Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tatsuoka & Leshchinsky (eds)
 ISBN 90 5410 358 2

Geosynthetic-reinforced soil retaining wall structures with short reinforcement and a rigid facing – Closure to the discussion by Mr Pierre Segrestin

F.Tatsuoka
Institute of Industrial Science, University of Tokyo, Japan

M.Tateyama & O.Murata
Railway Technical Research Institute, Tokyo, Japan

Y.Tamura
Technical Research Institute, Tokyu Construction Co., Ltd, Kanagawa, Japan

1 INTRODUCTION

Most of the questions raised in the discussion about the design concepts of the Geosynthetic-Reinforced Soil Retaining Wall structures with short reinforcement and a rigid facing (**the GRS-RW system**) have been fully answered in the keynote lecture and paper by Tatsuoka (1992), presented immediately after this symposium. The performance of GRS-RW system have been monitored using scientific means and has been reported by Murata et al. (1991) and Tatsuoka et al. (1992) previously, and Tateyama et al. (1993), Doi et al. (1993), Kanazawa et al. (1993) and Emura et al. (1993) in this volume.

Mr. Segrestin states in the opening of his discussion that "the papers and presentations by Tatsuoka and his colleagues openly attack the Reinforced Earth (or Terre Armee) technique." This statement is correct in a sense that any novel technique (the Terre Armee technique in this case) cannot remain best forever. That is, another technique having several advantages (the GRS-RW system in this case) over the Terre Armee technique has evolved. Certainly, the 'attack' is not personal nor is it commercial. It was presented in a factual framework, reporting the advantages of a new system.

In respond properly to the discussion, we will address some facts about the Terre Armee technique. The writers have refrained from doing so until now due to our very high admiration of this novel technique.

2. FACTS

2.1 Reinforced Earth (Terre Armee) in Japan

It appears that Mr. Segrestin does not like to look at the current practice in Japan. Rather, he may prefer to live in the memory land about the past situation in Japan enjoyed by the Terre Armee technique.

It is true that before the GRS-RW system was introduced in the Japanese market, the Terre Armee technique had dominated the construction market of important permanent reinforced-soil retaining wall structures. Indeed, many conventional geosynthetic-reinforced soil retaining walls had a wrapped-around wall face, and, therefore, they could not compete with Terre Armee walls; they are used mostly for temporary or less important permanent walls.

The past practice described in the above may have led to the claim by Mr. Segrestin that "the facts do not support the statement by Tatsuoka (1992a) that the GRS retaining walls are now replacing conventional Reinforced Earth retaining walls (i.e., Terre Armee walls) for the construction of permanent railway structures in Japan." Let us reiterate one more time that the statement is perfectly correct, although ten years ago, such a statement could only be a wish. At present (in 1993), we can even say that the use of the Terre Armee technique has been completely stopped in the construction of railway wall structures in Japan. However, the GRS-RW system is now widely used in place of Terre Armee technique and other conventional wall systems, and is scheduled to be used for many further projects. The total length of GRS-RW walls, typically having a height of 5 m, constructed using the GRS-RW system has reached 8 km by the end of April 1993, and will become 11 km by the end of 1994. Note that this length of railway retaining walls constructed for the last five years from 1988 is already about one third of the length of Terre Armee walls constructed for railways for the last twenty years. Namely, for railway structures, the current construction rate per year of GRS retaining walls is much higher than the average rate in the past of Terre Armee walls. This is partly because GRS retaining walls could be constructed in locations where the Terre Armee technique could not be used, e.g., bridge abutments or walls supporting large lateral load on its top or on the crest immediately behind the wall face.

One may realize how this change in practice has impacted the soil reinforcement market in Japan, especially when considering the near monopoly Terre Armee had in Japan. Considering only major civil engineering structures, the Terre Armee technique was first introduced to the Japanese market about twenty five years ago. The Japan National Railway, JNR (presently a group of Japan Railway Companies, JR Group) was the first nation-wide organization to use, about twenty years ago, the Terre Armee technique for several major projects. A design standard for Terre Armee was approved by JNR in 1978. Subsequently, Terre Armee was used by the Japan Highway Authority, and finally a design and construction manual was approved in 1982 by the Ministry of Construction.

The evolution of the GRS-RW system is now tracking a similar path. The GRS-RW system was developed jointly by a group of researchers from the University of Tokyo, Railway Technical Railway Research Institute of JR Group and Technical Research Institute of Tokyu Construction Co., Ltd. The draft of the design standard for the GRS-RW system for railway wall structures was prepared in 1989 and the standard was approved in 1992 by the Ministry of Transport. It is to be noted that in this standard, the design and construction methods of both GRS-RW system and Terre Armee wall system are specified parallel to each other so that each designer can select either system (or another technique). The fact is, though, that the Terre Armee technique has not been specified for any major construction project of railway wall structure, whereas the GRS-RW system has always been selected. The first GRS-RW project was in 1988, followed by two large scale projects in Nagoya in 1989 and at Amagasaki in 1990 (Tateyama et al., 1993, Kanazawa et al., 1993).

In 1992, the authors received an award for the development of this technique from Japanese Society of Soil Mechanics and Foundation Engineering. In March 1993, as the first private railway company, Seibu Railway Company successfully constructed an approach fill using the GRS-RW system in the northwest part of Tokyo City area. This approach fill, which will be connected to a high-rise RC structure for a rapid transit railway, has the maximum wall height of 6 m, a total length of 120 m and a width of 9 m (thus the total GRS retaining wall length of about 250 m), including three bridge abutments. Moreover, in 1993, the Japan Highway Authority has decided to adopt the GRS-RW system for widening the crest of part of the highway embankment between Kyoto and Osaka so as to increase the number of lane from four to six.

2.2 Length of reinforcements

Allowing for relatively short reinforcement is one of the advantages of the GRS-RW system over the Terre Armee technique. This is especially the case when steepening a gentle slope to a near-vertical wall without expanding outwards the construction site as well as significantly limiting the excavation into the slope. It is claimed in the discussion that the length of geosynthetic reinforcement used for the GRS-RW system is not significantly shorter than that for the metal strip reinforcement used in Terre Armee walls. However, the facts do not support this claim.

(1) Practice: For the small model walls in the laboratory and full-scale ones in the field, both constructed by using the GRS-

-RW system (Tatsuoka et al., 1989, Murata et al., 1991, Tatsuoka et al., 1992), the length of geosynthetic reinforcement was as short as 0.3 or 0.4 times the wall height (H). In the loading tests with a strip footing on the crest, in which the walls were loaded to failure, the actual failure load exceeded, by far, the design failure load. Based on the results of these experimental works combined with some theoretical analyses, the minimum allowable length of geosynthetic reinforcement was specified in the design standard for railway soil structures as the larger value of 0.35·H and 1.5 m. Indeed, this minimum allowable length is significantly smaller than what has been specified for Terre Armee walls.

There is no doubt that a GRS retaining wall becomes more stable as the reinforcement length increases. Therefore, considering the relatively short history of the GRS-RW system, designers have specified longer reinforcement when it did not lead to a deep excavation into an existing slope. Further, when the wall is to carry load from another structure (e.g., an electric pole or a bridge girder) the required length of reinforcement increases, exceeding the minimum allowable length of 0.35·H or 1.5 m. In fact the average reinforcement length for the GRS retaining walls constructed as bridge abutments is typically 1.0·H. Consequently, for the actually constructed GRS retaining walls, the reinforcement length has been generally longer than the minimum allowable length.

In addition, the reinforcement length relative to H tends to become larger as the wall becomes shorter; e.g., when H= 3 m, the minimum length length becomes 1.5 m, which is equal to 0.5·H. Excluding bridge abutments, the average length is approximately 0.5·H for the walls constructed so far; for cases where H= 5 m, the length is generally around 0.4·H with the smallest value so far being 0.35·H.

The first and revised versions of the design and construction manual for Terre Armee walls were published in 1982 and 1989, respectively, by Civil Engineering Research Center supervised by the Ministry of Construction (herein, they will be called **the CERC-manual**). According to the CERC-manual, in comparison with the GRS-RW system, the minimum allowable length of strip for Terre Armee walls is the larger value of 0.4·H (H= wall height) and 4 m for a range of 0.3·H from the wall base, while the length should be longer at higher levels. Although no major wall has been constructed in Japan, when used as a bridge abutment, the minimum allowable length is the larger value of 0.7·H and 7 m uniformly for the entire wall height. In the case of H= 5 m, which is the most typical height for highway embankments in Japan, the minimum allowable length of 4 m means 0.8·H, which is certainly much larger than the typical length used for GRS retaining walls under similar conditions. Only for exceptional cases of Terre Armee walls, under the condition that particularly good mechanical compaction of a selected high-quality backfill is performed, the CERC-manual allows a minimum length of 2.5 m. However, the rationale for this reduction in the minimum allowable length of reinforcement is not clear to us, since the above-mentioned specification is not an exception, but rather it is required for all ordinary Terre Armee walls.

In the design standard of Terre Armee walls for railway soil structures specified by JNR in 1978, the minimum allowable length was 4 m without allowing for the reduction to 2.5 m. In both the design standard revised by JNR in 1983 and the aforementioned design standard approved by the Ministry of Transport in 1992, the specification of the minimum allowable length has become similar to the one for highway structures; i.e., the standard minimum allowable length of reinforcement is 4 m, while allowing the use of the larger value of 0.4·H and 2.5 m as the exceptional case on the condition that particularly good mechanical compaction of backfill is performed. Note that this specification is slightly stricter than for highway Terre Armee walls in that the strip length should be larger than 0.4·H in all cases.

