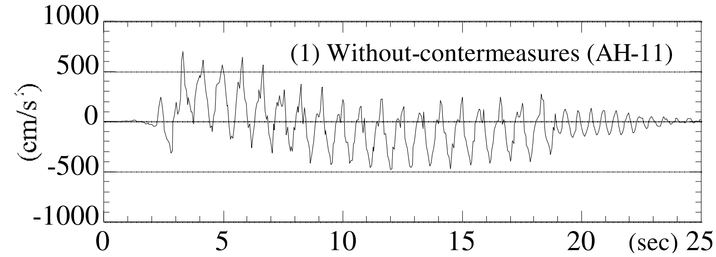

Figure 3 Input motion at the bottom of saturated liquefiable layer in prototype

4. TEST RESULTS

4.1 Lateral displacement

An average residual lateral displacement was 4.03m (8cm) as tabulated in Table 3. It is thought that the differences among the test cases were caused not only from fluctuation of manufacturing models but also from performances of the countermeasures.

Table 3. Maximum lateral displacement of caisson

Case	Residual Lateral Displacement of Caisson (m)
1. Without-countermeasures	3.3
2. Drain piles method	4.0
3. Earth retaining wall method	3.8
4. Streamlined shield block method	5.0

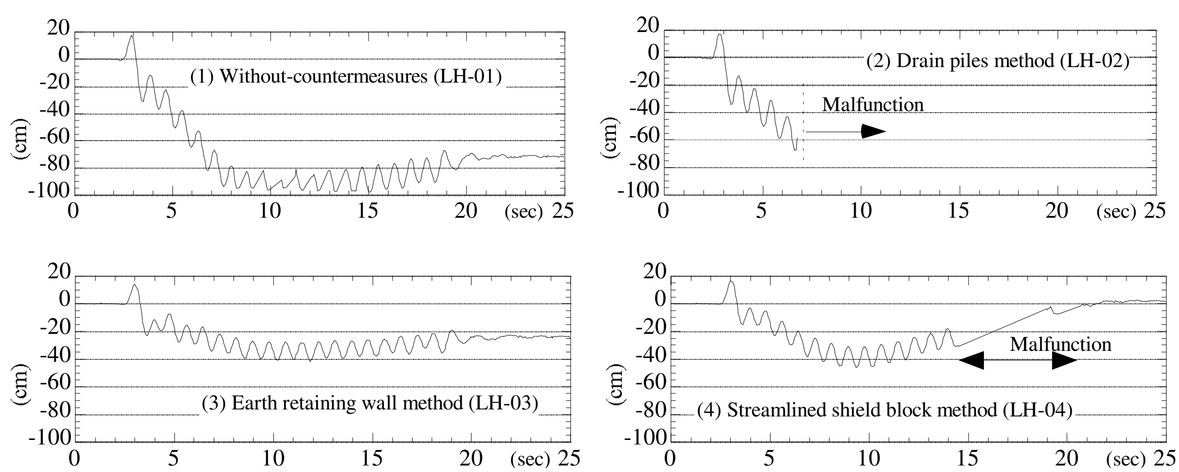

Figure 4 Time histories of lateral displacement

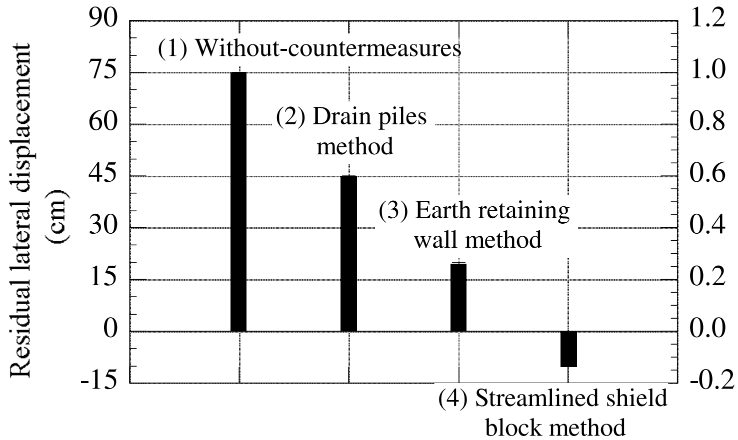

Figure 5 Comparison of residual lateral displacement of footings

Time histories of lateral displacement at the top of the footing during shaking are shown in Figure 4. The minus (-) sign indicates direction of lateral ground flow. The maximum lateral displacement is about 1m in the case of without countermeasures, which is almost 1/4 of that of the caisson. While the input motion was subjected from 2 second to 19 second, the lateral displacement progressed just after beginning of shaking. The maximum displacement was observed during the first half of shaking, and during the last half of shaking lateral displacement slightly decreased. This rebound of the lateral displacement would be affected by the elastic component of piles. The largest lateral displacement among test cases was observed at the case of without-countermeasures. The lateral displacements in case of the earth retaining wall and the streamlined shield block method are significantly decreased from that of without-countermeasures. In the case of without-countermeasures the foundation suffered lateral forces not only from the saturated layer but also from upper unsaturated layer, while in the case of the earth retaining wall the footing did not suffer earth pressures from upper unsaturated layers. The maximum displacement of the earth retaining wall was about 50% of that in case of without-countermeasures, which indicates that decrementation of flow forces along the unliquefiable layers is significantly effective for damage of foundations.
The vibration components on the displacement time histories were between 15 and 20 cm irrespective of the test cases. This result indicates that the countermeasures mainly act for mitigation of lateral residual deformations.

Figure 5 shows the residual displacements at the top of the footings. Those of any countermeasures were smaller than that of without-countermeasures on 75cm. The minimum displacement was observed at the streamlined shield block method, which was 12% of without countermeasures.

4.2 Pore water pressures

Figure 6 shows time histories of excess pore water pressure ratio in case of the drain piles method. The maximum excess pore water pressure ratio at WP-21 and WP-22 which were located at the outside of drain piles reached 1.0, which means the free field reached liquefaction. Meanwhile WP-23 that was located around drain piles did not reach 1.0 (except vibration components), which indicated that drain piles were effective for non-liquefaction. However, the excess pore water pressure ratio at WP-23 was built up to 0.5, it was thought that the stiffness of the ground below the footing became about 75% of the initial stiffness. In this state, it was impossible to prevent the lateral flows toward the caisson.

