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Full-scale lateral loading tests of column foundations in geosynthetic-reinforced soil retaining walls

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ABSTRACT: The stability against lateral loading of the foundation for such structures as electric poles constructed immediately behind the reinforced zone of geosynthetic-reinforced soil retaining wall (GRS-RW), used as permanent railway structures, was investigated by means of full-scale lateral loading tests. The test results demonstrate that the GRS retaining walls can withstand large concentrated outward lateral loads which far exceed the current design load. The mechanism is that the reinforced zone spreads the lateral load to a wide area in the backfill. It is also supported by a wide area of the rigid continuous facing, being supported (i.e., anchored) by the lateral resistance of many reinforcement layers located for the full wall height in a wide wall width.

1. INTRODUCTION

In comparison with a reinforced soil retaining wall having a conventional discrete panel facing (e.g., Terre Armee retaining wall), a geotextile-reinforced soil retaining wall (GRS-RW) having a full--height continuous rigid facing has a particular advantage in effective resistance against large outward lateral load from other structures constructed on the facing or behind the facing. In particular, for railway structures, it is extremely cost-saving to construct foundations of noise barrier fences, electric poles or other utility structures, subjected to large outward lateral force, directly on or behind the facing of GRS retaining wall. **Plate 1** shows a foundation for an electric pole constructed in the reinforced zone of geotextile-reinforced soil retaining wall for railway at Amagasaki, under

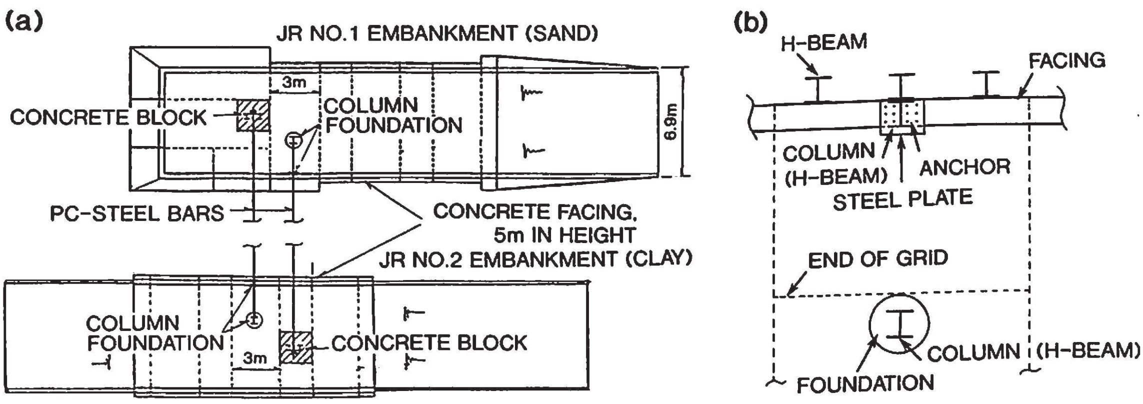

Fig. 1 Plan of the full-scale test embankments with GRS retaining wall segments with a clay or sand backfill soil: (a) general plan, and (b) details of test zone

