Earth retaining wall with a short geotextile and a rigid facing

Un mur de soutènement avec un géotextile court et un revêtement rigide

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SYNOPSIS Two 5m-high full-scale test embankments were constructed. They had near-vertical slopes, reinforced with various types of short geotextile. Each slope face was covered with a concrete layer. In laboratory, small models of reinforced earth retaining walls having different types of facings were failed by loading from the crest. The results clearly showed that facing rigidity increases the stability of the wall remarkedly.

INTRODUCTION

For a reinforced earth retaining wall having a vertical or near-vertical slope, its facing is usually not designed so as to explicitly contribute to the overall stability of the wall. This is because a long reinforcement extending beyond the potential failure plane is arranged so as to resist against the whole of the horizontal earth pressure acting to each soil layer. When metal strips are used, the length increases further because of its relatively smaller pull-out resistance. In this case, flexible facing structures such as metal skins or geotextile sheets or concrete panels having a compressive material in each spacing are used so that they can be compressed vertically in accordance with the compression of the back fill during its filling.

Different from the above conventional reinforcing method, the authors have been studying into another method using a short planar reinforcement (i.e., geotextile) and a rigid facing structure, as a more economical one. For this purpose, two full-scale test embankments to be used for railway were constructed in 1987 and 1988 (Fig.1). The geotextiles are either several different grid-type ones for cohesionless soil or several different sheet-type ones having a function of drainage for cohesive soil, typical of which is a non-woven geotextile reinforced with a stiffer inclusion. One of the advantages of using a planar reinforcement is that when compared with metal strips the anchoring length required for resisting the earth pressure can be much shorter because of a larger contact area with soil.

It was learnt that the possible damage to the connections between the rigid facing and the reinforcing members due to the compression of the back fill can be effectively avoided by the following two methods:
(1) Stage construction: As shown in Fig.1a, first the filling is completed with well compacting soils near the slope face using gabions placed at the shoulder of each soil layer. This construction method has also been successfully used for constructing other two full-scale test embankments using very compressive soft clay, reinforced with a non-woven geotextile (Tatsuoka and Yamauchi, 1986, Yamauchi et al., 1987, Tatsuoka et al., 1987, Nakamura et al., 1988). After the major part of post-construction compression of the back fill has been completed, a facing structure such as a thin unreinforced concrete layer is placed on the slope surface, ensuring the connection with the existing surface (Fig.1b).
(2) Using gabions as a buffer: Even when the back fill is compressed to some extent after placing a rigid facing structure, the relative settlement between them can be smoothened by using gabions.

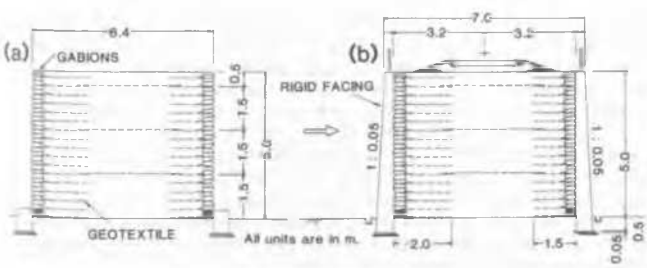

Fig.1 Cross-sections of two test embankments.

LABORATORY MODEL TESTS

In order to define various different kinds of rigidities of the facing structures and their effects, a series of laboratory small model tests were performed using different facing structures (Fig.2, Table 1):
(1) Type A facing was made of a latex rubber membrane with a thickness of 0.2mm and a tensile stiffness of about 300gf/cm. This facing laterally confined the back fill soil near the wall face only to a very limited extent, inducing its local compressive failure. This type of failure can induce the loss of the overall stability of the wall, as has been observed in a clay test embankment having the near-vertical flat slope covered only with non-woven geotextile sheets (Tatsuoka and Yamauchi 1986).
(2) Type B' facing was made of a tracing paper with a density per unit area of 170g/m^2 and a tensile stiffness of about 840kgf/cm. This facing had a local rigidity to some extent in the sense that it confined better the soil near the slope face by its larger stiffness when compared to Type A.
(3) Type B facing was made by piling up eleven rigid block components without connecting them to each other. Thus, each block had a sufficient local rigidity. However, this facing had a smooth back face with a friction angle of about seven degrees. Further, it had a 5mm thick soft material in each

