GEOSYNTHETIC-REINFORCED SOIL RETAINING WALLS FOR ABUTMENTS

CULEES DE PONT EN TERRE RENFORCEE PAR GEOTEXTILES

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SYNOPSIS: From loading tests on scaled and full-scale models it was found that facing rigidity increased the stability of the wall remarkably. Based on these results GRS-RWs with a continuous rigid facing were used for actual railway bridge abutments. Observations of these abutments during loading tests and actual repeated train loads showed that the GRS-RWs bridge abutments had been very stable.

INTRODUCTION

The authors proposed a reinforced soil retaining wall system using short geosynthetic sheets and a continuous rigid facing (Murata et al.,1991,1992, Tatsuoka et al.,1991). This type of facing is a cast-in-place lightly reinforced concrete layer placed directly on a geotextile wrapped-around wall face. To ensure its applicability to actual construction projects, we constructed two full-scale test embankments, then observed their long-term behavior and finally performed loading tests bringing them to failure. Further, a series of shaking table tests of scaled models were performed for the aseismic design. From these test results we found that the use of a continuous rigid facing was very effective to stabilize a reinforced wall and to reduce its deformation. It was also found that when a planar geotextile and a continuous rigid facing were used the reinforcement could be relatively short (say only 30% of the wall height) without losing its stability. Based on these results the authors proposed a design method which takes the effects of facing rigidity into account. And the total length of geosynthetic-reinforced soil retaining walls (GRS-RWs) of this type which support railway tracks now amounts to more than 6km. Furthermore, this type of GRS-RWs have been used to construct bridge abutments which support directly an over-road railway bridge girder at five places. At two places, the backfill and reinforcement were heavily instrumented to observe their behavior to confirm the safety of this new method and to establish a more rational design method.

This report describes the results of scaled model tests in the laboratory and full-scaled model tests, in connecting with the application to abutments , the behavior of these abutments in the field and its analysis.

SCALED AND FULL-SCALE MODELS

In the laboratory, model walls of sand, 52cm high, with different types of facings were brought to failure by loading on their crest (Tatsuoka et al.,1989). Fig.1 is a summary showing the results for the following four types of facing; a continuous rigid facing (Type D), a discrete panels facing having a rough back face(Type C), a discrete panels facing having a soft material in between vertical adjacent panels and a smooth back face so that the axial and shear forces and moment are not exerted in the facing (Type B) and a rubber membrane facing(Type A) . For each facing type, the wall was loaded with a 10cm-wide footing having a smooth base. The model grid reinforcement was designed not to rupture in tension. The length was 15 cm (only 30% of the wall height). In Fig.1, q_u is the ultimate average footing pressure when the footing was placed on the crest of reinforced

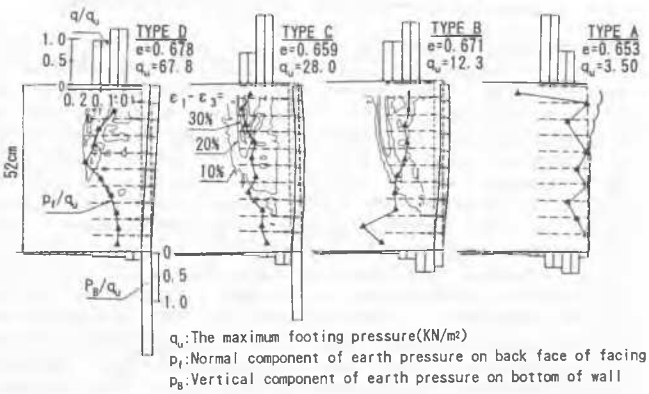

Fig.1 Pressure distributions on the bottom face of footing(p_B/q_u) and the back surface of facing(p_f/q_u), and the formation of wall

zone. The local footing pressure q for the two halves of footing and the earth pressure on the back face of the facing, the bottom of the wall(p_F and p_B) are divided by q_u. The values of p_F and p_B are due to the footing pressure. The contour lines show local shear strain ε_1-ε_3(%) observed at the failure. It may be seen that the wall became more stable as the facing rigidity increased and pressure working at the back face and the bottom of the facing increased as the facing rigidity increased. This means that larger part of the weight of the back fill and the footing load can be supported by the facing with adequate rigidity and thereby the resistance of wall against overturning and sliding out increases .

To confirm the findings described above, two full-scale test embankments (sand and clay backfill soils) were constructed in 1987 and 1988 (Murata et al.,1991,1992). After observing their behavior until the end of 1989, loading tests were carried out to bring them to failure. The following lessons were obtained:

(1) As far as the long-term behavior is concerned, the walls having a continuous rigid facing exhibited a very small deformation (an outward horizontal deflection at the crest of the wall was less than 3 mm for 18 months), while one test wall segment having a discrete panels facing exhibited a much larger deformation (about five times of the above).