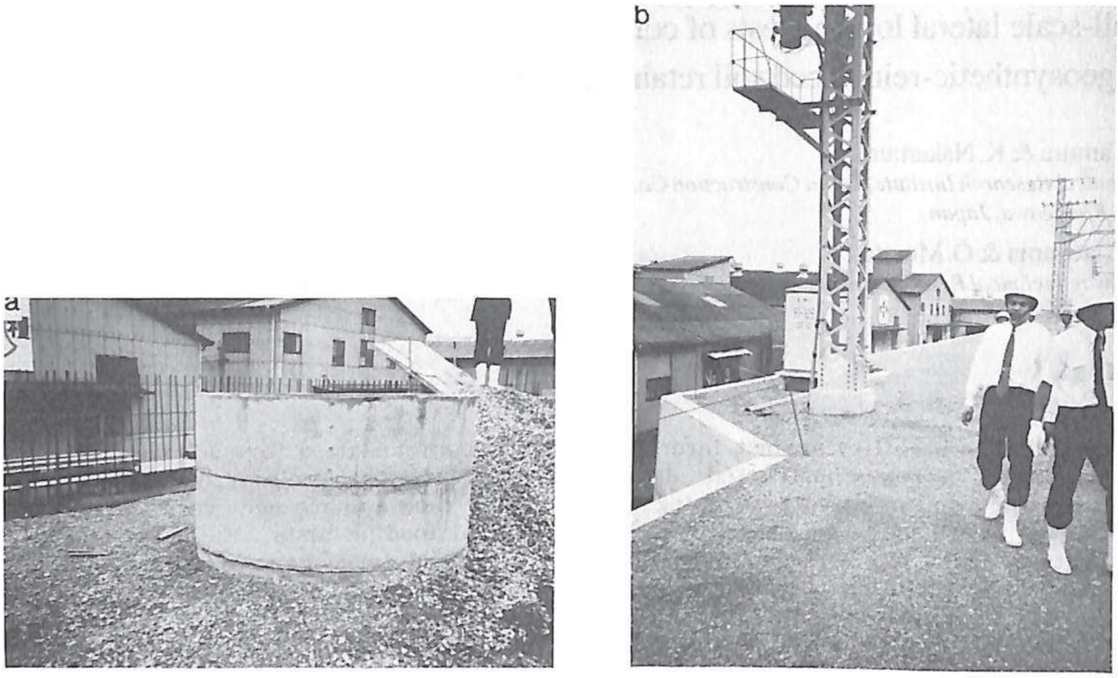

Plate 1 View of a foundation for an electric pole constructed in the reinforced zone of geotextile-reinforced soil retaining wall for railway, Amagasaki: a) under construction, and b) completed (Kanazawa et al., 1993 in this volume)

construction and completed. The design method for such construction as above is described in Horii et al. (1993) in this volume.

Conversely, flexible metal facings and discrete panel facings such as those used for Terre Armee retaining walls may not be able to support this type of load, and therefore some other, more complicated and expensive, measures may become necessary (Tatsuoka et al., 1993 in this volume). This is because reinforced soil retaining walls with a flexible or discrete panels facing should be designed so that equilibrium among the forces at each height be satisfied.

In order to examine the capability of GRS retaining wall to withstand large concentrated outward lateral load applied on the top of, or behind the facing, a model column foundation was constructed on the top of and immediately behind the reinforced zone of two full-scale GRS retaining walls. Then, lateral loading tests were performed. This paper describes the results of the loading tests on the former type of column foundations constructed behind the reinforced zone. The results for the column foundations on the top of facing are described in the companion paper (Tateyama et al., 1993 in this volume).

2. OUTLINE OF LOADING TESTS

2.1 Full-scale model GRS retaining walls

Fig. 1(a) shows the plan of the two test embankments with backfill soil of clay (Kanto Loam) or sand (Inagi sand) having several geosynthetic-reinforced soil retaining wall (GRS-RW) segments (Murata et al., 1991, Tatsuoka et al., 1991, 1992). The locations of column foundations are also shown in **Fig. 1(b)**. **Fig. 2** shows the cross-sections of the two test embankments. The backfill of the wall segments of the two embankments, in which the column foundations were constructed, had been

