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A REINFORCING METHOD FOR EARTH RETAINING WALLS USING SHORT REINFORCING MEMBERS AND A CONTINUOUS RIGID FACING

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ABSTRACT: To develop a reinforcing method suitable for most on-site soils, six full-scale test embankments having near vertical slopes were constructed, five using near-saturated clay and one using sand. This method uses relatively short sheets of a grid for sandy soils and a stiff woven/non-woven composite having a function of drainage for cohesive soils. Further, a continuous rigid facing is placed on the wrapped surface of the wall which has first been constructed with the aid of gabions. The long-term behavior and the loading test results of the last two embankments are described. The reconstruction of about 2.5km-long railway embankments has started in 1990.

INTRODUCTION

A study has been undertaken for the last decade to develop a reinforcing method which satisfies the following requirements:

(A) On-site soils as obtained from an excavation work, including sands with a high fines content and even near-saturated cohesive soils, can be used as the backfill soil. Their use will be a large cost saving, compared with the cost for the use of cohesionless soil transported from a remote place and the treatment of excavated soil.

(B) It can be used for the reconstruction of existing embankment making a gentle slope steep to produce a wider crest area, without a large amount of excavation work. Fig. 1 shows its example.

(C) Deformation of slope, especially the settlement at the crest, should be very small, so as to be used for railway embankments.

(D) It should be reasonably inexpensive so that it could be used for large lengths as, for example, railway or highway embankments.

PROPOSED REINFORCING METHOD

The first four 4-5.5m high full-scale test embankments having a slope between 0.2-0.5 in H to 1.0 in V with various facing types were constructed using a very problematic volcanic ash clay called Kanto loam, which had at the time of filling a high water content of 100-120%, a high degree of saturation of 83-90% and a low dry density of 0.55-0.65 g/cm^3 (7-9, 14). They were reinforced with one kind of spun-

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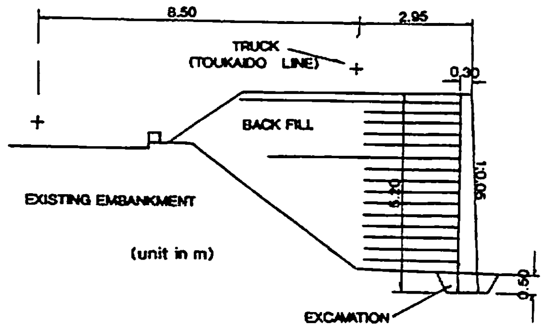
Fig. 1
Reconstruction of railway embankment at Amagasaki City (the height of wall is 5.2m)



-bond non-woven geotextile made of 100% polypropylene. Based on their long-term behavior together with laboratory model test results (10), the following method has been proposed (6, 10, 11).

(A) Planar geotextile sheets are used so as to reduce the required anchorage length by increasing the contact area with the backfill, compared to metal strips. While a stiff and high-strength grid is suitable for cohesionless soils, a composite of non-woven and woven geotextiles, among others, is suitable for cohesive soils to facilitate drainage and also to ensure high tensile rigidity and strength. The function of drainage is essential for the effective compaction of near-saturated cohesive soils and for both the high soil shear strength and high bond strength. The latter is achieved by maintaining suction (negative pore water pressure) in the backfill during heavy rainfall (3, 8) and by maintaining the backfill under drained conditions for load application. The importance of the drained condition was demonstrated by two drained triaxial compression tests performed on undisturbed Kanto loam taken from No. 2 embankment, either unreinforced or reinforced with the geotextile used for that embankment (Fig. 2a). The drained strength increased by reinforcing, while the initial rigidity decreased by large compression of the geotextile (Fig. 2a). On the other hand, in the undrained test (Fig. 2b, c), the maximum effective principal stress ratio increased by reinforcing as in the drained test, but the maximum deviator stress did not increase noticeably because of a larger positive excess pore water pressure developed due to large compression of the geotextile.

(B) Relatively short reinforcement members with a length of, say, 30% of the wall height are used so that it can be used for the reconstruction of embankment (Fig. 1). It has been confirmed that the reduction in the stability of the slope by using shorter reinforcements is compensated by using a planar geotextile (Item A) and using a continuous rigid facing (Item C) (7, 10, 11).

(C) A continuous rigid facing is placed directly on the wrapped wall surface for increasing the stability of wall and for reducing the lateral deformation and the settlement of the backfill, by enhancing the reinforced zone and the facing together to behave like a monolith. Its use also increases the resistance against mechanical damage, fire and the deterioration of geotextile when exposed to sun light.