The case presented on the left-hand side in Fig. 1 in the discussion, for which the strip length near the wall base is 0.4·H and equal to 2.5 m, is the above-mentioned special case. It may be noted, however, that this wall is for a railway track in a

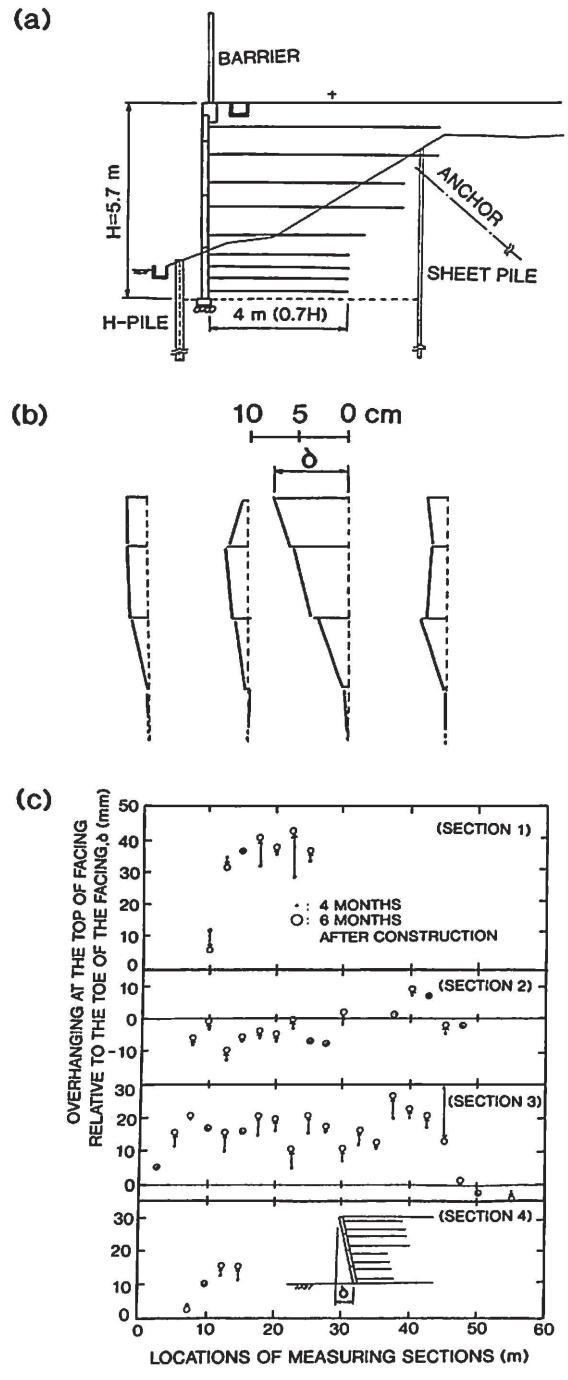


Fig. 7 Terre Armee Wall constructed in 1981 in Yamagata Prefecture; a) typical cross-section, b) several typical facing shapes measured in November 1991, c) overhanging δ at the top of facing in relation of the toe of wall measured four and six months after the construction of the wall.

yard (this information has been omitted from Fig. 1), a less demanding application. On the other hand, for the right-hand side wall, which supports a more important major railway track (this information was not given either in Fig. 1), the length is about 0.75·H even near the wall base, which was apparently achieved by excavating into the existing slope utilizing anchored sheet piles. Indeed, one must wonder why the rationale for the use of different reinforcement lengths for the two Terre Armee walls shown in Fig. 1 was not given in the discussion. Therefore, the left-hand side wall shown in Fig. 1 is a poor example to demonstrate that a strip length of 0.4·H is the value widely used for Terre Armee walls in Japan.

Further, our nation-wide survey of the Terre Armee railway walls to support major tracks shows that in all cases, the strips are longer than 0.4·H as typically shown in **Fig. 7a** (the figure number is sequential to that in the discussion). This wall was constructed in 1981 in Yamagata Prefecture with a strip length of 0.7·H even near the wall base, while excavating into the existing slope to a relatively large extent with the aid of anchored sheet piles.

<u>(2) Theory:</u> For a given design problem, the required total cross-sectional area of reinforcement per unit wall face is much larger for geosynthetics, because of its lower allowable strength (in terms of stress) than that of metal strips. This leads to the use of a planar geosynthetic rather than the use of geosynthetic strips. Because of the same reason, the number of reinforcement layers is usually larger for geosynthetics than for metal strips. In fact, a standard vertical spacing between reinforcement layers is 30 cm for the GRS retaining walls while it is 75 cm for Terre Armee walls. Consequently, because of both the reinforcement geometry (i.e., planar) and the smaller vertical spacing between reinforcement layers (i.e., smaller tensile force in each layer), the required anchorage length l_a (**Fig. 8**) for planar geosynthetic reinforcements becomes much smaller (i.e., exponentially smaller) than that for metal strips. The consequence of this smaller l_a value is a substantially smaller total length of reinforcement.

In **Figs. 8a** and **8b** (reproduced from Fig. 2 of the discussion), nearly the same anchorage length l_a is indicated for the Terre Armee wall and GRS retaining wall. However, based on the above discussion, this illustration is grossly misleading; a substantially smaller anchorage length l_a between lines B-B and C-C in Fig. 8b is much more representative for planar geosynthetic reinforcements. Typically, the required anchorage length for planar reinforcement is shorter by an order of magnitude (or even more) as compared to that for metal strips.

It was also claimed in the discussion that the horizontal distance at the crest between the facing and the maximum tension line, l_r, defining the width of the active zone, is larger by 0.3·H for 'extensible' geosynthetic reinforcements than for 'quasi-inextensible' metal strips, as illustrated in Fig. 8. Namely, according to the discussor, the maximum tension line is A-A for Terre Armee walls (Fig. 8a) and B-B for GRS retaining walls (Fig. 8b). This statement is not correct either.

First a maximum tension line in Fig. 8b (line B-B with a distance from the wall face of 0.6·H at the crest) has never been reported in the literature as an experimental fact even for geosynthetic-reinforced soil retaining walls having a wrapped-around face. For example, in a 12.6 m-high wrapped-around GRS retaining reported by Allen et al. (1992), the maximum tension line was located approximately along the Rankine failure line (i.e., the boundary of the so-called Coulomb wedge), for which the distance from the wall face was less than 0.3·H at the crest. The width of Coulomb wedge at the crest is equal to $H \cdot \tan(45° - \phi/2)$. Hence, a distance of 0.6·H means $\phi = 28°$. Although this very low value of ϕ may be used in design as a safe value, it is substantially lower than the actual value for well-compacted typical backfill soil under plane strain conditions. It is misleading to compare between the maximum tension line based on field observations in Terre Armee walls where real quality backfill used (i.e., Line A-A in Fig. 8a) and a failure line that is calculated using a Coulomb wedge together with a reduced ϕ (i.e., line B-B in Fig. 8b).

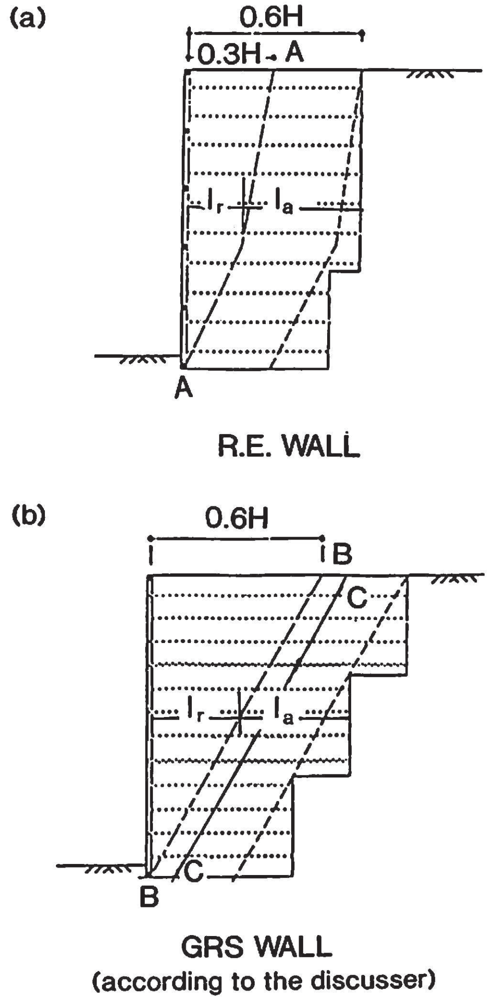


Fig. 8 Effect of reinforcement stiffness on the length of reinforcement according to the discussor (reproduced of Fig. 2 of the discussion)

In other words, line A-A is for an actual ϕ (typically 35 - 40°), whereas line B-B is a design $\phi_d = \tan^{-1}(\tan\phi/F_s)$ (i.e., a ϕ value that includes a factor of safety of 1.3 - 1.5). Of course, for a factored ϕ, a larger 'wedge' is predicted.