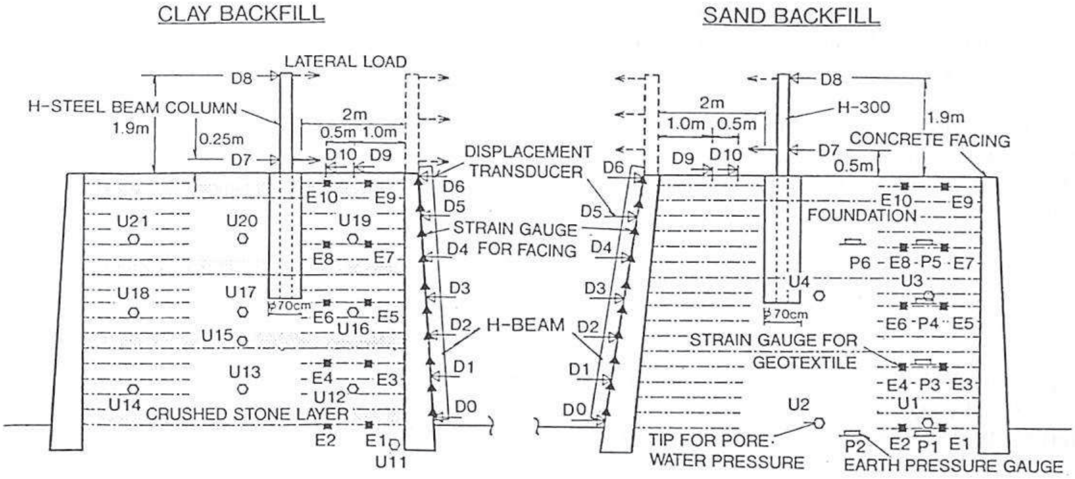

Fig. 2 Cross-sections of test GRS-RWs with column foundations

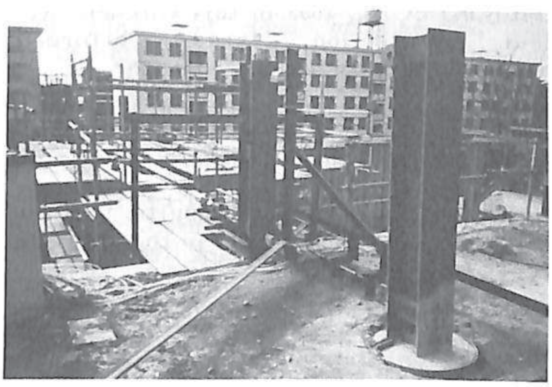

Plate 2 View of the two types of column foundations in the clay test embankment

reinforced by the same type of grid having a length of 2 m. The adjacent wall segments were separated from each other by using a friction-cut-layer at their boundary so as to perform the tests under plane strain conditions.

2.2 Model column foundations and the loading system

The following two kinds of model column foundations were constructed on each of the test wall segments (see **Plate 2**).

(1) Central columns, which are model column foundations located immediately behind the reinforced zone: This model foundation was constructed in the central zone on the crest of embankment. Their construction started with excavating a ditch to a depth of 2.5 m in the embankment very carefully so as not to damage the reinforcing material. Then, H-shaped steel beam with flange width and thickness of 30 cm and 1.5 cm, respectively, and a web depth of 27 cm (H-300) was installed in a hard cardboard cylindrical mold placed at the bottom of the ditch. Then, foundation concrete was poured in it and the space between the mold and the surrounding soil was back-filled. Finally the backfill soil was compacted. The concrete column foundations thus constructed were 70 cm in diameter and 2.5 m deep with a H-beam extruded 2 m above the crest of embankment. These columns simulate prototype electric poles as used along single track narrow gauge lines. Their design lateral load is 1 tf.

(2) Facing columns, which are model columns constructed on the top of the facing: After the loading tests on the central columns described above, a steel plate was attached by using anchor bolts to the central part of the top of the concrete facing (see Fig. 1b). Then, a 2-m high H-shaped steel beam was erected on top of the steel plate and welded in position. This type of model column represents an electric pole or noise barrier fence, for which the design lateral load is also 1 tf.

In the actual construction cases, column foundations are often constructed in the reinforced zone immediately behind the facing (see Plate 1). It was considered that the stability of column foundation in such a case can be inferred from the test results for the central and facing column models.

2.3 Loading testing method and measurements

Concentrated lateral loads were applied to each of the central and facing columns constructed as described above toward the other test embankment (see Fig. 1). To this end, cubical concrete blocks (2 m x 2 m x 2 m) were placed as a reaction body. Each column and the corresponding concrete block were connected with a PC-steel bar connected using a universal joint to the H-beam. A hydraulic jack fixed to a steel H-beam extruded from the concrete block was used to apply lateral load to the PC-steel bar.

To assess the effect of the loading moment on the behaviour of the columns, the lateral load was applied at different levels from the lowest to the highest level. For the H-shaped steel beam of the central column, the loading elevations were two, 1.9 m and 0.25 m or 0.5 m above the ground surface (see Fig. 2). For the H-shaped steel beam of the facing column, the lateral load was applied at three elevations, 0.25 m, 1.0 m, and 1.9 m above the top of the facing.

The lateral load was increased by an increment of 1 tf, and at each loading step, a constant load was maintained for 5 minutes. The loading tests of the central columns were ceased when it was considered that further loading at larger lateral load may result in noticeable effects on the result of subsequent loading tests. Subsequently, the loading tests for the facing columns were done until the peak lateral load was attained, which was about 20 tonf (Tateyama et al., 1993 in this volume). All data collection was performed automatically with a micro-computer.