The local and overall rigidity of facing can contribute to the stability of wall in various ways (7, 10), see Fig. 3. In fact, three walls of the test embankments made by wrapping around geotextile sheets (Type A in Fig. 3) deformed largely during natural heavy rainfalls and

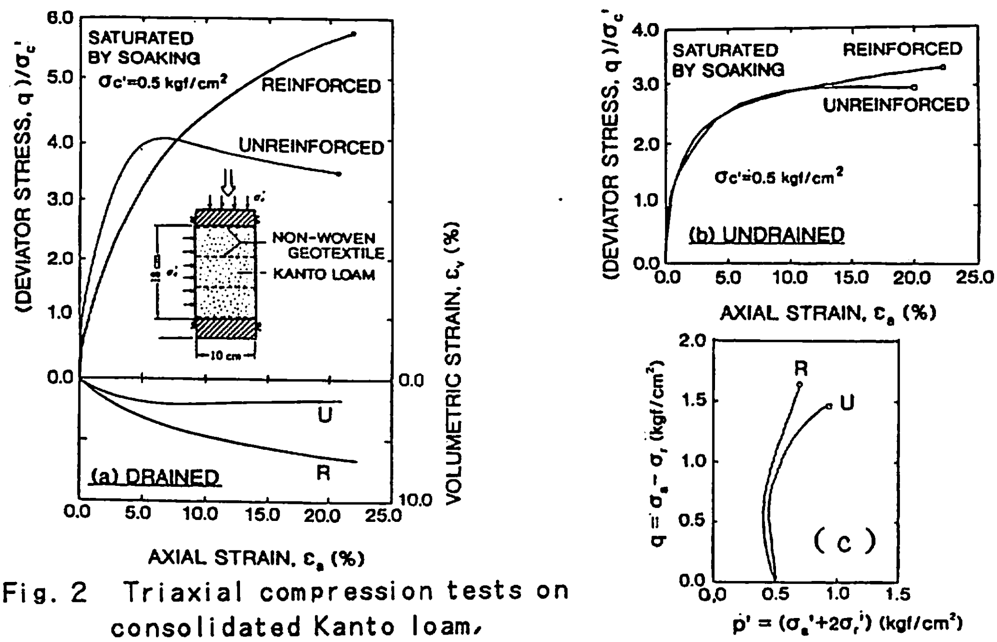
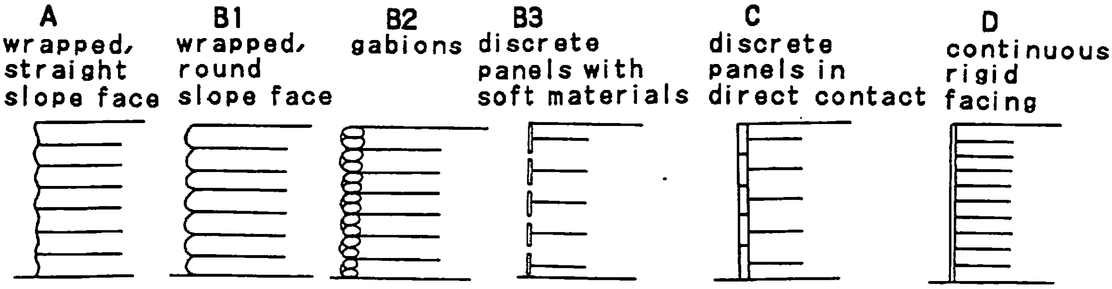
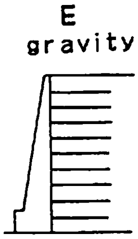


Fig. 2 Triaxial compression tests on consolidated Kanto loam, unreinforced or reinforced (performed by Yamauchi)

earthquakes. This was due to the local compressional failure in the soil layers behind the wall face, because of the lack of the confinement to the soil layers of the wrapping geotextile sheets. Other three walls was constructed with the aid of gabions (Type B2 in Fig. 3), and other two slopes with the aid of discrete concrete panels (Type C). On the wrapped surface of the other wall, an about 8cm-thick shotcrete layer was placed (Type D). Despite their steep slope of 0.2-0.3 in H to 1.0 in V, all these slopes with facings of Types B2, C and D were very stable against natural heavy rainfalls and earthquakes, due mainly to a high degree of compaction of the backfill near the slope face and the confinement by the facing. Further, in the laboratory, the model walls were loaded with a footing (see Fig. 14a). They were more stable in the order of facing types D, C, B and A. With Type D, the failure surface always passed through the toe of wall, while for the other facing types, particularly for Type A, it passed through the intermediate height of the slope face. Thamm et al. (13) also showed that for a 3.6 m-high wrapped reinforced wall (Type B1) with a near vertical wall face, a local failure occurred by concentrated loading on the crest only in the upper part of wall.