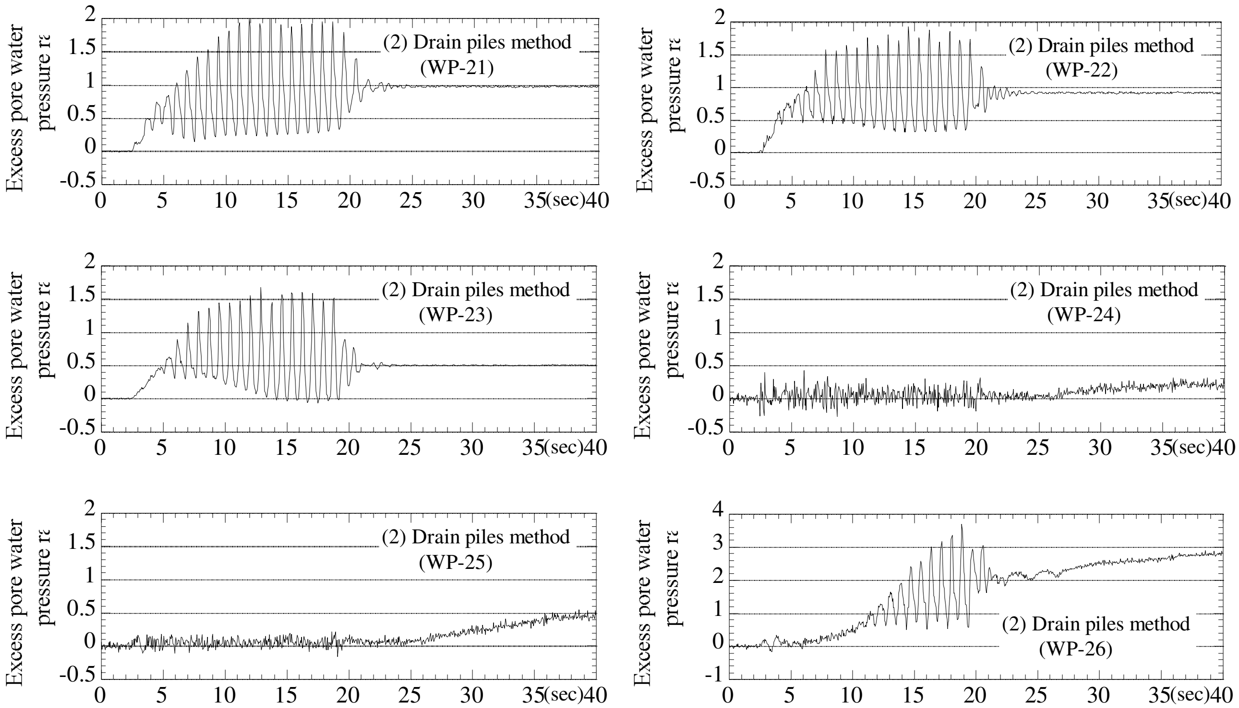

Figure 6 Time histories of excess pore water pressures

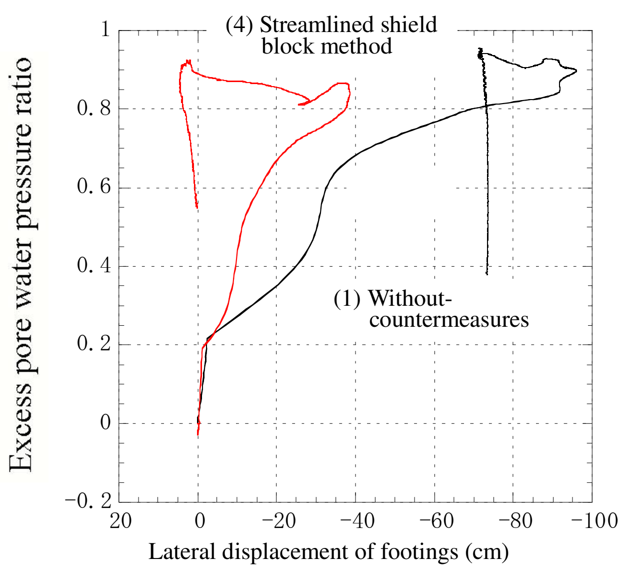

Figure 7 Relationship between lateral displacement and excess pore water pressure

As the result of liquefaction behind the caisson, the residual lateral displacement of the drain piles method occurred about 40cm, which was about 55% of the without-countermeasures. It was shown that the drain piles method can reduce the residual displacement, however it was not enough to prevent the lateral flows.

Regarding the excess pore water pressure in the unsaturated layers such as WP-24 and WP-25, the excess pore water pressures slightly built up at the place far from drain piles after the end of shaking of around 25 second. However, the excess pore water pressure in the gravel mat such as WP-26 is rapidly built up during shaking (after 7 second), eventually the excess pore water pressure ratio exceeded over 3. It is indicated that drain piles are effective for feeding pressurized water that lead to decrease the stiffness of unsaturated layers. In this centrifuge test, the lateral residual displacement had already reached the maximum displacement as described later using Figure 7, which was not able to reduce the deformation effectively.

Figure 7 shows the relationship between lateral displacement of the footing and excess pore water pressure (residual component) at the saturated layer of the backfill ground. Both without countermeasures and the streamlined shield block method the footing moved to the caisson in proportion to the excess pore water pressure by the time when the excess pore water pressure ratio built up to about 0.2. It is to be noted that both two cases the residual lateral displacement reached 50% of the maximum displacement when the excess pore water pressure ratio built up to about 0.7. When the pore water pressure ratio became between 0.7 and 0.9, the residual displacement extremely progressed up to the maximum displacement, eventually rebound deformation is initiated. Especially, the rebound of lateral displacement of the streamlined shield was significant. It was shown that the streamlined shield makes the footing to set back to the original position. The residual displacement was slightly progressed toward the caisson during dissipation of the pore water pressure. Difference between without-countermeasures and the streamlined shield block method is obviously observed when the pore water pressure ratio built up to 0.2, especially up to 0.7.

4.3 Strains of piles

Time histories of bending strains of piles together with the yield strain of 1.4x10-3 are shown in figure 8. In case of the without-countermeasures the maximum strains exceeded the yield strain at both pile head and lower boundary. On the other hand, in cases of both the earth retaining wall method and the streamlined shield block method the maximum strains of all portion of the pile were kept within elastic condition. In case of the drain piles method the maximum strain did not exceed the yield strain except one at the boundary.