Secondly, provided that the backfill soil is compacted very well, the retaining length l_r is controlled primarily by the facing rigidity, unless the reinforcement stiffness is very low (as in the usual cases). This point has been discussed in detail by Tatsuoka (1992), which included

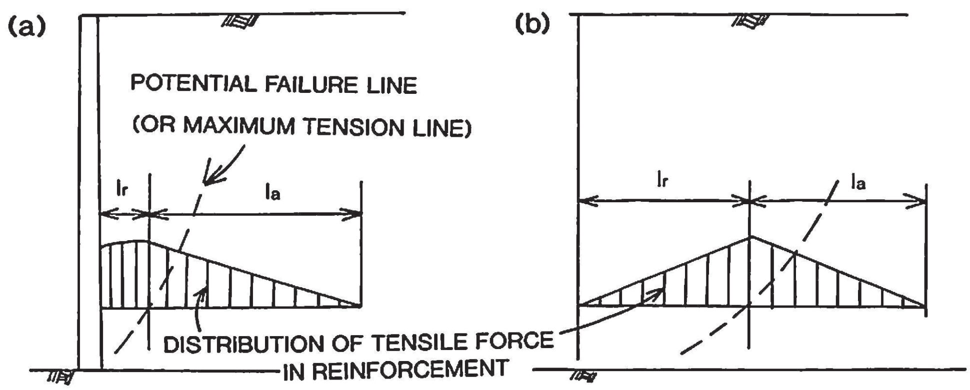


Fig. 9 Schematic diagram illustrating the effect of facing rigidity on the length of reinforcement; a) rigid facing, b) flexible facing

numerous laboratory and field data. At lower levels of a wall (**Fig. 9a**), when the facing rigidity is stiff enough to activate some earth pressure behind the facing, the length l_r can be very short to resist the earth pressure thrust exerted by the unreinforced zone. On the other hand, when the facing is too flexible to activate a sufficiently large earth pressure behind the facing, only a small tensile force develops in the reinforcement at the connection to the facing (**Fig. 9b**). Therefore, to balance with a large anchorage length l_a required to resist the earth pressure thrust, the length l_r should be long enough too; namely, in this case, the length l_r is least influenced by the reinforcement stiffness. It should also be noted that with a very flexible facing, the length l_r could become even larger for strip reinforcement than for planar reinforcement. This is due to a smaller total cross-sectional area of reinforcement. In this case, in addition to the danger of local compressional failure in soil near the wall face, the required total length of metal strip reinforcement will become much longer than what was shown in Fig. 2a (or Fig. 8a). Under the same conditions, the required total length could be much shorter for planar geosynthetic reinforcement.

It should be further noted that when a facing of flexible metal or discrete panels is used (e.g., those used in Terre Armee walls), the shear load cannot be transmitted through the facing. Therefore, each reinforcement layer should be long enough to extend beyond the potential failure plane (i.e., the Rankine failure line) at each and every level. However, when a full-height rigid facing is used so that the facing can transmit part of the thrusting earth pressure from higher to lower levels in the wall, the reinforcement layers at the higher levels can be as short as not extending beyond the potential failure line and without loosing the stability of wall (n.b., in view of the above, it is likely that if a full-height concrete facing is used for a Terre Armee wall, the length of reinforcement could be reduced to some extent). This is another factor in which the reinforcement length for the GRS-RW system could be shorter than that for Terre Armee walls. This point has been discussed by Tatsuoka (1992) in relation to Fig. 3.2.

2.3 Use of fine-grained soil backfills

The discussor claims that the same poor quality on-site soils which can be used for the GRS-RW system can be used equally for Terre Armee walls. Then the discussor points out that "only the systems using drainage (actually draining) composite reinforcements are exempted from the potential need for extra drain means."

The first point has been a subject of argument among many researchers and engineers for a long time, and thus we do not like to repeat it. The writers believe that when used in either a soil having a large amount of fines or a clayey soil, especially under near saturated conditions, metal strips cannot be better than some geosynthetics because of many inherent problems associated with metals, for example possible corrosion of metallic reinforcement (e.g., Blight and Dane, 1989), and the possible reduction in bond strength due to pore water pressure development at the interface between the soil and the reinforcement.

The second statement is perfectly correct. It seems that the discussor is not aware of the fact that full-scale tests on retaining walls have been successfully conducted using the GRS-RW system and a backfill of a nearly saturated volcanic ash clay (called Kanto loam) which had a natural water content of about 100 %. In one of the walls constructed at the site of Railway Technical Research Institute, the backfill soil was reinforced with a nonwoven-woven composite geosynthetic having dual functions of tensile-reinforcing and drainage (Murata et al., 1991, Tatsuoka et al., 1992). Indeed, the method suggested by the discussor has already been used for the GRS--RW system. It is hard to believe that metal strips can be placed in such a clay (or even in a 'better' clayey soil) without problems. Special treatment, such as coating the metal strips with epoxy, makes the economics of such walls less attractive.

It should be noted that the GRS-RW system does not limit the reinforcement type to grids, but it may use such a composite as described above for poor quality soils. It is in particular the case when the backfill drainage is needed for better compaction of the backfill and for keeping the backfill under 'dry' conditions during heavy rainfall while maintaining a high negative pore pressure (Tatsuoka and Yamauchi, 1986, Murata et al., 1991). Indeed, good compaction is one of the crucial factors when a low quality soil is used for the backfill; in other words, a low-quality soil is defined because of the difficulty of attaining good compaction. Therefore, apart from the problem of the corrosion of metallic reinforcement, a near-saturated fine-grained soil could become a good backfill soil when compacted to a sufficiently dense state. For good compaction of fine-grained backfill with high-water content, both drainage from the interior of soil and restraint from lateral spreading when vertically compressed are essential. For these purposes, placing layers of stiff composite geosynthetic sheets having a drainage function with a relatively small vertical spacing (i.e., typically 30 cm) is substantially more effective than placing layers of metallic strip reinforcement with a relatively large vertical and horizontal spacing (i.e., typically 75 cm for Terre Armee walls). In fact, the construction of railway GRS retaining walls with an average height of 2~3 m and a total length of 2,000 m using a backfill soil of high-water content clayey soil (with a fines content of about 75 %), reinforced with a composite geosynthetic reinforcement, is now being scheduled for construction.

The writers are aware of some cases in Japan in which a clayey soil was used for the backfill of Terre Armee walls. In these cases, each metal strip layer was surrounded by a thin layer of cohesionless soil, while each clayey layer was sandwiched between two vertically adjacent cohesionless soil layers. This construction method is not simple and needs a very strict quality control. The writers believe that today there is no reason to use metal strips when a poor quality on-site soil is to be used for the backfill soil.

2.4 Bridge abutments

It is claimed in the discussion that a 7.5 m high test Terre Armee wall abutment constructed in France is 'stronger' than the 5 m high test GRS retaining wall with a sand backfill reported by Tatsuoka et al. (1992). It is indeed meaningless and misleading to compare the ultimate strengths of different wall structures designed under different conditions and for different purposes.

We were able to construct a more stable test GRS retaining wall than the Terre Armee wall described by the discussor by increasing the length, strength and the number of layers of reinforcement. However, the main purposes of these loading tests were (1) to ensure that even a GRS retaining wall having a very short reinforcement (30 % and 40 % of the wall height) can be stable enough against the design load, and (2) to confirm the positive effects of facing rigidity. The test results clearly showed that the wall segment having a continuous rigid facing was much stronger than the other wall segment having a discrete panels facing under otherwise the same conditions (Murata et al., 1991, Tatsuoka et al., 1992).

The discussor recognized the fact that a bridge abutment of Terre Armee wall which supports directly the load from a bridge girder has not been constructed in Japan except for two small ones. This fact was also reported by a supervising engineer of Terra Armee walls in Japan (Hotta, 1991). In a recent case reported by Hotta (1991), the load from a bridge girder is supported by two independent conventional piers at both ends, each pier being supported by a pile foundation constructed immediately in front of the Terre Armee wall so that the bridge load is **not** transmitted to the Terre Armee wall. Although this wall was called "a composite-type Terre Armee bridge abutment", it is obvious that this does not really belong to the category of bridge abutment discussed herein.

According to the discussor, the reasons why Terre Armee bridge abutments have not been used by the Japanese railway and highway authorities are; (1) "unwillingness to see bridges directly supported anything else than stiff columns and piles," and (2) "concerns with respect to the effects of earthquakes", and these were nothing to do with the facing rigidity problem. Conversely, the writers of this closure, including the members of one of Japan Railway companies, link these two reasons directly to the lack of overall facing rigidity of both flexible metal facing and discrete concrete panels facing used for Terre Armee walls. As discussed in detail by Tatsuoka et al. (1992), Tatsuoka (1992) and Jewell (1992), for concentrated vertical and outward horizontal load applied on the top of facing or on the crest of backfill immediately behind the facing, the wall becomes less stable as the facing rigidity decreases.

Reflecting on this fact, in the current CERC manual (1989), concentrated outward lateral load P_h should be resisted only by part of the reinforcement layers arranged for a small range $y_o = 2(B' + C)$ from the crest (**Fig. 10a**). Note that such concentrated load as described above becomes larger under seismic conditions. **Fig. 10(b)** illustrates a similar design method. Indeed, many Japanese engineers are very much reluctant to rely on the resistance against such concentrated load only by several reinforcement layers located near the crest in Terre Armee walls (Figs. 10a and 10b).

Conversely, a full-height rigid facing used by the GRS-RW system helps resisting such concentrated loads by distributing the load over the full wall height and the supporting ground (**Fig. 10c**). Hence, this load is carried by the backfill and all the geosynthetic layers. Consequently, more effective use of the soil-reinforcement system is achieved with far less load concentrated in reinforcement layers. Based on the results of the loading tests of full-scale GRS retaining walls reported by Tateyama et al. (1993a) and Tamura et al. (1993) in this volume, it is specified in the design standard of the GRS-RW system that the outward lateral load P_h is distributed to the full height of wall (Fig. 10c).

