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Keynote lecture: Roles of facing rigidity in soil reinforcing

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ABSTRACT: Rigid facings have often been used for important permanent reinforced and nailed soil structures mainly for higher durability, better aesthetics and easier construction. It is discussed based on many laboratory and field data that both local and overall facing rigidities can also help in reducing the deformation while increasing the stability of the structures. Namely, large tensile force in reinforcement at the connections to the back face of a rigid facing can confine effectively the soil immediately behind the wall face, which can in turn increase the stability of the structures; soil reinforcing methods cannot be best characterized by a largely reduced earth pressure on the back face of the facing. Further, axial force, shear force and bending moment transmitted through a full-height continuous rigid facing can restrain a premature local failure in a reinforced soil structure and thereby, can increase the stability of the structure. A method to take into account the effects of facing rigidity in limit equilibrium-based stability analyses is discussed. For reinforced soil retaining walls, as a method to alleviate some potential disadvantages associated with using a full--height unit facing, a stage construction by casting in place a concrete facing directly on a completed wrapped-around wall face is suggested. It is also shown that for minimizing the deformation during excavation of nailed walls, occasionally a pre-propping structure is constructed immediately behind the wall face before the excavation proceeds.

1. INTRODUCTION

No doubt the most characteristic and important manufactured structural component for tensile-reinforced soil structures are reinforcing members. Therefore, the material properties, the length, the spatial density, the direction and so on of reinforcement are essential design factors in soil reinforcing. Together with the design and construction methods of soil reinforced soil structures and the principles underlying their design, the issue of reinforcement has been reviewed by many researchers (e.g., Bolton, 1991 Ingold, 1982, Jewell, 1991, Mitchell, 1982, Schlosser and Juran, 1980, 1983, Schlosser, 1990, Schlosser and De Buhan, 1991, Stocker and Riedlner, 1990, Gassler, 1991). In this note, however, these issues are touched only in relation to the effects of the facing rigidity. Herein, the terminology <u>reinforced soil</u> means "slopes or retaining walls with a backfill soil reinforced during embanking and <u>nailed soil</u> means "those with an existing soil reinforced during excavation".

The effects of local and overall facing rigidities are not taken into account explicitly in most (not all) of the current stability analysis methods used for reinforced and nailed soil structures, probably based on the assumption that the facing of a reinforced or nailed soil structure "does not play a major role in the overall structural stability" (Bruce and Jewell, 1986). Rather, the issue of the facing has been discussed mostly from a viewpoint of durability and aesthetics of wall face, and construction convenience (e.g., Jones, 1985, 1991, Jones et al., 1988, O'Rourke and Jones, 1990). However, attention herein is directed primarily to the mechanical roles of the facing in reducing the deformation while increasing the stability of reinforced and nailed soil structures. The issue of the effects of facing rigidity in stability analyses is also discussed.

2. A BRIEF HISTORICAL REVIEW

The two fundamental advantages of rein-

forced soil retaining walls over the conventional retaining walls such as RC gravity-type and cantilever type wall structures which have often been advocated are:
1) that only very small earth pressure is activated on the back face of facing, which is much smaller than the active earth pressure activated when the backfill is not reinforced, and
2) that the wall is flexible enough to accommodate relatively large deformation (particularly unequal settlement) of the supporting ground.

With respect to the first point, Vidal (1978, pp.24) stated for the facings of Reinforced Earth (R. E.) retaining walls (i.e., Terre Armee retaining walls) that "if we could put in place one layer of grains in contact with one layer of reinforcement, then one layer of grains, and so on, we should not have any need for a facing. The facing retains the grains located near the exterior between two layers of reinforcement; it corresponds to a very local problem, and is not important." Probably, this statement was made to clearly contrast this new technology with the conventional retaining walls.

With respect to the second point, when constructed on relatively soft ground, a conventional RC retaining wall, which is much more rigid than reinforced soil retaining walls, may need to be supported by a pile foundation to avoid damage to the wall structures. In contrast, a pile foundation becomes unnecessary for most reinforced soil structures because of their flexibility, which leads to large cost saving and shortening of construction period. Then for ensuring the global flexibility of the wall, the facing should also be flexible enough. In addition, the facing should be axially compressible also so as to accommodate the possible relative settlement between the facing and the backfill so that the connections between the reinforcement and the back face of facing are not damaged.

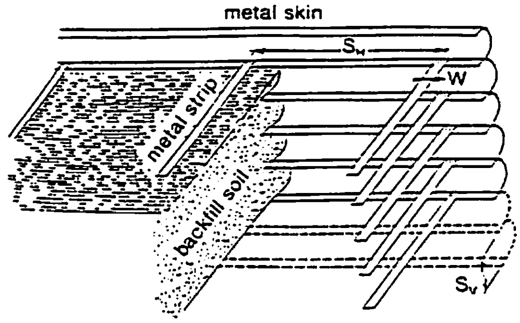

Fig. 2.1 Metal facing for R. E. retaining walls (Vidal, 1978).

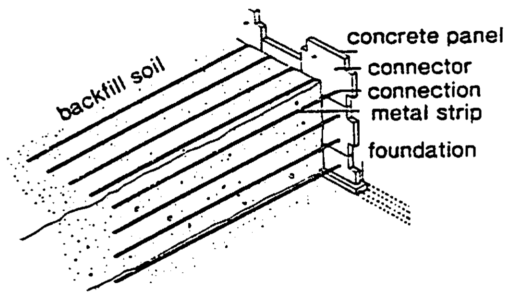

Fig. 2.2 Standardized cruciform concrete panels facing for R. E. retaining walls (Vidal, 1978).

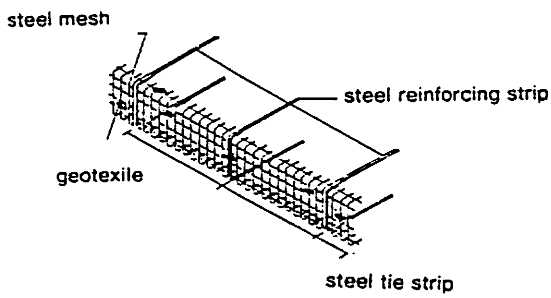

Fig. 2.3 Terratrel facing panel for R. E. retaining walls (Smith, 1991).

<u>Reinforced Earth (R. E.) retaining walls:</u> When based on the above-mentioned two rationales, metal facings (Fig. 2.1) used at its developing stage, which was called the concertina construction by Jones (1985), can be considered as a proper type of facing due to its high flexibility. However, currently only standardized cruciform concrete panels (Fig. 2.2) are being used, despite that this type of facing is much less deformable. For the reasons, Vidal (1978, pp.25) stated that "I began with steel facings in an elliptic-cylindric shape. · · · · The steel facings have been quite satisfactory. But progressively, I have replaced steel with precast concrete panels which are better looking." Schlosser (1990, pp.352) addressed another reason that the cruciform facing panel type "allows various architectural possibilities, including curved facings." In order to maintain the necessary deformability, in particular the axial compressibility of concrete panels facings, a compressive material, usually a piece of cork, is inserted at each joint between vertically adjacent panels, which was called the Telescope construction method by Jones (1985).

As will be discussed later in this note, discrete concrete panels facings are better than metal facings also for decreasing the deformation while increasing the stability of the wall. As far as the author is aware, however, the above point has not been referred to as the major reason for the replacement of facing type as mentioned earlier. Interestingly, a soft facing type called Terratrel (Fig. 2.3, Smith, 1991) has been introduced to R. E. retaining walls to replace conventional discrete concrete panels facings for lower construction cost, vegetation when needed, for a construction speed faster than wrapped-around geotextile walls (but equivalent to the discrete concrete panels facings), and for the construction of steep slopes rather than vertical walls. This is stiff steel facing elements with the top and bottom of each element being connected to reinforcement metal strips. Seemingly, although this new type soft facings are as flexible as the old type metal facings, possible effects of the decrease in the facing rigidity on the stability of wall have not been addressed. Interestingly, the combination of a soft facing with metal strip reinforcement is in contrast with another combination of a discrete or continuous rigid facing with planar geotextile reinforcement (Table 2.1).

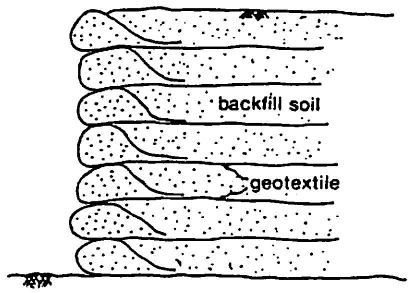

Fig. 2.4 Typical wrapped-around geosynthetic-reinforced soil retaining wall.

Table 2.1. Rough classification of reinforced-soil retaining walls based facing rigidity and reinforcement shape

		Facing type	
		Soft[1]	Rigid[2]
Reinforcement	Srip	Terre Armee with Terratrel	Terre Armee with a concrete panels facing
	Planar	GRS-RW with a wrapped-around wall face	GRS-RW with a concrete panels facing

1: Types B1 or B2, having the local rigidity (see Fig. 5.1 and Table 5.1).
2: Types C or D or E, which have local and overall rigidities. More detailed discussions on the facing rigidity are given in Section 5.

Geosythetic-reinforced soil retaining walls (GRS-RWs): It seems that the above mentioned design principle for the facing has been adopted in most of the design methods (but not all) for GRS-RWs. Namely, "facing is only necessary to prevent erosion of the slope and to keep the front layer in place" (Veldhuijezen van Zanten, 1985, pp.431). Or "for vertical structures a facing is required. The function of the facing is to stop erosion of the fill and to provide a suitable architectural treatment to the structure" (Jones, 1985, pp.68).

Many of the GRS-RWs constructed so far have a wrapped-around wall face, for which the end of geosynthetic sheet is wrapped upwards around the end face of soil layer forming a round or flat shape (Fig. 2.4). It has been noticed, however, that this type of wall face is not adequate for permanent structures, because of some or all of the following disadvantages (e.g., Wichter and Gay, 1986): 1) the deformation of the wall face and the soil immediately behind the wall face could be too large to be acceptable as typically reported by Allen et al. (1992), 2) the long-term durability against vandalism (mechanical damage and firing) and the UV sunlight is not sufficient, 3) the wall face may not be aesthetically pleasing, particularly when constructed in urban areas, and 4) the step-by-step construction of wrapped-around face is relatively slow and labor-intensive. However, a wrapped-around wall face is still adequate for temporary and/or less-important walls as those located in most rural settings (Keller, 1990).

Indeed, the above four points are the primary criticisms against the performance of GRS-RWs having a wrapped-around wall face. The other group of criticisms is against the performance of geosynthetic as reinforcement, which includes that (e.g., Schlosser, 1990): 5) geosythetic reinforcement is too extensible, 6) long-term creep deformation may not be acceptable, 7) chemical deterioration may be large when placed in soil, and 8) it may be damaged too much during the compaction of soil layers, in particular when using a large-diameter angular material. These criticisms have always been controversial issues in contrast to those against metal strip reinforcement, which include the problem of corrosion, a relatively small contact area with the surrounding soil, no function of drainage, therefore relatively strict restriction to the soil type for the backfill (i.e., only cohesionless soils with a small amount of fines) and so on. Many researchers on one hand (e.g., Jewell, 1991, Koerner et al., 1980, 1985, 1989) consider that these problems 4) through 8) could not be actual serious problems when properly designed.

Because of the first group of problems 1) through 4), however, wrapped-around wall faces are often protected by shotcreting or planting (e.g., Resl et al., 1988). For relatively important permanent GRS-RWs, more rigid, more durable and more aesthetically acceptable facings are used more often, which include; a) pre-cast discrete concrete panels facings as used for R. E. retaining walls (e.g., Piggs and McCafferty, 1984), b) continuous rigid facings of full-height pre-cast concrete panel with the back face connected to geotextile reinforcement layers; e.g., for bridge abutments of GRS-RW (Fig. 2.5, McCaul, 1992), or inverted Y-shaped RC retaining walls having a degree of self-standing capability with a backfill reinforced with a polymer grid (Yamanouchi, 1986), c) full-height pre-cast concrete panels separated from the wrapped-around wall face (e.g., Fig. 2.6, Gourc et al., 1991, Gourc and Matichard, 1992), and d) facings made of a stack of modular unreinforced concrete blocks with decorated outer face (Fig. 2.7, Simac et al., 1990), or cellular facings (e.g., Fig. 2.8, Gourc et al., 1991, Gourc and Matichard, 1992). However, the mechanical contributions of facing rigidity are not taken into account explicitly in the stability analyses for most of these GRS-RWs, presumably by considering that it is conservative and may be justified from its relatively short history of actual construction. On the other hand, recently in Japan, GRS-RWs were constructed as important permanent railway structures (Tatsuoka et al., 1992, Murata et al., 1992). The walls have a cast-in-place concrete facing placed directly on the wrapped--around wall face after the full height of wall had been constructed with the aid of gabions placed at the shoulder of each soil layer (Fig. 2.9). This type of facing was adopted not only for better durability and aesthetics of wall face and easier construction, but also by expecting contributions of local and overall rigidities of facing to the stability of the wall.

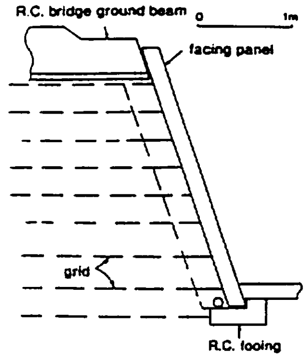

Fig. 2.5 Continuous rigid facing of pre-cast concrete full-height panel for geosynthetic-reinforced bridge abutments (McCaul, 1992).

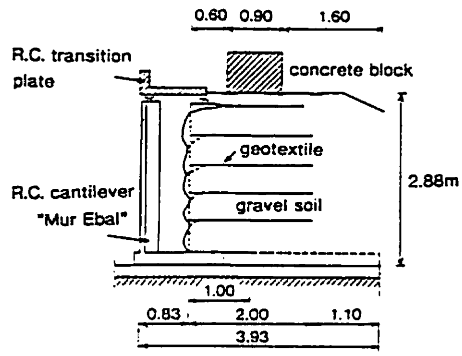

Fig. 2.6 Pre-cast full-height concrete panel facing separated from the wrapped-around wall face of GRS-RW : Model for loading test (Thamm et al., 1990, see Fig. 6.9 for the wall after loading).

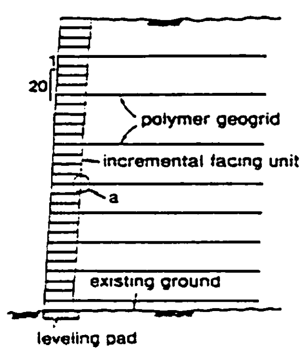

Fig. 2.7 Concrete blocks facing (Simac et al., 1990).

Nailed retaining structures: For soil nailing, since soils are generally stiffer having a larger cohesion and finished slopes are less steeper, the need for a rigid facing is generally lower when compared with reinforced soil structures. For most temporary nailed retaining walls, a relatively thin shotcrete lining with one layer of wire mesh and small steel bearing platens placed on the shotcreted surface are sufficient to accommodate earth pressure in the back of wall face and head loadings of nails respectively, in contrast to substantial bearing arrangements needed for prestressed ground anchorages (Bruce and Jewell, 1986). Correspondingly, the effects of facing rigidity are not taken into account in the stability analyses.

However, more rigid facings have been used occasionally in the following three cases: 1) For permanent important walls, shotcrete layers may not be aesthetically pleasing and/or not sufficient for the long-term support of the cutting. In some cases, a facing of pre-cast concrete unit or cast-in-place concrete layer was placed on the shotcrete face (e.g., Gasseler, 1991, Tateyama et al., 1992). Bruce and Jewell (1987) reported a case at Versailles--Chantier in France, where a gravity retaining wall was constructed on a 22 m-high completed nailed slope having a 70° slope angle. 2) When the soil is not strong enough, such as uncemented sands, delayed shotcreting after the excavation may lead to a local compressional failure in the soil immediately behind the exposed slope face, which may lead to the overall failure of slope. 3) The horizontal outward displacement at the top of the excavation is usually of the order of 0.1 % to 0.3 % in the ratio to the wall height for drilled and grouted nails (Bruce and Jewell, 1987, Schlosser and Unterreiner, 1991, Schlosser and De Buhan, 1991) or 0.2 % to 0.4 % (Stocker and Riedinger, 1990). These values are in general larger than those for well anchored structures, and may not be acceptable, for example, when the slope is supporting a building or a railway track. In some cases of the latter two 2) and 3), in particular when the soil is not very strong, a pre-propping structure such as a

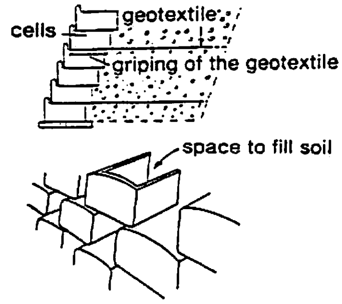

Fig. 2.8 Cellular facing (Gourc et al., 1991, Gourc and Matichard, 1992)

cement-treated soil diaphragm was constructed immediately behind the wall face before the excavation proceeded. (e.g., Bruce and Jewell, 1987, Woonward, 1991, Tateyama et al., 1992).

In this note, based on many laboratory and field data, it will be discussed that the structure of facing and its related design issues become more important, even equally important to the issues of reinforcement; 1) as the slope becomes steeper, 2) as the soil becomes weaker, in particular as the cohesion or natural cementing becomes smaller, 3) as the load on the crest of slope or wall becomes larger, and/or is located closer to the shoulder of slope or wall, 4) as the vertical and horizontal spacings, and the density and length of reinforcement becomes larger and shorter, respectively, 5) as allowable deformation of wall becomes smaller, 6) as the structure becomes more important, and 7) as the designated life time of structure becomes longer. Generally speaking, the failure of a soil structure is always more-or-less

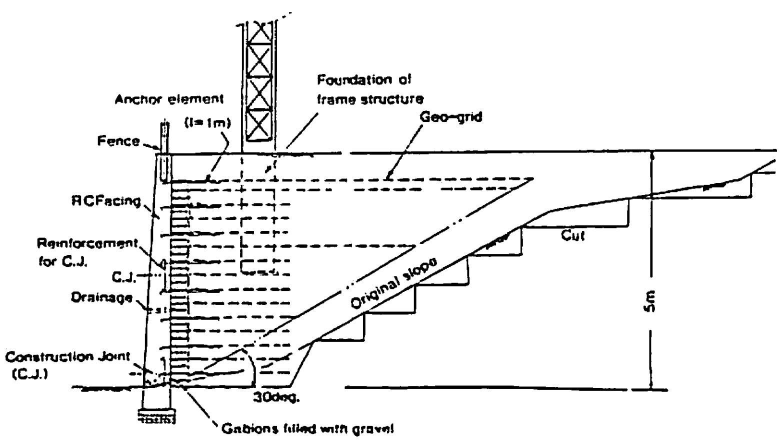

Fig. 2.9 Cross-section of geosynthetic-reinforced soil retaining wall with a slightly inclined wall face at Shinkansen yard, Nagoya (Tatsuoka et al., 1992).

progressive starting from a local failure. Therefore, as stress concentration and the resulting initiation of premature local failure can be restrained effectively by using a rigid facing, the failing zone becomes larger, resulting in larger failure load and larger stability of the structure. In this note, some potential disadvantages when using a rigid facing which should be alleviated are also discussed.

3. MECHANICAL ROLES OF FACING RIGIDITY

Some differences in both principles and details exist among the currently used various limit equilibrium-based stability analysis methods for reinforced and nailed retaining structures; for example whether failure plane is fixed (e.g., R.E. retaining walls) or not fixed while seeking the minimum safety factor (e.g., the two-wedge method for GRS-RWs and nailed retaining walls), or whether failure plane is two straight lines resembling a log-spiral (R.E. retaining walls), a log-spiral (Leshchinsky and Volk, 1986), an arbitrary two straight lines with one intermediate vertical interface line (the two-wedge method), a single straight line, part of circle, or others. Commonly among these, the global stability for overturning is not examined on the premise that the reinforcement is long enough to prevent this mode of failure from taking place, while it is assumed that the whole horizontal outward thrust by the earth pressure activated at the failure plane is resisted by the tensile resistance of reinforcement. The following two methods are being used.

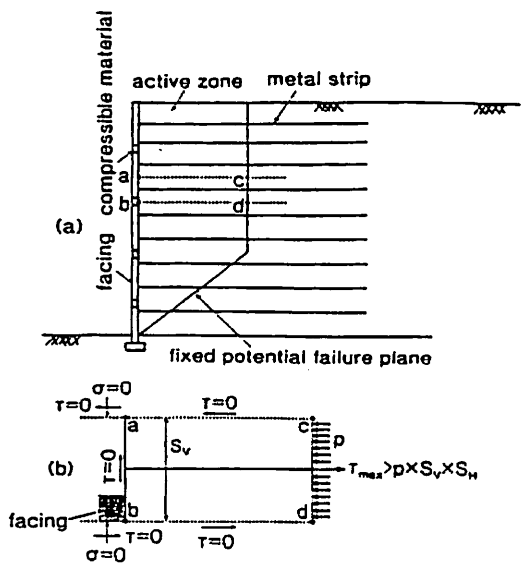

Fig. 3.1 Illustration of the lower bound design method (for the case of R. E. retaining walls).

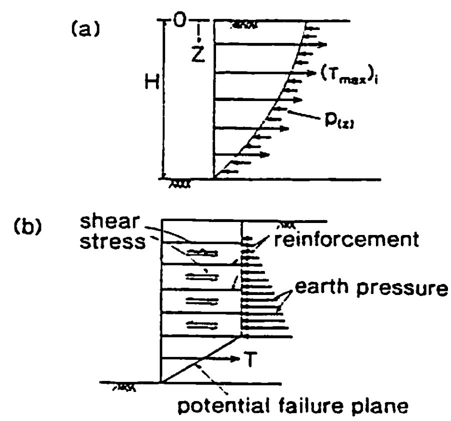

Fig. 3.2 Illustration of the upper bound design method.