(D) A stage construction method (Fig. 4) is employed. First the wall is constructed with the aid of gabions, wrapped around them with geotextile sheets. After major part of the compression of the backfill and/or the supporting ground has occurred, a continuous rigid facing is placed by either of the following methods or another: (a) A lightly steel-reinforced concrete slab is placed directly on the wrapped slope face so that the slab does not separate from the slope face. (b) A lightly steel-reinforced precast concrete plate is erected, with leaving some space between its back face and the slope face. Or discrete modules (e.g., concrete blocks) are piled up and later tightened up by penetrating reinforcement bars through them. The space is sub-





FACING TYPE	A	B	C	D	E
LOCAL RIGIDITY	×	○	○	○	○
OVERALL VERTICAL RIGIDITY	×	×	○	○	○
OVERALL SHEAR RIGIDITY	×	×	○	○	○
OVERALL BENDING RIGIDITY	×	×	×	○	○
RESISTANCE BY WALL WEIGHT	×	×	×	×	○

○; with this function
×; without this function

Fig. 3 Various types of facing

sequently filled with fresh concrete. In this method, the final height of the wall is controlled by the stable height of the wrapped wall without a continuous rigid facing.

Several other methods were proposed to alleviate problems associated with the use of a continuous rigid facing (Fig. 5):
(a) In Fixed type (Fig. 5a), when the backfill is compacted and filled up with reinforcements connected to the back face of facing, relative settlement may occur between the facing and the backfill due to the compression of the backfill and/or the supporting ground and this may damage the connection between the reinforcements and the facing. Therefore, sometimes heavy compaction immediately behind the facing is avoided, but it would not be a final solution and also on the expense of reducing the stability of wall. On the other hand, in Separated type (Fig. 5b) (1), the facing is not in contact with the wrapped wall. In Sliding type (Fig. 5c) (4), the reinforcements are permitted to slide relative to the facing by means of slideable attachments. In Compressional layer type (2, 5, 15), the reinforcements are not connected to the facing and a compressive layer is placed on the back face of facing. The stage construction method is free from this problem. Note also that in this method, if large relative settlement occurs after the wall is completed, the gabions are expected to elimi-

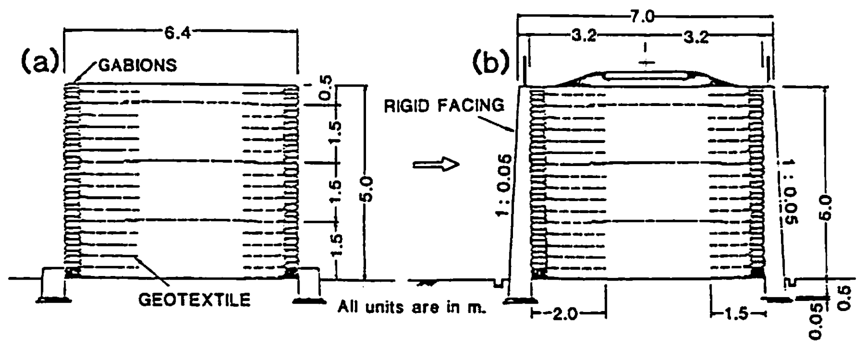


Fig. 4 Stage construction method used for JR No.1 embankment (sand)

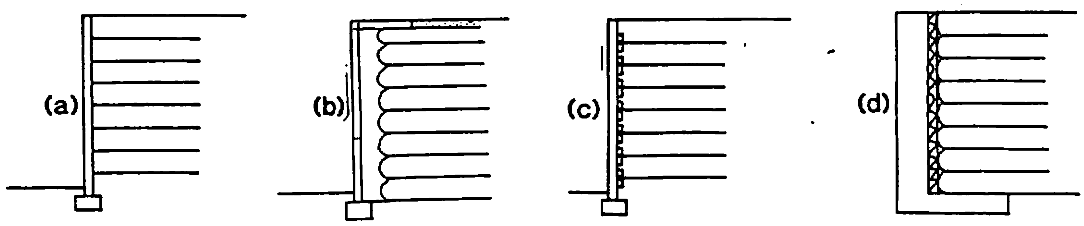


Fig. 5 Reinforcing methods using a continuous rigid facing
(a) Fixed type with a facing fully propped at filling,
(b) Separated type (1),
(c) Sliding type with a facing fully propped at filling (4), and
(d) Type using a compressional layer on the rear of facing fully propped or fixed at filling (2, 5, 15)

nate a large degree of stress concentration to the connection between the facing and the reinforcements.