(2) In the loading tests performed by using a footing on the crest of embankment, the wall segments with a continuous rigid facing were very stable. The wall with a discrete panels facing exhibited smaller resistance than the others wall segments having a continuous rigid facing.

(3) The wall was stronger against loading from the crest of the reinforce zone (Fig.1) than against the loading immediately behind the reinforced zone as in the laboratory tests .

From the above, it was considered that when a continuous rigid facing is used and the wall is adequately designed, GRS-RWs can be constructed safely and will perform very satisfactorily as the permanent important structures. Considering a very high strength against concentrated load close to the wall face , it was decided to use this type of GRS-RW for bridge abutments, on which heavy loads will act near the crest (as the front loading in Fig.1), allowing only a very small deformation of the wall.

GEOSYNTHETIC-REINFORCED SOIL BRIDGE ABUTMENTS

The actual construction was started at three sites in Japan in 1989 and 1990. At Biwazima in Nagoya City, the crest area of the yard for the bullet train (Sinkan-Sen) was made wider by reconstructing by means of GRS-RWs the relatively gentle slope of the existing embankment with an average height of 5m and a total length of 930m. In Amagasaki city next to Osaka city, part of the existing railway embankment was reconstructed to add two tracks to the existing four tracks for one of the busiest and most important railways in Japan, Tohkaido Line. These GRS-RWs constructed at the three sites are all permanent important structures, which allow only a very small residual and dynamic deformation. It is specified that the largest allowable vertical displacement of the track during the passing of train should be 1mm or less. At one place at Nagoya site and four places at Amagasaki site, the GRS-RWs were constructed also as bridge abutments, supporting a simple-beam bridge girder through RC blocks placed on the crest of the wall near the wall face (Fig.2). This railway bridge was to be constructed next to the existing one. This additional bridge could not be constructed by a box girder, since its construction requires the closing of the road, which was not allowed at this site. And conventional reinforced concrete bridge abutments require a deep foundation, which was not adopted either because this could not be constructed unless the road was closed, this would cost much higher and would produce large noise. After all, the proposed GRS-RW system was adopted, since the above-mentioned problems can be solved by this method. It is to be noted that in the current design framework of railway structures in Japan, reinforced soil bridge abutments became acceptable only with the use of a continuous rigid facing, since this type of facing could resist effectively against concentrated vertical and horizontal loads from the girder activating at the crest near the wall face. Considering the importance of the structure, a relatively strong grid with a rupture strength of 6 tonf/m was used. In addition, a high-quality selected well-graded gravel was carefully compacted to a dry density of as high as about 2.2 g/cm^3. And the facing was constructed using a larger amount of steel reinforcement than other ordinary portions of the GRS-RW.

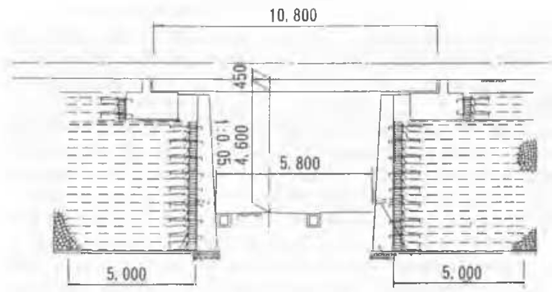

Fig.2 Cross-section of geosynthetic-reinforced soil bridge abutments at Shinkansen Yard,Nagoya

LOADING TEST ON GEOSYNTHETIC-REINFORCED SOIL BRIDGE ABUTMENTS AT NAGOYA SITE

In the backfill, measuring instruments were set as shown in Fig.3. A static load of 30 tonf, which was the total weight of a loaded dump truck and a compaction tired roller, was applied on the girder immediately above the concrete block having a base area of 2 m by 10 m. The static load was equivalent to the design train load. Fig.4 shows the earth pressure and the strain of the reinforcement when the earth pressure was maximum during the loading. These values due to the static load were much smaller than those during construction. Further the settlement of the block was less than 0.1mm and almost recoverable. The average vertical strain as obtained by dividing the footing settlement by the footing width (2 m) was as small as 0.01%. Fig.5 shows the hysteresis loops of the relationships between the vertical earth pressure and the tensile strain at three points