<u>The Rankine method (Lee et al., 1973) or the stress method (Bolton and Pang, 1982)</u>: This method may be called the lower bound method in contrast to the upper bound method which is explained next (Bolton and Pang, 1982). This method includes the design method for R. E. retaining walls. The global stability for horizontal sliding out is examined by considering the horizontal equilibrium in a single representative critical horizontal soil layer including one reinforcement layer at any depth (Fig. 3.1):

T_{max} (force per strip) $> p\cdot S_V\cdot S_H$ (for strip reinforcement) (3-1a)

T_{max} (force per width) $> p\cdot S_V$ (for planar reinforcement) (3-1b)

where T_{max} and p are the maximum available tensile force along the concerned reinforcement layer and the horizontal earth pressure, both activated at the potential failure plane, and S_V and S_H are the vertical and horizontal spacings, respectively, of reinforcement. When the equilibrium in any one layer is not satisfied, the structure is judged to fail. This method, therefore, is the safe side design, since the actual failure is always more-or-less progressive redistributing stresses from the firstly failed zone to the adjacent zones. However, when the failure of reinforcing members is sudden exhibiting a smaller post peak strength (n.b., the residual tensile strength of reinforcement is zero), a high degree of the stress redistribution without losing the overall stability of the wall cannot be attained. Therefore, this method may be adequate when the failure is catastrophic as caused by the tensile rupture of in-extensive reinforcement having a very small failure strain. This method is on the safe side also from the point that it is not inherently assumed that the whole active zone behaves like one monolith, because Eq. (3-1) assumes no horizontal shear stresses mobilized both along the mid-height horizontal plane in each soil layer (i.e., along the planes **a-c** and **b-d** in Fig. 3.1) and at joints between vertically adjacent panels.

<u>The Coulomb method (Lee et al., 1973) or the upper bound method (Bolton and Pang, 1982):</u> Typical of this method is the two-wedge method used for GRS-RWs (e.g., Jewell 1991, Jewell et al., 1984) and for nailed retaining walls (e.g., Gassler and Gudehus, 1981, Gassler, 1988, Stocker and Riedinger, 1990). The overall equilibrium between the total earth pressure and the total available tensile force of the reinforcement layers crossing a potential failure plane is examined for the whole wall height as (**Fig. 3.2**):

$$\Sigma (T_{max})_i \text{ (force per one column of strip)} > \int p(z)\cdot dz\} \cdot S_H \quad \text{(for strip reinforcement)} \quad (3\text{-}2a)$$

$$\Sigma (T_{max})_i \text{ (force per width)} > \int p(z)\cdot dz \quad \text{(for planar reinforcement)} \quad (3\text{-}2b)$$

where $(T_{max})_i$ and $p(z)$ are the maximum available tensile force along the i-th reinforcement layer and the horizontal earth pressure at the elevation z, both activated at the failure plane. Generally when the material has a trend of post-peak softening, the peak strength is not mobilized at the same time along the potential failure plane. Therefore, if the peak strengths of soil and reinforcing member are used in this type of stability analysis, the result is always on the unsafe side, of which the degree increases as the degree of post-peak softening increases. This issue in the bearing capacity of footing on unreinforced and reinforced level grounds and slopes of cohesionless soil has been discussed in detail by Tatsuoka et al. (1991), Huang and Tatsuoka (1990, 1992) and Huang et al. (1991). Therefore, the use of this method may in principle be adequate only when the failure is not catastrophic as in the case where the rate of post-peak reduction in the strength of reinforcing member is small or moderate as in the case where the failure is caused by the tensile rupture of extensive reinforcing member or by the pull-out failure of reinforcement. In addition, in some cases, even when Eq. (3-2) is satisfied, the reinforcement layers at higher elevations may not be extended into the stationary anchoring zone beyond the potential failure plane (**Fig. 3.2b**). In this case, it is assumed that horizontal shear stresses which are mobilized along the horizontal planes in the active zone are large enough to transmit the earth pressure activated at higher elevations towards the reinforced zone at lower elevations. Namely, this method expects that the active zone behaves more-or-less like a monolith, while the lower bound method does not.

Considering the above two points, it is reasonable to use reduced strengths of soil and reinforcement, such as the residual shear strength of soil (e.g., Jewell, 1991 for GRS-RWs and Gassler and Gudehus, 1981, for nailed retaining walls) and the allowable tensile rapture strength of reinforcement divided by a material safety factor as in most of the current design methods. Therefore, overall safety factors thus obtained tend to underestimate the actual values, as supported by many observations for actually constructed walls in which the measured tensile strains in reinforcing members are very low. For example, measured strains in geogrid and geotextile in GRS-RWs were generally less than 1 % (e.g., Simac et al., 1990, Allen et al., 1992), which are much less than the values assumed in the design (say 5 %). Fukuda et al. (1986) also reported that measured strains in the geo-grid for a 7 m-high GRS-RW with a sand (Shirasu) backfill were as small as 0.15 - 0.3 %, which were only 9 - 26 % of the values estimated by the limit equilibrium-based design method by Jewell et al. (1984).

<u>Roles of earth pressure activated on the back face of facing:</u> It is common for the above two methods that the possibility of local compressional failure in the soil immediately behind the wall face is not examined. This assumption is justified, however, only when soil is strong enough while a rigid facing is not used, or when this type of local failure is prevented by applying sufficient confinement by using an adequate rigid facing. Namely, without this confinement, the soil immediately behind the wall face, in particular cohesionless soil, may locally collapse, which may induce a progressive failure towards the inside (**Fig. 3.3**), and may finally trigger the overall failure of the wall.

A sufficient amount of earth pressure on the back face of the facing is needed also for retaining the active zone, particularly at lower elevations. Although this earth pressure is implicitly assumed to exist in the current design methods, it is not always the case. The actual maximum available tensile force in the reinforcement T_{max} is obtained as:

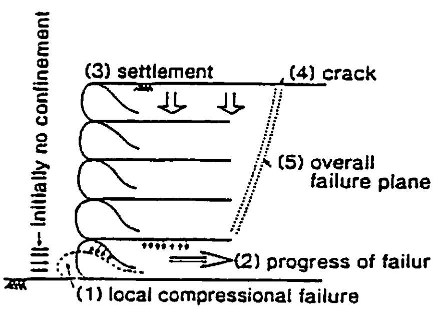

Fig. 3.3 Illustration for progressive failure starting from a local compressional failure of soil immediately behind the wall face.

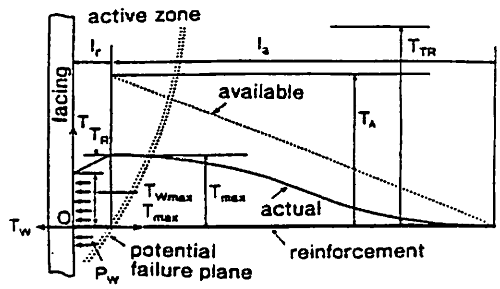

Fig. 3.4 Schematic diagram to explain T_{TR}, T_A, T_R, $T_{W.max}$ (when $T_{TR} > T_A > (T_R + T_{W.max})$).

T_{max}= minimum of $\{T_{TR}, T_A, T_R+T_{W.max}\}$ (3-3)

Namely, the value of T_{max} is equal to the smallest one among the following three components (Fig. 3.4); a) the tensile rupture strength T_{TR} of reinforcing member, b) the maximum available anchoring strength T_A, which is the sum of the bond strength which can be mobilized on the surface of reinforcement for the length l_a embedded in the stationary anchoring zone, and c) the sum $(T_R+T_{W.max})$. T_R is the maximum available retaining force, which is the sum of the available bond strength on the surface of reinforcement for the length l_r embedded in the active zone between the connection at the back face of the facing and the potential failure plane, and $T_{W.max}$ is the maximum available tensile force in reinforcement which can be mobilized at the connection to the back face of facing, which is equal to the maximum available earth pressure (i.e., the passive earth pressure) integrated for the part of the back face of facing surrounding the reinforcement layer concerned. If the reinforcement is not connected to a rigid facing, or the wall face is not covered with any rigid facing, T_W= 0.0. If the value of $T_R+T_{W.max}$ is the smallest among the three components as illustrated in Fig. 3.4, T_{max} becomes equal to $T_R+T_{W.max}$. This situation is more likely to occur at lower elevations in a wall, because of shorter lengths of l_r. Then, the possible failure mode is such that by the activated earth pressure, the active zone is pushed out getting apart from the stationary anchoring part of the backfill or slope leaving reinforcing members anchored in them. When such sufficiently large earth pressure p_w can be activated on the back face of the facing so that T_{max} is controlled by T_A or T_{TR}, it is considered that this type of facing has local rigidity.

Ideal distribution of tensile force in the reinforcement: Fig. 3.5 illustrates the two extreme distributions A and B of reinforcement tensile force T. Note that no active zone exists, for example when a rigid facing is pulled outwards by external force applied directly to the facing and the maximum tensile force T_{max} is activated at the back face of the facing. For the pattern A, the tensile force in reinforcement T in the active zone is constant and equal to the maximum tensile force $(T_{max})_A$, and beyond the potential failure plane, T decreases towards zero at the deepest end. In most cases, the tensile force in reinforcement at the back face of the facing T_W

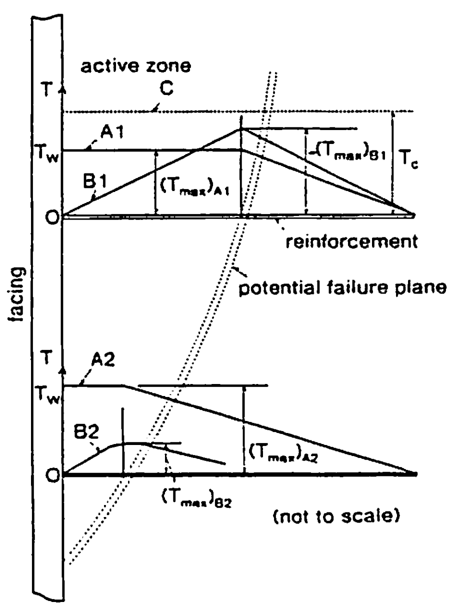

Fig. 3.5 Schematic diagram showing two distributions (the patterns A and B) of the tensile force in reinforcement for a reinforced or nailed soil structure, and the pattern C for an anchored structure.

may be much smaller than its maximum available value $T_{w,max}$. On the other hand, for the pattern B, as approaching to the back face of the facing, T decreases from the maximum tensile force $(T_{max})_B$ at the potential failure plane to zero at the back face of the facing; i.e., T_w= 0.0. Actual patterns may be between the two, and may become closer to the pattern A; 1) as the facing rigidity increases, 2) as the elevation of reinforcement layer concerned becomes lower, 3) as the degree of compaction of soils in the active zone, particularly near the back face of the facing, becomes better, 4) as the reinforcement is placed better-stretched, and 5) as the total stiffness of reinforcement layer becomes higher.

The pattern B is realized only when the wall face is exposed to the atmosphere, or when a flat wall face of each soil layer is covered with a flexible sheet with zero local rigidity, or when reinforcement is not connected to the back face of a rigid facing or a conventional retaining wall structure. On the other hand, Jewell (1990; 7.2.1 Influence of the face) pointed out that the pattern A is possible for "an ideal facing". He further considered that as the pattern changes from A to B, the value of T_{max} at higher elevations increases; namely, referring to Fig. 3.5, along the potential failure plane, the reduction from $(T_{max})_{A2}$ to $(T_{max})_{B2}$ at lower elevations should be compensated by the increase from $(T_{max})_{A1}$ to $(T_{max})_{B1}$ at higher elevations. In other words, the decrease in T near the facing means the decrease in the horizontal stress σ_h in the soil near the facing, which in turn reduces the vertical stress σ_v, and it should be compensated by the increase in σ_v in deeper places in a similar way to the change for T from the pattern A1 to the pattern B1. Then, the horizontal stress near and at the potential failure plane increases, which in turn increases T_{max}.

The pattern B is more preferable from a viewpoint of one of the most characteristic features of reinforced soil retaining walls that only very small earth pressure is activated on the back face of the facing. On the contrary, for a less deformable and more stable active zone, which leads to a more stable wall, the pattern A is more preferable, since this can apply large confining pressure to the active zone. Indeed, the pattern A is similar to that along a tendon in an anchored structure (the pattern C, Fig. 3.5), for which a rather uniform horizontal confinement corresponding to a constant tensile tendon force T_c is applied to the adjacent soil along the whole length of tendon. The magnitude of T_c depends on the magnitude of initial preload and subsequent deformation of the wall. On the other hand, on the back face of a conventional RC retaining wall structure, the active earth pressure or a similar value may be activated. Therefore, for both anchored soil structures and the backfill of conventional RC retaining walls, a local compressional failure is unlikely to occur in the soil immediately behind the wall face.

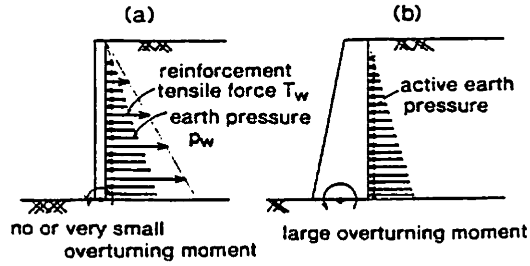

Fig. 3.6 Schematic diagram of the earth pressure and force distribution on the back face of (a) the facing for reinforced or nailed structures, and (b) the conventional RC retaining walls (smooth back faces of the facing are assumed).

In summary, the pattern A with T_{max} as large as possible is most preferable from a viewpoint of the stability of reinforced and nailed soil structures. In that case, the earth pressure in the active zone may range from a value not too much smaller than the active earth pressure to the value similar to the pressure at rest depending on the details of wall structure and construction procedures. To this end, the facing should be rigid enough. One may consider that then the first advantage of the reinforced earth retaining walls over the conventional RC retaining walls is lost, noting that large earth pressure p_w on the back face of a conventional RC wall structure means large overturning moment around the bottom of wall structure (Fig. 3.6b). For the case of reinforced and nailed soil structures, however, the increase in p_w can be balanced by the corresponding increase in the tensile force T_w in reinforcement at the connections to the back face of facing, therefore, it does not mean the increase in the overturning moment around the facing bottom (Fig. 3.6a). Therefore, for a similar earth pressure distribution along the wall height, the structure of a full-height continuous rigid facing for a reinforced or nailed soil structure can be much simpler than that for a conventional RC retaining wall structure. Namely a facing is supported by reinforcement layers at many elevations with a span being much smaller than that for a conventional RC retaining wall. This is like the comparison between a cantilever having a large free length and a continuous beam supported at many points with a small span. In short, soil reinforcing methods cannot be best characterized by a large reduction in the earth pressure acting on the back face of facing.

4. OBSERVATIONS OF EARTH PRESSURE ON THE BACK FACE OF RIGID FACING

R. E. retaining walls: Schlosser (1990) reported the distributions of tensile force along reinforcement in R. E. retaining walls having a standard discrete concrete panels facing. **Fig. 4.1a** shows the records during back-filling and **Fig. 4.1b** shows those in a full-scale model test having a relatively short reinforcement. Common among the above, the ratio of the reinforcement tensile force T_w at the back face of facing to the maximum tensile force T_{max} is generally large, not smaller than 50 %, and is larger at lower elevations in the wall. A result of a FEM analysis (**Fig. 4.1c**) also shows clearly that the ratio T_w/T_{max} increases with facing rigidity. Schlosser (1990) stated that "for standard reinforced concrete facings panels, and for depths greater than 0.6H, it was found that the ratio can approach one". These results clearly indicate that discrete concrete panels facings can actually confine firmly the active zone with earth pressure nearly equal to the active earth pressure. Bolton and Pang (1982) also showed that in centrifuge tests of a model R.E. retaining wall having a metal facing, the ratio T_w/T_{max} ranged from 50 % to about 100 %, which suggests that prototype metal facings for R. E. retaining walls also have a degree of local rigidity.

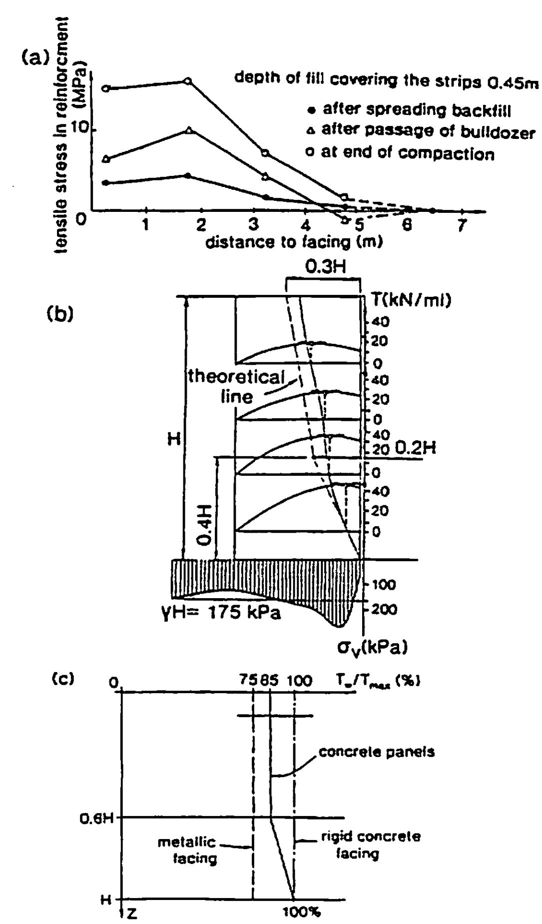

Fig. 4.1 Distribution of tensile force along reinforcement for R. E. retaining walls; (a) measured during back-filling, (b) measured for a test wall (H= 6 m), and (c) the effect of facing rigidity by a FEM analysis (Schlosser, 1990).

Geosynthetic-reinforced soil retaining walls (GRS-RWs): **Fig. 4.2** shows strain distributions along non-woven geotextile layers at various times in a clay GRS-RW having a precast discrete panels facing, which is part of 5.5 m-high Test Embankment No. 3 (Nakamura et al., 1988, Tatsuoka et al., 1991, 1992). Each panel is connected to one layer of reinforcement. It may be seen that a degree of confining pressure is mobilized on the back face of the facing. The wall has been very stable for more than six years. On the other hand, **Fig. 4.3** (Allen et al., 1992) shows that at higher elevations in a 12.6 m-high retaining wall having a sand backfill reinforced with a non-woven geotextile, the tensile strain in the geotextile drops to a very small value as approaching the wrapped-around wall face, which presumably corresponds to a reported relatively large deformation at the wall face (Type B1). Fukuda et al. (1986) observed strains in geogrid layers in a 7 m-high GRS-RW with a sand (Shirasu) backfill reinforced with a polymer geogrid having a vertical spacing of 1 m (**Fig. 4.4**). The wall had an average slope of 1:0.2 in V:H with a Type B2 facing made by wrapping around a stack of four sand backs for each sand layer. A trend of the drop in the grid strain as approaching the wall face may be seen. The degree of drop, however, is more moderate than that seen in Fig. 4.3, probably due to that the Type B2 facing was more rigid.

The ideal pattern A of reinforcement force distribution (Fig. 3.5) can be seen seemingly best in the result of a laboratory test on a 1 m-high GRS-RW model with a dry sand backfill (ϕ = 38°, D_r= 45 %) reinforced with only one layer of polymer geogrid, which was connected to the back face of the facing (**Fig. 4.5**, Fukuda et al., 1986). The unit full-height facing hinged at the bottom was propped during filling. An equilibrium was obtained after the facing was rotated to an angle of 1/330 upon the release of the propping. As seen from the relations denoted by the letter **a**, reinforcement tensile strain is almost constant in the active zone, while it decreases with distance from the facing. Further, the earth pressure on the back face of the facing is almost equal to the active earth pressure. In contrast to the above, when the grid is not connected to the back face of the facing, at an angle of 1/300 of the facing rotation, a pattern B appeared; i.e., the tensile strain in the grid at the back face of the facing is zero (the relation denoted by **b**). The earth pressure on the back face of the facing is reduced largely from the active earth pressure, but the facing being supported by some external propping.

Nailed retaining walls: **Fig. 4.6** shows the result from a full-scale test of nailed retaining wall constructed in a natural dense sand (Gassler and Gudehus, 1981, Stocker et al., 1979). It is reported that the earth pressure due to the self-weight of soil amounted to about 50 % of the estimated Coulomb value (**Fig. 4.6a**), while the earth pressure due to surface loading reached about 70 % of the Coulomb value (**Fig. 4.6b**). Corresponding to the above, large tensile force due to surface loading was mobilized near the wall face (**Fig. 4.6c**), which was the maximum along the length of each nail at lower elevations as the failure plane approached the wall face. Summarizing many observations in Germany since 1975, Stocker and Riedinger (1990) stated that in the design, "the earth pressure onto the wall face may be assumed with a uniform rectangular distribution", and "its magnitude is on the order of 0.4 to 0.7 times the

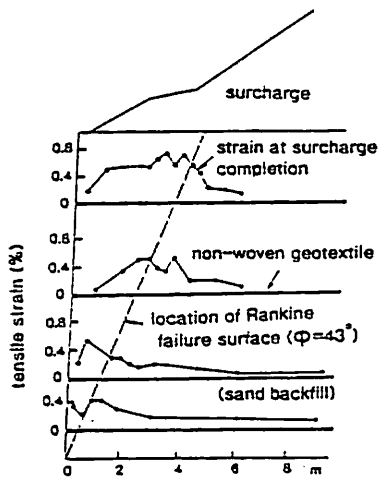

Fig. 4.3 Strain distribution along reinforcement in a GRS-RW (simplified from Fig. 10 of Allen et al., 1992).

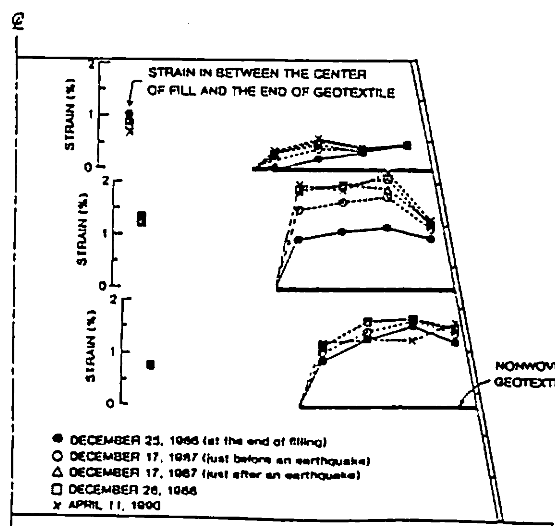

Fig. 4.2 Strain distributions along nonwoven geotextile layers in the wall with a clay backfill having a discrete panel facing (Fig. 8, Tatsuoka et al., 1991).