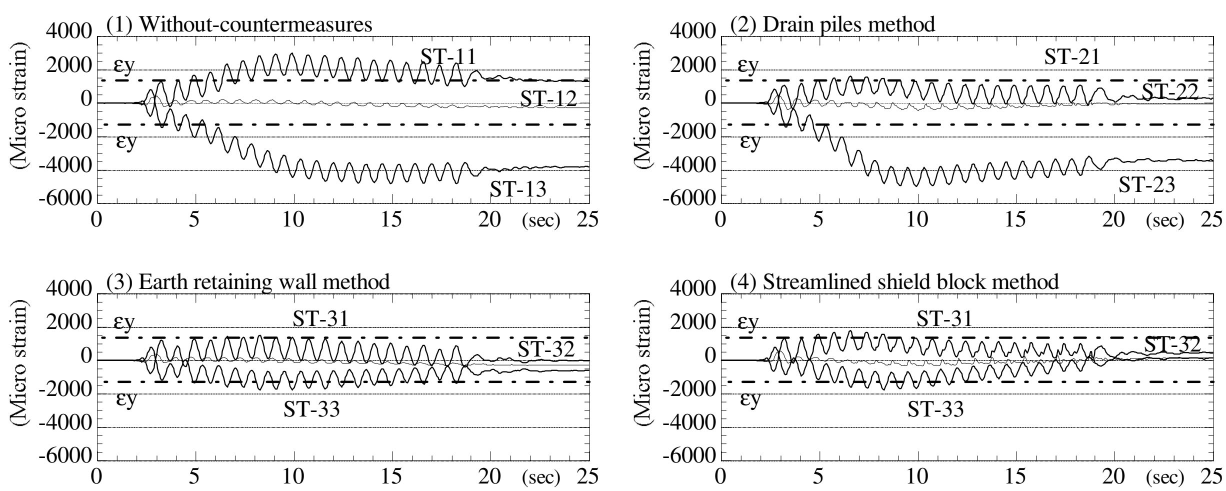

Figure 8 Time histories of bending strains

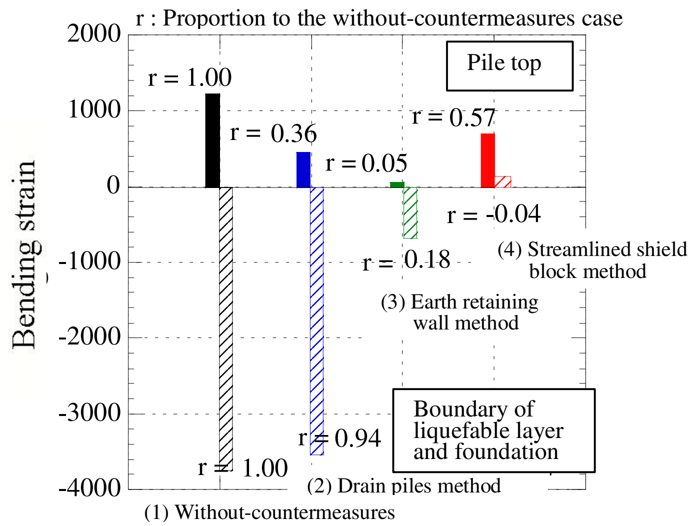

Figure 9 Comparison on the maximum bending strain

Figure 9 shows the comparison of residual bending strains ratio (proportion to without countermeasures) among all test cases. Regarding to the strains at the pile head, any countermeasures can reduce the residual bending strain at least 60% of without countermeasures. Especially both the earth retaining wall method and the streamlined shield block method can reduce bending strains more than 20% of without countermeasures.

5. CONCLUSIONS

The dynamic centrifuge test on three countermeasures against lateral ground flow caused by liquefaction was carried out. These countermeasures are designed with reducing the external forces to the footings. The earth retaining wall method and the streamlined shield block method are directly able to reduce the lateral flow force to the footings. As a result of the tests, the streamlined shield block method indicated remarkable decrementation of residual lateral displacement and bending strain of piles. It is noted that the prevention mechanism of the streamlined shield were not only to reduce the lateral forces due to the ground flow caused by liquefaction but also to set back to the original position. This countermeasure does not require any soil improvement to prevent ground liquefaction, so that the cost performance of construction would be higher than other countermeasures.

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