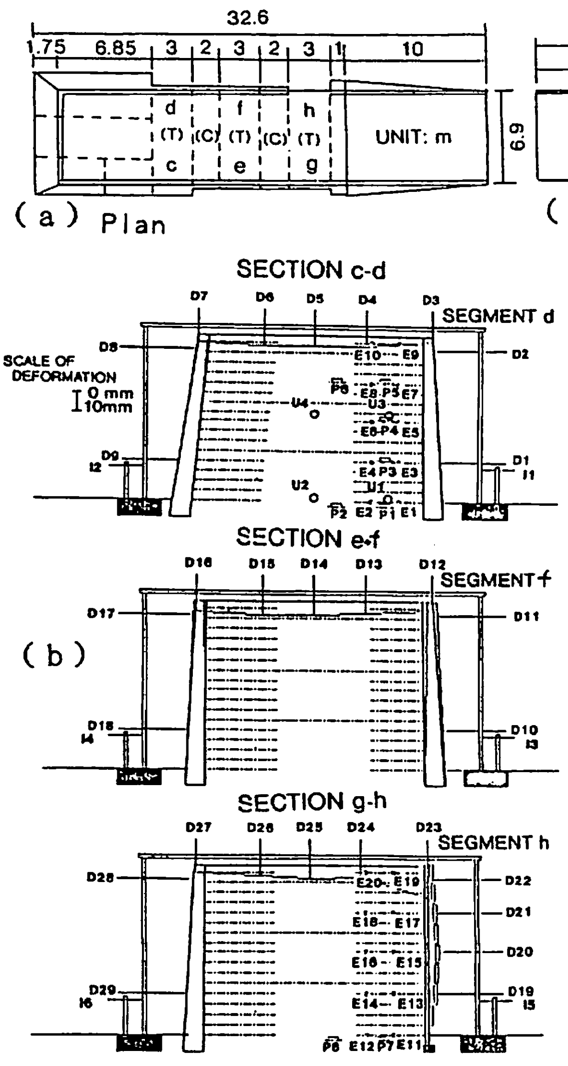
(b) By compacting heavily the backfill near the facing, the facing and the reinforced zone together can behave like a monolith, thus the wall becomes more stable and less deformable. However, if the facing is not well supported during filling the backfill, the compaction immediately behind the facing should be avoided so as not to induce excessive lateral deformation of the facing (15). In Separated type, heavy compaction can be achieved by using a temporary inflation system placed between the facing and the wrapped wall face (1). In the stage construction method, the soil near the crest of slope can be compacted heavily with a compaction machine.

(c) The development of large tensile stress in the reinforcements is essential for their efficient use. In Fixed and Sliding types, when the facing is well fixed during filling the backfill, only small tensile strain may be developed in the reinforcing members with large earth pressure activated on the back face of facing. Some amount of tensile strain is mobilized only by removing the support of the facing after the full height of wall is completed. Thus, when a relatively extensible geotextile is used, relatively large displacement of facing may occur, inducing some unpreferable large relative settlement between the backfill and the facing. In Separated type, a sufficient amount of tensile strain may be mobilized during the heavy compaction of backfill. In Compressional layer type, by collapsing the compressional layer during compacting the backfill, some amount of tensile strain may be mobilized in the reinforcing members with smaller earth pressure activated on the facing. In the stage construction method, sufficient amount of the tensile strain can be developed during filling the backfill and very small earth pressure would be activated on the facing placed subsequently.

(d) Fixed type facing and the facing made by the stage construction method have all the kinds of facing rigidity shown in Fig. 3, while the facings of Sliding type and Compressional layer type have only some of them. While it is on the safe side to ignore in the design their contribution to the stability of wall, at the same time it is less economical. Thus, this contribution is to be duly relied on.

TEST EMBANKMENTS JR NOS. 1 AND 2

JR (Japan Railway Co.) embankments Nos. 1 and 2 were constructed by the stage construction method from the end of 1987 and after the observation of their behavior by the end of 1989 (6), loading tests were carried out in 1990 to bring them to failure. JR No. 1 (Fig. 6) used a sand having a mean diameter of 0.2mm and a fines content of 16%. The reinforcement was a grid consisting of members made of polyester, covered with PVC to increase its durability, having a rectangular crosssection of 0.9mm x 3mm with an aperture of 20mm. The grid had a tensile rupture strength of 2.8 tonf/m and an initial tensile modulus of 1.0 tonf/m at an elongation of 5% at a strain rate of 5 %/min. JR No. 1 had six test segments; five had a continuous rigid facing of unreinforced concrete slab with two horizontal lightly reinforced construction joints in each, with some amount of the gravity resistance (Type E in Fig. 3). Only one had a facing of discrete panels (Type C in