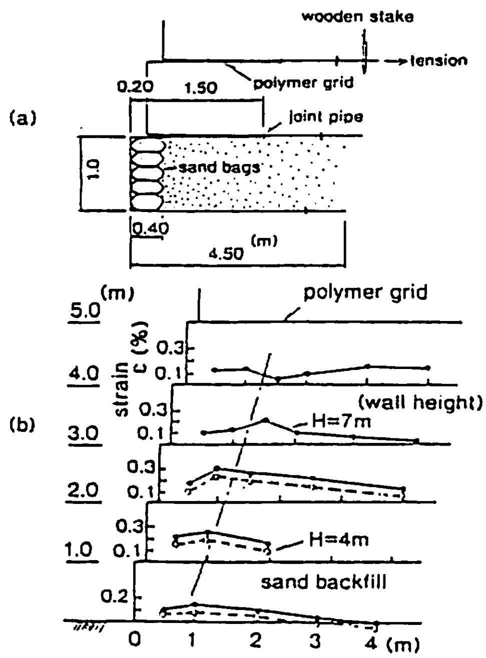

Fig. 4.4 Strain distribution in the grid in a 7 m-high GRS-RW (Fukuda et al., 1986)

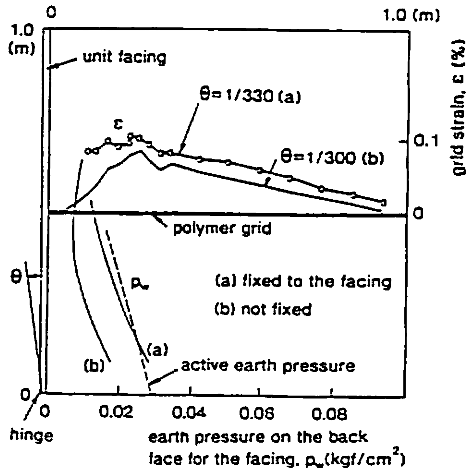

Fig. 4.5 Tensile strain in the grid and earth pressure on the back face: relations a for the grid connected to the back face of the facing, and relations b for the grid unconnected to the facing (Fukuda et al., 1986)

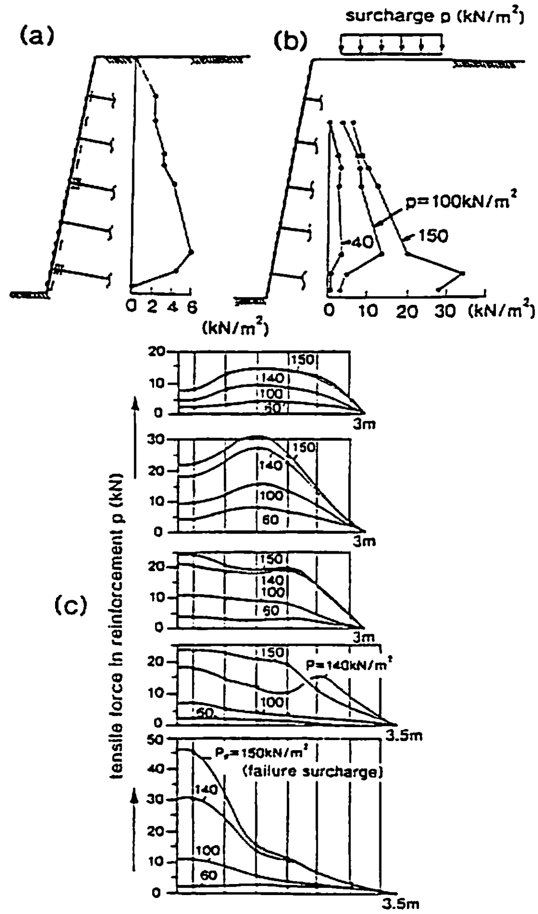

Fig. 4.6 Results of a full-scale test of nailed retaining wall in sand; (a) earth pressure on the back face of the wall due to the dead weight of soil, (b) earth pressure due to the surface load (Gassler and Gudehus, 1981), and (c) the corresponding distribution of tensile force in reinforcement due to the surface load (Stocker et al., 1979).

active Coulomb's earth pressure." Despite that this order of earth pressure is less than the active earth pressure, it is certainly sufficient to confine the soil immediately behind the wall face so as to resist against overburden. At the same time, this relatively high earth pressure can be realized only with such a facing as a shotcrete lining which is strong and rigid enough to resist against this magnitude of earth pressure. In the German practice, shotcrete linings are designed for earth pressure of 85 % of the active Coulomb's earth pressure at the back face of the wall (Stocker and Riedinger, 1990).

Schlosser and De Buhan (1991) reported, on the other hand, that for the same vertical spacing S_v (= 1.5 to 2 m) of reinforcement, the ratio T_w/T_{max} is about 0.25 for soil nailing, which is smaller than about 0.8 for R. E. retaining walls (presumably both values are due to the self-weight of soil). They argued that the former smaller value is due to that the soil in the vicinity of the exposed face yields outwards to some extent before installing nails. This would suggests that if a sufficient amount of confining pressure can be applied by some pre-propping measures made before the excavation proceeds, the ratio T_w/T_{max} will increase, which will decrease the deformation of slope as discussed in Section 8.

In most of the current stability analyses for nailed retaining walls, the term $T_H + T_{w,max}$ is not included in Eq. (3-3), assuming implicitly that a sufficient amount of the earth pressure is activated onto the back face of wall which is needed for retaining the active zone (e.g., see Fig. 17 of Stocker and Riedinger 1990). Again, it should be emphasized that this assumption is justified only when the heads of nails are connected to a facing which is strong and rigid enough.

The importance of the local rigidity of facing for the stability of nailed retaining walls has been demonstrated inversely by several reported cases of local compressional failures in unreinforced cohesionless soil layers not covered with a shotcrete lining following an excavation phase. Plumelle et al. (1991, 1991a) reported a full scale model test (Fig. 4.7). In this

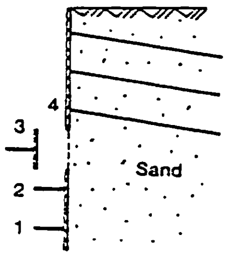

Fig. 4.7 Test layout of a full-scale nailed sand retaining wall (Plumelle et al., 1991, 1991a).

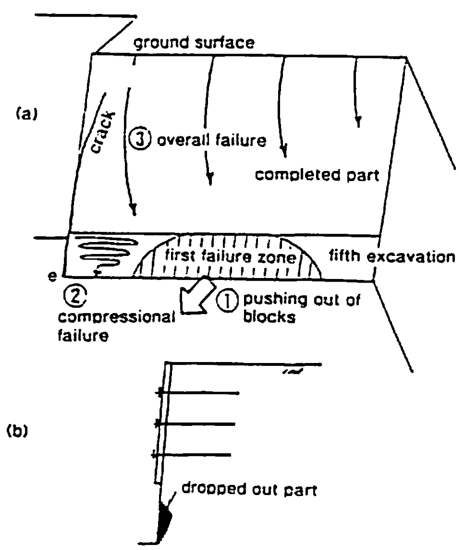

Fig. 4.8 (a) Sketch showing a failure in a nailed wall under excavation, and (b) schematic diagram showing a local failure (Kimura, 1989).

test, 1 m-high facing panels Nos. 1, 2 and 3 had been supporting the unreinforced part of sand backfill located immediately below the reinforced part confined with the facing No. 4. They were removed one by one in the sequence of 3, 2 and 1. When the panel 1 was removed, a local compressional failure occurred in the soil, which led to the overall failure of the wall.

Fig. 4.8 shows a similar failure occurred during the excavation for a temporary nailed retaining wall in a partially saturated uncemented sand of the Pleistocene Era (Kimura, 1989). The sand was relatively weak with estimated values of c= 0.06 kgf /cm^2 and ϕ = 32°. Part of the cohesion was quickly lost by drying while exposing to the atmosphere after excavation. The depth of one excavation was 1.5 m. After having excavated the fifth layer from the top but before nailing, when placing a shotcrete lining on the exposed surface for a horizontal length of 36 m, a local compressional failure started in the sand immediately behind the wall face as denoted by 1, which triggered another local failure in the adjacent zone (2), which in turn induced the global failure in the entire wall (3). Summarizing this and other several similar failures at this site, Kimura (1989) stated that the occurrence of local failure in un-nailed soil layer became more often as the excavation depth increased, and accordingly the scale of failure became larger. The occurrence of this type of failure may be restrained largely by; 1) limiting the horizontal length of exposed excavated soil layer (in the above case, a length of 36 m may have been too large), 2) reducing one cut depth, or 3) casting a shotcrete facing as soon as possible after the excavation of each soil layer. In some practice, a shotcrete lining is casted after installing nails. however, this procedure would be adequate only for relatively stiff soils having a degree of cohesion.

5. CLASSIFICATION OF FACING TYPES

The local and global stability of reinforced and nailed soil structures increases not only by local rigidity of facing but also various kinds of overall rigidities (Table 5.1). Namely, 1) by **overall axial rigidity**, axial force can be transmitted through a facing, 2) by **overall shear rigidity**, shear stress can be transmitted through a facing and, 3) by **overall bending rigidity**, bending moment can be transmitted through a facing. Note that when a facing has overall bending rigidity, it has also overall axial and shear rigidity. 4) Some facings may have a degree of **gravity resistance** by their own weight, which also increases the stability of the structures.

Various facing types are classified according to the degree of facing rigidity summarized in **Table 5.1** and **Fig. 5.1** (both revised from those presented in Tatsuoka et al., 1989, 1992). In this note, Type A facings are those for which a flat soil face is wrapped-around with a geotextile sheet. The flat soil face can be constructed only by using a clay having a degree of cohesion (see Figs. 6.5 through 6.7). The facing types B1, B2 and B3 have different degrees of **local rigidity**. Type B1 is wrapped--around wall faces with each soil layer compacted so that the end face has a stable rounded shape (n.b., this is not simple to be achieved in actual con-

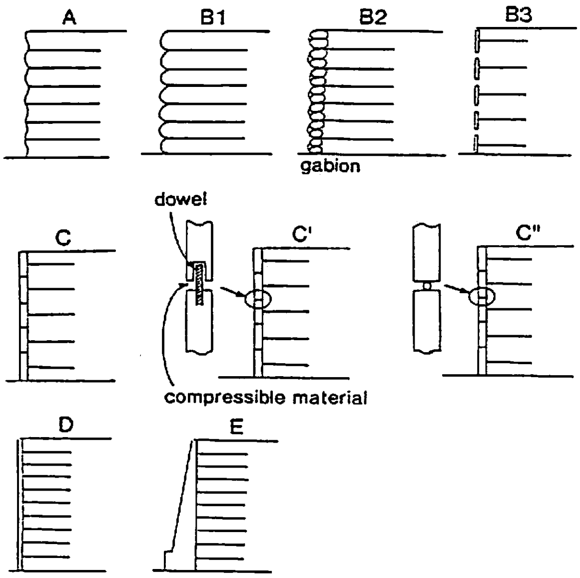

Fig. 5.1 Schematic diagram showing various facing types for vertical walls.

Table 5.1 Classification of facing types according to the facing rigidity

FACING TYPE / FACING RIGIDITY	A	B1	B2	B3	C	C'	C"	D	E
Local rigidiy	×	△	□	○	○	○	○	○	○
Overall axial rigidity	×	×	×	×	○	×	○	○	○
Overall shear rigidity	×	×	×	×	○	○	×	○	○
Overall bending rigidity	×	×	×	×	×	×	×	○	○
Gravity resistance	×	×	×	×	×	×	×	×	○

× means that this facing type lacks this kind of facing rigidiy.
△ means that this facing type has this facing rigidiy only slightly.
□ means that this facing type has this facing rigidiy moderately.
○ means that this facing type has this facing rigidiy sufficiently.

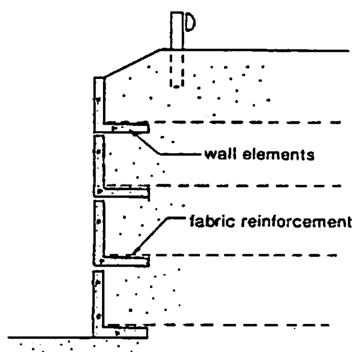

Fig. 5.2 Fabric-reinforced soil retaining wall with a L-shaped block facing (Broms, 1978)

struction). Type B1 has the least local rigidity. Typical of Type B2 is metal skin facings used for R. E. retaining walls, which has larger local rigidity than Type B1. Typical of Type B3, which has the largest local rigidity among Types B1-B3, is discrete precast concrete panels facings having a compressible material at joints between vertically adjacent panels while without using shear dowels at the joints. If the L-shape blocks (Fig. 5.2, Broms, 1978, 1988) are placed so that the contacts between vertically adjacent blocks are very loose, this type of facing is classified into Type B3.

Typical of Type C is precast discrete panels facings having a rough back face while adjacent panels are directly in contact with each other so that the axial and shear load can be transmitted through the facing. Type C facings, therefore, have overall axial and shear rigidities in addition to the local rigidity. The standard cruciform concrete panel facings for R. E. retaining walls have a soft material at the joints between vertically adjacent panels so that only a small amount of axial force is transmitted through the facing. However, since out-of-plane relative horizontal displacement between vertically adjacent panels is prevented by using dowels, the facing has overall shear rigidity, but not very large overall bending rigidity. Therefore, this type of facing is different for Type C facings, and may be classified into Type C'. Concerning the overall axial rigidity of facing, as the wall height increases, a facing of small size blocks such as modular concrete blocks may buckle by the axial force in the facing and the earth pressure activated on the back face of the facing. In that case, the buckled part of the facing has lost the confinement to the soil. On the other hand, each discrete concrete panel of the facing for R. E. retaining walls is usually supported by two layers of reinforcement (Fig. 2.2), which provides each panel with large resistance against rotation, which leads to large resistance of the whole facing against overall buckling. Finally if moment and shear force cannot be transmitted between vertically adjacent discrete panels while only axial force is transmitted, this type of facing can be classified into Type C".

Full-height precast or cast-in-place concrete facings which are connected to reinforcement at the back face have both local rigidity, and overall axial, shear and bending rigidities. This type of continuous rigid facing is classified into Type D. Even a thin lightly steel-reinforced concrete panel is unlikely to easily buckle under large axial force in the facing by being supported by both many reinforcement layers at many elevations and the earth pressure activated continuously along the wall height (**Fig. 3.6a**). This is similar to the case that a long slender steel pile has large resistance against the buckling by axial force when placed under ground (e.g., Bjerrum, 1957).

Type E facings are full-height continuous rigid ones having a degree of gravity resistance as the conventional gravity type retaining walls, but to a lessor extent. Full-height cast-in-place concrete facings having some slope angle at the outer wall face directly placed on a vertical wall face of a wrapped-around GRS-RW (Fig. 2.9) and those casted on an inclined surface of shotcrete lining of a nailed retaining wall have a degree of gravity resistance (Bruce and Jewell, 1987b, Tateyama et al., 1992), and therefore, can be classified into Type E. Tronderblock precast concrete facing (**Fig. 5.3**, Jones, 1988, 1991, Knutson 1990) has also a degree of gravity resistance, while the overall bending rigidity may not be very large.

The classification of facing types discussed above depends on the mechanical properties of facing in the vertical direction. The horizontal longitudinal overall rigidity is another factor which contributes to the stability of wall, particularly against concentrated load (or point load), for example, by a long foundation as shown in Fig. 2.9. The facing shown in Fig. 2.9 has this type of longitudinal overall rigidity as demonstrated by the results of full-scale concentrated loading tests as described in the next section. Timber facings as used by State of Colorado Department of Transportation (Wu, 1991) have not a sufficient degree of overall bending rigidity in the vertical direction similarly to discrete panels facings, but they have a larger degree of horizontal longitudinal rigidity than discrete panels facings.

Effects of facing rigidity in stability analyses: The effects of facing rigidity on the stability of reinforced and nailed soil retaining walls are considered herein based on a two-wedge method (**Fig. 5.4**), which is one of its many variations. In this method, the smallest safety factor is seeked by changing arbitrarily the locations of Point A, which is always located on the back face of the facing, and Point B, and the angles θ_A and θ_B. The plane BF between the two blocks is assumed to be vertical. The smaller one of the two safety factors; the one for sliding out of the two active blocks along the failure planes ABC and the other for overturning of the block ABED together with the facing about

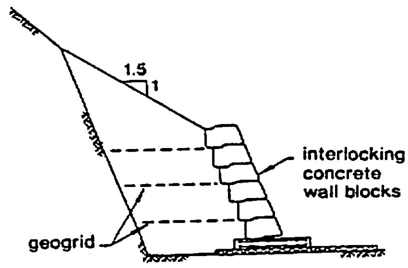

Fig. 5.3 Tronderblock precast concrete facing (Jones 1988, 1991, Knutson, 1990)

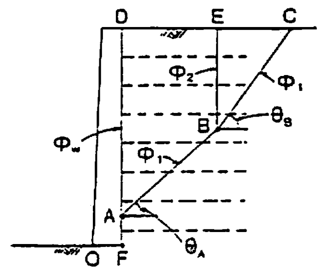

Fig. 5.4 Two-wedge stability analysis method based on the limit equilibrium method for reinforced and nailed soil retaing walls.

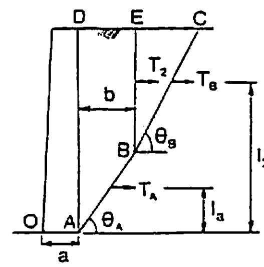

Fig. 5.5 Force components working in the reinforcement.

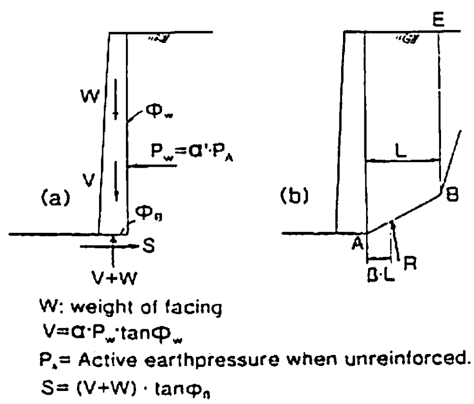

Fig. 5.6 Coefficients α and β to take into account the contribution of the facing rigidity to the stability of the wall.

the toe of facing. Point O, is the safety factor for the wall concerned. The tensile force in reinforcement contributes to the stability of the wall, first by the increase in the resisting horizontal force $T_A + T_B$ and that in the moment $T_A \cdot l_a + T_2 \cdot l_2$ by the tensile force itself (**Fig. 5.5**); each of T_A, T_B and T_2 is the sum of tensile force of several reinforcement layers crossing each segment of failure plane. The second component is the increase in the soil shear strength due to the increase in the normal force on the failure planes AB, BC and BE by the tensile force in reinforcement, $T_A \cdot \sin\theta_A \cdot \tan\phi_1$, $T_B \cdot \sin\theta_B \cdot \tan\phi_1$ and $T_2 \cdot \tan\phi_2$. These increase the resistance against horizontal thrust by $(T_A \cdot \sin\theta_A \cdot \tan\phi_1) \cdot \cos\theta_A + (T_B \cdot \sin\theta_B \cdot \tan\phi_1) \cdot \cos\theta_B$ and that against overturning moment by $(T_A \cdot \sin\theta_A \cdot \tan\phi_1) \cdot a \cdot \sin\theta_A + T_2 \cdot \tan\phi_2 \cdot (a + b)$.

In this method, it is considered that the facing rigidity increases the stability of wall in the following three ways:
- 1) For Types D and E facings having a sufficient amount of overall bending and shear rigidities to support the combination of earth pressure and tensile force in reinforcement activated on the back face of facing, **Point A** is forced to be located at the facing bottom (**Fig. 5.5**). On the other hand, for flexible or incremental facings as Types A - C'', **Point A** may be located at an intermediate elevation of the facing.
- 2) For facings of Type C,. D or E having a sufficient amount of overall axial rigidity, part of the weight of backfill is transmitted to the facing through the frictional force on the back face. According to Clough and Duncan (1971), the reduction of the active earth pressure due to the full wall friction is about 10 % for conventional RC retaining walls having a unreinforced backfill. Then, referring to **Fig. 5.6a**, the axial force V working in the facing may be expressed as:

$$V = \alpha \cdot P_W \cdot \tan\phi_w, \quad P_W = \alpha' \cdot P_A \qquad (5\text{-}1)$$

where P_W is the total earth pressure activated on the back face of the facing, ϕ_w is the wall friction angle, and P_A is the active earth pressure activated on the back face of the facing when the backfill is not reinforced. The laboratory and field tests (Tatsuoka et al., 1989, 1992, Murata et al., 1991, 1992) showed that the coefficient α' is nearly equal to zero for Type A facings, while it is nearly equal to 1.0 for facings having sufficient local rigidity as Types B3 - E facings. Herein, $\alpha' = 1.0$ is used. The coefficient α increases as the overall axial rigidity increases, from zero for Types A and B facings to 1.0 for Types C - E. When the supporting foundation of the facing is able to support the load transmitted from the facing, the increase in α leads to the increase in the reaction V, which in turn develops the reaction S. The reaction S can be transmitted through a facing by the overall shear rigidity of facing. The reactions V and S the decrease the reaction R on the failure plane AB (**Fig. 5.6b**). All of these components increase the stability of the wall against both sliding out and over-turning. Jewell et al. (1992) analysed the effect of the friction on the back face of a full-height facing on the gross horizontal required force for the equilibrium of a geosynthetic-reinforced sand retaining wall. They showed that by the front facing friction, the required reinforcement force could be reduced by as much as about 40 %.
- 3) It is likely that as the facing rigidity increases, the soil behind the back face of facing becomes stronger due to a larger confining effect (therefore, this factor is due to both local and overall facing rigidity). It is also likely that as the facing rigidity increases, a larger amount of stress is transferred from the soil near the facing to the facing. Therefore, the location of the reaction R may be a complicated function of the facing rigidity (**Fig. 5.6b**). The horizontal distance of the location of R from the back face of facing may be expressed as $\beta \cdot L$, where L is the horizontal length of the failure plane AB. The effect of this last factor is the most ambiguous among these three factors

Tatsuoka et al. (1991) and Murata et al. (1992) showed that this method of stability analysis could explain the effect of the facing rigidity as observed in the laboratory and field tests of GRS-RWs.