It is further stated in the discussion that "obviously, using even more flexible, extensive or prone to creep geosynthetic reinforcements cannot allay (or reduce) these reservations (to use GRS-RW bridge abutments)." Reality has proven that this statement is totally wrong. Thirteen railway GRS-RW bridge abutments, with a wall height between 2 m and 6 m, have already been constructed to support railway bridge girders, and the performance of all of them is very satisfactory. The potential problem of creep deformation of geosynthetic reinforcement is also exaggerated in the discussion. In fact, to the best knowledge of the writers, no geosynthetic-reinforced soil retaining walls including those constructed by the GRS-RW system had a serious problem due to large creep deformations of geosynthetic. Simple creep tests ignore the interaction between soil and geosynthetic layers and the subsequent relaxation. Consequently, these tests tend to imply creep that is much more severe than reality is. Further, Wu and Helwany (1993) correctly pointed out that it may be incorrect to ignore the creep deformation of soil which may, for some soils, deform faster than the geosynthetic reinforcement.

3 WHAT ARE DRAWBACKS OF THE GRS-RW SYSTEM ?

3.1 Sliding at the wall base

In the discussion, it is suggested that a mechanically stabilized earth structure (e.g., the reinforced zone in a GRS retaining wall) having a width of 0.3 times the wall height H is very close to sliding at its base when the friction angle ϕ at the base is equal to 26° (Fig. 3 of the discussion). This suggestion is based on a somewhat un-realistic assumption that there exists no friction at the vertical boundary between the reinforced zone and the unreinforced backfill soil. In fact, with ϕ = 26° along the vertical boundary (i.e., interwedge), the critical length for sliding at the base becomes $0.23 \cdot H$ under the same conditions as shown in Fig. 3. The minimum allowable reinforcement length of $0.35 \cdot H$ in current specification is made recognizing that this type of sliding does not occur when a 20 tons compaction plant is operating at the crest of the wall without a rigid facing.

For completed GRS retaining walls, the weight of the rigid facing helps resisting the sliding along its base; for the typical arrangements shown in **Fig. 11**, while ignoring the soil-wall friction and the passive earth pressure at the front face of the embedded part of facing and its leveling pad, the critical reinforcement length L_c for sliding failure becomes $0.15 \cdot H$ for H= 5 m (i.e., the facing base plays a role in resisting slide). Also, in the actual applications of the GRS-RW system, for reasons described in Section 2.2, the reinforcement length of about $0.4 \cdot H$ has been used for a wall height of about 5 m. Moreover, in the design of GRS retaining walls, the writers and designers in charge check all possible

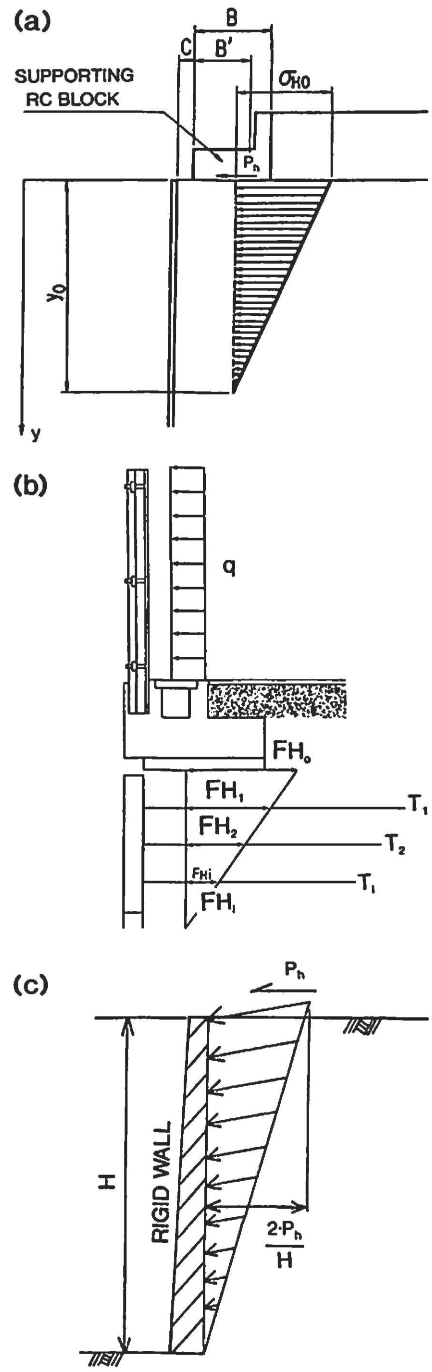


Fig. 10 Design methods for concentrated outward lateral load on the crest of wall immediately behind the wall face; a) Terre Armee wall bridge abutment (reproduced from CERC manual (1989), b) Terre Armee wall having a tall noise barrier wall, and c) GRS retaining wall with a rigid facing.

failure modes, including sliding at the base while ignoring the passive resistance in front. If the reinforcement length is not enough to prevent sliding failure at the wall base, the writers would increase as necessary the reinforcement length so as to have an ample safety factor (= 2.0 under live loading state, and 1.25 under earthquake loading state). Therefore, the writers do not see any possibility of direct slide along the wall base as illustrated in Fig. 3 for the GRS-RW system.

3.2 Proper use of both reinforcement and rigid facing

The discussor considers that it is more advantageous to "simply provide at each and every level the necessary ad-hoc reinforcements" without using a full height rigid facing. As discussed in relation to Fig. 10, it is obvious that reinforcement arrangement without using a rigid facing (as mentioned above) is not effective in resisting concentrated load on the top of facing and on the crest immediately behind the wall face. The use of a full-height rigid facing is much more effectively in stabilizing the wall than the use of a discrete panels facing, as pointed out by Jewell (1992).

The discussor claims repeatedly that the

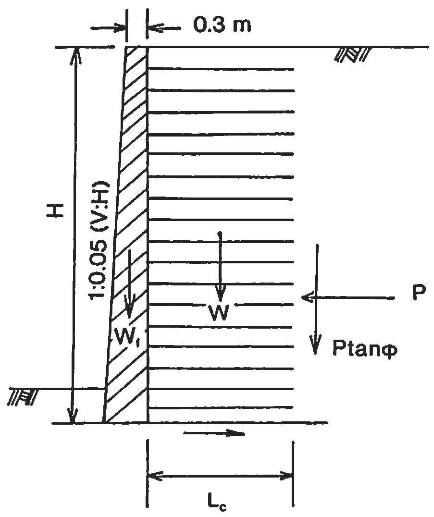


Fig. 11 Sliding along the base of GRS retaining wall

use of rigid facing is 'expensive', therefore its use is not advantageous. This type of discussion should be made from the view point of the cost/performance ratio (i.e., the cost effectiveness). In this regard, Terre Armee company has developed a **soft** facing element made of steel mesh called Terratrel, the top and bottom ends of which are connected to conventional metal strip reinforcements (Smith, 1991). It has been claimed that Terratrel facing is cheaper than the conventional discrete panels facing (Smith, 1991). Despite the above, it does not appear to the writers that only because of economics, this soft facing Terratrel is going to replace the conventional discrete panels facing for permanent important Terre Armee walls. Not only the discrete panels facing is more convenient for construction, more aesthetically pleasing and more durable, but it also contributes largely to the stability of Terre Armee walls. As discussed in detail by Tatsuoka (1992), when compared with both the conventional flexible metal facing and the new soft facing Terratrel, the discrete concrete panels facing has a much better degree of local rigidity and some degree of overall rigidity, both being very important for the wall stability. The writers wonder why the contribution of facing rigidity to the stability of Terre Armee wall has not been clearly addressed by Terre Armee Company. It is misleading and even unsafe to advocate that the structural role of the facing for Terre Armee is only for preventing the spilling out of backfill soil immediately behind the facing. The writers consider that the foremost advantage of Terre Armee walls using a discrete panel facing over wrapped-around GRS retaining walls exists in the substantially larger facing rigidity, not in the use of stiff metal strip reinforcement.

After all, the construction cost of facing per wall area for discrete panels of Terre Armee wall is about 50 % higher than for the cast-in-place concrete facings for the GRS-RW system in Japan. When the construction cost of placing layers of gabions behind the rigid facing is included for the GRS-RW system, the cost of facing becomes very similar for the two wall systems. Then, when a concrete facing, whether discrete panels or full-height one, is to be used, why not take advantage of the overall facing rigidity so as to increase the stability of the wall while decreasing its deformation.

3.3 Resistance at the base of full-height rigid facing for the GRS-RW system

Fig. 5 shows the stress concentration at the base of a continuous rigid facing of a small scale GRS retaining wall model. This stress is at the state of failure caused by the loading of a strip footing on the crest immediately behind the wall face. This stress is very high. The discussor claims that this test result demonstrates that the GRS-RW system requires very high bearing capacity of the supporting ground beneath the rigid facing. The writers should repeat that this is a model test performed to examine the strength of GRS retaining wall with a full height rigid facing. Indeed, the bearing capacity in terms of concentrated load applied at the crest of this model wall is about 35 times higher than the design load obtained by scaling the design load for an equivalent proto-type 5 m-high bridge abutment supporting a railway bridge girder with a span length of 9.5 m, as constructed in Nagoya (Tatsuoka et al., 1992, Tateyama et al., 1993 in this volume). In full scale GRS retaining walls under working load conditions, the writers do not expect such high stress as observed in the model GRS retaining walls at the state of failure.

The writers count on the bearing capacity of the foundation soil to some degree if it is available. Then, the discussor suggests that "simply opening a trench at the toe of the (GRS retaining) wall might have disastrous consequences." It should be reminded that the full scale GRS retaining walls are maintained similarly to conventional permanent gravity-type RC retaining wall structures. This means that a trench opening or similar excavation at the toe of the wall, which may decrease the bearing capacity, is not permitted, although if it is done locally, it will not actually endanger the wall. Furthermore, since the rigid facing is continuous not only along the wall height, but also in the longitudinal direction, thus, even if a trench is excavated at the toe of the wall for a limited length, it will hardly endanger the facing and wall.