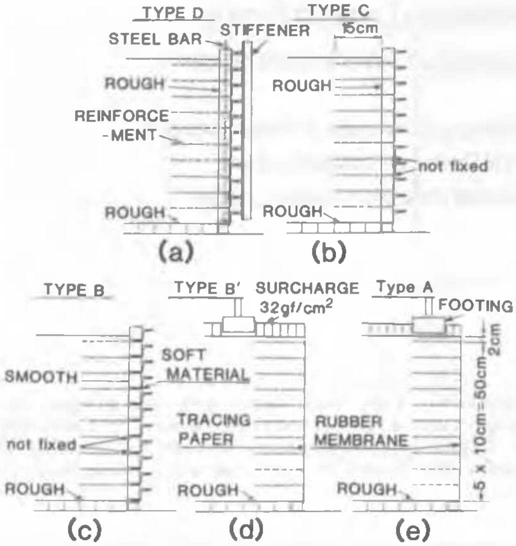

Fig.2 Cross-sections of model walls.

Table 1 Classification of facing types.

FUNCTION OF FACING STRUCTURE	FACING TYPE				
	A	B,B'	C	D	E
LOCAL RIGIDITY	NO	YES	YES	YES	YES
OVERALL VERTICAL RIGIDITY	NO	NO	YES	YES	YES
OVERALL BENDING STIFFNESS	NO	NO	NO	YES	YES
RESISTANCE BY WALL WEIGHT	NO	NO	NO	NO	YES

horizontal spacing between blocks so that it had no overall vertical rigidity in the sense that vertical compressive forces be not activated within the facing. This type of facing is a sort of simulation of the Reinforced Earth retaining wall using discrete concrete panels.

(4) Type C facing was made by piling up eleven rigid block components as Type B. However, it had a rough back face without including the soft material in each spacing. Thus, this facing had a vertical rigidity. These blocks were not fixed to each other. Consequently, this facing had a very low resistance against overall bending forces.

(5) Type D facing consisted of the same components as Type C, but these were tightly connected to each other by means of both the steel bars penetrating them and the outside stiffeners (Fig.2a and Fig.3). Thus, this type of wall had an overall bending stiffness.

The bottoms of these facings were hinged on the rigid base of the sand box, preventing their sliding out. The failure of wall due to the failure of base ground also was beyond the scope of this study.

The prototype facing structures as shown in Fig.1b have additionally a resistance against earth pressure due to its weight and are classified as Type E, as listed in Table 1.

Model reinforcement members were of grid-type, made of phosphor-bronze strips (Fig.5). Each strip is 3mm wide and 0.1mm thick and has a bending stiffness EI of 3kgf·cm². The tensile forces in the strips were measured by means of strain gages. The length was

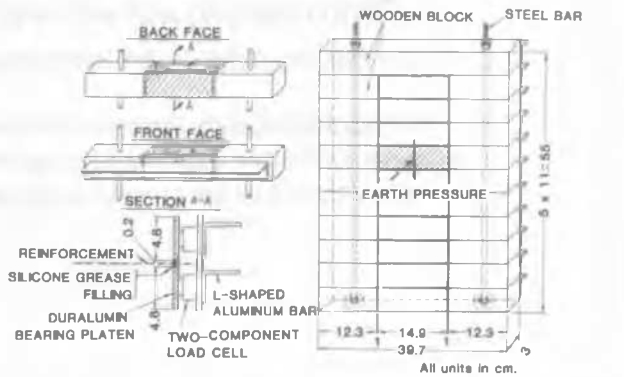

Fig.3 Details of model facing (Type D).

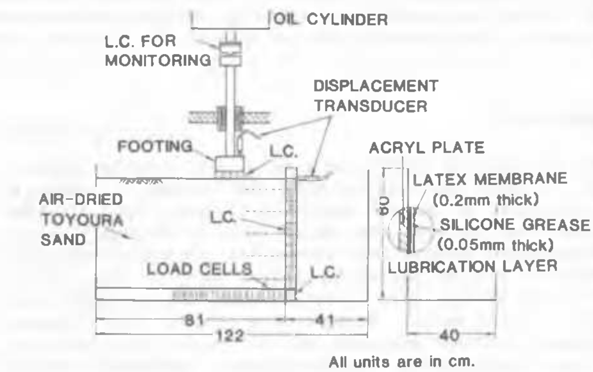

Fig.4 Loading method.