Fig. 5.7 illustrates the effects of facing rigidity on GRS-RWs as obtained by the two-wedge method described above under the following conditions: the wall face slope is 1:0.05 in V:H, the wall height is 5 m, c= 0, $\phi = 35°$ and $\gamma_t = 2.0$ gf/cm^3 for the backfill soil, the vertical spacing and length of geotextile are 0.3 m and 2.0 m, the tensile rupture strength is 3 tf/m, and the bond friction is equal to ϕ. The surcharge is applied for a width B equal to 2 m either on the reinforced zone as denoted by FL or immediately behind the reinforced zone as denoted by BL. Failure plane does not enter the supporting ground. The effect of the weight of facing is not taken into account. It is assumed that the facing has a sufficient local rigidity to prevent a local compressional failure. It is also as-

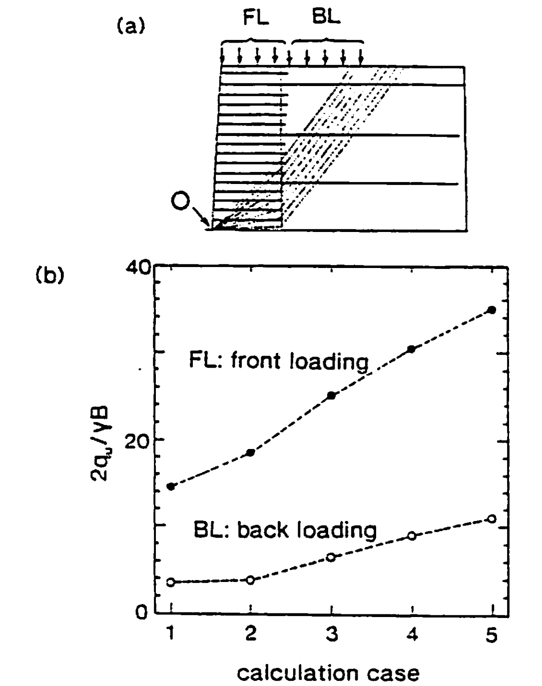

Fig. 5.7 Change in the safety factor for GRS-RWs with the change in the facing rigidity obtained by a two-wedge method (Murata, 1992)

sumed that the value of β is 1/3 for flexible facings of Type A, and decreases as the facing rigidity increases. The following values of α and β are used:

Case	Facing type	Location of Point A	α	β	Failure mode which controls safety factor
1	B	①	0.0	1/3	Sliding out
2	C	②	1.0	1/3	Sliding out
3	-	Point O	0.0	1/3	Overturning
4	D	Point O	1.0	1/3	Overturning
5	D	Point O	1.0	0.0	Overturning

① and ② : Point A is allowed to be located either at any elevation or at every 60 cm increment along the wall height, respectively. Point O is the front edge of the bottom of the wall.

The case 3 corresponds to the case where the reactions at the bottom of a Type D facing cannot be mobilized by some reason, for example, by excavation below or in front of the facing. In Fig. 5.7a, some typical trial failure planes are shown. Fig. 5.7b shows the ultimate surcharge values in terms of $2q_u/(\gamma \cdot B)$. The failure mode, sliding out or overturning, which controls the safety factor in each case, is indicated in the above table. It may be seen that the wall becomes more stable as the facing rigidity increases as expressed by the restriction to the location of Point A and the designated values of α and β.

6. EFFECTS OF FACING RIGIDITY OBSERVED IN LABORATORY AND FIELD TESTS

Typical model test: The effects of facing rigidity were observed typically in the loading tests on 50 cm-high GRS-RW models performed in the laboratory reported by Tatsuoka et al. (1989) (Fig. 6.1). The backfill soil was air-dried Toyoura Sand reinforced with a model grid designed so that a tensile rupture failure would not occur. The facings used are: Type A made of a latex rubber membrane, Type B' made of tracing paper with a small degree of local rigidity, Type B consisting of a stack of wooden blocks having a smooth back face with a compressible material at the joints between vertically adjacent blocks so as to minimize the overall axial rigidity of facing, Type C consisting of a stack of wooden blocks which were in direct contact with each other having a rough back face, and Type D of a full-height continuous rigid unit. These model walls were loaded to failure by means of a 10 cm-wide strip footing with a smooth base. For each type of wall, both **Front loading** from the top of the reinforced zone and **Back loading** from immediately behind the reinforced zone were performed. From the maximum average footing pressures q_u presented in Fig. 6.1, it may be seen that for each loading pattern, the walls were more stable in the order of the facing types D, C, B, B' and A in accordance with the degree of facing rigidity. In particular, for the wall with a Type D facing, the failure surface passed through the facing bottom, whereas for the other facing types, particularly for Type A, the failure surface passed through the wall face at an intermediate elevation. The ratio of the earth pressure p_f on the back face of the facing to q_u did not change significantly with the facing rigidity (except for Type A facing), which means that the value of p_f increased at a large rate with the facing rigidity. For the reaction p_B at the bottom of the facing, both the magnitude and the ratio to q_u increased substantially with the facing rigidity. Fig. 6.2 shows tensile stress per unit wall width (= 1 cm) in the reinforcement activated by q_u. It may be seen that the tensile force, particularly at the connections at the back face of the

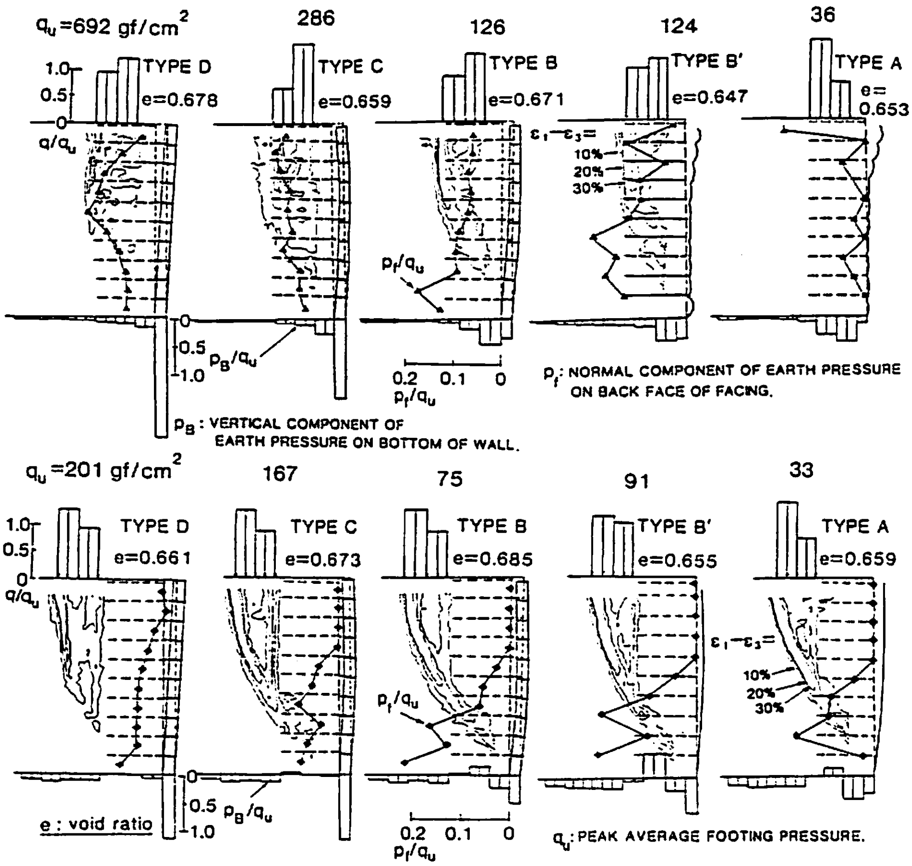

Fig. 6.1 Loading tests of small models of reinforced sand retaining wall having different types of facing (Tatsuoka et al., 1989); e: the mean void ratio of the backfill, the numeral indicated for each contour means shear strain $\varepsilon_1 - \varepsilon_3$ (%) occurred until the peak footing load was activated, q_u; the peak average footing pressure, q: the peak average pressure for each half of the footing base, p_f: the earth pressure at the back face of the facing activated by q_u, and p_B: the pressure at the base of wall activated by q_u.

facing, increased with the facing rigidity. It may also be seen that the location of the largest tensile force T_{max} along each reinforcement layer approached the back face of the facing as the facing rigidity increased. These results indicate that all of the components of the facing rigidity as classified in Table 5.1 can contribute to the stability of the wall.

The effects of facing rigidity on the stability of reinforced and nailed soil structures have been observed also in many other field and laboratory tests as shown below. Further, it has been noticed that the stability of many actually constructed reinforced and nailed soil structures and full-scale and laboratory models which had various degrees of facing rigidity was stronger than anticipated when based on stability analyses ignoring the effects of facing rigidity.

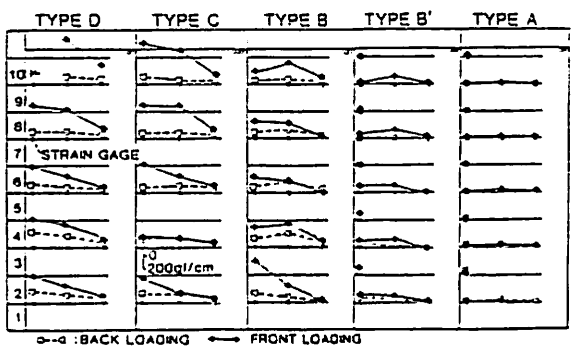

Fig. 6.2 Tensile stress per unit wall width (= 1cm) in the reinforcement activated by q_u (*: the strain gauges were broken due to excessive strains) (Tatsuoka et al., 1989).

R. E. retaining walls: Schlosser and Juran (1978, pp.206) stated that "although the model (metal) facing is rather flexible, it has a given rigidity which does influence the values of the model height at failure. It is essential to measure this effect when analysing the results of reduced scale models." Juran and Schlosser (1978) also inferred based on a theoretical analysis that the overall axial rigidity of the (metal) facing increases the stability of R. E. retaining wall, noting that a log-spiral failure plane as observed in model and prototype walls (Fig. 6.3) can be realized only with some vertical reaction at the bottom of the facing. On the other hand, it is very likely that relatively large axial force can be transmitted through precast discrete concrete panels facings, since a compressible material placed between vertically adjacent panels may not be very soft, and the facing is difficult to buckle by the axial force with each panel being supported with two reinforcement layers.

Bolton and Pang (1982) found in their centrifuge model tests a overall reduction in vertical soil stress behind the facing when a relatively stiff facing made of articulated aluminum panels (presumably between Type B3 and Type C), rather than a flexible foil facing (Type B2) was used. They stated as to their test results that "a suitable hypothesis is (a) stiff articulated facing panels can develop substantial shear stresses on their buried surfaces, (b) the magnitude of these shear stresses is variable, depending on the magnitude of relative vertical displacements, including those due to the closing of the irregular gaps between panels, and (c) that where stiff panels are supported at their base by a stiff foundation, the thrust accumulated in a stack of panels can be sufficient to reduce the vertical stresses over a wide area, by as much as 25 % in the region of greatest stress (pp.359)." They concluded, however, that this effect of facing rigidity "could not be expected to be a reliable and predictable source of strength to the designer unless extraordinary measures were taken on site to control, for example, facing joint sealants and spacers. The simple analysis (which ignores this effect) is therefore recommended for design (pp.366)." To the present author, the above conclusion suggests inversely that if the facing is rigid enough as Type D or E and the foundation of the facing is stiff enough, the effect of the facing rigidity can be taken into account in the design.

Jaber (1989) also found that in his centrifuge tests of 15.2 cm-high R. E. retaining

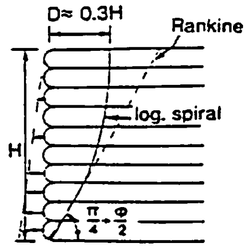

Fig. 6.3 Log-spiral failure plane and wall face displacement typically observed in a R. E. retaining wall (modified from Fig. 4 of Schlosser and Juran, 1980)

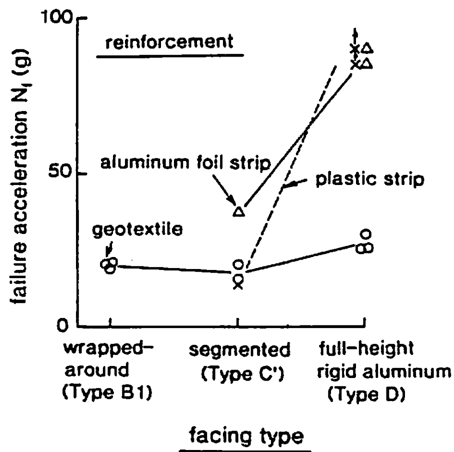

Fig. 6.4 Centrifuge model test data by Jaber (1989) showing the effect of the facing rigidity on the stability of reinforced soil retaining wall models.

wall models with a sand backfill, "full height rigid facing panel walls are much stronger than segmented facing walls" (Fig. 6.4). Namely, when reinforced with a woven-geotextile, the increase in the failure acceleration was moderate as the facing rigidity increased from a wrapped-around wall face (Type B1) to a segmented plastic facing (Type C') and further to a full-height rigid aluminum facing (Type D). When reinforced with aluminum or plastic strips, the increase was significant. Jaber concluded, however, that full height rigid facing panel walls have "disadvantages as they are more difficult to construct." To the present author, this conclusion also suggests that the use of full-height rigid facing is very beneficial, if potential construction problems can be solved.

Geosynthetic-reinforced soil retaining walls (GRS-RWs)

1) Walls with a Type A facing: The 4 m-high test embankment with a backfill soil of

volcanic ash clay reinforced with a non-woven geotextile reported by Tatsuoka et al. (1986, 1986a) had two wrapped-around wall faces with flat soil faces (Type A facing, **Fig. 6.5a**). Of the two walls, the one having an initial vertical spacing of the geotextile layers of 0.8 m failed by a series of heavy rain falls due to the loss of suction (namely the loss of soil strength) by wetting and also due to the lack of the confinement to the soil by the use of a Type A facing. The failure started from a local compressional failure in the zone immediately behind the wall face in the lowest soil layer, which progressed toward the deeper zone as illustrated in Fig. 3.3 and in turn triggered the settlement and outward displacement of the upper relatively undeformed part of the wall while forming vertical cracks from the crest of the central unreinforced zone. A very similar type of failure occurred in a similar 5.5 m-high clay wall having a Type A facing of Test Embankment No. 3 (**Fig. 6.5b**). For that embankment, another wall having a Type C facing (Fig. 4.2) and the other having a Type E facing were very stable under otherwise similar conditions. Further, two 5.2 m-high walls having a Type B2 facing of Test Embankment No. 2 also was very stable for a long duration under the similar conditions as the cases described above (Tatsuoka and Yamauchi, 1986a, Nakamura et al., 1988, Tatsuoka et al., 1991).

Goodings (1989) performed centrifuge tests of 152 mm-high GRS-RW models with a backfill of Speswhite kaolin (S_r= 90 %) either unreinforced or reinforced with a nonwoven geotextile having a tensile strength of 0.7 kN/m (**Fig. 6.6**). The wall face of each clay layer was flat and wrapped-around with a geotextile (Type A). The failure acceleration was doubled by reinforcing for the case shown in Fig. 6.6. In that case, as the acceleration increased, the failure started from local compression in the lower soil layers, which was greatest nearest the front of the wall. This local failure resulted in the outwards bulging of the wall face forming a rounded face for each layer, which led to uneven support for the upper less-compressed portion of the wall, which led, in turn, to considerable forward tilting of those upper portions accompanying a tension crack, which finally triggered the overall failure of the wall breaking geotextiles in the process. This failure mode is surprisingly similar to that observed for the full-scale clay walls shown in Fig. 6.5. Since the similar full-scale clay walls with a rigid facing were much more stable (Fig. 6.8), it is very probable that if a rigid facing connected to reinforcement had been used, the centrifuge clay models would have been much stronger.

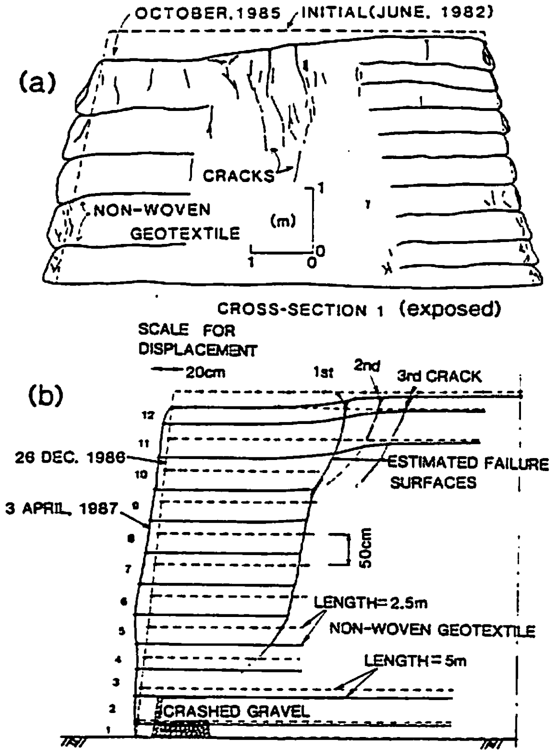

Fig. 6.5 Cross-section of failed clay wall having a Type A facing; (a) exposed cross-section of Test Embankment No. 1, and (b) estimated cross-section of Test Embankment No. 3 (Tatsuoka et al., 1986, 1986a, 1991, Nakamura et al., 1988).

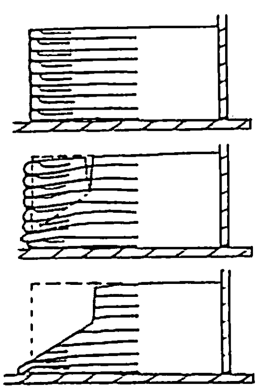

Fig. 6.6 Result of typical centrifuge test showing the sequence of failure for intermediate reinforced clay wall (Series I models) (Goodings, 1989).

Wichter et al. (1986) constructed a 4.5 m-high GRS-RW model (Fig. 6.7) attempting

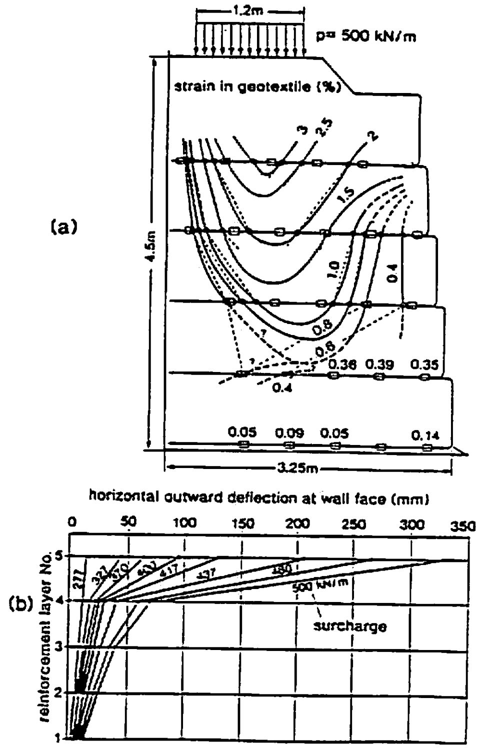

Fig. 6.7 Wrapped-around wall face (Type A) after loading from the crest of the test wall; (a) contours of stains in the geotextile layers, and (b) the outward displacement at the wall face (Wichter et al., 1986).

to reinforce a low-quality soil (weathered clay marl with c= 0.4 kgf/cm^2 and ϕ = 21.5° compacted to γ_t= 2.0 tf/cm^3) with a geotextile considering that a planar geotextile sheet has a much larger contact area with the surrounding soil than metal strips as used for R. E. retaining walls. The wrapped-around wall face had a flat soil face (Type A), which was maintained after construction by a cohesion of the soil. It is reported that the construction of this type of wall face is not easy even in the laboratory, particularly for cohesionless soils. By loading from the crest of the reinforced zone, large deformation occurred only locally at higher elevations in the wall (Fig. 6.7b). At the same time, a very small amount of strain was mobilized in the geotextiles near the wall face (Fig. 6.7a), which is in contrast to the result presented in Fig. 6.2. This behaviour may be not only due to a high compressibility of the soil, but also due to zero local rigidity of the Type A facing used. The latter point is supported also by the fact that a 5 m-high GRS-RW with a clay reinforced with a nonwoven/woven geotextile composite having a Type E facing did not exhibit a local failure when loaded from the crest (**Fig. 6.8**).

2) Walls with a facing having a different degree of rigidity (Types B and C facings): Thamm et al. (1991) performed a loading test on a 2.9 m-high wrapped-around GRS-RW with a near vertical wall face of Type B1 (**Fig. 6.9**). The gravelly sand backfill (c'= 0.08 kgf/cm^2, ϕ = 39° and γ = 2.0 gf/cm^3) was reinforced with a nonwoven-geotextile. By loading from the crest, a local failure occurred only in the upper part of the wall while the observed failure plane was shallower than that predicted by a theory. Bastic (1991b, pp.92-91) claimed that for a similar loading condition, a R. E. retaining wall with a standard cruciform discrete concrete panels facing (Type C') was much stronger than this GRS-RW, because the metal strip be much stiffer than the geotextile. The author believes that this difference is not due totally to the difference in the stiffness of reinforcement, but due largely to the different facing rigidities between the Type B1 and C' facings.