3. RESULTS FOR CENTRAL COLUMNS

Figs. 3 and 4 show the results for the center columns constructed in the clay and

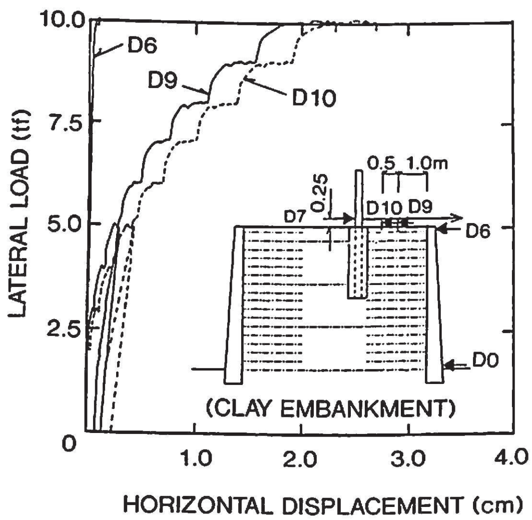

Fig. 3 Relationships between the lateral load and the horizontal displacements at several points on the crest of wall; load applied at 0.25 m from the ground surface, the central column in the clay embankment

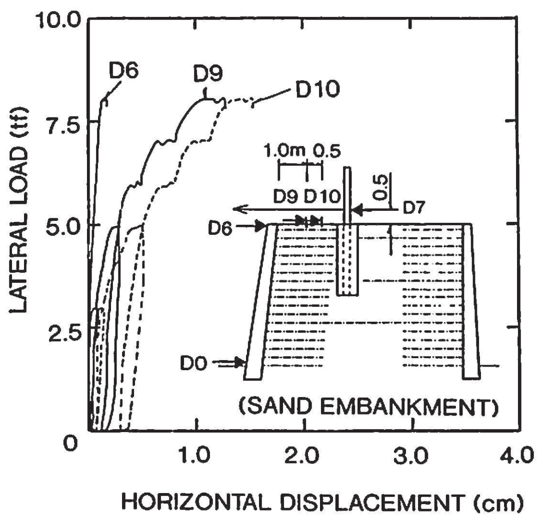

Fig. 4 Relationships between the lateral load and the horizontal displacements at several points on the crest of wall; load applied at 0.5 m from the ground surface, the central column in the sand embankment

sand embankments, respectively. Presented are the relationships between the lateral load applied at a level of 0.25 m or 0.5 m above the ground surface and the outward lateral displacements, measured at several locations along a line in the direction of loading on the crest of the wall. It may be seen that in both cases, even at the maximum applied lateral load (10 tonf and 8 tonf, which far exceeded the design load of 1 tonf), the displacement of the facing was extremely small: a value between 1 and 2 mm at the top of the facing (D6). The displacement at the bottom of the facing (D0) was negligible.

It may also be seen that at the maximum applied lateral load, the outward lateral displacements at distances of 0.5 and 1.0 m from the back edge of the top of the facing (D9 and D10) was 2 to 3 cm and 1 to 2 cm for the clay and sand embankments, respectively. These values were noticeably larger than those at the facing, and smaller than those at the column (as shown in **Figs.** 5 and 6). The results indicate that the outward lateral displacement on the crest of wall decreased as the distance from the column increased, showing that the backfill soil between the facing and the column was compressed, while being retained effectively by the rigid facing.

These results clearly indicate that against such lateral loads, the strength of the GRS retaining walls was so large that the stability of the central columns was not controlled by the stability of the wall. Rather, the bearing capacity of the soil surrounding the column controlled the stability of the columns. It will be shown later that the bearing capacity of the surrounding soil increased by the presence of reinforcement.

The behaviour of the loaded central columns was examined in more details. **Fig.** 5 shows for the central column in the clay embankment the relationships between the displacement of the column at the level of load application (0.25 m above the ground surface, D7) and the lateral load applied either near the bottom or the top of the column (0.25 m or 1.9 m above the ground surface; T0 and T2 as shown in the inset figure of Fig. 6). It may be seen that for an identical lateral load, the displacement was larger when the load was applied at a higher level of the column.