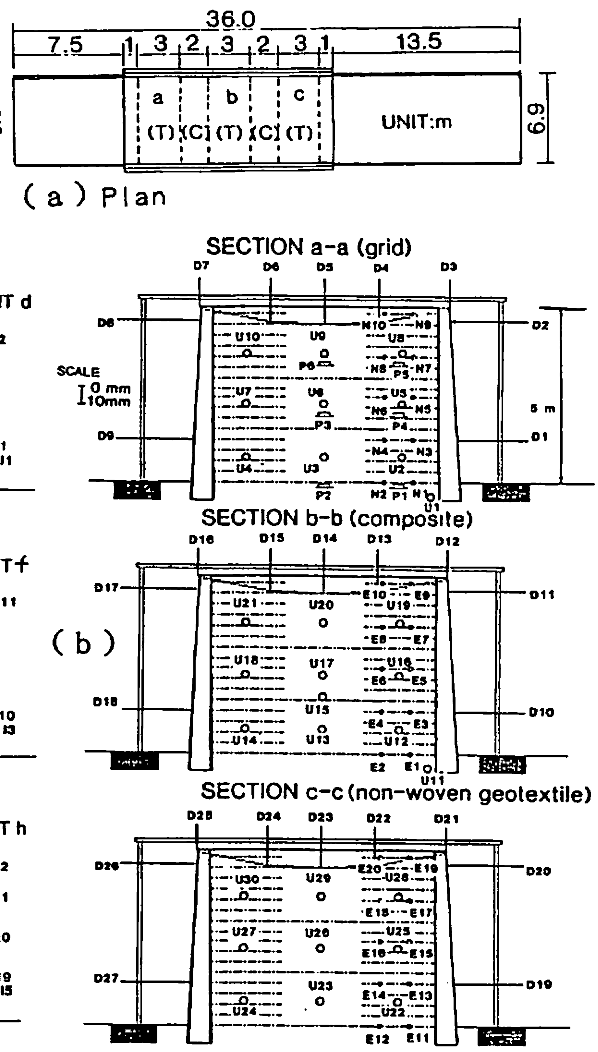


Fig. 6 JR No.1 embankment (sand) Fig. 7 JR No.2 embankment (clay)
(T: test section, and C: control section)

Fig. 3). Each panel had an area of 0.6m x 0.6m and a weight of 40 kgf.

JR No. 2 (Fig. 7) used Kanto loam with a water content of 120-130% and a degree of saturation of about 90% and the dry density of 0.55-0.60 g/cm^3. All the six test segments had a continuous rigid facing. Two test segments in Section a-a were reinforced with a grid as used for JR No. 1, sandwiched between gravel drainage layers. Other two in Section b-b used a composite of high tensile-rigidity woven geotextile sandwiched between non-woven geotextile sheets with a rupture strength of 1.8 tonf/m and a initial tensile modulus of 1.4 tonf/m at a 5% strain. The other two in Section c-c used a non-woven geotextile with a rupture strength of 0.7 tonf/m as used for the first four clay test embankments.

Figs. 6b and 7b show the cross-sections before and after deformation (until Feb. 1989) (note that the scales for embankment and deformation differ 60 times). All the test segments having a continuous rigid facing exhibited very good performance with a very small settlement of 1cm or less at the center of the crest over one and a half years after the construction (see also Fig. 8). Only Segment h of JR No.1 with a discrete-panel facing deformed relatively largely. Correspondingly, tensile strain in the reinforcements in Segment h was larger than that in Segment d (Fig. 8a). It was also found that while using the same kind of soil, Kanto loam, the deformation of JR No. 2 was much smaller than that of the walls with a facing of Type B2 of the fist four test embankments (7-9), primarily because of the additional use of a continuous rigid facing. The effect of facing type shown above is well in accordance with the laboratory model test results (10).

Fig. 9 shows the vertical distribution of hydraulic potential h (= the pore water pressure u - the gravity potential $\gamma_w \cdot z$) at the time when the antecedent precipitation index (API) was maximum. A larger API means a more wet condition (8). It was found that suction was maintained in the clay fill JR No. 2 with larger value in the reinforced zones, while almost no suction in the sand fill JR No. 1. A positive value of dh/dz means downwards percolation of pore water and vise versa. It can be deduced that in the reinforced zones, water percolated from both the crest and bottom of fill, which seems to indicate that the reinforced zones were always kept "dry". It is important that this clay test embankment JR No. 2 also has been very stable despite several times of natural heavy rainfalls.

Three sections of JR No.1 were loaded at their crests using a footing with a 2m x 3m base (Figs. 10 and 11). Each 3m-wide test section was separated from each other by a 2m-wide control section in between. A layer of two plywood sheets with a layer of grease in between had been placed between two sections to achieve plane strain conditions. The length of geotextile was 2m, except in Segment f (1.5m). The footing was located at a setback of 2m from the edge of the backfill of Segments d, f and h with the center of footing off the center of the crest by 15cm. Of the two segments in each section, the one which was considered weaker in advance failed (i.e., Segments d, f and h). The effect of the different length of geotextile between Segments d and f and that of the different facing types between Segments d and h may be seen in Fig. 11. Note that the horizontal displacement of the wall was largest at the top of wall for Segments d and f, but not for Segment h (see Fig. 10). Therefore, if the largest horizontal displacement along he wall height is plotted in Fig. 11, the difference between these segments becomes larger. Segments d and f yielded when a crack appeared in the upper construction joint (see Figs. 10 and 11). Therefore, it seems that if the bending rigidity of the construction joints