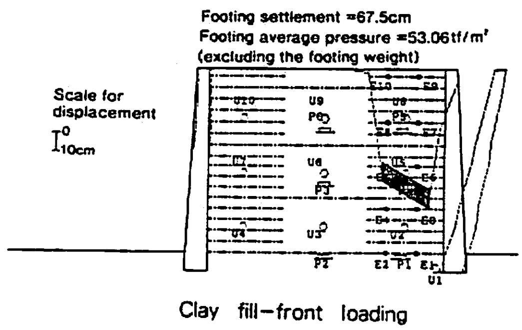

Fig. 6.8 Deformation of a GRS-RW with a clay backfill by loading test: the displacement of the footing is exaggerated (Tatsuoka et al., 1992).

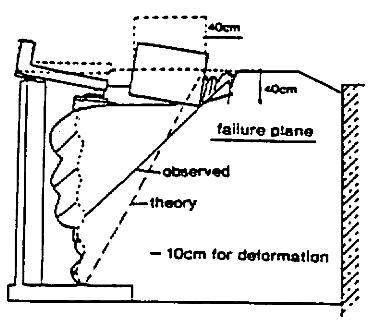

Fig. 6.9 Wrapped-around wall face (Type B1) after loading from the crest of the test wall (Thamm et al., 1990, see Fig. 2.6 for the wall dimensions before loading).

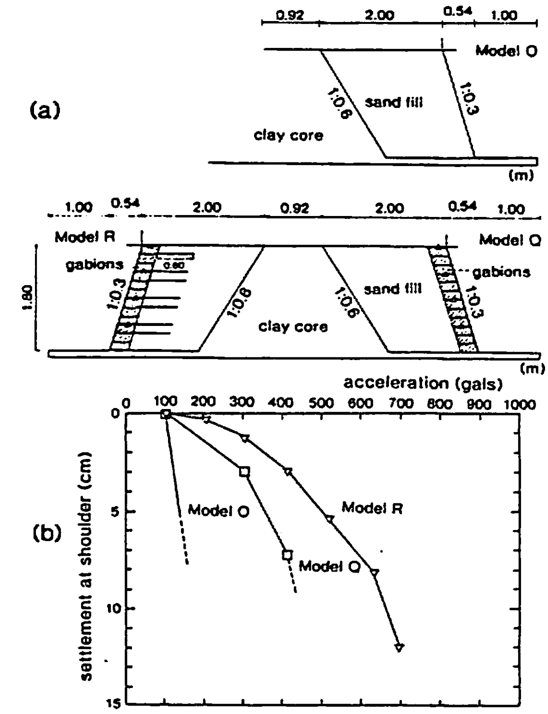

Fig. 6.10 Effects of the use of a gabions facing and geotextile-reinforcing on the stability of sand walls under seismic loading conditions; (a) the test arrangements, and (b) the settlement at the shoulder of the slope versus the horizontal acceleration at the shaking table (Koga et al., 1992).

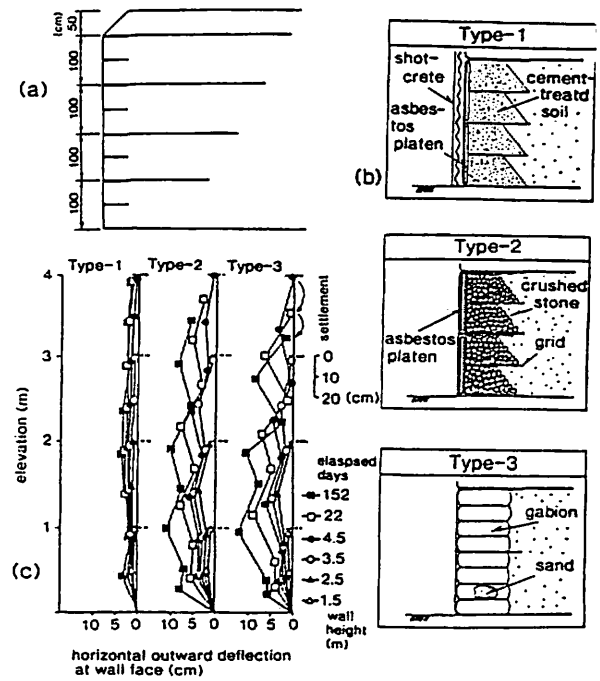

Fig. 6.11 Long-term observations of three GRS-RWs having different types of facing; (a) wall configurations, (b) facing types and (c) the displacement at the wall face (Nakane et al., 1990).

Koga et al. (1992) studied the seismic stability of GRS-RWs by performing shaking tests on 1.8 m-high walls with a slope of 1:0.3 (V:H) (**Fig. 6.10a**). While the core was made of a well compacted silty clay, the slope sand had a mean diameter of 0.25 mm and a fines content of 2.4 % with c= 0.21 kgf/cm^2 and ϕ = 33.5°. A series of horizontal shaking was applied by increasing step-by-step the acceleration at 4 Hz for a duration of 10 seconds at each step. Only by the use of a gabions facing (the gabion wall model **Q**), the stability increased substantially compared to the unreinforced model O (**Fig. 6.10b**). This would be presumably due to both a large compressive strength of the gabions facing at lower elevations where stress was concentrated by the tendency of over-turning of the wall and the effect of the confinement of the gabions to the sand behind them. Resl et al. (1988) suggested that the "gravity wall" effect by a stack of gabions as shown above can be expected also for the soil zone wrapped all around by a geotextile sheet at the shoulder of wrapped-around wall. In the test by Koga et al. (1992), when reinforced with a relatively short non-woven geotextile (the model **R**), the stability increased further.

Miki et al. (1992) performed a full-scale failure test of a 6.25 m-high geogrid-reinforced sand retaining wall with a wall surface slope of 1:0.2 in V:H, in which the layers of geogrid were successively disconnected at several points along the length by an electric method. They showed a significant effect of the use of gabions at the wall face on the stability of wall; namely, a limit equilibrium-based stability analysis ignoring the existence of gabions unduly underestimated the stability of the wall. They proposed a method of limit equilibrium-based stability analysis considering a stack of gabions as a monolith to take into account the effect of gabions.

Nakane et al. (1990) observed for a long duration the behaviour of three 4.5 m-high GRS-RWs having different facing types (**Fig. 6.11**). The backfill was a wet sand backfill (γ_t= 1.89 gf/cm^3, c= 0.08kgf/cm^2 and ϕ =24°) reinforced with a polymer grid. The facing type 1 had, for each soil layer, a stack of four cement-treated sand blocks having a layer of shotcrete on the wall face with two thin asbestos platens in between them (presumably imperfect Type D facing). The facing type 2 had a layer of crushed gravel reinforced with a very short complementary geotextile covered with two thin asbestos platens (presumably Type B1). The type 3 had a stack of gabions (Type B2). The type 1 facing exhibited fairly well, whereas the deformation at the wall face of the other two types was too large to accept. Naemura et al. (1991) confirmed the above result by performing a similar test on two 5 m-high sand walls having two different facing types (**Fig. 6.12**), which were the type 1 of a stack of L-shaped concrete blocks (presumably Type C facing) and the type 2 facing of Type B2. The deformation at the wall face when completed and after applying a surcharge of 0.1 kgf/cm^2 on the crest was larger for the less rigid type 2 facing.

Kawasaki et al. (1990) performed loading tests on three 5 m-high test walls having different types of facing. The backfill was sand having about 13 % fines content with a water content w= 16 % compacted to γ_d= 1.43 gf/cm^3 (a degree of compaction= 90 %) (**Fig. 6.13a**). The first wall I had a wrapped-around facing constructed with the aid of gabions at each soil layer (Type B2 facing), and was reinforced with a FRP grid. The number of reinforcement layers was only two relying on its very large stiffness and strength. The soil zone immediately below the footing was densely reinforced with a short geotextile so as to avoid a local failure in that zone. The second wall II had a 1 m-thick facing made of a cement-treated sand having a design compressive strength of 20 kgf/cm^2 at a curing time of 28 days (imperfect Type E). The third wall III had a thicker cement-treated sand wall as a conventional gravity type retaining wall, while the backfill was unreinforced. When loaded

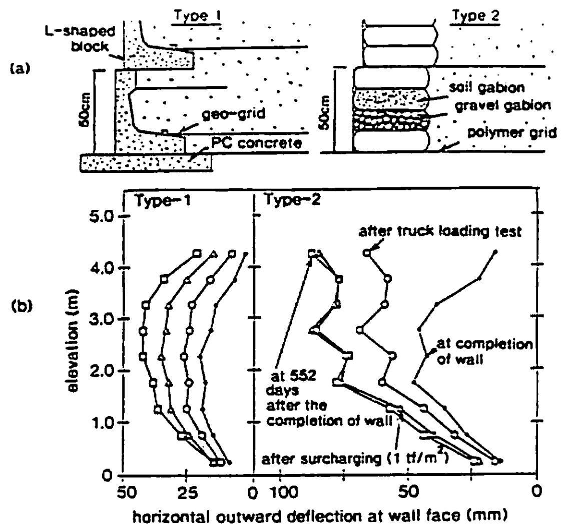

Fig. 6.12 Long-term observations of two GRS-RWs having different types of facing: (a) facing types and (b) the displacement at the wall face (Naemura et al., 1991).

from the crest (Fig. 6.13a), Wall I exhibited a sign of local failure with a failure surface intersecting with the wall face between the top two reinforcement layers (**Fig. 6.13b**), probably due to the too large vertical spacing of reinforcement layers. In Wall II, however, this type of local failure did not occur presumably due to the increase in the facing rigidity. It was also the case for Wall III. This result indicates clearly that in order to make the stability of the GRS-RWs equivalent to that of the conventional retaining walls, it may not be sufficient to reinforce the backfill, but a proper rigid facing should also be used.

For a similar reinforcement stiffness, measured deformation of a 6 m high GRS-RW with a backfill sand reinforced with a grid hav-

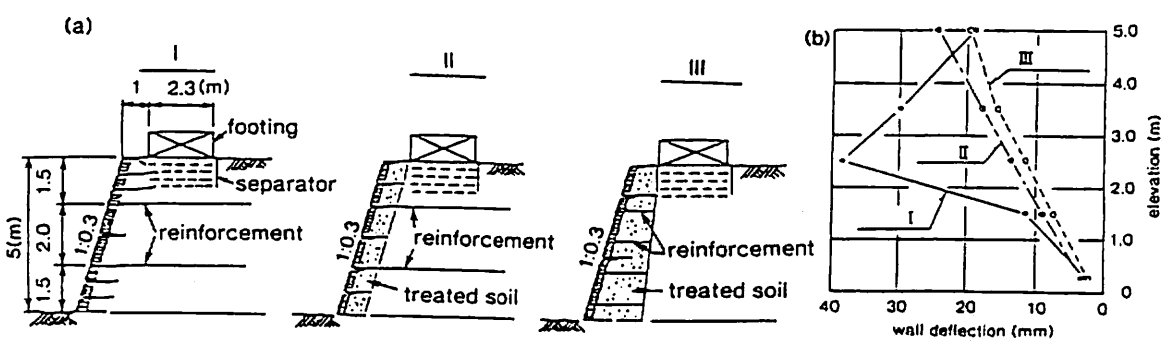

Fig. 6.13 Three test walls loaded from the crest; (a) wall configurations, and (b) the outward horizontal displacement at the wall face by loading of q= 1.3 kgf/cm^2 on an area with a width of 2.3 m on the crest (Kawasaki et al., 1990).

ing a Type C facing of modular concrete blocks (Fig. 2.7, Simac et al., 1990) was as small as 0.3 % of the wall height, when compared to about 1 % for a 12.6 m-high wrapped-around GRS-RW with a backfill sand reinforced with a non-woven geotextile (Allen et al., 1992). Gourc et al. (1991) performed plane strain model tests using GRS-RW models with a backfill of 3 - 5 mm-diameter small aluminum cylinders reinforced with a heat-bonded non-woven geotextile. Under otherwise similar conditions, the failure height of the models having a cellular facing of Type C (Fig. 2.8) was larger by about 30 % than that for the models having a wrapped-around facing of Type B1. The failure was associated with the bulging of wall face, which was to a larger extent for the wrapped-around facing with a failure plane intersecting with the wall face at an intermediate elevation. They also showed that the reaction at the bottom of the cellular facing should be taken into account in stability analyses to explain the test result.

Wu (1992), Claybourn and Wu (1992) and Wu et al. (1992) compared the class A predic-

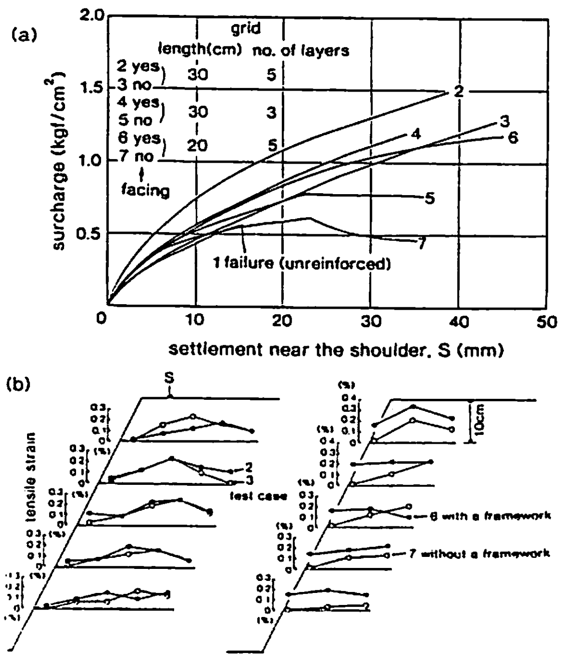

Fig. 6.14 Effects of slope confinement on sand slopes reinforced with a grid with the slope face either un-protected or protected; (a) relationships between the pressure q and the settlement S of the plate closest the slope shoulder, and (b) tensile strains in the grid at q= 1.0 kgf/cm^2 (for Test ⑦, strains after the peak). (Torilhara et al., 1991).

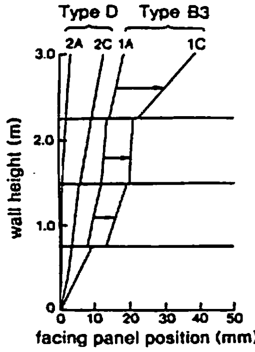

Fig. 6.15 Comparison of the wall facing position before and after surcharging for model GRS-RWs (Bathurst et al., 1988).

tions with the results of the loading tests on two 3 m-high plane strain GRS-RW models with a sand or clay backfill reinforced with a woven-geotextile (Denver Walls). The facing was a stack of 14 cm-high by 10 cm-wide wooden blocks (Type C) and the reinforcement length was relatively short (55 % of the wall height). The walls were brought to failure by applying surcharge from their crests. For the sand wall, despite the use of the residual angle of friction determined by triaxial compression tests (35 degrees), all of the six limit equilibrium-based stability analysis methods currently used in the United States under-predicted the actual failure surcharge by 1/4 - 1/40 times. This discrepancy between the experiments and the analyses would be due largely to that the effects of the facing rigidity are not taken into account in these design methods. In contrast, some of the FEM analyses taking into account the effects of facing rigidity could predict the failure surcharge reasonably.

Summarizing the above, it is apparent that walls having a facing of Type B or C is more stable than those having a Type A facing. This difference could not be evaluated by stability analyses which do not take into account the effects of facing rigidity. When using Type B or C facings, however, depending on loading conditions in each case, the deformation of wall face could be still too large and may not be stable enough against load on the crest, particularly against concentrated load near the wall face.

3) Walls with a facing of Type D or E: Tatsuoka et al. (1991, 1992) and Murata et al. (1991, 1992) showed that a 5 m-high full-scale GRS-RW with a sand backfill having a Type E facing has been much more stable for a long duration more than three

years and was also stronger against loading from the crest, compared to the one having a Type C facing (see Fig. 6.8). Also in the shaking table tests on 1.0 m-high reduced scaled GRS-RW models with a sand backfill, the wall having a Type D facing was much more stable than those having a Type C facing (Murata et al., 1992).

To examine the effect of the confinement applied onto the slope surface, Toriihara et al. (1989, 1990, 1991) performed laboratory loading tests on 60 cm-high models reinforced with either steel bars or a relatively short (either 20 cm or 30 cm) polymer grid having a rapture strength of 1.8 tf/m at an elongation of 14 % (**Fig. 6.14**). In both cases, the backfill was a partially saturated sand of w= 5% compacted to D_r= 80 % to have ϕ = 33.2°. The slope was 1:0.5 in V:H either unprotected (Type A) or protected with a model framework with an aperture of 10 cm (thus, imperfect Type D). Uniform load applied for a length of 62.5 cm on the crest of the slope was increased step-by-step. The slope became stronger not only by reinforcing, but also by applying a confinement (**Fig. 6.14a**; Tests ② versus ③, ④ versus ⑤, and ⑥ versus ⑦). By the confinement, the tensile strain in grid immediately behind the slope surface increased from zero or nearly zero to a relatively large value, which was nearly equal to the maximum value along each grid layer when the reinforcement length was as short as 20 cm (1/3 of the slope height) (**Fig. 6.14b**). Further, Toriihara et al. (1991) suggested based on the test results that the reinforcement length can be made relatively short by placing a framework on the slope face without losing the stability of slope as have been found for GRS-RWs with a near vertical wall face (Tatsuoka et al., 1991, 1992, Murata et al., 1991, 1992).

Barthust et al. (1988) constructed three 3 m-high polymer grid-reinforced sand test retaining walls, two of which had either a facing of 0.75 m-high timber bulkhead panels with a compressible foam placed at each horizontal joint between panels (Type B3) or a full-height timber bulkhead panel facing (Type D), both being propped during the construction of the wall. The walls were brought to failure by uniform surcharging on the crest of the backfill. Near the connections at the back face of the Type D facing, the largest tensile strain or the value close to that was mobilized in each reinforcement layer. That is, the Type D facing provided more restraint to the wall system than the Type B3 facing (Fig. 6.15). They also observed that both before and after surcharging, in the vicinity of the facing at the bottom of the wall, a large amount of vertical soil stresses was transferred to the facing, which was about 25 % even for the Type B3 facing.

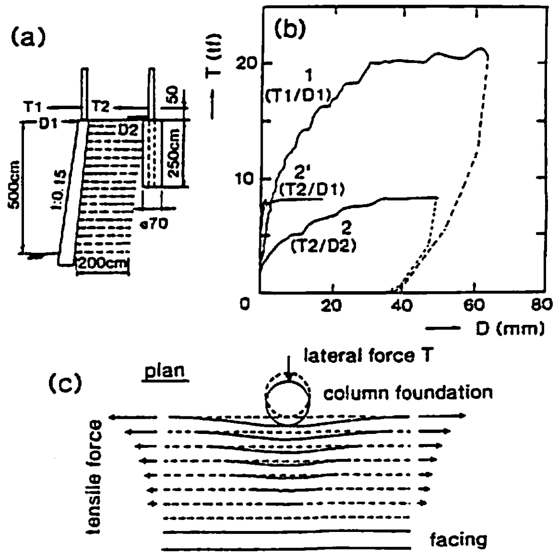

Fig. 6.16 Lateral loading tests of the foundation either fixed on the top of the facing (case ①) or placed immediately behind the reinforced zone (case ②) in a GRS-RW with a sand backfill: (a) test configurations, (b) the relationships between the lateral load T and the horizontal displacement D, and (c) schematic diagram showing the deformation of grid in the case ② (Tateyama et al., 1992a, Tamura et al., 1992).

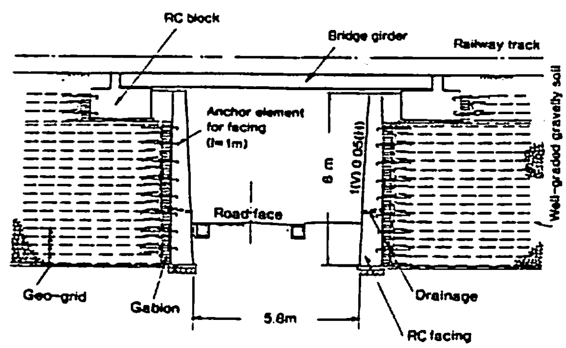

Fig. 6.17 Cross-section of GRS-RWs bridge abutments at Shinkansen Yard, Nagoya (Tatsuoka et al., 1992).

Tateyama et al. (1992a) and Tamura et al. (1992) examined the stability of a column foundation as seen in Fig. 2.9. They performed lateral loading tests on a full scale model foundation constructed either on the crest of the facing (case ①) or immediately behind the reinforced zone (case ②) in two 5 m-high GRS-RW full scale models having a Type E facing of full--height cast-in-place concrete layer (**Fig. 6.16a**). The backfill soil was sand and clay. **Fig. 6.16b** shows the results for the sand wall reinforced with a grid having a tensile rupture strength of 2.8 tf/m. For the both cases, the ultimate lateral load was much higher than the assumed design load of 1 tonf. Further, the foundation was much stronger when fixed to the top of the facing (the relation 1) than when constructed in the unreinforced zone immediately behind the reinforced zone (the relation 2). This is because when fixed on the top of the facing, the lateral load is spread to a wide zone due to the facing rigidity and resisted by the tensile force mobilized in the multiple reinforcement layers. On the other hand, when constructed behind the reinforced zone, only compressive strains were mobilized in the members of the reinforcement (grid) in parallel to the direction of loading (i.e., in the direction perpendicular to the wall face). By the outward displacement of the foundation, however, tensile force are developed in the members of the grid in the direction perpendicular to the loading direction as illustrated in Fig. **6.16c**, spreading the foundation load to the whole height of the wall for a relatively large width, which led to large resistance of the column foundation against lateral load. Further, the displacement at the top of the facing was very small (the relation 2' in Fig. 6.16b). In short, due to several kinds of three dimensional effects of both continuous rigid facing and grid reinforcement, this type of GRS-RWs can retain a column foundation from which relatively large concentrated horizontal load is transmitted.