Conversely, according to a supervising engineer of Terre Armee walls in Japan (Hatta, 1991), 73 walls out of a total of 3,913 Terre Armee walls, which had been constructed in Japan by May 1991, exhibited unsatisfactory behaviour; namely as high as nearly 2 % of all of the Terre Armee walls had problems ! Among them, ten walls had problems due to excessive settlement (seemingly unequal settlement) of the foundation. This fact indicates that the settlement which may occur due to the opening of a trench at the toe of Terre Armee wall may unstabilize the facing, or at least may induce mis-alignment of discrete panels. The writers wonder whether excavating a trench at the toe of a Terre Armee wall is allowed (the writers think it should not be allowed !). In fact, the CERC-manual requires a minimum depth of the foundation to be 40 cm so as to support the axial force from the facing. This provision is made considering also possible scouring in front of the foundation (page 143). It appears therefore that this statement "simply opening a trench at the toe of the wall might have disastrous consequences" is more pertinent to Terre Armee walls.

In this connection, it is stated in the CERC manual that "since the role of facing is only to retain the soil grains located immediately behind the facing, the deformation of facing will not seriously endanger the wall unless the deformation of facing and the damage to it are not very serious" (pages 131-132). Indeed, this is a typical example of mis-understanding the structural role of facing.

For the GRS-RW system, when the ground condition is very poor, some ground improvement is made below the facing and the base of the reinforced zone adjacent to the facing (e.g., cement-treated soil columns; see Fig. 3 of Doi et al., 1993 in this volume). It should be reminded, however, that since a rigid facing is cast-in-place after the major deformation and settlement of the backfill and supporting ground has occurred, large stress concentration on the facing is avoided.

On the other hand, for Terre Armee walls, the facing is erected simultaneously with the construction of backfill. Therefore, if the compression of the untreated supporting poor ground due to the weight of wall is too large, it may result in relative settlement between the facing and the backfill which cannot be absorbed by the compression of the compressible material placed between vertically adjacent panels. In fact, many Terre Armee walls had problems when constructed on poor ground as has been reported by Hatta (1992): among total 73 cases of Terre Armee walls with problems, 21 cases were due to local failure, sliding failure and settlement of untreated supporting poor ground. Furthermore, the relative movement between the facing and the backfill may result in over-stressing of the reinforcement at its connection to the facing. This stress concentration might be quite large; however, since it is a function of relative movement, it is difficult to predict its magnitude and therefore, assume a safe design value. It is therefore dangerous to advocate that Terre Armee walls can withstand large deformation of the supporting poor ground.

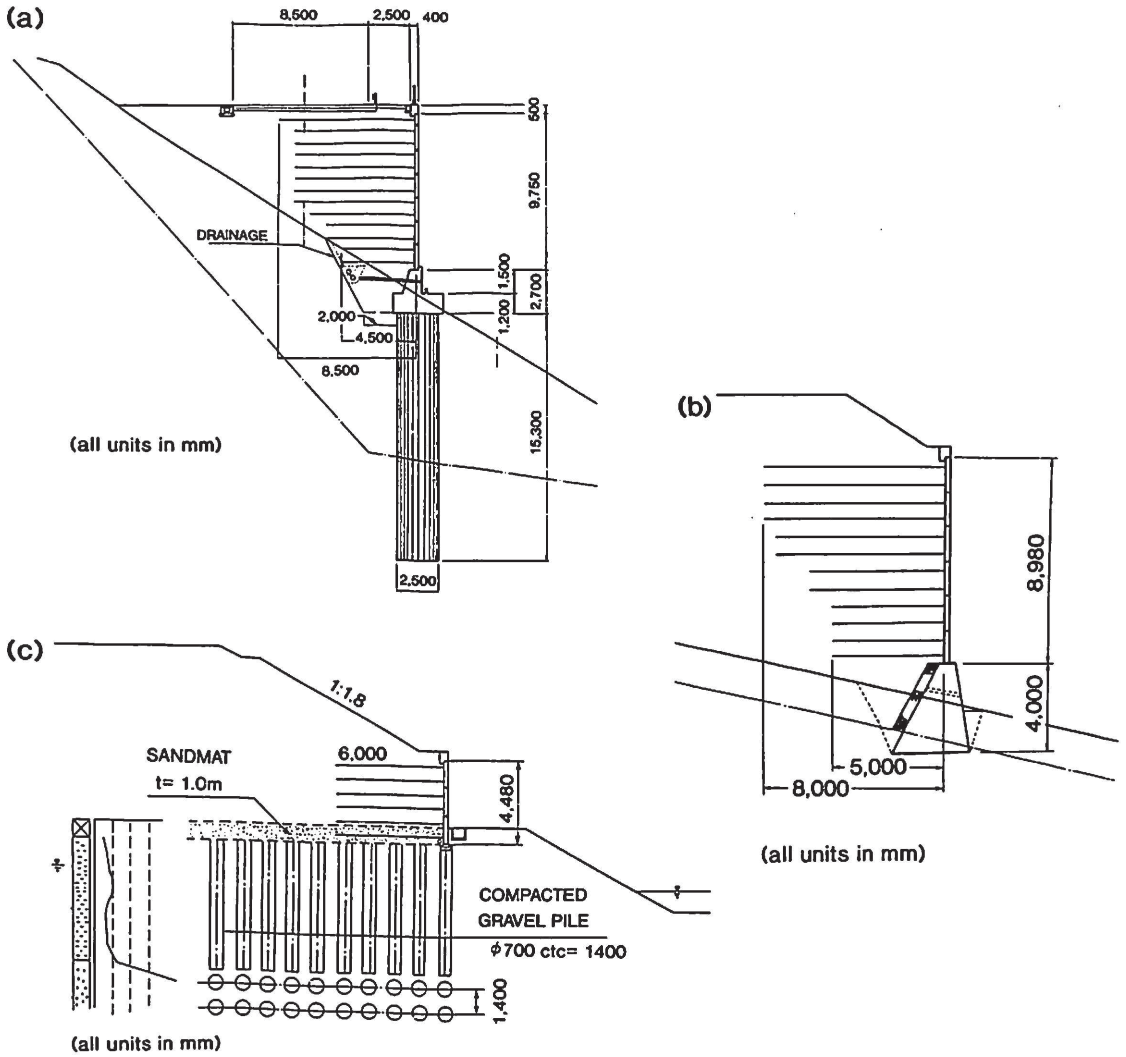
Despite the above, the discussor claims that Terre Armee walls are "able to even handle the case of grounds with poor bearing capacity". The writers are aware of the fact that when the supporting ground for a Terre Armee wall is poor, it is a common practice in Japan to use an enlarged or deep footing or to improve the ground by some proper methods (it is indeed a wise practice). In fact, **Fig. 12** shows several examples of Terre Armee walls for which the facing is supported by a deep or relatively large footing, or the poor ground has been improved. Indeed, the CERC manual presumes that when the supporting ground is poor, the ground has been treated before constructing the foundation for facing (page 145). In light of the belief that Terre Armee walls can withstand poor foundation, the writers wonder how the discussor can explain why the provisions described in Fig. 12 are necessary (the writers consider that they are necessary).

3.4 Usefúlness of rigid facing

The discussor claims that it is dangerous to count on the load carried by the rigid facing of GRS retaining wall. On the other hand, he claims that "a rigid facing does not play any structural rule" by quoting Tatsuoka et al. (1992) "since a continuous rigid facing is constructed after the full height of wall is constructed, very small earth pressure would be activated on the back face of facing." The writers have to admit that this statement is weak; instead, the writers should have stated that "as a rigid facing is cast-in-place, the earth pressure which has been acting on the back of gabions is transferred to the back of the rigid facing. When compared to the earth pressure at the higher levels in the wall due to concentrated load acting on the crest, this earth pressure is very small."

The rigid facing of GRS retaining wall is not 'garnish' (as the discussor claims). This is so in particular (1) when a GRS retaining wall serves as a bridge abutment, and (2) when the wall should carry concentrated load at the top of the facing or on the crest immediately behind the wall face (e.g., a foundation of an electric pole or frame or a noise barrier wall). **Fig. 13** illustrates some examples of the second case. Tamura et al. (1993a) and Tateyama et al. (1993) reported the results of full-scale loading tests showing that the full-height rigid facing can support very large outward lateral load applied at the top of facing and on the crest immediately behind the wall face. Note that for the GRS-RW system, even in such cases, the facing needs not be very thick (i.e., 30 cm at the top of facing), since the facing is supported by many reinforcement layers at the back face (see Fig. 3.6 in Tatsuoka, 1992). In fact, the 30 cm thickness is to improve workability where concrete has to be poured into forms.