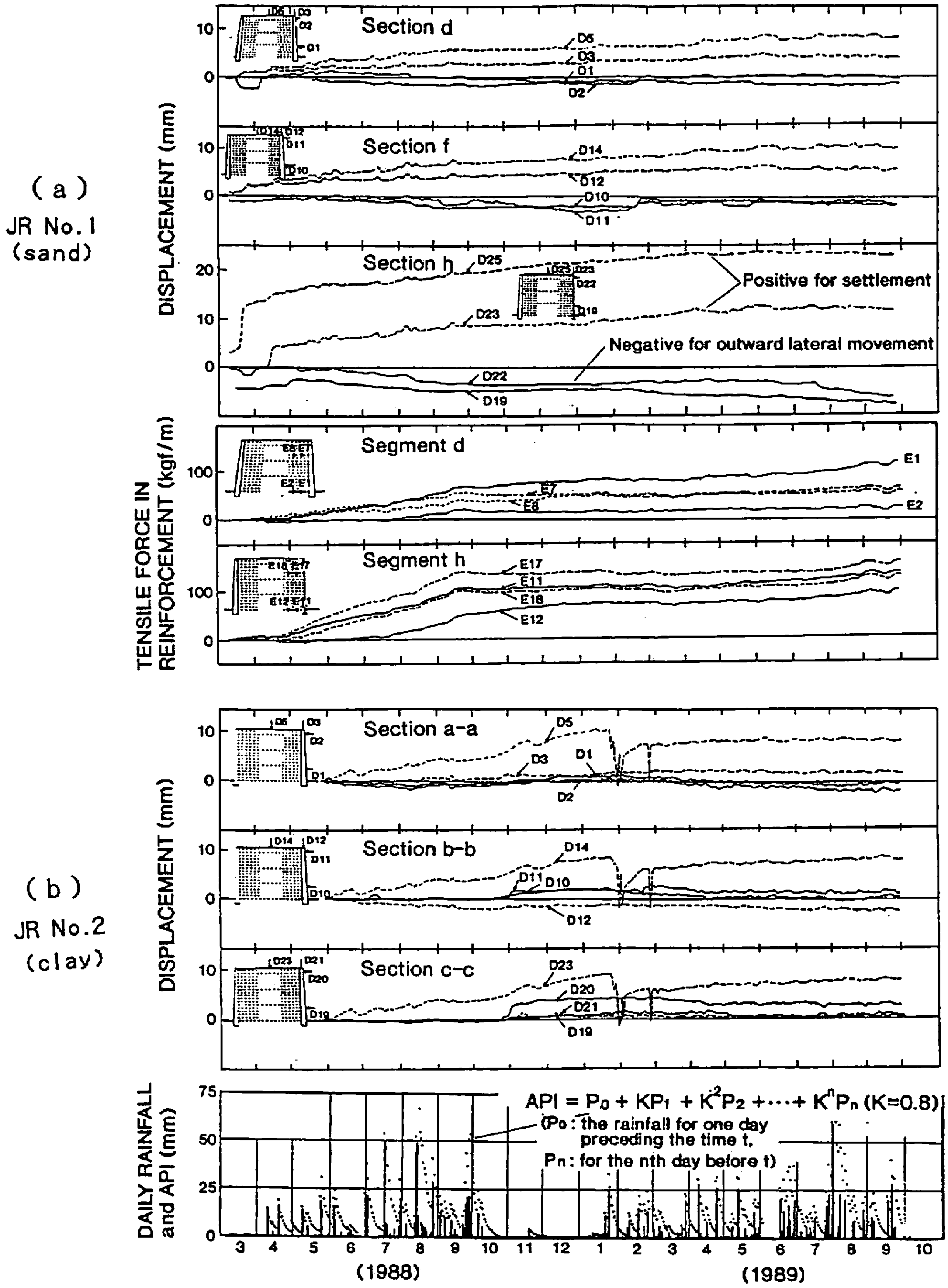


Fig. 8 Long-term behavior of (a) JR No.1 and (b) JR No.2

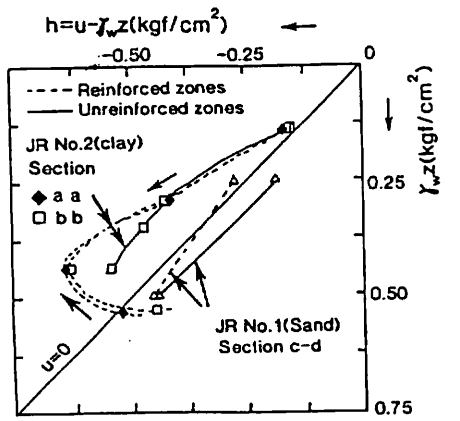
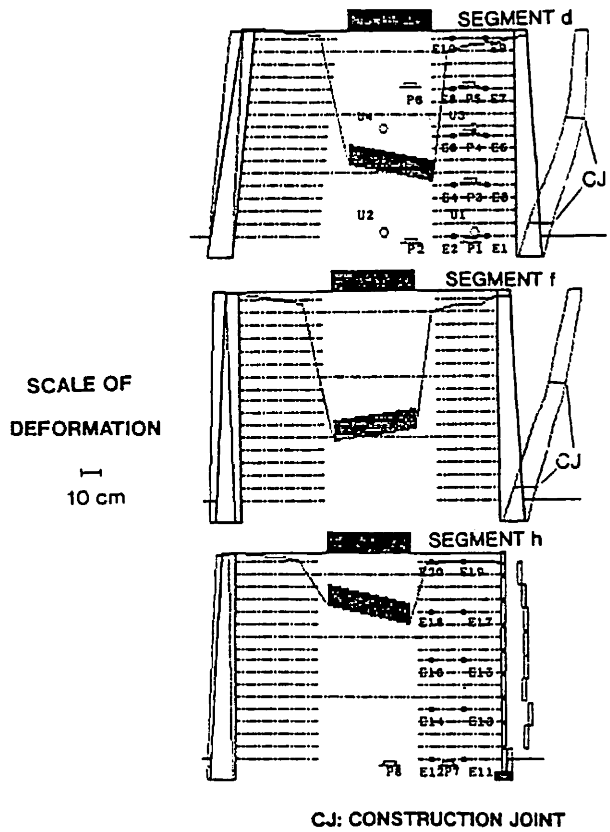