In summary, many data presented above indicate that under otherwise identical conditions, GRS-RWs with a Type D or E facing are more stable than those having a less rigid facing against the self-weight of soil and the load applied on the crest of backfill or the top of facing. **Fig. 6.17** shows the cross-section of a GRS-RW bridge abutmets having a Type E facing (Tatsuoka et al., 1992). This full-height facing, which is continuous and rigid not only in the wall height direction but also in the longitudinal direction, can resist concentrated vertical and horizontal load applied immediately behind or onto the facing much more effectively than the other types of facing.

<u>Nailed soil structures:</u> Guttierresz and Tatsuoka (1988) performed loading tests on three model sand slopes which were either unreinforced, or reinforced with ten relatively long phospher bronze strips (30 cm--long, 3 mm-wide and 0.5 mm-thick) for a width of 40 cm of the sand box, or reinforced as above but with a facing (**Fig. 6.18**). The other testing conditions were the same as the tests described in Fig. 6.1. The slopes were loaded with a 10 cm-wide footing having a smooth base. By reinforcing, the slope became much stronger (**Fig. 6.18a**) and the failure plane seen in the unreinforced slope was separated into deeper and shallower ones (**Fig. 6.18b** and **c**). The strength of the reinforced slope was controlled by the shallower failure plane, as seen from the fact that the footing pressure near the slope face did not increase by reinforcing (**Fig. 6.18d**). This shallow-seated failure took place since the active zone near the slope face could not be effectively retained only by reinforcing. By using a facing to which the head of the reinforcement was connected, the slope became stronger (the actual rate of the increase in the failure load is larger than that seen in Fig. 6.1a when considering the difference in the void ratio e of the sand). Correspondingly, the sallower failure plane disappeared (Fig. 6.18b ③), the tensile force in the reinforcement at the connection to the back face of the facing increased from zero to some value (Fig. 6.18c), and the pressure near the toe on the footing base (i.e., near the shoulder of the slope) increased largely (Fig. 6.18d). This result suggests that also for nailed soil structures as for reinforced soil structures, a failure in the shallow zone near the slope or wall face may be difficult to prevent without using a rigid facing. In fact, the role of facing becomes more important as the slope becomes steeper.

Sonomura et al. (1991) performed centrifuge tests on 25 cm-high vertical retaining wall of a wet Silica Sand (a maximum diameter of 0.3 mm and w= 8 % compacted to γ_t= 1.66 gf/cm^3) nailed with relatively short and long piano wire having a diameter of 0.38 mm and 1 mm, respectively, with a surface made rough by glueing sand particles (**Fig. 6.19a**). A small bearing plate was attached to the head of each reinforcement; namely the wall face was partially covered with a confining material (i.e., between Types A and B). The wall became more stable by reinforcing as seen from the increase in the failure acceleration (**Fig. 6.19b**). However, in all the reinforced walls, a failure plane passed through the wall face at some intermediate elevation, presumably starting

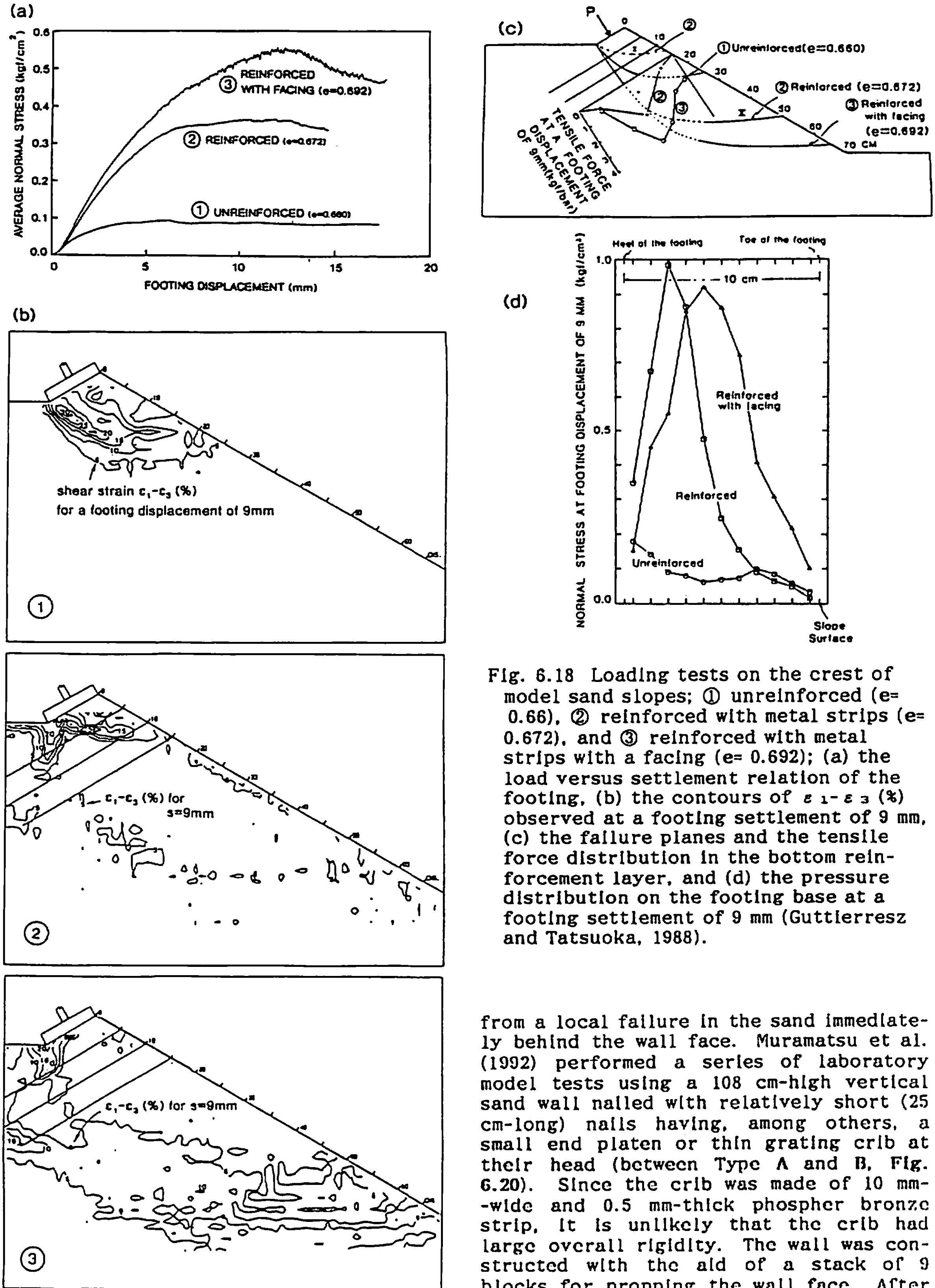

Fig. 6.18 Loading tests on the crest of model sand slopes; ① unreinforced (e= 0.66), ② reinforced with metal strips (e= 0.672), and ③ reinforced with metal strips with a facing (e= 0.692); (a) the load versus settlement relation of the footing, (b) the contours of $\varepsilon_1-\varepsilon_3$ (%) observed at a footing settlement of 9 mm, (c) the failure planes and the tensile force distribution in the bottom reinforcement layer, and (d) the pressure distribution on the footing base at a footing settlement of 9 mm (Guttierresz and Tatsuoka, 1988).

from a local failure in the sand immediately behind the wall face. Muramatsu et al. (1992) performed a series of laboratory model tests using a 108 cm-high vertical sand wall nailed with relatively short (25 cm-long) nails having, among others, a small end platen or thin grating crib at their head (between Type A and B, Fig. 6.20). Since the crib was made of 10 mm--wide and 0.5 mm-thick phospher bronze strip, it is unlikely that the crib had large overall rigidity. The wall was constructed with the aid of a stack of 9 blocks for propping the wall face. After applying a surcharge of 0.009 kgf/cm^2, the wall was brought to failure by releasing the blocks one-by-one from the top. In most cases, a failure occurred accompanying a failure plane passing through the wall face at an intermediate elevation. It is very probable that in the both cases shown in Figs. 6.19 and 6.20, if the wall face had been covered with a continuous rigid facing, the walls would have been much stronger.

A similar local failure was observed in field full-scale loading tests on 5 m-high natural partially saturated sand walls with a slope of 1:0.2 in V:H (Fig. 6.21, Nishikawa et al., 1991). The sand was of the Pleistocene Era having γ_t= 1.706 gf/cm^3 and c= 0.23 kgf/cm^2 and ϕ = 39.9° by UU triaxial compression tests. The walls were either unreinforced or reinforced with 15 cm-wide and 0.45 cm-thick steel strips installed with a special boring method at a horizontal spacing of 1.5 m with a 30 cm x 30 cm end bearing plate at the wall face. The wall faces, however, were not covered with a shotcrete lining. For loading from its crest through an area of 6 m^2, the slope became stronger by reinforcing, while the deformation of the wall face was restrained to some extent by the end platen. However, the failure of the reinforced slopes was still very local while occurred only at higher elevations as in the unreinforced slope. Also in this case, it is very probable that the wall would have been much stronger if a continuously rigid facing had been used.

Loading tests similar to the above had been performed by Okuzono et al. (1985, 1986) and Nagao et al. (1988) on nailed retaining walls reinforced with nails having a 20 cm x 20 cm steel end bearing platen at each head, while the wall faces of a slope of 1:0.3 in V:H were not covered with a shotcrete lining. The walls were (a) 7.5 m-high excavated in a Pleistocene Era sand (γ_t= 2.0 gf/cm^3, and c= 0.4 kgf/cm^2 and ϕ = 35° by UU TC tests) and (b) 3 m-high excavated in a deposit of intact volcanic ash clay called Kanto loam. The walls were loaded from the crest either with a 1.8 m-wide footing with a setback of 1.1 m from the wall shoulder for the sand walls, or with a 1.5 m-wide footing with a setback of 0.5 m for the clay walls. In both the cases, a local compressional failure was observed in the soil behind the wall face near the crest of the wall. It was suggested by the authors of the papers that this type of local failure can be prevented by using a proper rigid facing. Indeed, in the loading tests on the nailed sand retaining walls with a properly designed shotcrete lining on the wall face reported by Gassler and Gudehus (1981) and Stocker et al. (1979, 1991), such a local failure in the soil behind the wall face as above was not observed (Figs. 4.6b and c).

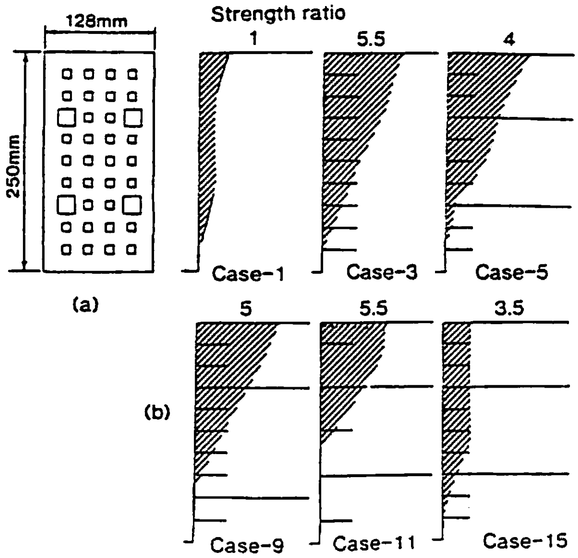

Fig. 6.19 Centrifuge tests of sand walls reinforced with steel bars; (a) model dimensions, and (b) the failure pattern (the numerals means the ratio of the failure acceleration to that for the unreinforced wall of Case 1, and the shaded zones indicate the failed zones) (Sonomura et al., 1991).

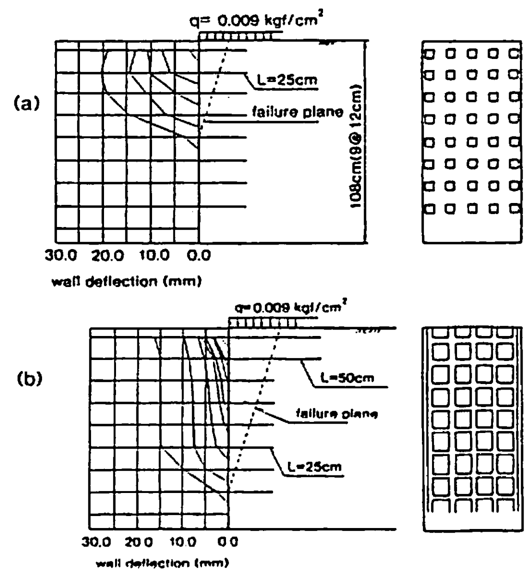

Fig. 6.20 (a) Wall face arrangements, and (b) failure patterns seen in two model nailed sand walls upon the release of propping blocks from the top (Muramatsu et al., 1992).

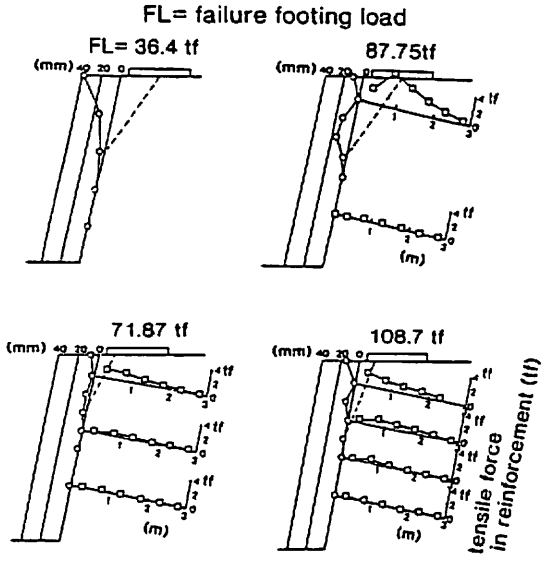

Fig. 6.21 Full-scale loading tests on un-nailed and nailed natural sand slopes (Nishikawa et al., 1991).

Muramatsu et al. (1992) also observed the effect of facing rigidity on the stability of a 9.5 m high temporary nailed wall with a slope angle of 1:0.2 in V:H excavated in a Talus Cone deposit including a volcanic sedimentary soil layer. The slope faces of the two test segments located adjacent to each other had either a 10 cm-thick shotcrete lining or a shotcrete lining plus a crib of 20 cm x 20 cm concrete beams at vertical and horizontal center-to-center spacings of 1.5 m, to which the heads of nails were fixed. Despite that the soil condition was worse for the crib wall, the deformation of the crib wall was smaller. Muramatsu (1992) further observed that in an excavation work in a deposit of mechanically highly disturbed but not weathered Tertiary sedimentary rock, different 7 m-high vertical nailed walls exhibited different deformation at the wall face due to the different facing types under similar other conditions. Namely, the wall having an ordinary concrete lining exhibited a outward displacement at the shoulder of the cut of 0.34 % of the wall height, which was about twice of 0.19 % for the other wall for which H-beams were extended downwards by welding piece by piece while fixing the heads of nails to them according to the progress of excavation. In addition, the deformation of the latter wall, for which the facing has apparently larger rigidity, was less localized.

Summarizing the above, it is apparent that nailed slopes also become more stable by using a more rigid facing, which is to a larger degree as nails become shorter, as slope becomes steeper and as load applied on the crest is located closer to the wall or slope face.

7. CONSTRUCTION METHODS OF A FULL-HEIGHT CONTINUOUS RIGID FACING FOR REINFORCED SOIL RETAINING WALLS

Potential problems: The following two major problems should be solved:

1) When a full-height facing unit (e.g., precast RC panel) is erected before the backfill is filled up and the soil is compacted with reinforcement connected to the back face of the facing (Fig. 7.1a), the connections between the reinforcement and the back face of the facing may be damaged due to the relative settlement between them (in some cases, also facing and reinforcement). This is also the case when geotextile reinforcement is sandwiched between facing modular blocks (Fig. 2.7); i.e., the reinforcement may be seriously damaged by over-stressing at the upper edges at the back face of blocks (in the zone a in Fig. 2.7). On the other hand, Fukuoka et al. (1986) constructed a 5 m-high vertical retaining wall with a relatively loosely compacted volcanic ash clay Kanto loam retained with a multiple anchoring system with the heads of the anchor rods fixed to

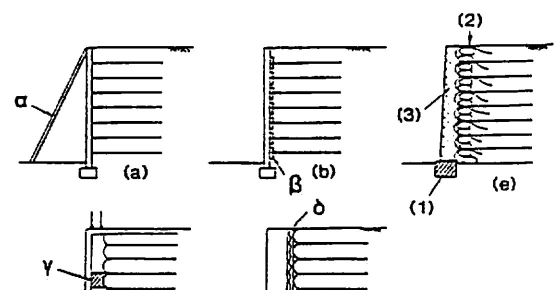

Fig. 7.1 Several methods using a continuous rigid facing: (a) Fixed type facing fully propped during filling and compacting the backfill, (b) the use of sliders for a facing fully propped during filling and compacting the backfill, (c) separated type facing, (d) the use of a compressible layer placed on the rear of facing fully propped during constructing the backfill, and (e) stage construction (the numbers mean the construction sequences).

a group of full-height vertical RC columns at centers of 1 m attached onto the wall face. Although any sign of the failure of the wall was not detected, by a relative settlement between the backfill and the columns, the anchor rods were bent at the places immediately behind the back face of the columns, typically to a vertical deflection of 20 cm in accordance with the amount of relative settlement. Further, Kutara et al. (1986) proposed a method to decrease the amount of bump, which is the relative settlement between the ground surface immediately behind the the crest of bridge abutment and the bridge abutment, by attaching grid layers to the back face of the abutment. A large model test using a 2.3 m-high cantilever RC retaining wall was performed, in which the whole of the backfill was made to settle down by collapsing a specially prepared 20 cm-thick layer at the bottom of the backfill. It was found that this method is very effective to reduce the amount of bump, but the largest strain was developed in the grid at the connections to the back face of the retaining wall, which was as large as about 0.7 %. Indeed, this result demonstrates the detrimental effects of the relative settlement between the facing and the backfill on the reinforcement.

2) Although the development of large tensile stress in reinforcement is essential for its efficient use, when a full-height rigid facing is firmly propped as soil layers are compacted (as denoted by α in Fig. 7.1a), only small tensile strains may be mobilized in the reinforcing members. Some amount of tensile strain may be mobilized only when the support of the facing was released after the full height of wall is completed. In this case, relatively large delayed outwards displacement of the facing may occur; for example, Bathurst (1992) reported about 1 % delayed outward displacement at the top of a unit facing of 7 m-high grid-reinforced sand retaining wall constructed by this method. This post-construction displacement may also induce some undesirable relative settlement between the backfill and the facing. Further, large earth pressure may be activated on the back face of a firmly propped facing when compacting the soil.

Several methods as shown in Fig. 7.1 have been used to alleviate either or both of these two problems:

a) To alleviate both the problems when using a <u>**fixed type**</u> facing (**Fig. 7.1a**), it is often recommended to avoid heavy compaction of soil around the connections and immediately behind the facing. It is, however, at the expense of reducing the stability of wall and, therefore, would not be a good solution. It has also been attempted to incline the facing inward to some extent before filling, and as each layer of backfill soil is compacted, the facing is inclined outward (Fishman et al., 1991). However, this method would complicate the construction work.

b) By means of a <u>slider</u> (denoted by β in **Fig. 7.1b**), reinforcement connected to the back face of a unit facing is permitted to slide relative to the facing when relative settlement occurs between the facing and the backfill during and after the construction of wall (e.g., Jones, 1985). It could be argued whether this arrangement makes the facing less stable, which in turn makes the backfill more deformable when compared with the case in which the reinforcement is tightly connected to the back face of the facing. Further study will be necessary to quantify the positive and negative effects of using sliders.

c) A <u>separated type</u> unit facing, which is erected before backfilling, is not in contact with the wrapped-around wall face both during and after the construction of wall (Delmas et al., 1988). To make the reinforced zone more stable and less deformable and to develop sufficiently large tensile strains in reinforcement, the backfill is heavily compacted with the aid of a flexible temporary inflation system (denoted by γ in **Fig. 7.1c**) placed between the back face of the facing and the wrapped-around wall face. However, the facing does not contribute directly to the stability of the wall (see Fig. 6.9).

d) A <u>compressible layer</u> is placed vertical on the back face of the facing (denoted by δ in **Fig. 7.1d**), while the reinforcement is not connected to the back face of a facing or a RC wall structure which has been erected before back-filling (Edgar et al., 1989, McGown et al., 1988, Wu and Helwany, 1990, Horvath, 1991). It is expected that during compacting the backfill, some amount of tensile strain be mobilized in the reinforcement by the collapse of the compressible layer, while smaller earth pressure be activated on the facing or a RC wall structure. The same mechanism can be expected also when surcharging on the crest of the backfill. This method is adequate when a GRS-RW is constructed behind an independent structure such as a conventional RC bridge abutment and it is required to reduce the earth pressure on the back face of the structure as much as possible. In case a unit panel facing is to be laterally supported by being connected to the wall face of backfill as for most reinforced soil retaining walls, the advantage described above may be balanced by a disadvantage that the deformation of the backfill cannot be efficiently restrained despite of the use of a rigid facing. Further, the stability of the wall may not increase sufficiently either. On the contrary, as discussed in Section 3.5, under a given loading condition, the deformation of backfill can be reduced by permitting the earth pressure on the back face of the facing to develop as much as possible by using a more rigid facing.

e) The stage construction method (Fig. 7.1e) may alleviate both of the two potential problems associated with the use of a continuous rigid facing. In this method, first a strip foundation of the facing (denoted by 1 in Fig. 7.1e) is constructed. Subsequently, the backfill is filled up while compacting the soil near the wall face as much as possible by means of a compaction machine with the aid of gabions (sand bags) placed on the shoulder of each soil layer being wrapped-around with a geotextile sheet (denoted by 2). A strong connection between gabions and reinforcement is essential for ensuring the stability of temporary "gabion wall". Brand (1992) reported a case of failure of a 6 m-high gabion wall with a cohesionless soil backfill reinforced with a grid. It is reported that the wall failed due to both a too large vertical separation of grid of 0.9 m and a too small length of grid of 1.8 m in the upper portions of the wall, and the relatively weak connection between the reinforcement and the gabions without wrapping-around the gabions geotextile sheet. A pattern of reinforcement force distribution as much as closer to the pattern A (Fig. 3.5), or the pattern shown in Fig. 4.1a, may be achieved by such heavy compaction as pushing out noticeably the gabions. An irregular shape of wall face produced as constructed will be masked by casting in place a concrete facing on it after the full height of wall is completed. As a temporary facing, in place of gabions, a steel mesh framework as shown in Fig. 2.3 may be used. In any case, the temporary facing should be rigid enough to resist against the earth pressure required to prevent a local compressional failure in the soil near the wall face and at the same time should be flexible enough to yield as tensile strain can be developed in the reinforcement during filling and compacting the backfill. After major part of the compression of backfill and the supporting ground has occurred subsequently to the construction of the wall to full design height, a full-height continuous rigid facing (denoted by 3 in Fig. 7.1e) is placed directly on the wall face. In this way, possible damage to the rigid facing and the connections between the facing and the reinforcement can be avoided.