The writers wonder which claim the discussor would like to adopt; (a) a full-height rigid facing of the GRS-RW system is only 'garnish' and useless as a structural component, or (b) it is dangerous to count on the structural role of full-height rigid facing for the stability of wall. It is not



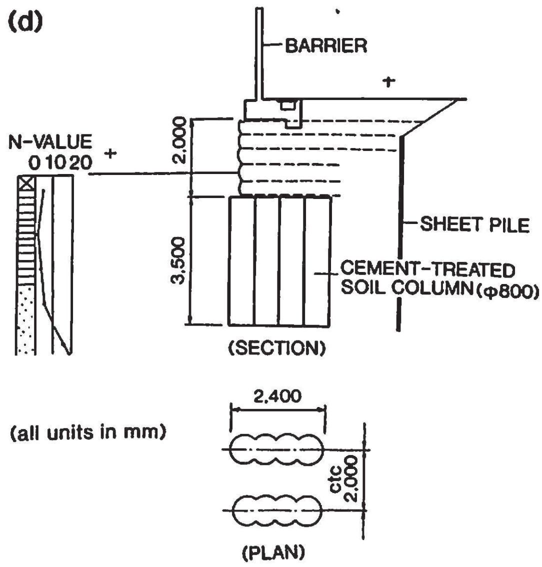


Fig. 12 Several examples of Terre Armee walls: (a) Facing supported by a deep footing, Substitute road for Akigawa Dam, Gifu Prefecture, (b) facing supported by a relatively large footing, Ohshima-Tachibana Route, Yamaguchi Prefecture, (c) poor ground improved by Gravel-Compaction Pile method, Kitayoshima-Aitani Route, Fukushima Prefecture, and (d) Terre Armee wall for railway on poor ground improved by cement-treated soil columns, Koga, Ibaraki Prefecture (Figs. a to c were reproduced from Terre Armee Association, 1989).

logical to criticize something from two contradicting points of view. The fact is that for the GRS-RW system, the writers count on the structural role of full--height rigid wall for the stability of wall in a rational way.

3.5 Some drawbacks of discrete panels facing

In comparison with full-height rigid facings used for the GRS-RW system, some drawbacks of discrete panels facings should be noted. First, in order to alleviate the problem of concentrated surcharge load from some other structure which cannot be directly applied to the facing or on the crest of the wall immediately behind the wall face of Terre Armee wall, several methods have been adopted as shown in the following examples. These methods are rather complicated.

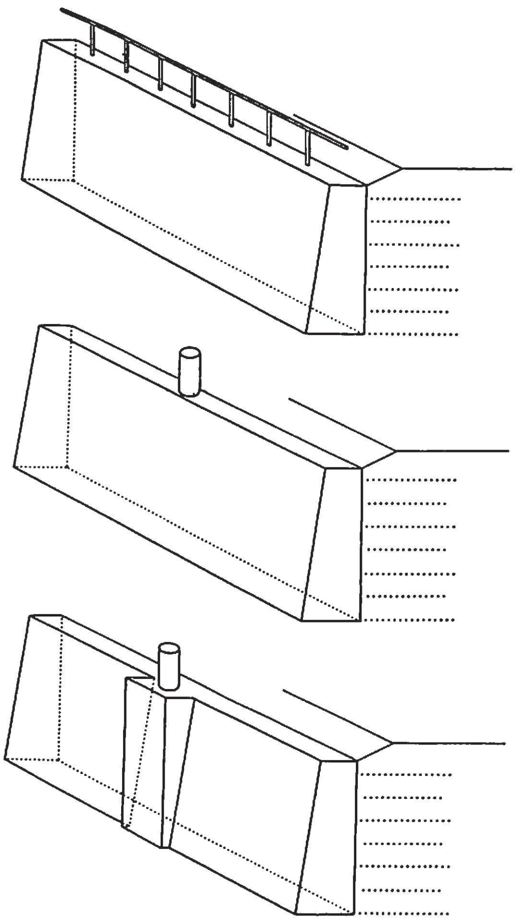


Fig. 13 Some examples of full-height rigid facing of GRS retaining wall

Fig. 14(a) shows the case where a completely independent conventional RC wall structure, having unreinforced backfill soil, was constructed to support an electric pole. The Terre Armee wall was constructed being split for that purpose. Another example is shown in **Fig. 14(b)**. A concave wall face was formed so as to leave a space for a pier foundation of an electric pole, by-passing the Terre Armee wall. In the next example (**Fig. 14c**), the space for installing a pier foundation of an electric pole is made by using a specially designed metal frame of which both ends are connected to strips. **Fig. 14(d)** shows another example for reconstruction of highway embankment. This case called for the construction of a noise barrier wall, which is as high as 5 m. A very large foundation is needed on the crest when using the Terre Armee technique. In such a case, the GRS-RW system would certainly be much more appropriate, and has actually been adopted in place of Terre Armee wall.

The second problem is the difficulty to achieve a good alignment of wall face. Namely, with a discrete panels facing, it is very difficult to compact the soil immediately behind the back face of each discrete panel while achieving an acceptable alignment between horizontal and vertically adjacent panels. When the backfill soil immediately behind each panel is mechanically compacted very well, the top of the panel tends to inevitably move outwards (**Fig. 15**). Therefore, before compaction, each panel should be battered inward slightly so that the panel becomes vertical after compaction. This technique needs a very sophisticated and difficult control. Another way to get around this problem is to compact only slightly the soil immediately behind each panel while setting the panel vertical. The writers hope that the second measure is less popular in practice, since it may result in a delayed outward displacement of the wall face and delayed settlement of the soil immediately behind the facing, neither of which are desirable.

Fig. 7(b) shows the shape of several sections of the wall face randomly selected at a horizontal spacing of 50 m, measured in November 1992, of the Terre Armee wall constructed in December 1983. The top of the facing has generally tilted outwards beyond the toe of the wall face. This may be largely due to post-construction deformation of the wall face, as evidenced by the fact that the wall face was still moving between the times of four and six months after the completion of wall construction (**Fig. 7c**). Indeed, 23 problem cases with Terre Armee walls reported by Hatta (1992) were due to the mis-alignment of the wall face.

It is interesting to note what is seemingly reflecting on the difficulty with the wall face alignment. The CERC manual allows a large tolerance of mis-alignment at the top: it is as much as ± 0.03 times the wall height for wall heights less than 5 m and $\pm$ 30 cm for larger wall heights (page 222). Such large amount of mis-alignment is not acceptable for GRS retaining walls, as well as conventional RC retaining walls. In fact, Ogawa et al. (1993), who are supervising engineers of the Terre Armee technique, honestly reported that many Terre Armee walls constructed in the west part of Japan exhibited very large deformation (**Fig. 16**, Ogawa et al., 1993); indeed, the largest outwards displacement at the top of facing was nearly 40 cm (not 40 mm) for a wall height of 6 to 8 m, even exceeding the tolerance. It is interesting that Ogawa et al. reported also that generally the displacement was larger as the fines content increased in the backfill soil. They also reported that usually each discrete panel is set battered inwards 1 % and the soil for a 1.5 m width from the facing is compacted only lightly. Then, it is likely that larger outward displacement reflects on better compaction of the soil immediately behind the facing, while zero or inwards displacement of the facing means poorer compaction of the soil.

On the other hand, for GRS retaining walls, a heavy mechanical compaction equipment has been operated even on gabions placed at the edge of each layer, allowing the wall face to move freely outwards as shown in **Plate 1**. A large degree of mis-alignment of temporary face is allowed, since it is covered subsequently with a cast-in-placed rigid facing (**Plate 2**).

Finally, it has also been reported by Hatta (1992) that 11 problem cases of Terre Armee walls were due to wash out of backfill sand during rainfall. The writers suspect that in some cases, this wash out occurred due to a gap between discrete panels.

3.6 Fundamental differences in design concepts between Terre Armee technique and GRS-RW system

Most of the discussion points are concerned with the structural role of facing for the stability of reinforced soil retaining wall. In this respect, the two main features advocated as the advantages for Terre Armee walls may be summarized as follows:

1) Only very small earth pressure is activated on the back face of facing. Accordingly, the structural role of the facing is only "to retain the soil grains located near the exterior between the two layers of reinforcement, and is not important" (Vidal, 1978).
2) The complete walls are flexible enough to accommodate relatively large deformation of the supporting ground when it occurs.

However, the writers consider that the advocating the above-mentioned features is in general misleading (Tatsuoka, 1992).

Regarding the first feature, the earth pressure acting on the back face of facing should be large enough, on the order of the active earth pressure in unreinforced soil, so that active zone between the facing and the potential failure surface is well confined. This pressure will counter-balance the thrusting earth pressure from the unreinforced part of backfill and external load (i.e., will produce a system in equilibrium). It should be emphasized that the facts do not support this first feature for Terra Armee walls; i.e., earth pressure close to the active earth pressure is acting on the back face of discrete concrete panels facing of Terre Armee wall as shown in Fig. 4.1 of Tatsuoka (1992), which was reproduced from Schlosser (1990). Namely, when the facing has some