Fig. 9 Relationship between h and the γw·z



SEGMENT	d	f	h
SETTLEMENT OF FOOTING (cm)	69.4	68.3	29.8
FOOTING PRESSURE (tonf/m²)	57.5	50.0	37.5

Fig. 10 Deformation of JR No.1 by loading test

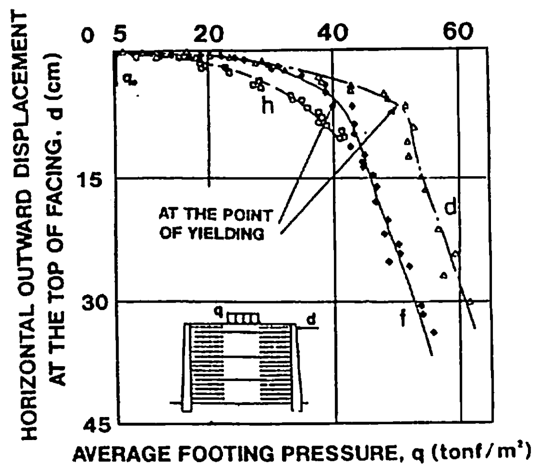


Fig. 11 Load-displacement relations Segments d, f and h of JR No. 1 by loading test
q_0: the pressure due to the weight of the loading apparatus (5tonf/m²)

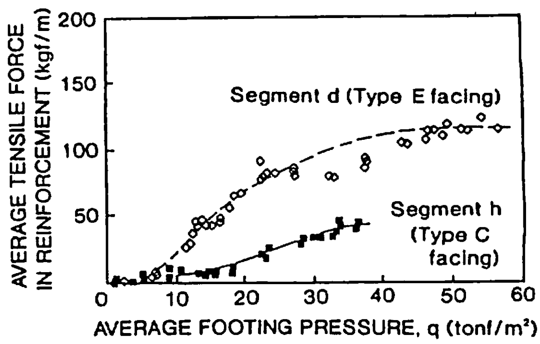


Fig. 12 Average tensile strain developed due to loading, JR No. 1

had been larger, the strength of the walls would have been larger. Indeed, this behavior shows the important role of the overall rigidity of facing for the stability of reinforced walls. The pattern of deformation of each facing seen in Fig. 10 was found very similar to that observed in the corresponding laboratory model test (10). The reinforcement functioned more effectively in Segment d having a Type E facing than in Segment h having a Type C facing (Fig. 12). This was due to that for such type of facing (Types D and E), larger earth pressure can be activated on the facing, as observed in the laboratory model tests reported in (10).

A simplified stability analysis by the two-wedge limit equilibrium method was performed for the laboratory model walls with a Type D facing (Fig. 14a) (10) and Segment d with a Type E facing of JR No. 1 by using the following assumptions (see Fig. 13). (a) Failure occurs by over-turning, since for the dimensions of this case, both experimentally and analytically, the safety factor was smaller for over-turning than for the failure by sliding out. (b) The safety factor is defined as SF= $\sum Ti \cdot Xi / \Delta M$. Ti is the tensile force in the reinforcement working at the height Xi on the failure surface abd. ΔM is the resisting moment additionally required to stabilize the corresponding unreinforced wall. To obtain ΔM, the friction angles δ_w and δ_f between the two wedges and on the back face of facing, respectively, are equal to the soil friction angle ϕ, while their actual values may be smaller than ϕ (note δ_f= 0 for the facing types A and B in Fig. 3). ΔM is also a function of either the location and magnitude of the reaction R on the failure surface ab (Fig. 14a), or the distribution pattern of earth pressure on the back face of facing (these two conditions are not independent of each other). This time, the latter was given (Fig. 13b and c). (c) The effect of reinforcing is reflected directly in the change of failure plane from the Coulomb failure plane as for unreinforced slopes to a deeper one, and also in the increase in the shear