For the delayed placement of a full-height continuous rigid facing, either of the following methods may be employed: **i)** A layer of shotcrete layer with one or two layers of wire mesh as used for nailed soil structures is casted on the wrapped-around wall face. However, this method may not be aesthetically pleasing. Ng and Mak (1988) adopted this method to construct a temporary 14 m-high wall having a 70° slope angle using a locally available granular material reinforced with a polymer grid, which supported a haul bridge at the shoulder of the wall. The wall was constructed with the aid of sand bags at the shoulder of each soil layer. In the design, the effect of facing rigidity was not considered. **ii)** A lightly steel-reinforced concrete facing is casted in place directly over the wrapped-around wall face so that the facing does not separate from the wall face (Figs. 2.9 and 6.17). **iii)** A lightly steel-reinforced pre-cast concrete plate unit is erected, leaving a space between its back face and the wall face, and the space is filled with concrete subsequently. **iv)** Discrete modules (e.g., concrete blocks or stone blocks) are stacked and subsequently tightened up for example by penetrating reinforcement bars through them. Subsequently, the space between the back face of the blocks and the wrapped-around wall face is filled with concrete.

In the stage construction method, it is not necessary to consider the problem of propping firmly the facing against large earth pressure during back-filling. Further, since a unit facing and reinforcing members are finally tightly connected to each other, the facing has all of the facing rigidities shown in Table 5.1. Note that Jones (1988) has already described a similar stage construction method used for a bridge construction in U. K., whereas the advantages and the construction details were not given. Allen et al. (1992) also suggested a stage construction method based on their experience; namely since the deformation of the wrapped-around wall face of a 12.6 m-high GRS-RW was not small, if rigid face panels are to be used, "it may be advisable to attach the panels to the wall face after it has been constructed to full design height". Note also that, in case large relative settlement occurs between a facing and backfill after the facing is casted, a stack of gabions is expected to prevent over-stressing at the connections between the facing and the reinforcement by functioning as a buffer.

When fresh concrete is casted directly on a geotextile-wrapped-around face, part of the geotextile strength may be lost with time by a high pH and the presence of calcium ions from concrete. For the walls shown in Figs. 2.9 and 6.17, a Vinylon grid, which is known to be strong against the above mentioned effects, while covered with PVC for protection, was used in place of a polyester grid covered with PVC used for the test embankments (Fig. 6.16).

GRS-RWs to be constructed by the stage construction method should be designed so that they are temporarily stable in the absence of a continuous rigid facing. If the walls will support some external load after a continuous rigid facing is constructed, the safety factors of the wall before and after the placement of facing can be adjusted to be similar. However, if the maximum height of the wall is controlled by the stable height of the temporary wrapped-around wall, the safety factor for the completed walls with a continuous rigid facing will be certainly higher than that for the uncompleted wall. From these points, the construction of GRS-RWs by the stage construction method is most adequate when the completed walls are subjected to some external load, or when the walls will be used as important permanent structures which permit only small wall deformation as in the cases shown in Figs. 2.9 and 6.17.

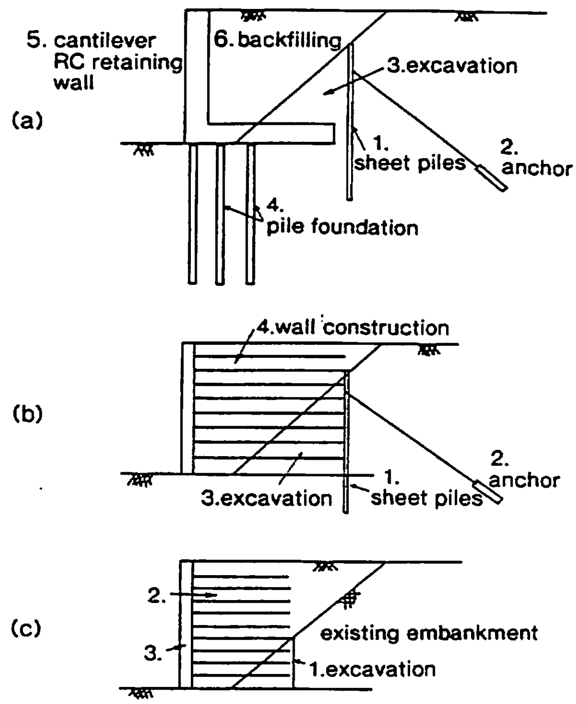

Fig. 7.2 Reconstruction of existing slope by three methods: (a) a conventional RC cantilever retaining wall, (b) a reinforced soil retaining wall having a relatively long reinforcement, and (c) a GRS-RW having a continuous rigid facing and a short geotextile (the numbers indicate the construction sequences).

<u>The use of relatively short reinforcement without losing the stability of wall:</u> Obviously, it is on the safe side to ignore in the design the contribution of facing rigidity to the stability of reinforced soil structures, even if it exists. This is the case in most of the current design methods. At the same time, this method is less economical and may be too conservative, in particular when a Type E or D facing is properly used. On the other hand, there are many cases where the use of short reinforcement is very much beneficial. Fig. 7.2a illustrates the reconstruction of an existing slope by means of a conventional RC cantilever retaining wall. This method, however, requires a large amount of excavation. Further, to ensure the stability of the slope during re-construction, particularly when only very small deformation of the slope is allowed, sheet piles may be installed, which are subsequently anchored before the excavation proceeds. A reinforced soil retaining wall (Fig. 7.2b) may be cheaper when a pile foundation is needed for the RC cantilever retaining wall. However, if relatively long reinforcement with a length, say of the order of the wall height, is used, both a very large amount of excavation and anchored steel-piles may also be needed. The construction cost can be reduced largely if such works as above are eliminated by making the reinforcement length relatively short, say 40 % of the wall height (Fig. 7.2c, see also Fig. 2.9). Note that it has also been studied whether relatively short metal strips could be used also for R. E. retaining walls (see Fig. 4.1b).

It is the author's view that in case the backfill soil has not a sufficiently large cohesion (n.b., this is most cases), the use of relatively short reinforcement can be justified only when a full-height continuous rigid facing is used. Namely, referring to Fig. 3.2b, when the potential failure plane does not intersect with the reinforcement at higher elevations, the earth pressure activated on the back face of the reinforced zone at the higher elevations should be transmitted to the lower portion of the wall. A continuous rigid facing can enhance this mechanism by supporting part of the load transmitted from the backfill. In addition, the reinforcement length can be reduced by using planar reinforcement, since it requires only a very short anchoring length.

The above point has been validated by several laboratory and field tests (Tatsuoka et al., 1988, 1991, 1992, Murata et al., 1991, 1992, Toriihara et al., 1991). Kawakami and Abe (1978) has already shown that several 1 m-high GRS-RW models having a sand backfill with a facing of Type B3 - C'' or Type D were very stable when loaded from the crest, despite that the reinforcement length was much less than the wall height (even down to 10 % of the wall height). Further, as reported by Tatsuoka et al. (1992) and Murata et al. (1992), railway embankments with a typical height of 5 m for a length of more than 6 km have been reconstructed to GRS-RWs having a Type D or E facing by means of the stage construction method (see Figs. 2.9 and 6.17).

One may consider that when the supporting ground is soft, a pile foundation may be required for supporting the reaction at the facing bottom (denoted by V + W and S in Fig. 5.6), which is indeed the lose of one of the primary advantages of reinforced soil structures over the conventional RC retaining walls. However, since the bearing capacity of the ground required in this case is much less than that required for a conventional RC retaining wall structures, a strip foundation having some width and depth as used for R. E. retaining wall is usually sufficient unless the ground is so soft that a failure could occur in the supporting ground by the weight of the backfill. Even in case a micro-pile system or a improved soil diaphragm is to be used to support the reaction from the facing as have been employed at several places in Japan, problems associated with the relative settlement between the facing and the backfill can certainly be avoided by placing a continuous rigid facing after most of the deformation of the supporting ground due to the weight of backfill has occurred.

One may also consider that it is dangerous to rely on the contribution of the overall rigidity of a full-height continuous rigid facing, since the bearing capacity of the ground supporting the facing may be lost by excavation below or in front of the facing in the future. This view may be reasonable if the wall is damaged seriously by excavation, in particular if the wall fails in a catastrophic manner. Note however, that for such GRS-RWs as shown in Figs. 2.9 and 6.17, the walls were very stable at the temporary state before casting a continuous rigid facing. Further, a 1/2 reduced scaled model of a 5 m-high test GRS-RW having a sand backfill shown in Fig. 6.16 was constructed on a liquefiable sand placed on a large shaking table (Tateyama et al., 1990). The wall was very stable even not exhibiting any sign of failure after the ground on both sides of the wall had liquefied by shaking while each facing of Type E was hanging on each wrapped-around wall face. In addition, for the prototype walls, the longitudinal continuity of facing will help the wall in resisting against such excavation unless it is for a very large length. Rather, the effects of such excavation will be more detrimental to other types of walls not having a continuous rigid facing such as wrapped-around walls or those having a discrete panels facing. Indeed, it is apparent that excavation below or in front of the foundation of wall should not be permitted whether a given reinforced soil structure has a continuous rigid facing or not.

8. SOIL NAILING WITH A MINIMUM SLOPE DEFLECTION

For most temporary nailed retaining walls and most of the permanent ones which are not particularly important, a shotcrete lining is sufficient for the stability of wall. However, in opposite to reinforced soil structures, the ground to be nailed may already be supporting an important structure which is sensitive to deformation, for example, a foundation of a building or a railway track. In that case, allowable outward horizontal displacement at the shoulder of the wall may be very small as less than 0.1 % of the wall height, which is much smaller than those observed in ordinary nailed retaining walls. To alleviate this problem, the following several methods have been used for pre-propping before the excavation proceeds.

Grout-columns and micro-piles (Nicholson and Wycliffe-Jones, 1984): At the PPG building site in Pittsburgh, USA, a 12-13 m-depth excavation was made by soil nailing in the close proximity of existing buildings. Immediately behind the face of the nailed retaining wall to be made afterwards, at the line of the foundation of an existing building, before the excavation proceeded, fully cased holes of 12.7 cm in diameter at 25-30 cm centers were drilled in a cohesionless granular soil and pressure-grouted after the insertion of a steel reinforcing bar. At the same site, as a more reliable method for another building, in addition to a grout columns, groups of micro-piles were installed to support directly about half of the total vertical load of the foundation.

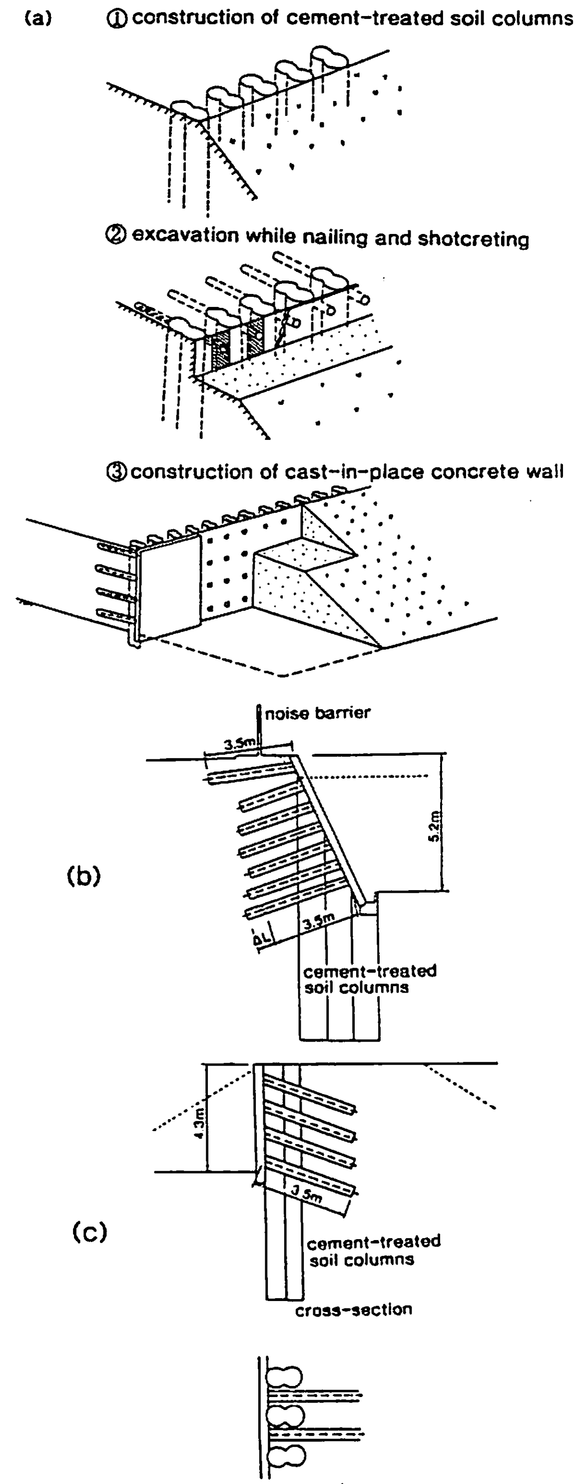

Fig. 8.1 Pre-propping by improved soil columns to minimize the deformation of nailed soil wall: (a) schematic diagram, and the cases using large diameter nails at (b) Akabane and (c)Rokuzizo (Tateyama, 1992).

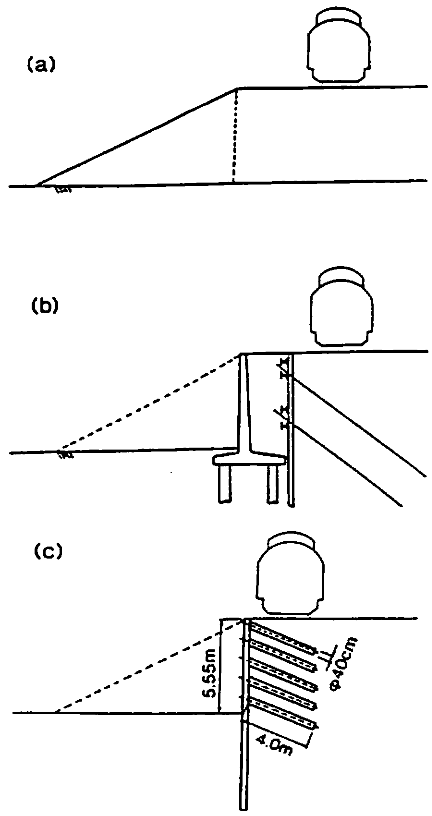

Fig. 8.2 Excavation in a slope supporting a railway track at Ikebukuro, Tokyo; (a) slope before excavation, (b) a conventional method by anchoring (a cantilever retaining wall is not constructed where the wall is temporary), and (c) soil nailing using large-diameter nails, prepropped by H-beams to minimize the deformation of nailed wall (a continuous rigid concrete facing is casted on the wall face where the wall is a permanent structure) (Taniguchi et al., 1992, Tateyama, 1992).

<u>Improved soil diaphragm:</u> In Osaka, Japan, a gentle slope of railway embankment made of a cohesionless soil supporting several very busy railway tracks was excavated by soil nailing to a 4-7 m high and 300 m long near-vertical permanent wall (Tateyama et al., 1992). To keep the deformation of the slope to be very small, before the excavation proceeded, groups of cement-treated soil columns were made by an in-situ mixing method from near the crest of the slope as illustrated in Fig. 8.1a. After the full height of the cut was excavated while nailing the slope and shotcreting the excavated slope surface, a 30 cm-thick lightly steel-reinforced cast-in-place concrete layer was constructed on the surface of shotcrete lining. Similar excavations were

made at Akabane (**Fig. 8.1b**, Tateyama, 1992) and at Rokuzizo in Kyoto (**Fig. 8.1c**, Tateyama, 1992). In the latter two cases, as nails, a FRP rod was installed at the center of the 40 cm-diameter cement-treated 'nails' formed by simultaneous drilling and cement-mixing. This method was adopted considering that the soils of most railway embankments are usually soft enough for such large-diameter drilling, and the pull-out resistance of each nail can be increased by increasing the nail diameter, therefore, the nail length can be reduced.

Installation of steel H-beams: At Ikebukuro in Tokyo, a temporary wall was excavated for a new track in a slope of intact volcanic ash clay (Kanto loam) supporting a busy railway track (**Fig. 8.2a**, Taniguchi et al., 1992, Tateyama, 1992). In case the conventional method as shown in **Fig. 8.2b** is used, too long anchorage extending the boundary between the railway and the adjacent private property should be used. To alleviate this problem and to shorten the construction period, the soil nailing method using large-diameter 'nails' was adopted, while a line of H-beams was installed before the excavation proceeded to minimize the deformation of the nailed wall (**Fig. 8.2c**).

For all the cases described above, drilling for the nails was executed through the pre-propping structure so that it became part of the facing after the wall was completed. It has been reported for all the cases that neither the excavation itself nor the pre-propping activities caused any delectable damage or movement to adjacent structures.

9. SUMMARY AND CONCLUSIONS

Many of the reinforced soil structures so far constructed have a more-or-less deformable and flexible facing, and most of the current design stability analysis methods ignore the effects of facing rigidity. This is in accordance with one of the primary advantages of reinforced soil structures which are widely accepted; namely the facing does not need to support such large earth pressure as a conventional retaining wall does. It has been discussed in this note, however, that methods of soil reinforcing cannot be best characterized by a large reduction in the earth pressure on the back face of wall from the Coulomb's active earth pressure. Of course, no doubt the most important feature is that the thrusting earth pressure is resisted by tensile reinforcement.

Many available data from laboratory and field tests show that as a facing becomes more rigid, the earth pressure acting on the back face of facing increases even to more than the Coulomb's active earth pressure which is activated when the backfill is not reinforced. It has also been observed that this large earth pressure confines the soil immediately behind the facing, which decreases the deformation and increases the ultimate stability of the wall. The local facing rigidity is defined as the ability of facing to activate a sufficiently large earth pressure on the back face of facing as required.

The overall rigidities of facing are classified into the overall axial, shear and bending rigidities, which contribute to the stability of the wall in different ways. In this note, a method to take into account the effects of these various facing rigidities in limit equilibrium-based stability analyses is suggested. Yet, more research is required before the more confident use of a continuous rigid facing relying on the effects of facing rigidity on the stability of wall can be expanded to the wider and critical applications.

It has also been discussed that a large degree of flexibility is not necessarily a preferable property for completed reinforced soil structures, although this property is required during the construction of wall to accommodate possible large deformation of the supporting ground so that a deep foundation becomes unnecessary. One of the possible compromises would be that walls are made as much as flexible during construction, whereas they are made stiff enough before opened to service. One of the solution would be the stage construction by casting in place a full-height continuous rigid concrete facing on a deformable wall face which has been constructed to design full height by some measures, for example by the aid of gabions placed on the shoulder of each soil layer.

Also for nailed soil retaining structures, many of the existing data indicate that a nailed wall becomes less deformable and more stable as the facing rigidity increases. In particular, a nailed wall excavated in a cohesionless soil without a shotcrete lining is prone to local compressional failures in the soil immediately behind the exposed wall face. When the allowable deformation of the wall is very small, for example when a wall is supporting structures directly or when a wall is in the close proximity of existing structures, the wall should be made stiff enough by means of a pre-propping method before the excavation proceeds, for example, by constructing a system of improved soil columns or by installing micro-piles or steel H-beams immediately behind the wall face.

Both for reinforced and nailed soil structures, whether a rigid facing is to be used and the level of facing depend on the allowable deformation, the importance, the design life time, the required aesthetical level, among others, of each structure. Namely, for temporary structures which allow relatively large deformation, a low class facing (e.g., wrapped-around facings or gabion facings for reinforced soil retaining walls and shotcrete facings for nailed soil retaining walls) may be sufficient. It will also be the case for such permanent structures as constructed in rural areas or for those which are less important in the scale and in the influence when failed. For more important permanent structures, however, such a higher-level facing as discrete concrete panels facings used for Terre Armee retaining walls may be required, which is more durable, aesthetically more pleasing and rigid enough to restrain the deformation of wall face. In more critical applications such as bridge abutments subjected to external load close to the wall face, a full-height continuous rigid facing may be required to ensure their long-term stability and to allow only small deformation of the structure.

Finally, it should be pointed out that a full-height continuous rigid facing to be used for an important permanent reinforced or nailed retaining wall can be much simpler and lighter than a conventional RC retaining wall structure. This is because large earth pressure which may be activated on the back face of the facing is supported by many reinforcement layers with a small span. This is indeed the first advantage for soil reinforcing method.

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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.

Balzer,E., Matichard,Y. and Thamm,B.R. 1991. Geotextile reinforced abutment: full scale test and theory, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.47-57.

Bastick,M.J. 1991. Reinforced earth narrow walls and abutments correlation of measured performance with design, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.59-63.

Bastic,M.J. 1991a. Soil nailing and Reinforced Earth, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.255-259.

Bastick,M.J. 1991b. Discussion in Session Summary summarized by Smith,R.J.H., Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.91-92.

Bathurst,R.J., Benjamin,D.J. an Jarrett,P.M. 1988. Laboratory study of geogrid reinforced soil walls, Proc. of Symp. on Geosynthetics for Soil Improvement, by GE Div., ASCE, Geotechnical Special Publication No.18, pp.178-192.

Bathurst,R.J. 1992. Case study of a monitored propped panel wall, Geosynthetic-Reinforced Soil Retaining Walls, Wu.(ed.), Balkema, pp.159-166.