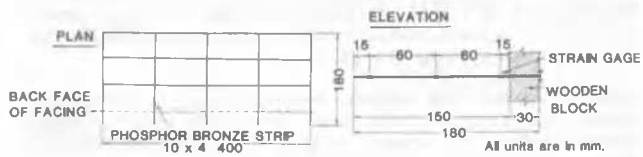

Fig.5 Model reinforcement.

15cm, which was only 29% the internal wall height, 52cm. This length is much smaller than that used for the Reinforced Earth retaining wall (i.e., of the order of 100% the wall height).

These models do not satisfy sufficiently the similitude rule. However, it was considered that the behavior of the different models reflects the comparative variation in the behavior of the corresponding prototype ones which have been created by the different facing types.

Each model was constructed in a sand box (Fig.4) by the following construction sequences as for prototype walls. For Types B, C and D, the surface of the previous sand layer was made flat and then a reinforcement layer was placed on it. Subsequently, a wall block component was placed in its position on the previous block with being supported by a temporary support. The reinforcement was connected to the block. Then, a layer of air-dried Toyoura sand was placed by pluviating through air at a controlled fall height so that a homogeneous dense back fill be made. Toyoura sand is a fine uniform sub-angular to angular sand (D_{50}=0.16mm, and the coefficient of uniformity=1.46). The deformation and strength characteristics have been thoroughly investigated.

This procedure was repeated until the whole height of wall was constructed. Then, only for Type D facing, blocks were tightly connected to each other.

For Types B' and A, the whole height of the facing was first fixed to a temporary support. Each reinforcing layer was placed on the flattened surface of the previous sand layer, with being connected by means of a hook to a transverse metal strip glued to the back face of facing. The whole height of the wall was completed by repeating this procedure.

In each test, before the temporary support was removed, a surcharge of $32gf/cm^2$ was placed on the crest of the back fill as shown in Figs 2(d) and 2(e), in order to increase pressure levels in the model for more accurate measurements of loads and stresses. At this stage, no footing load was applied. Each lateral inner surface of the sand box was lubricated (see Fig.4). The use of the surcharge increases the normal stress in the grease layer, resulting in a smaller apparent friction angle. On the outside surface of the membrane, grids with a spacing of 1cm were printed. The displacements of the nodes of grid were read to an accuracy of about 20μm by reading them on the pictures taken occasionally during each test. Then, the strain in each 1cm square element was obtained.

Horizontal (normal) and vertical (tangential) components of the earth pressure working on the central third of each of the ten wall blocks were measured by means of a two-component load cell (see Fig.3). For Types A and B', the earth pressures on the back faces of facings were estimated from the tensile forces in the reinforcements at their connections with the facings. The earth pressure distribution on the bottom of the facing and the back fill was measured by using eleven load cells in a similar way.

TEST RESULTS

As shown in Figs 2(d) and 2(e), each model was brought to failure by applying a vertical load on a part of the crest by means of a guided strip footing with a width B of 10cm having a lubricated base at an axial displacement rate of 0.08~0.10mm/min. The footing load was measured by means of two two-component load cells located at the central third of the footing. The following two types of loading methods were used for each type of the models. (a) Front loading; as shown in Fig.2(e), the footing was located above the reinforced zone with the heel of footing above the back of reinforced zone. (b) Back loading; as shown in Fig.2(d), the footing was located behind the reinforced zone with the toe of footing above the back of reinforced zone. These loading conditions were employed to induce (a) vertically compressive failure of the reinforced zone, and (b) overturning of the reinforced zone behaving like a monolith.