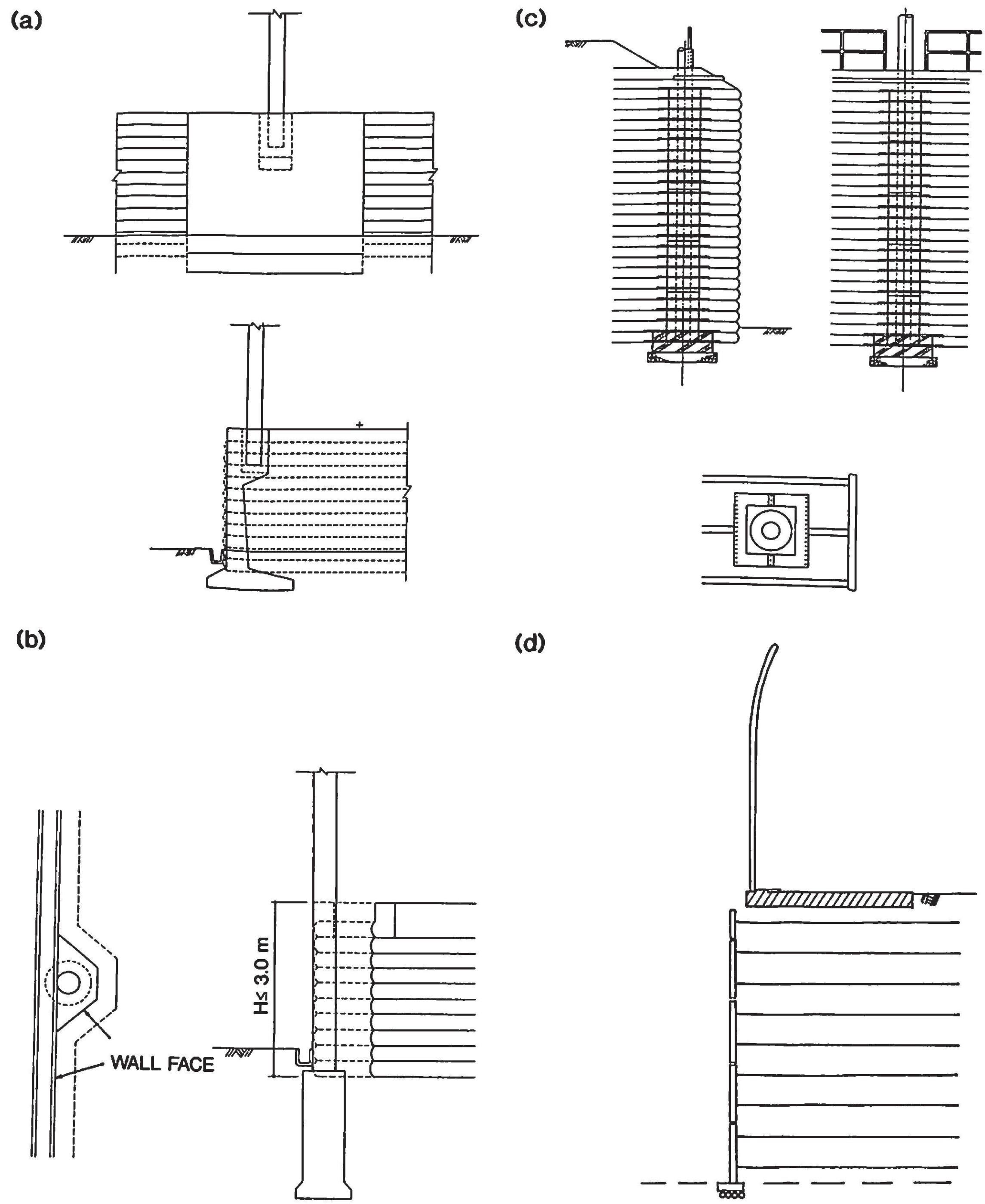


Fig. 14 Some examples of Terre Armee walls to accommodate a foundation of structure

degree of rigidity (as with both discrete concrete panels facings of Terre Armee wall and full-height rigid facings of the GRS retaining walls), relatively large earth pressure tends to act on the back face of facing.

Regarding the second feature, it is true that a reinforced soil retaining wall under construction should be flexible enough to accommodate potential relatively large deformation of the supporting ground. However, a completed wall should be rigid enough when used as an important permanent structure so that serviceability is satisfied. As discussed by Tatsuoka (1992), the rigidity of facing can contribute largely to this purpose.

In short, the GRS-RW system is a sort of

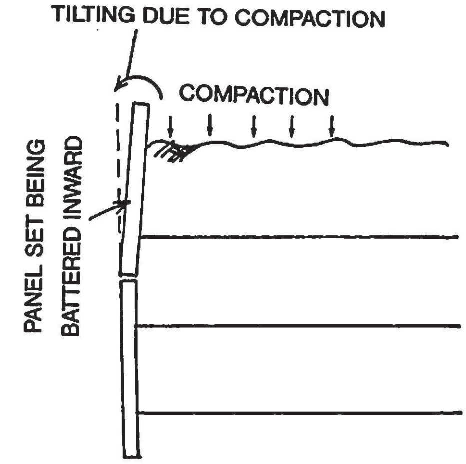
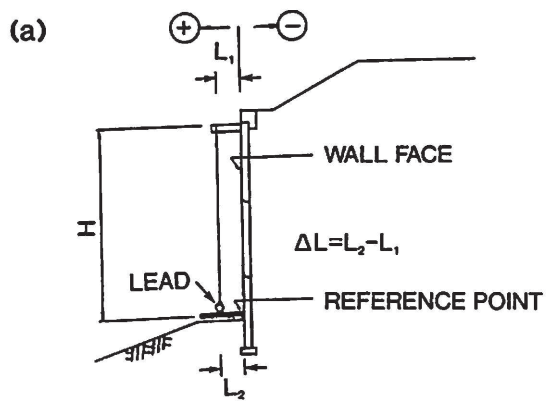


Fig. 15 Schematic diagram showing complicated arrangements to ensure wall facing alignment for Terre Armee wall.



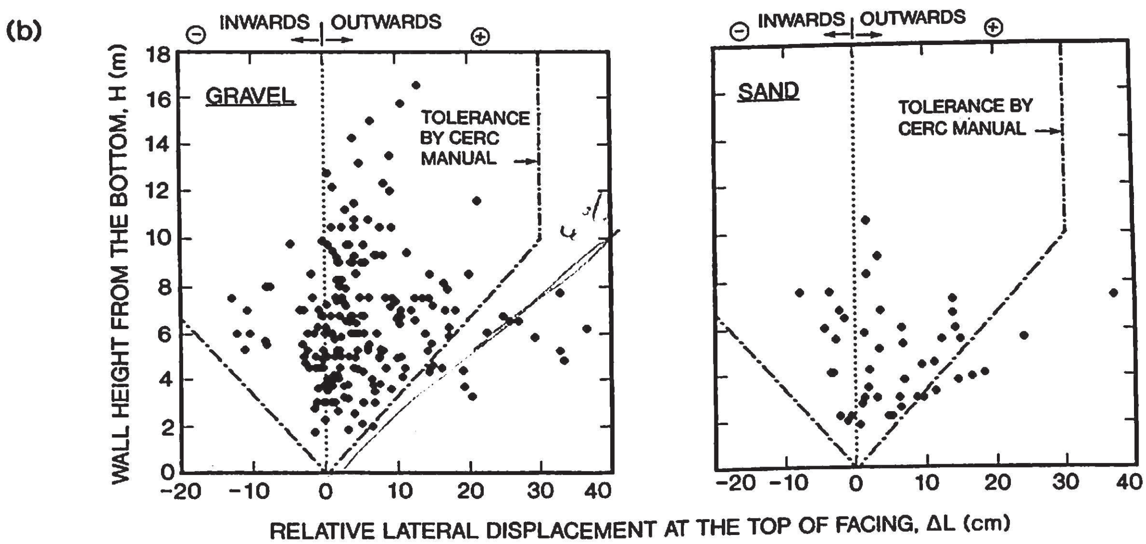


Fig. 16 Lateral displacement at the top of the facing relative to the toe of the facing for completed Terre Armee walls constructed in the west area of Japan: backfill soil are gravel and sand (Ogawa et al., 1993).

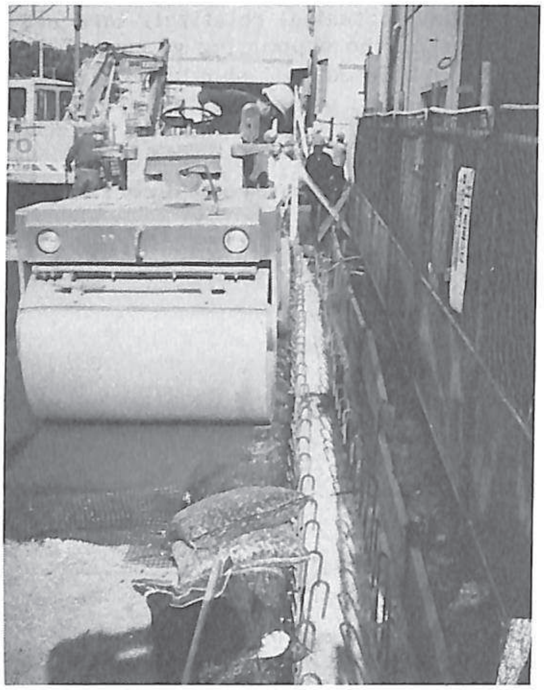


Plate 1 (left) Mechanical compaction until the wall face by using a large compaction plant for a GRS retaining wall in Tokyo for Seibu railway.

"hybrid system" as claimed in the discussion, taking advantage of the facing rigidity for the stability of wall.