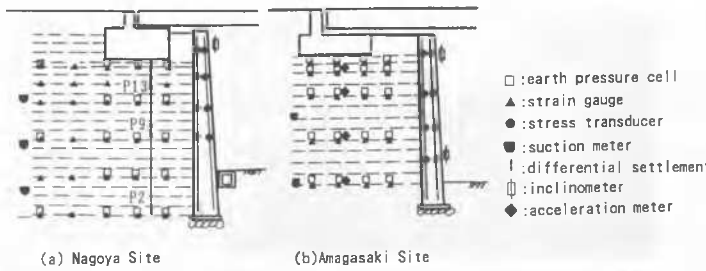

Fig.3 Arrangement of measuring instruments

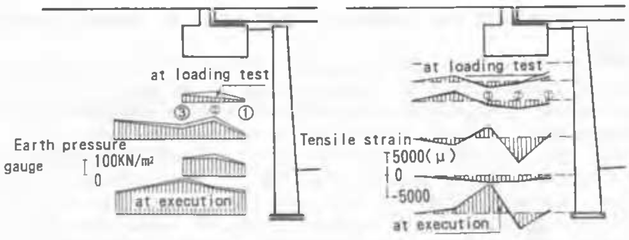

Fig.4 Earth pressure and strain of reinforcements by loading test, Nagoya site

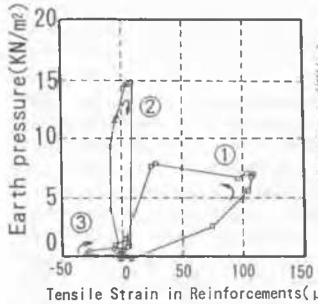

Fig.5 Relationship between of the earth pressure and the strain of the reinforcements

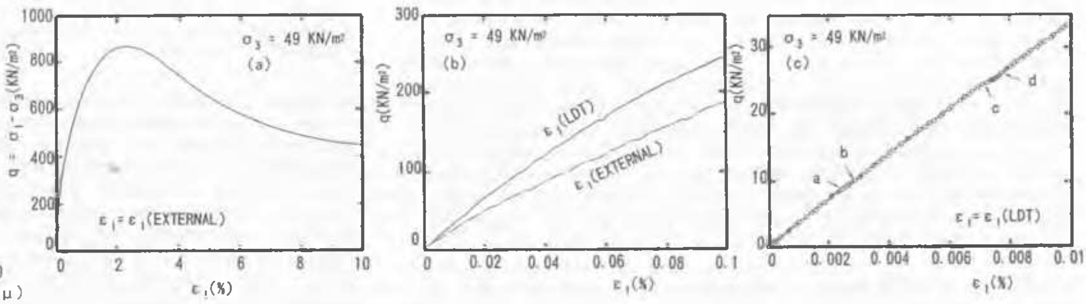

Fig.6 Stress-strain relationships in three scales from a triaxial compression test on the backfill gravel used for GRS bridge abutment in Nagoya

along the grid at a depth of 90 cm from the bottom of the concrete block, caused by one cycle of static loading. It may be seen that the strain was very small and recoverable with the largest being about 0.01%. All these results shown above indicate that under the static load, the backfill and reinforcement was far before the failure condition and almost in the elastic condition.

The above behavior of the GRS abutment was analyzed by the FEM based on the result of triaxial compression test using the backfill material(D_{50} =4.2 mm, U_0=61.2, sub-angular grain shape and a dry unit weight r_d of 2.23 gf/cm^3). The diameter and height of the specimen were 30 cm and 60 cm, respectively. Fig.6 shows typical results, where external means the axial strains obtained from the displacement of the specimen cap, which include the effect of bedding error at the top and bottom ends of the specimen, while LDT means those measured locally along the lateral surface of the specimen. In this test, some small unload/reload cycles were applied (between the points a and b, and between c and d in Fig.6c). As seen from Fig.6(c), at strains less than 0.01%, which is the strain level in the backfill during the static loading test, the initial Young's modulus E_{max} is as high as more than 300 MN/m^2 and the behavior is almost perfectly elastic. For the non-linear FEM analysis, the test results were modeled, in which E_{max} be in proportion to the mean stress $(\sigma_1+\sigma_3)/2$. In the FEM analysis, the actual construction sequence was simulated; i.e.,the filling of gravel layer and placement the reinforcement, and their repetitions, the placement of a cast-in-place concrete facing, the placement of the concrete block and the bridge girder, and finally the static loading. No special boundary element was used between the back face of the facing and the backfill considering small strain behavior.

Fig.7 compares the measured and calculated earth pressure at three in the backfill during filling elevations. A very good agreement may be seen. Fig.8 shows the comparison of the settlement in the backfill below the center of the concrete block. Despite some discrepancy between the measured and simulated values, the fact that the measured settlement was extremely small(only 0.1 mm at the base of the concrete block) was simulated fairly well.