Figs 6(a) and 6(b) respectively show the relationships between the average footing pressure q and the footing settlement for the two loading methods. Note the difference in the scale for q in these two figures. Fig.7 compares the peak average footing pressures q_u in the normalized form $2q_u/(\gamma\cdot B)$ where γ is the unit weight of back fill, excluding Type B'. It may be seen that the strengths of the different models are remarkedly different in each loading pattern; i.e., the wall having a more rigid facing structure is stronger. It may further be seen that the effects of facing rigidity is more remarkable when being loaded on the reinforced zone. In particular, in the case of front loading, the wall with the Type A facing

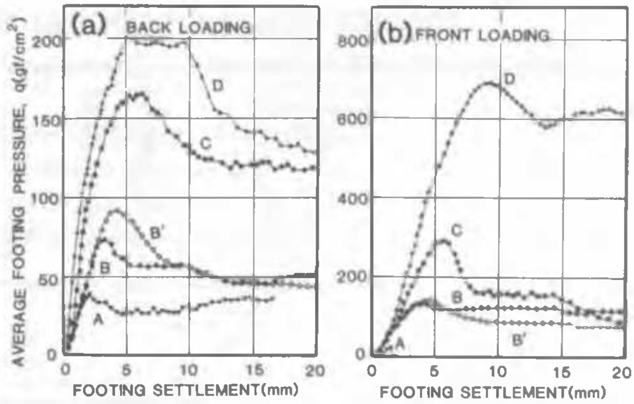

Fig.6 Behavior of different walls.

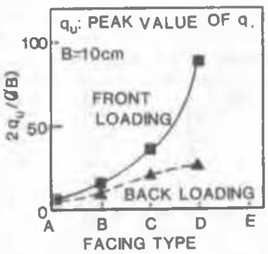

Fig.7 Peak footing pressure versus facing type.

collapsed partially only by removing the temporary support after applying a surcharge of $32gf/cm^2$ between the footing and the slope face on the crest before applying any footing load. These results suggest that especially when a concentrated load is applied on the shoulder as in the case of an abutment for a bridge, a more rigid facing structure such as Type D is recommended. Figs 8 and 9 show (a) the strain fields (contours of $\varepsilon_1-\varepsilon_3$) observed at a footing settlement of 15mm, (b) the distributions of the horizontal normal earth pressure p_f, divided by q_u, working on the back face of facing induced by peak footing load, and (c) the distributions of the vertical pressure p_B, divided by q_u, on the bottom of model induced by peak footing load. In Fig.8, the strain field for Type A is not shown, since at a footing displacement of 2mm, the failure of wall occurred.

The following points may be seen:
(a) For a facing having a smaller degree of rigidity, shear band(s) were formed more clearly within the reinforced zone, associated with larger deformation of facing. In particular, in the case of back loading (Fig.9), for Type A facing, a distinct shear band starts from the footing heel and ends at an intermediate height of facing, whereas for Type D facing the shear band starting from the footing heel goes downwards and then heads for the heel of the reinforced zone.
(b) For a facing having a larger degree of rigidity, the center of earth pressure on the back face of facing is located higher. This indicates that with a larger degree of facing rigidity, the back fill near the facing is better confined. This finding is supported by the distribution of tensile forces in the reinforcements induced by peak footing load (Fig.10). It may be seen that as the degree of facing rigidity increases, the tensile forces near the back face of facing increase.

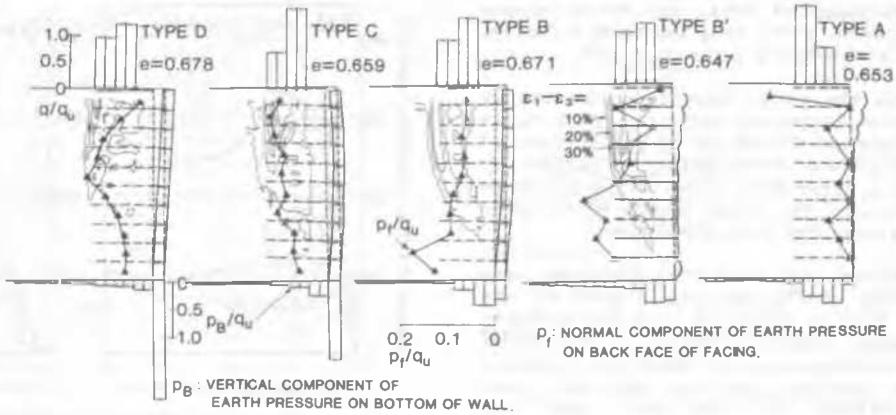

Fig.8 Test results when loaded on the reinforced zone. Pressures are due to peak footing load only.