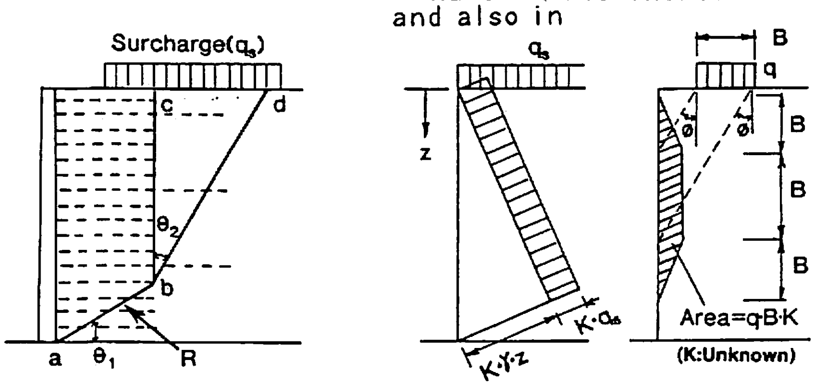
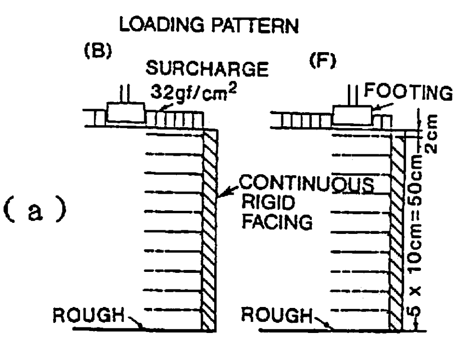


Fig. 13 Assumptions used in the limit equilibrium method



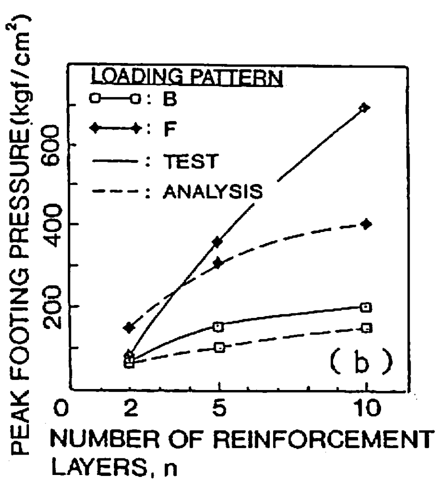


Fig. 14 Laboratory reinforced wall models of Toyoura sand (10):
(a) dimensions and loading method (the case of n= 10 shown)
(b) results of stability analysis

strength of soil due to the increase in the normal force caused by the reinforcement tensile force. (d) Ti is obtained by integrating the bond stress for the anchorage length: $\int 2\cdot\sigma_v\cdot\tan\phi\cdot dl$. σ_v is the normal stress acting on the plane of reinforcement (= the self weight of soil plus the uniform surface pressure q_o plus the pressure by the footing pressure distributed within the backfill at an angle of ϕ relative to the vertical). For the laboratory model tests, Ti was always less than its rupture strength, while for JR No.1, this was upper-bounded by the rupture strength (i.e., 2.8 tonf/m). (e) The failure plane ab always passes through the toe of wall, because of its sufficiently large overall rigidity of the facing types D and E. The smallest value safety factor SFmin was seeked by changing the location of the point b and the angles θ_1 and θ_2. The footing load for SFmin= 1 was then obtained. (f) ϕ of Toyoura sand for the laboratory model tests was obtained from the plane strain compression tests (12). The strength of sand of JR No. 1 was assumed as ϕ= 30° and 35°.

For the laboratory model tests, experimentally and analytically, the footing pressure which bring the wall to failure increased as the number of reinforcement layers, n, increased (Fig. 14b). In the case of n= 10, despite the use of short reinforcement, the wall without the footing pressure was very stable, and it failed only by at a very large footing pressure. The discrepancy of the measured and calculated values is due partly to the assumption concerning the pressure distribution pattern of the earth pressure on the back face of facing. Namely, the gravity center of the measured earth pressure was lower than the assumed ones (Figs. 13b and c). The maximum footing pressure computed for Segment d of JR No. 1 are 48 and 70 tonf/m² for ϕ= 30° and 35°, which are not very different from the measured value. A more refined analysis using measured values of ϕ is now underway.

namely R is closer to the toe of wall.

A series of shaking table tests of the scaled models (1/10 and 1/2) of Jr. No. 1 also were performed for the aseimic design. Further, JR No.2 clay embankment was loaded in late 1990. In July 1990, to support railway trucks, the reconstruction of 5-7m high existing embankments with a total length of about 2.5km has started at three sites by using the reinforcing method described in this paper.

CONCLUSIONS

A reinforcing method for walls has been described, which is characterized by the use of on-site soils, the use of relatively short geotextile sheets (having a drainage function for cohesive soils) and the stage construction of continuous rigid facing placed on the wrappped surface of the wall constructed with the aid of gabions.

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