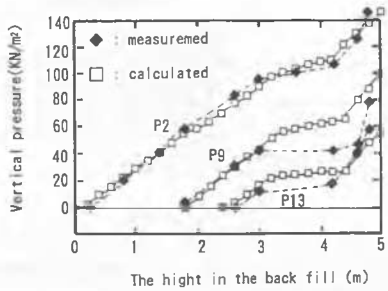

Fig.7 Comparison of measured and calculated vertical earth pressure by the static loading (see Fig.3(a))

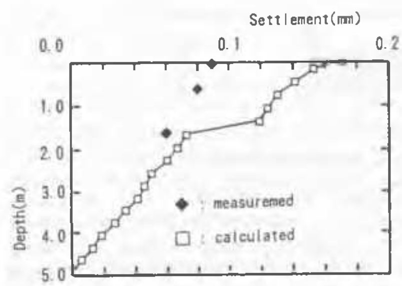

Fig.8 Comparison of measured and calculated settlement by static loading

MEASUREMENTS OF DYNAMIC RESPONSE TO ACTUAL TRAIN

For one of the GRS bridge abutments constructed at three sites at Amagasaki (Plate 1) , the dynamic response to actual train passing was measured. Fig.9 shows the increase in the vertical earth pressure in the backfill and the reinforcement axial strain at the moment when the largest wheel load was recorded during the first passing of train, which was slightly different from the moment when the maximum acceleration was recorded. The maximum acceleration was about 0.2g and about 0.1g in the vertical and horizontal directions . The vertical stress decreased with depth with the value at the bottom of the backfill being about a half of that at the top. The stress distribution with depth was at an angle of about 30^{0} relative to the vertical. Relatively large strains were recorded only in the first and second layers of grid from the top. The dynamic reinforcement force was compressive and tensile when the train load was approaching and leaving, respectively, the measurement point. The maximum tension was only about 5 kN/m, which means that the stress condition in the grid was far before the failure. Fig.10 shows the time history of the settlement of the railway track after the start of train passing at both side GRS abutments at one location, different from that shown in Fig.9. The number of passing of train (about 10 cars times 20 tonf per train) running at a speed faster than 100 km/h (Plate 1). is on average about 100 times per day. It may be seen that despite some initial settlement, the settlement has ceased after several days. However the total amount of settlement so far recorded is larger than that observed for static loading at Nagoya site. For Amagasaki site although both the degree of compaction and the quality of the backfill gravel were in generally satisfactory, they were relatively worse when compared with Nagoya site by several practical reasons. Nonetheless, any problems have been reported.
All these results described above indicate that the GRS bridge abutments so far constructed have been very stable for actual dynamic loads by train passing.

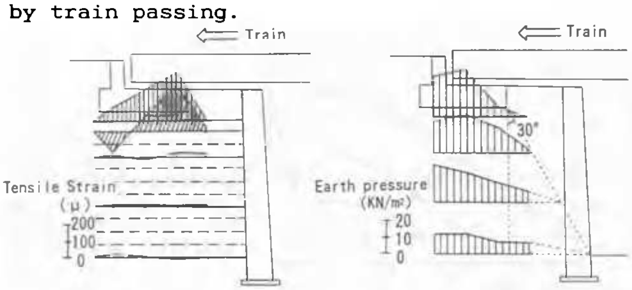

Fig.9 Earth pressure and strain of reinforcements due to train load

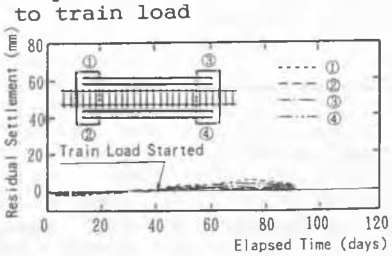

Fig.10 Settlement at the railway track by train passing at the two sides GRS abutments, Amagasaki

Plate.1 View of the train running on the geosynthetic-reinforced soil bridge abutments at Amagasaki adjacent to Osaka city (one of the busiest and most important railways in Japan, Tohkaido Line)

CONCLUSIONS

A geosynthetic-reinforced soil retaining wall (GRS-RW) system with a continuous rigid cast-in-place concrete panel has been used for the reconstruction railway embankments for more than 6 km to increase the crest area to support new railway tracks. These GRS-RWs are permanent important structures; at Amagasaki, trains of running at a speed faster than 100 km/h (Plate 1). In addition, they were used as a railway bridge abutment at four locations in place of the conventional RC abutments to directly support the bridge girder and the live load. The behavior of the abutment against static and dynamic loads were recorded and analyzed. The results showed very satisfactory performance of these GRS bridge abutments. This was due partly to that a selected backfill material is compacted very well as verified by the FEM analysis, not only due to the characteristic features of this GRS-RW system.

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