3.7 Degradation of geosynthetic reinforcement

The writers are aware of the fact that prolonged high alkaline environment created by the concrete is detrimental to polyester fibers. However, it is the discussor's misunderstanding to claim that polyester grid has been used as the reinforcement for the GRS retaining walls constructed for railways. The fact is that the writers used a grid of polyester fibers coated with PVC protection only for the full-scale test embankments constructed at the site of Railway Technical Research Institute (Murata et al., 1991, Tatsuoka et al., 1992). However, the writers are using a grid of Vinylon fibers covered with PVC protection for all the "real life" GRS retaining walls. Vinylon is known to have high resistance to alkaline. **Fig. 17** shows the test results of tensile failure tests of three materials (Vinylon is the trade mark of PVA). In these tests, three materials

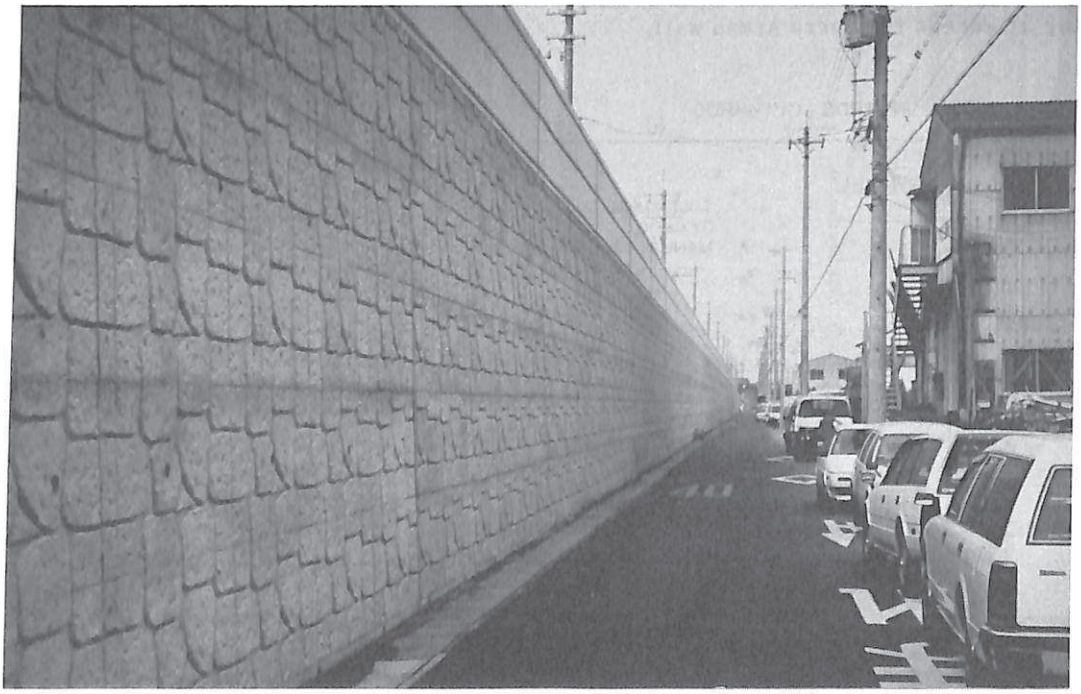


Plate 2 A view of the wall face of GRS retaining wall in Nagoya.

were tested before and after soaking for given periods in cement extract at 80° C. It may be seen that Vinylon grid did not deteriorate with time at all. Indeed, the capability to resist the effects of alkaline was one of the most important features considered when selecting the geosynthetic type to be used for the GRS-RW system.

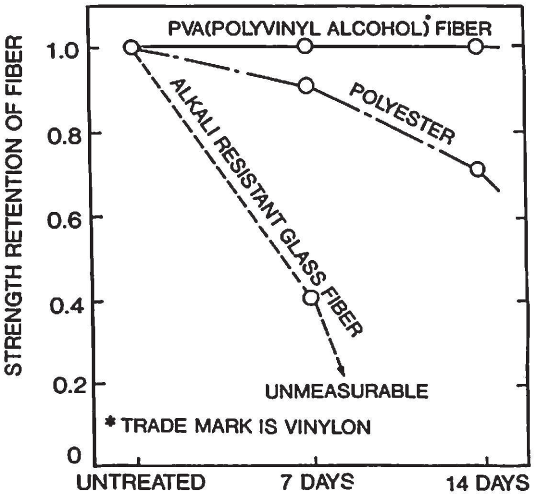


Fig. 17 Test results of tensile failure tests of three materials (Vinylon is the trade mark of PVA).

4. SUMMARY AND CONCLUSIONS

The Terre Armee technique is no doubt one of the most novel inventions in the evolution of earth retaining walls. It is shown in this closure, however, that the use of metal strip reinforcement and discrete panels facing is no longer the best solution nowadays. It has been demonstrated that a better soil reinforcing method for constructing retaining walls is the GRS-RW system, which uses a planar reinforcement (i.e., a wide variety of geosynthetics including grid and non-woven/woven composite geosynthetics) combined with a full-height continuous rigid facing. This facing is cast-in-place after a full height wall is constructed with the aid of gabions (or alternative methods which very likely will be developed in the future) placed at the edge of each soil layer.

The final two lines of the main text of the discussion state that "GRS walls cannot pretend to compete in this same category (with Terre Armee retaining walls)." Indeed, the writers are not pretending to compete in the same category with the Terre Armee walls, but maintaining that the GRS-RW system is different from Terre Armee in many of its important features. Because of that, the GRS-RW system is superior to Terre Armee walls in many respects. The GRS-RW system has been and will continue to be used for many important construction projects in Japan in place of Terre Armee retaining walls, as well as conventional RC retaining walls.

ACKNOWLEDGEMENTS

The writers appreciate kind and critical review of the manuscript by Dr. H.I. Ling, Prof. Dov Leshchinsky, University of Delaware, USA, and Prof. Jonathan T.H. Wu, University of Colorado, USA.

REFERENCES

Allen,T.M., Christopher,B.R. and Holtz,R.D. 1992. Performance of a 12.6m high geotextile wall in Seatle, Washington, Geosynthetic-Reinforced Soil Retaining Walls, Wu. (ed.), Balkema, pp.81-100.

Blight,G.E. and Dane,M.S.W. 1989. Deterioration of a wall complex con structed of Reinforced Earth, Geotechnique 39, No.1, pp.47-53.

Civil Engineering Research Center, 1989. Design and construction manual for Terre Armee walls (in Japanese).

Doi,Y., Tateyama,M., Murata,O. 1993. Construction of a geotextile-reinforced soil retaining wall to reconstruct a railway embankment at Rokujizo, Kyoto, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tokyo, Nov. 1992 (Tatsuoka & Leshchinsky ed.), Balkema, this volume.

Emura,Y., Tateyama,M. and Murata,O. 1993. Geosynthetic-reinforced soil retaining walls for full-reconstruction of railway embankments at Aso, Kyushu, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil

Retaining Walls, Tokyo, Nov.1992 (Tatsuoka & Leshchinsky ed.), Balkema, this volume.

Japan Terre Armee Association. 1989. Construction case records of retaining walls by Terre Armee technique.

Jewell,R.A. 1992. Links between the testing, modelling and design of reinforced soil, Keynote Lecture, Preprint, International Symposium on Earth Reinforcement Practice, IS Kyushu '92 (also printed in Volume II published in 1993).

Hatta,T. 1991. Present state of Terre Armee technique, The Foundation Engineering & Equipment, Monthly Journal Kisoko, pp.40-45 (in Japanese).

Hotta,M. 1991. Case record of Terre Armee composite bridge abutment, The Foundation Engineering & Equipment, Monthly Journal Kisoko, pp.110-114 (in Japanese).

Horii,K., Kishida,H., Tateyama,M. and Tatsuoka,F. 1993. Design and computer program for geosynthetic-reinforced soil retaining wall for railway embankments, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tokyo, Nov. 1992, this volume.

Kanazawa,Y., Murata,O. and Tateyama,M. 1993: Geosynthetic-reinforced retaining walls for reconstructing railway embankment at Amagasaki, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tokyo, Nov. 1992 (Tatsuoka & Leshchinsky ed.), Balkema, this volume.

Murata,O., Tateyama,M, and Tatsuoka,F., Nakamura, K. and Tamura, Y. 1991. A reinforcing method for earth retaining walls using short reinforcing members and a continuous rigid facing, Proc. ASCE Geotech. Engrg Congress 1991, Geotech. Special Publication 27, pp.935-946.

Ogawa,N., Hori,T., Enzitsu,K. and Ueno,H. 1993. Displacement measurements of Terre Armee wall (Part I), Proc. the 28th Japan National Conf. on Soil Mechanics and Foundation Engineering, Kobe, Vol.II, pp.2757-2758 (in Japanese).

Schlosser,F. 1990. Mechanically stabilized earth retaining structures in Europe, Design and Performance of Earth Retaining Stuctures, Geotechnical Special Publication No.25, ASCE, Lambe and Hansen (ed.), pp.347-378.

Smith,R.J.H. 1991. The development of Terratrel for the construction of steel slopes, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, London, pp.135-139.

Tateyama,M., Murata,O., Watanabe,K. & Tatsuoka,F. 1993. Geosynthetic-reinforced retaining walls for bullet train yard in Nagoya, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tokyo, Nov. 1992 (Tatsuoka & Leshchinsky ed.), Balkema, this volume.

Tateyama,M., Murata,O., Tamura,Y., Nakamura,K., Tatsuoka,F. and Nakaya,T. 1993a. Lateral loading tests of full-scale model column foundations on geosynthetic-reinforced soil retaining walls, Proc. of Inter. Sympo. on Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tokyo, Nov. 1992 (Tatsuoka & Leshchinsky ed.), Balkema, this volume.

Tatsuoka,F. and Yamauchi,H. 1986. A reinforcing method for steep clay slopes using a non-woven geotextile, Geotextiles and Geomembranes 4, pp.241-268.

Tatsuoka,F. Tateyama,M. and Murata,O. 1988. Earth retaining wall with a short geotextile and a rigid facing, Proc. 12th Int. Conf. on SMFE, Rio de Janeiro, pp.1311-1314.

Tatsuoka,F. 1992. Roles of facing rigidity in soil reinforcing, Keynote Lecture, Preprint, International Symposium on Earth Reinforcement Practice, IS Kyushu '92 (also printed in Volume II published in 1993).

Tatsuoka,F. 1992a. Permanent geosynthetic reinforced soil retaining walls for railway embankments in Japan, IGS News, Vol.8, No.1, pp.7.

Ministry of Transport. 1992. Design Standard for Earthworks and Soil structures for Railway Structures.

Vidal,H. 1978. The development and future of Reinforced Earth, Keynote Address, Proc. of Sympo. on Earth Reinforcement, ASCE, pp.1-61.

Wu,J.T.H. and Helwany,M.B. 1993. A performance test for long-term soil-geosynthetic interaction, submitted to Journal of Geotech. Engineering, ASCE for review.