The data shown in **Fig.** 5 were replotted relating the overturning moment and the lateral displacement at D7 (**Fig.** 6). The

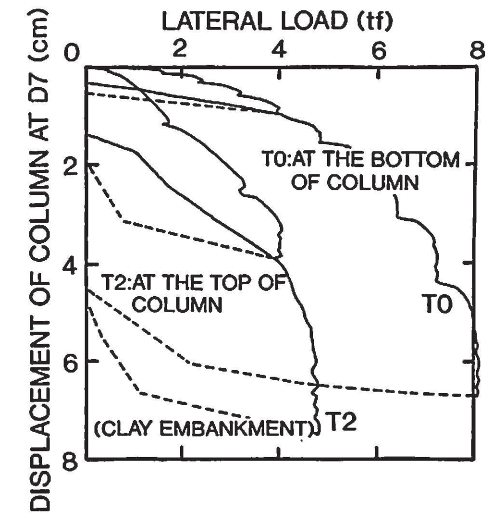

Fig. 5 Relationship between lateral load and lateral displacement, the central column in the clay embankment

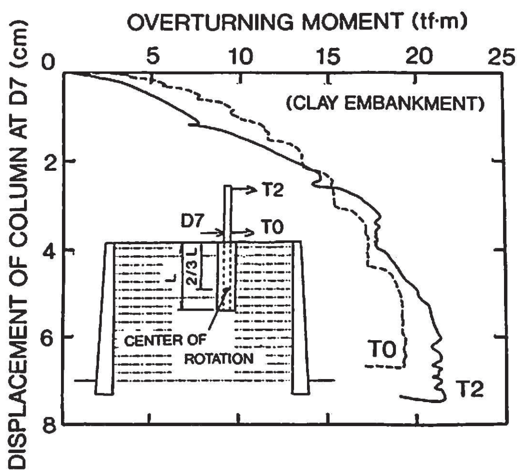

Fig. 6 Relationship between overturning moment about the one third of the embedded length of the foundation from the bottom end and the lateral outward displacement (D7), the central column in the clay embankment

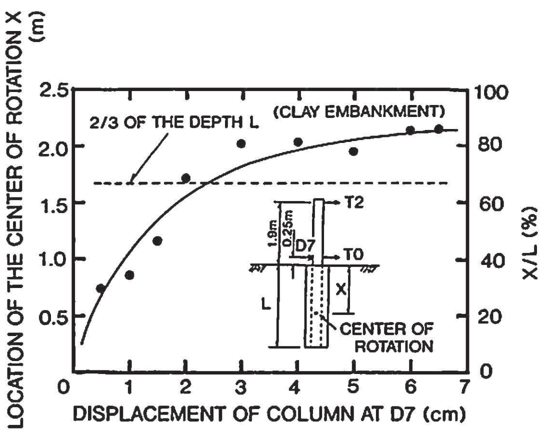

Fig. 7 Change in the center of rotation with lateral loading, the central column in the clay embankment

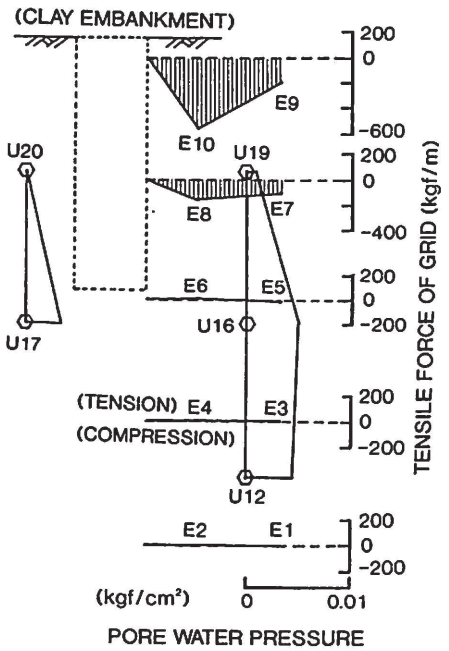

Fig. 8 Development of the compressive force in the axial members in the grid and positive pore water pressure in the backfill due to the lateral loading, the central column in the clay embankment

overturning moment was defined tentatively about a point at one third of the embedded length of the column foundation measured from its bottom end. It may be seen that the general trend between the two relations is similar for the loading at two different levels. However, more detailed observation of this result shows that the two curves deviate from each other, for which the trend changes with loading.