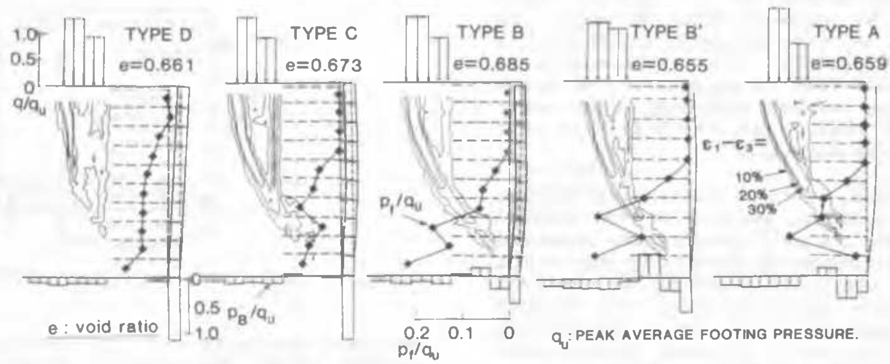

Fig.9 Test results when loaded behind the reinforced zone.

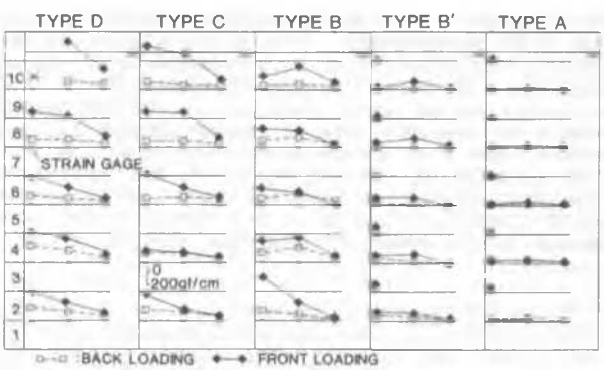

Fig.10 Tensile forces in reinforcements at peak footing load in gf/(unit wall width = 1cm) (* gages were broken due to excessive strains).

(c) For Types D and C facings, the vertical force working at the bottom of facing is much larger than the vertical pressures working at the bottom surface of reinforced zone, whereas for Type B it is not the case. This means that a larger part of the weight of the back fill and the footing load is supported by the facing and thereby the resistance of wall against overturning about its toe increases as well. It was further found that on the bottom of the reinforced zone and the facing, the angle of friction was mobilized only slightly. This means that when such short reinforcements are used, overturning is more likely to occur than the horizontal sliding out.

CONCLUSIONS

It has been demonstrated by the laboratory small model tests that a vertical retaining earth wall reinforced with a short planar reinforcement becomes very stable by using a rigid facing. Damage due to relative settlements between the rigid facing and the back fill can be avoided by filling up the wall using gabions and subsequent placement of rigid facing, as demonstrated by the construction of two full-scale test embankments.

REFERENCES

Nakamura, K., Tatsuoka, F., Tamura, Y., Iwasaki, K. and Yamauchi, H. (1988). Roles of facing steep clay slopes with a non-woven geotextile, Proc. of Int., Geotechnical Symp. on Theory and Practice of Earth Reinforcement, Fukuoka, to appear.

Tatsuoka, F. and Yamauchi, H. (1986). A reinforcing method for steep clay slopes with a non-woven fabric, Geotextiles and Geomembranes, 4, 241-268.

Tatsuoka, F., Nakamura, K., Iwasaki, K., Tamura, Y. and Yamauchi, H. (1987). Behavior of steep clay embankments reinforced with a non-woven geotextile having various face structures, Proc. of Post Vienna Conference on Geotextiles, 387-403, Singapore.

Yamauchi, H., Tatsuoka, F., Nakamura, K., Tamura, Y. and Iwasaki, K. (1987). Stability of steep clay embankments reinforced with a non-woven geotextile, ditto, 370-386, Singapore.