Bjerrum,L. 1957. Norwegian experience with steel pile foundations to rock, Jour. of the Boston Society of Civil Engineers, Vol.44, No.3, July, pp.155-198.

Bolton,M.D. and Pang,P.L.R. 1982. Collapse limit states of reinforced earth retaining walls, Geotechnique 32, No.4, pp.349-367.

Bolton,M.J. 1991. Reinforced soil: laboratory testing and modelling, S-O-A Report, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.287-298.

Brand,A.H. 1992. Gabions and geogrids, Civil Engineering, Sept. pp.65-67.

Broms,B.B. 1978. Design of fabric reinforced retaining structures, Proc. of Sympo. on Earth Reinforcement, ASCE, pp.282-303.

Broms,B.B. 1988. Fabric reinforced retaining walls, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu'88, Fukuoka, pp.3-31

Bruce,D.A. and Jewell,R.A. 1986. Soil nailing: Application and practice-Part 1, Ground Engineering, Vol.19, No.11, pp.10-15.

Bruce,D.A. and Jewell,R.A. 1987. Soil nailing: Application and practice-Part 2, Ground Engineering, Vol.20, No.1, pp.21-33.

Claybourn,A.F. and Wu,J.T.H. 1992. Failure

loads of the Denver walls by current design methods, Geosynthetic-Reinforced Soil Retaining Walls, Wu.(ed.), Balkema, pp.61-77.
Clough,G.W. and Duncan,J.M. 1971. Finite element analyses of retaining wall behavior, Jour. SMF Div., Proc. ASCE, Vol.97, No.SM12, pp.1657-1673.
Delmas,Ph., Grourc,J.P., Bilvet,J.C. and Maticard,Y. 1988. Geotextile-reinforced retaining structures: A few instrumented examples, Proc. Int. Geotech. Sympo. on Theory and Practice on Earth Reinforcement, IS Kyushu, Balkema, pp.511-516.
Edgar,T.V., Puckett,J.A. and D'Spain,R.B. 1989. Effects of geotextiles on lateral pressure and deformation in highway embankments, Geotextiles and Geomembranes, Vol.8, No.4, pp.33-38.
Fishman,K.L., Desai,C.S. and Berg,R.R. 1991. Geosynthetic-reinforced soil wall: 4-year history, Transportation Research Record No.1330, pp.30-39.
Fukuda,N., Yamanouchi,T. and Miura,N. 1989. Comparative studies of design and construction of a steep reinforced embankment, Geotextiles and Geomembranes, Vol.4, pp.269-284.
Fukuoka,M., Imamura,Y. and Nishimura,J. 1986: Fabric-faced retaining wall with multiple anchors, Geotextiles and Geomembranes, Vol.4, pp.207-221.
Gassler,G. and Gudehus,G. 1981. Soil nailing -Some aspects of a new technique, Proc. 10th Int. Conf. on SMFE, Stockholm, Vol.3, pp.665-669.
Gassler,G. 1988. Soil nailing - theoretical basis and practical design, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu'88, Fukuoka, pp.283-288.
Gassler,G. 1991. In-situ techniques of reinforced soil; S-O-A Report, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.185-196.
Gourc,J.P., Gotteland,P. and Wilson-Jones,H. 1991. Cellular retaining walls reinforced by geosynthetics: behaviour and design, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.41-45.
Gourc,J.-P. and Matichard,Y. 1992. Development of geotextile reinforcement techniques in France- Application to retaining structures, Geosynthetic-Reinforced Soil Retaining Walls, Wu (ed.), Balkema, pp.131-152.
Goodings,D.J. 1989. Effects of poorly draining backfill on geotextiles for earth reinforcement of vertical soil slopes, Final Report to the Maryland Dept. of Transportation State Highway Administration Bureau of Research.
Guttierresz,V. and Tatsuoka,F. 1988. Roles of facing in reinforcing cohesionless soil slopes by means of metal strips, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu '88, Fukuoka, pp.289-294.
Huang,C.-C. and Tatsuoka,F. 1990. Bearing capacity of reinforced horizontal sandy ground, Geotextiles and Geomembranes, Vol.9, pp.51-82.
Huang,C.-C., Tatsuoka,F. and Sato,Y. 1991. Reinforcing a sand slope supporting a footing using steel bars, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.323-328.
Huang,C.-C. and Tatsuoka,F. 1992. A comparative study of stability analysis methods for reinforced sand slope, Proc. Int. Sympo. on Earth Reinforcement Practice, IS Kyushu '92, Fukuoka, pp.191-196.
Horvath,J.S. (1991. Using geosynthetics to reduce surcharge-induced stresses on rigid earth-retaining structures, Transportation Research Record No.1330, pp.47-53.
Ingold,T.S. 1982. Reinforced earth, Thomas Telford Ltd, London.
Jaber,M.B. 1989. Behavour of reinforeced soil walls in centrifuge tests, Ph.D. Thesis, Univ. of California Berkeley.
Jewell,R.A., Paine,N. and Woods,R.I. 1984. Design methods for steep reinforced embankments, Polymer Grid Reinforcement, McGown et al.(ed.), Thomas Telford, pp.70-81.
Jewell,R.A. 1985. Limit equilibrium analysis of reinofrced soil walls, Proc. 11th ICSMFE, San Francisco,Vol.5, pp.1705-1708.
Jewell,R.A. 1990. Strength and deformation in reinforced soil design, Keynote Paper, Proc. 4th Int. Conf. on Geotextiles, Geomembranes and Related Materials.
Jewell,R.A. 1991. Application of revised design charts for steep reinforeced slope, Geotextiles and Geomembranes 10, pp.203-233.
Jewell,R.A., Burd,H.J. and Milligan,G.W.E. 1992. Predicting the effect of boundary forces on the behaviour of reinforced soil walls, Preprints of Predictive Soil Mechanics, Proceedings of the Wroth Memorial Symposium, Oxford, 1992, pp.272-283.
Jones,C.J.F.P. 1985. Earth reinforcement and soil structures, Butterworths.
Jones,C.J.F.P., McGown,A. and Varney,D.J. 1988. Construction methods, Economics and specifications, The Application of Polymeric Reinforcement in Soil Retaining Structures, Jarret and McGown (ed.), NATO ASI Series, Kluwer Academic Publishers, pp.573-611.
Jones,C.J.F.P. (1991. Construction influences of the performance of reinforced soil structures, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.97-116.
Juran,I. and Schlosser,F. 1978. Theoretical analysis of failure in reinforced earth

structure, Proc. of Sympo. on Earth Reinforcement, ASCE, pp.528-555.

Kawakami,H. and Abe,H. 1978. Reinforcing of concrete retaining and bloc walls, Proc. on Symposium on Soil Reinforcing and Stabilizing Techniques, Sydney, pp.137-151.

Kawasaki,H., Hirai,T., Furukawa,O., Sato,M., Sekijima,K. and Kawabe,M. 1990. Large-scaled test on the reinforced embankment with FRP geotextiles, Proc. 25th Japan National Conf. on SMFE, JSSMFE, pp.1981-1984 (in Japanese).

Keller,G.R. 1990. Alternative wall and reinforced fill experiences on forest road, Design and Performance of Earth Retaining Stuctures, Geotechnical Special Publication No.25, ASCE, Lambe and Hansen (ed.), pp.155-169.

Koerner,R.M. and Welsh,J.P. 1980. Construction and geotechnical engineering using synthetic fabrics, John and Welley & Sons.

Koerner,R.M. 1985. Designing with geosynthetics, Prentice-Hall.

Koerner,R.M.(ed.) 1989. Durability and aging of geosynthetics, Elsevier Applied Science.

Kimura,H. 1989. On temporary stability of the cut slope before reinforcing with shotcrete and rock bolts, Proc. 21th Sympo. on Rock Mechanics, Committee on Rock Mechanics, Japanese Society for Civil Engineers (in Japanese).

Knutson,A.F. 1990. Reinforced soil retaining structures, Norwegian experiences, Proc. 4 th Int. Conf. on Geotextiles and Geomembranes and Related Products, Vol.1, pp.87-91.

Koga,Y., Washida,S. and Ito,Y. 1992. Report of the experiment as to the seismic stability of reinforced embankment, Report of Public Works Research Institute No.3066, Ministry of Construction (In Japanese).

Kutara,K., Gomadou,M. and Takeuchi,T. 1986. Deformation analysis of geotextiles in soils using the finite element method, Geotextiles and Geomembranes, Vol.4, pp.191-205.

Laba,J.T., Kennedy,J.B. and Seymour,P.H. 1984. Reinforced earth retaining wall under vertical and horizontal strip loading, Canadian Geotechnical Journal 21, pp.407-418.

Lee,K.L., Adams,B.D. and Vaganeron,J.-M.J. 1973. Reinforced earth retaining walls, Jour. SMF Div, Proc. of ASCE, Vol.99, No.SM10, Oct., pp.745-763.

Leshchinsky,D. and Volk,J.C. 1986. Predictive equation for the stability of geotextile reinforced earth structure, Proc. Third Int. Conf. on Geotextiles, Vienna, Vol.II, pp.383-388.

McCaul,C. and Snowdon,R.A. 1991. Early performance of a reinforced earth bridge abutment, Performance of Reinforced Soil Structures, Thomas Telford, London, pp.87-94.

McGown,A., Andrews,Z.Z. and Murray,R.T. 1988. Controlling yielding of the lateral boundaries of soil retaining structures, Proc. Sympo. of Geosynthetics for Soil Improvement, GE Div., ASCE, pp.193-210.

Miki,H., Kudo,K., Taki,M., Fukuda,N, Iwasaki,K. and Nishimura,J. 1992. Experimental study on full scale geogrid reinforced embankment model, Proc. Int. Sympo. on Earth Reinforced Practice, IS Kyushu '92, Fukuoka, pp.269-274.

Mitchell,J.K. (1982. Soil improvement, S-O-A Report for Session 12, Proc. Int. Conf. On SMFE, Stockholm, Vol.4, pp.509-565.

Muramatsu,M. Nagura,K., Seoka,T., and Suami, K. 1992. Stability analysis for reinforced cut slopes with a rigid facing, Proc. Int. Sympo. on Earth Reinforcement Practice, IS Kyushu '92, Fukuoka.

Muramatsu,M. 1992. Private communication.

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.

Murata,O., Tateyama,M. and Tatsuoka,F. 1992. Loading tests of geosynthetic-reinforced soil retaining walls and their stability analyses, Proc. Int. Sympo. on Earth Reinforced Practice, IS Kyushu '92, Fukuoka, pp.385-390.

Murata,O. 1992. Private communication.

Naemura,S., Onodera,S., Maruo,S. and Nakane, A. 1991. Case record of test steep slopes having different facing types, The Foundation Engineering & Equipment (Kisoko), Nov., Vol.19, No.11, pp.68-74 (in Japanese).

Nagao,A., Kitamura,T. and Mizutani,J. 1988. Field experiment on reinforced earth and its evaluation using FEM analysis, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu'88, Fukuoka, pp.329-334.

Nakamura,K., Tamura,Y., Tatsuoka,F., Iwasaki, K. and Yamauchi,H. 1988. Roles of facings in reinforcing steep clay slopes with a non-woven geotextile, Proc. Int. Geotech. Sympo. on Theory and Practice on Earth Reinforcement, IS Kyushu, Japan, Balkema, pp.553-558.

Nakane,A., Nagaoka,T., Onodera,S. and Shimada,I. 1990. Long-term observation of reinforced retaining structure with geotextile, Proc. 25th Japan National Conf. on SMFE, JSSMFE, pp.2013-2016 (in Japanese).

Ng,H.Y. and Mak,C.H. 1988. A 14 metre high reinforced soil embankment as the abutment of a steel bridge in Tuen Mun, Hong Kong, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu'88, Fukuoka, pp.449-454.

Nicholson,P.J. and Wycliffe-Jones,P.T. 1984.

Soil nailing: an earth reinforcement system for support of excavation, Proc. of Int. Conf. on In Situ Soil and Rock Reinforcement, Paris, Oct. (as referred by Bruce and Jewell, 1987).
Nishikawa,J., Chida,S., Kitagawa,J., Suzuki,K. and Iwasa,N. 1991. Field loading test of reinforced slopes with steel strips, Proc. 25th Japan National Conf. on SMFE, JSSMFE, pp.2133-2136 (in Japanese)
Okuzono,S., Okuhara,M., Nagao,A., Uehara,S. and Innami,S. 1985. In-situ loading test of reinforced slope with steel bars - Influence of steel bar length for reinforcing effect, Proc. 20th Japan National Conf. on SMFE, JSSMFE, pp.1353-1356 (in Japanese).
Okuzono,S., Sano,N., Uehara,S. and Innami,S. 1986. In-situ loading test of reinforced slope with steel bars - Influence of bar length for reinforcing effect at clayey ground, Proc. 21th Japan National Conf. on SMFE, JSSMFE, pp.1907-1910 (in Japanese).
O'Rourke,T.D. and Jones,C.J.F.P. 1990. Overview of earth retention systems: 1970-1990, Design and Performance of Earth Retaining Stuctures, Geotechnical Special Publication No.25, ASCE, Lambe and Hansen (ed.), pp.22-51.
Piggs,D.R. and McCafferty,W.R. 1984. The design and construction of a reinforced soil retaining wall at Loa Southwick, Sunderland, Polymer Grid Reinforcement, Thomas Telford.
Plumelle,C. and Schlosser,F. 1991. A French national research project on soil nailings: Clouteree, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, pp.219-223.
Plumelle,C., Schlosser,F., Delage,P. and Knochenmus,G. 1991a. French national research project on soil nailing: CLOUTERRE, Design and Performance of Earth Retaining Stuctures, Geotechnical Special Publication No.25, ASCE, Edited by Lambe and Hansen, pp.660-675.
Resl,S., Schneider,H. and Werner,G. 1988. Theory abd practice of 'Reinofrcing' steep slopes with nonwoven geotextiles, Proc. of Int. Geotechnical Sympo. on Theory and Practice of Earth Reinforcing, IS Kyushu'88, Fukuoka, pp.565-570.
Schlosser,F. and Juran,I. 1980. Design parameters for artificial improved soils, General Report for Session 8, Proc. 7th European Regional Conference on SMFE, Vol.4, pp.197-224.
Schlosser,F. and Juran,I. 1983. Behaviour of reinforced earth retaining walls from model studies, Chapter 6, 3.3.3 Modelling the Facing, Developments in SMFE-1, Applied Science Publishers, pp.197-229.
Schlosser,F. 1990. Mechanically stbilized 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.
Schlosser,F. and Unterreiner,P. 1991. Soil nailing in France: research and practice, Transportation Research Record, No.1330, Soils, Geology, and Foundations, Behaviour of Jointed Rock Masses and Reinforced Soil Stuctures, pp.72-79.
Schlosser,F. and De Buhan,P. 1991. Theory and design related to the performance of reinforced soil, S-O-A Report, Performance of Reinforced Soil Structures, McGown et al.(ed.), Thomas Telford, London, pp.1-14.
Simac,M.R., Christopher,B.R. and Bonczkiewics,C. 1990. Instrumented field performance of a 6 m geogrid soil wall, Proc. 4th Int. Conf. on Geotextiles and Geomembranes and Related Products, Vol.1, pp.53-59
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.
Sonomura,S., Onodera,S., Ebe,T. and Nakai,S. 1991. Centrifuge model tests on soil reinforcing (Part 1), Proc. 46th Annual Conf. of the Japanese Society for Civil Engineers, Vol.III, pp.742-743 (in Japanese).
Stocker,M., Korber,G.W., Gassler,G. and Gudehus,G. 1979. Soil nailing, C.R. Coll. Int. Renforcement des Sols, Paris, pp.469-474.
Stocker,M. and Riediner,G. 1990. The bearing behaviour of nailed retaining structures, Design and Performance of Earth Retaining Stuctures, Geotechnical Special Publication No.25, ASCE, Lambe and Hansen (ed.), pp.612-628.
Tamura,Y., Tateyama,M., Murata,O., Tatsuoka, F. and Nakaya,T. 1992. Full-scale horizontal loading tests on column structures in reinforced embankments with a rigid facing, Proc. 27th Japan National Conf. on SMFE, JSSMFE, Kochi, pp.2127-2130 (in Japanese).
Taniguchi,Y., Tateyama,M., Saito,T., Toriihara,M., Kami,C. 1992: Application of soil nailing with pre-propping for a temorary wall, Proc. of the 47th Annual Conf. on the Japan Society of Civil Engineers, III, pp.1214-1215.
Tateyama,M., Murata,O. and Tatsuoka,F. 1990. Shaking table test on embankment with short reinforcement and rigid facing, Proc. 25th Japan National Conf. on SMFE, pp.2023-2026 (in Japanese).
Tateyama,M., Tamura,Y., Tarumi,H. and Tatsuoka,F. 1992. Permanent cut of an embankment slope by soil nailing allowing very small deformation, Proc. Int. Sympo. on Earth Reinforcement Practice (IS Kyushu '92), Fukuoka, pp.555-560.
Tateyama,M., Murata,O., Tamura,Y., Tatsuoka, F. and Nakaya,T. 1992a: Full-scale horizontal loading tests on crest of rigid facing

for the reinforced embankments, Proc. 27th Japan National Conf. on SMFE, JSSMFE, Kochi, pp.2153-2156 (in Japanese).
Tateyama,M., 1992. Private communication.
Tatsuoka,F., Ando,H., Iwasaki,K. and Nakamura,K. 1986. Performance of clay test embankments reinforced with a non-woven goetextile, Proc. of third Int. Conf. on Geotextiles, Vienna, Vol.3, pp.355-360.
Tatsuoka,F. and Yamauchi,H. 1986a. A reinforcing method for steep clay slopes with a non-woven geotextile, Geotextiles and Geomembranes Vol.4, pp.241-268.
Tatsuoka,F., Tamura,Y. Nakamura,K., Iwasaki,K. and Yamauchi,H. 1987. Behaviour of steep clay embankments reinforced with a non-woven geotextile having various face structures, Proc. the Post Vienna Conf. on Geotextiles, Singapore, pp.387-403.
Tatsuoka,F. and Yamauchi,H. 1986. A reinforcing method for steep clay slopes using a non-woven geotextile, Geotextiles and Geomembranes, Vol.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., Murata,O., Tateyama,M., Nakamura,K., Tamura,Y., Ling,H.I., Iwasaki, K. and Yamauchi,H. 1991. Reinforcing steep clay slopes with a non-woven geotextile, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.141-146.
Tatsuoka,F., Tateyama,M. and Murata,O. 1989. Earth retaining wall with a short geotextile and a rigid facing, Proc. 12th ICSMFE, 2: 1311-1314.
Tatsuoka,F., Murata,O. and Tateyama,M. 1992. Permanent geosynthetic-reinforced soil retaining walls used for railway embank ment in Japan, Geosynthetic-Reinforced Soil Retaining Walls, Wu.(ed.), Balkema, pp.101-130.
Tensor Cooperation 1991. Design guidelines for Tensor geogrid reinforced soil walls with modular concrete facing units, Tensor Technical Note, August.
Thamm,B.R., Krieger,B. and Krieger,J. 1990. Full-scale test on a geotextile reinforced retaining structure, Proc. 4th Int. Conf. on Geotextilesand Geomembranes and Related Products, Vol.1, pp.3-8.
Thamm,B.R., Krieger,B. and Lesniewska,D. 1991. Full scale test of a geotextile reinforced soil wall, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.341-345.
Toriihara,M., Yamamoto,A. and Hirama,K. 1990. Model test of slope reinforced with steel bars (part IV), Proc. 25th Japan National Conf. on SMFE, JSSMFE, pp.2059-2060 (in Japansese).
Toriihara,M., Matsuomoto,S. and Hirama,K. 1989. Model test of reinforced embankments with geotextiles, Proc. 24th Japan National Conf. on SMFE, JSSMFE, pp.1853-1854 (in Japanese).
Toriihara,M., Yamamoto,A. and Hirama,K. 1991. Model test of reinforced embankments with short length geotextiles, Proc. 26th Japan National Conf. on SMFE, JSSMFE, pp.2183-2184 (in Japanese).
Veldhuijzen van Zanten (ed.) 1985. Geotextiles and geomembranes in civil engineering, Balkema, pp.431.
Vidal,H. 1978. The development and future of Reinforced Earth, Keynote Adress, Proc. of Sympo. on Earth Reinforecement, ASCE, pp.1-61.
Yamanouchi,T. 1986. Historical review of geotextiles in Japan, Geotextiles and Geomembranes, Vol.4, pp.165-178.
Yamauchi,H., Tatsuoka,F., Nakamura,K., Tamura,Y. and Iwasaki,K. 1987. Stability of steep clay embankments reinforced with a non-woven geotextile, Proc. Post Vien Conf. on Geotextiles, pp.370-386.
Wichter,L., Risseeuw,P. and Gay,G. 1986. Large-scale test on the bearing behaviour of a woven-reinforced earth wall, Proc. of third Int. Conf. on Geotextiles, Vienna, Vol.IV, pp.1073-1078.
Wichter,L. and Gay,G. 1986. Design, dimensioning, and construction of geotextile-soil retaining wall, Proc. of 8th Doanu-europe Conf. on SMFE, Nurnberg.
Woodward,S.J. 1991. Modification of soil nailed wall deflections, Performance of Reinforced Soil Structures, McGown et al. (ed.), Thomas Telford, pp.231-234.
Wu,J.T.H. and Helwany,H.B. 1990. Alleviating bridge approach settlement with geosynthetic reinforcement, Proc. 4th Inter. Conf. on Geotextiles and Geomembranes and Related Materials, Vol.1, pp.107-11.
Wu,J.T.H. 1992. Measured behaviour of the Denver walls, Geosynthetic-Reinforced Soil Retaining Walls, Wu.(ed.), Balkema, pp.31-41.
Wu,J.T.H., Qi,X, Chou,N., Ksouri,I. and Helwany,M.B. 1992. Comparisons of predictions for the Denver walls, Geosynthetic-Reinforced Soil Retaining Walls, Wu. (ed.), Balkema, pp.43-60.