Fig. 7 shows the relationship between the level X of the center of rotation defined downward from the ground level and the outward lateral displacement D7. For this plot, the center of rotation was obtained so that the overturning moment becomes the same at an identical displacement at D7 for the loading at the two levels. It is seen that the center of rotation moved downward as the lateral displacement (or load) increased, indicating the ground yielded progressively from the level near the ground surface to the deeper levels.

The above point can also be seen from **Fig. 8**; with lateral loading of the central column, only the incremental compressive force has developed in the axial members of grid in the direction perpendicular to the facing (or in the direction of lateral load) in the two reinforcement layers from the top, located at the depth of the embedded foundation. Note that the forces in the grid shown in Fig. 8 are incremental representing the change in grid force due to the lateral loading.

The data presented above show that to increase the stability of this type of column against such lateral loading, it is most effective to increase the bearing capacity of the adjacent soil, particularly of that near the ground surface. The observation of the ground surface between the column and the facing showed that tensile cracks developed in the radial direction from the column (**Fig. 9**). The directions parallel and perpendicular to the cracks show the directions of the major and minor principal strains (the compressive and tensile strains), respectively. As the direction of the tensile strains conformed with the direction of the longitudinal members of the grid, the members performed as tensile reinforcement to increase the bearing capacity of the ground against such lateral loading as illustrated in **Fig. 10**. This implies that the concentrated lateral load applied to the ground was effectively spread to a wider area in the ground towards the back face of the facing. This factor with grid reinforcement is of significant advantage over metal strip reinforcement. Because of the overall rigidity of facing, the lateral load activated at the facing could be retained by the axial members of reinforcement layers located in a wider area on both sides of the column (although only the compressive strains were developed in the axial members of the reinforcement layers adjacent to the column).

The compression of soil between the column and the facing can be inferred also from the development of the positive pore water pressure in this zone (Fig. 8). This fact also indicates that the facing retained very effectively this type of concentrated lateral loading. This point can also be seen from the behaviour of the facing as described below.

The facings constructed for the two test embankments were cast-in-place unreinforced plain concrete and had horizontal construction joints at two levels. It was observed that during the loading tests with a strip footing on three test wall segments of the sand test embankment, the failure of the construction joints controlled the strength of the walls (Murata et al., 1991, Tatsuoka et al., 1991). Based on these observations, it is specified in the current design standard for the GRS-RW system (Ministry of Transport, 1992) that the facing be lightly steel-reinforced, designed to retain earth pressure as large as the one activated in unreinforced backfill soil. To simulate the facings actually used for GRS retaining walls, after the construction of these test embankments, the facing of one test wall segment of the clay test embankment was stiffened to the level of the facings of the actual GRS retaining walls by attaching three H-shaped steel beams on the front face of the facing. Then, loading tests on the test wall segment with a strip footing was performed. Subsequently, for the lateral loading tests described in this paper, other two facings of the two test wall segments of the sand and clay test embankments (see Fig. 1) were made stiffer by using three H-shaped steel beams (**Fig. 11a**).

The strains in the flange in contact with the front face of the facing of the H-beam were measured by attaching strain gauges on both sides of the flange. **Fig. 11(b)** shows the compressive strains (averaged values of the strains on the both sides of the flange) along the wall height, measured when the peak lateral load was applied to the central columns in the clay and sand embankments. These strains reflect on the axial strains along the front face of the concrete facing. These results indicate that the full height of facing was bending with the top most displaced outward, acting as a cantilever while spreading the lateral load activated at the upper levels of facing to the full height of the wall.

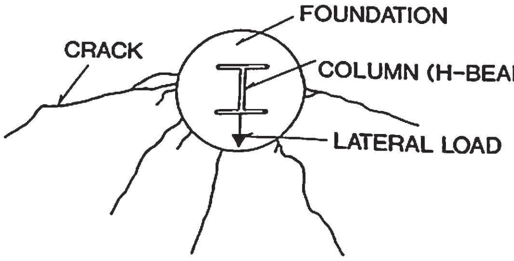

Fig. 9 Sketch of the ground surface adjacent to the central column at the peak lateral load, the sand embankment

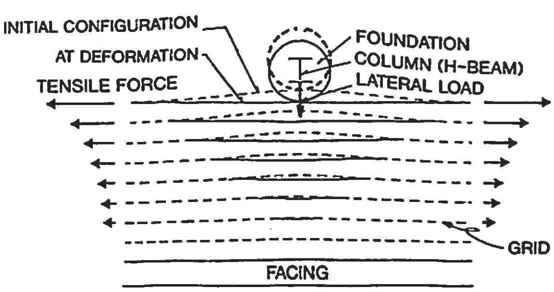

Fig. 10 Schematic diagram showing the function of the longitudinal members of grid as tensile reinforcement to resist the lateral load from the column, the central column in the clay embankment

4. DISCUSSION

It is inferred from the test results described above that the column foundation constructed immediately behind the reinforced zone in GRS retaining walls be more

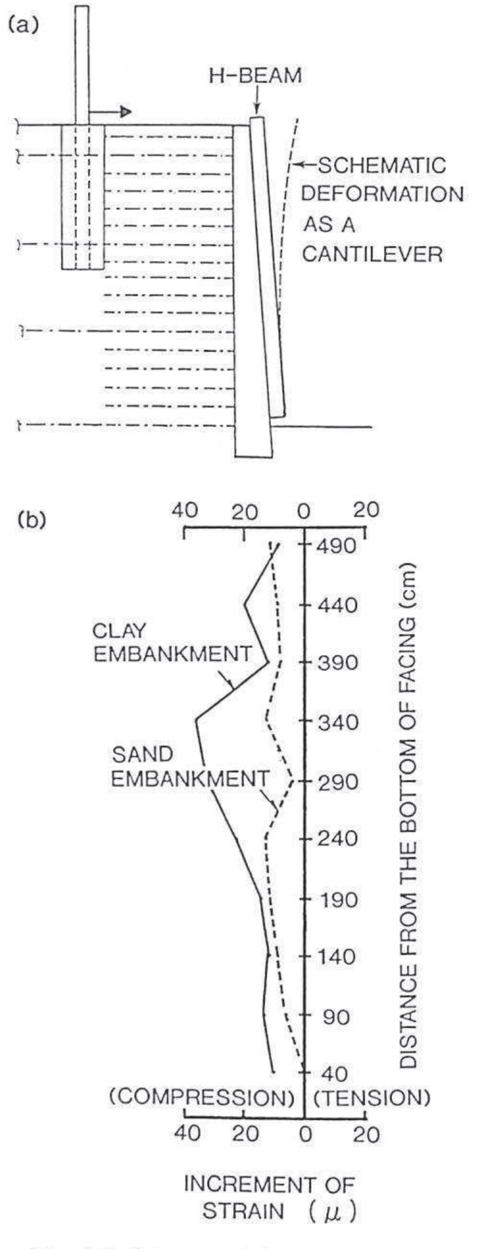

Fig. 11 (a) H-beam with strain gauges on the flange in contact with the front face of facing and (b) compressive strains in the flange observed at the peal lateral load applied to the central columns

stable against outward lateral load than when constructed in level unreinforced ground.

Further, as reported in the companion paper (Tateyama et al., 1993 in this volume), for the similar outward lateral loading condition, the facing column model constructed directly on the top of facing was more stable than the central column model constructed immediately behind the reinforced zone. It can be inferred, therefore, that the column foundation will become more stable when constructed in the reinforced zone (closer to the facing) than when constructed immediately behind the reinforced zone.

5. CONCLUSION

The results of the full-scale lateral load tests described in this paper showed that the geosynthetic-reinforced soil retaining wall can retain a column foundation constructed in the zone behind the facing to which very large outward lateral load (about ten times larger than the design lateral load of 1 tonf and moment of 2 tonf·m) is applied. Namely, the strength of the GRS retaining wall itself does not control the stability of the column. Rather, the column foundation becomes more stable than it is when constructed in unreinforced level ground. Three dimensional effects by both continuous rigid facing and grid reinforcement (having axial and longitudinal members) contributed to the increase in the stability of such a column foundation. In particular, the tensile reinforcing mechanism by the longitudinal members of grid helped in increasing the bearing capacity of the backfill soil against the concentrated lateral load from the column.

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