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Technical report – Wall structures session

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ABSTRACT: The discussion points of the session on reinforced soil retaining wall (RW) structures are summarized. They are 1) the reinforcing mechanism that is relevant to reinforced RWs; 2) technical issues related to the major structural components of reinforced soil RWs, which are reinforcement (material and geometry); roles of facing; effects of backfill soil type; and effects of subsoil conditions; 3) roles of physical experiments in the laboratory and the field; 4) roles of numerical analysis of the deformation and stability of reinforced soil RWs and issues of design procedures; and 5) seismic stability of reinforced soil RWs.

1 INTRODUCTION

The following technical topics related to the reinforced soil retaining wall (RW) technology were discussed in this long session with an allocated time of four hours:

1) the reinforcing mechanism that is relevant to reinforced RWs;
2) technical issues related to the major structural components of reinforced soil RWs, in particular;
   a) effects of the constituting material and geometry of reinforcement members;
   b) roles of facing; effects of backfill material type; and effects of subsoil conditions
3) roles of physical experiments on reinforced soil RW models in the laboratory and the field;
4) roles of numerical analysis of the deformation and stability of reinforced soil RWs and design procedures; and
5) seismic stability of reinforced soil RWs.

Among a number of the papers that were prepared for this session, eleven papers were presented during the session, covering these sub-topics.

In the following, the background and present situations of these discussion points are summarized, together with comments of the author (as the chairman). The discussions made during the session and the responses of the authors to them are summarized separately by the discussion leader, Prof. CHEW Soon-Hoe of the National University of Singapore.

2 REINFORCING MECHANISM RELEVANT TO REINFORCED SOIL RETAINING WALLS

2.1 Definition of reinforced soil retaining walls

In the opinion of the author, reinforced soil walls and reinforced slopes are defined as follows:

1) *In the structural point:* With reinforced soil walls, the active zone located outside the potential failure plane, having the smallest safety factor in a reinforced soil RW, could not remain sufficiently stable and stiff without being supported with a relevant rigid facing structure on which some large earth pressure could act. On the other hand, with reinforced slopes, the active zone in a reinforced slope could remain stable enough without using such a facing structure as used for reinforced soil walls, although reinforced slopes become more stable and less deformable by being supported with a relevant rigid facing structure under otherwise the same conditions.
2) *In the analytical point:* The limit equilibrium stability analysis based on the earth pressure is relevant to the practical design of reinforced soil walls. On the other hand, the limit equilibrium stability analysis based on the overall moment is usually relevant to the design of reinforced slopes. It should be noted however that the theoretical basis and the framework of analysis pro-

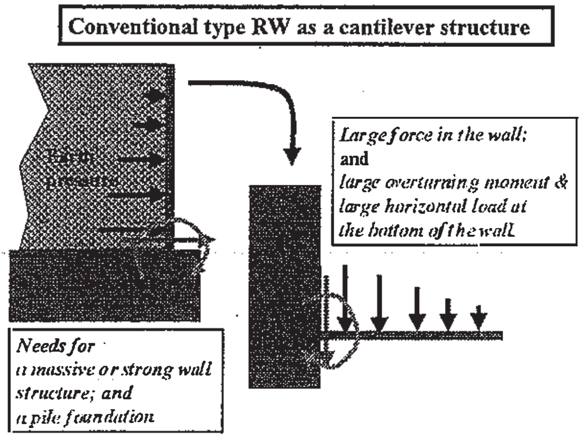


Figure 1 Illustration of conventional type RWs

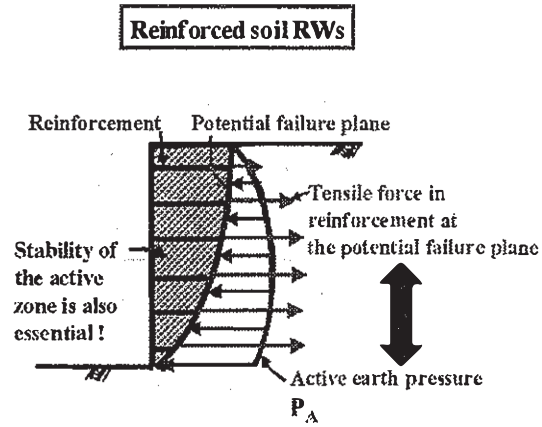


Fig. 2 Reinforcing mechanism in reinforced soil RWs

cedure for such earth pressure and slope stability problems are common in the nature.

The stability and deformation of reinforced soil structure changes continuously with continuous changes in the slope angle at the wall or slope face under otherwise the same conditions. Despite the above, reinforced soil structures are classified into separated categories (i.e., reinforced walls and reinforced slopes) in routine engineering practice. A slope angle of 70 degrees is often used as the boundary between the two types of reinforced soil structures (Jewell et al., 1984; Jewell, 1985). This separation criterion is however only for convenience in routine design. For example, two reinforced soil structures having slope angles of 69 and 71 degrees should be designed in nearly the same way under otherwise the same conditions. However, in the "handbook" engineering practice, by following the above-mentioned separation criterion, an unduly jump or discontinuity may be introduced in the design of these two reinforced soil structures.

In one of the reinforced soil structure projects that the author knows, a relatively high was designed and constructed to support a highway road. The reinforced soil structure was designed to consist of a so-called slope part and a so-called wall. The lower part was a "reinforced soil RW" with reinforcement layers of a non-woven and woven geotextile composite arranged at a relatively small vertical spacing having a somehow rigid facing structure. On the other hand, the upper and overlying part was a "reinforced slope" with reinforcement layers of the same composite as the one used for the wall, but arranged at a much larger vertical spacing while not having any facing structure. So, the arrangements of reinforcement layers and the facing conditions changed largely and suddenly at the boundary between the wall and the slope. The actual problem with this structure was that, despite that the backfill was a relatively high-water content clayey soil, the vertical spacing between reinforcement layers in the upper "reinforced slope" was too large to achieve a sufficiently high compacted dry density of the backfill and to ensure a sufficiently high restraint to the lateral deformation of the backfill in the zone near the slope face, which resulted into too large unequal residual settlements at the road face.

With reinforced walls, the backfill is usually relatively heavily reinforced and a relatively rigid facing structure is used by considering that the backfill having a nearly vertical wall face cannot be stable without using these measures. These measures can also contribute to a high degree of compaction of the backfill and a better restraint of wall deformation. On the other hand, reinforced slopes are relatively lightly reinforced while not using a facing structure due to a lower requirement for increasing the stability of the backfill. Such arrangements as above could result in a lower compaction density of the backfill, which may result into larger vertical residual compression of the backfill. It seems that the serviceability limit design evaluating the deformation of structure at working loads should be employed to avoid such a problem as described above. It is required to develop such a serviceability limit design procedure for reinforced soil structures.

2.2 Reinforcing mechanism relevant to reinforced soil RWs

In the author's opinion, the most important and essential difference in the basic mechanism between conventional type RWs and reinforced soil RWs is the following:

__*Conventional type RWs*__ **(Figure 1)**: Conventional type RWs are basically cantilever structures with the facing structure alone resisting against the earth pressure acting on its back face. In addition, the sta-

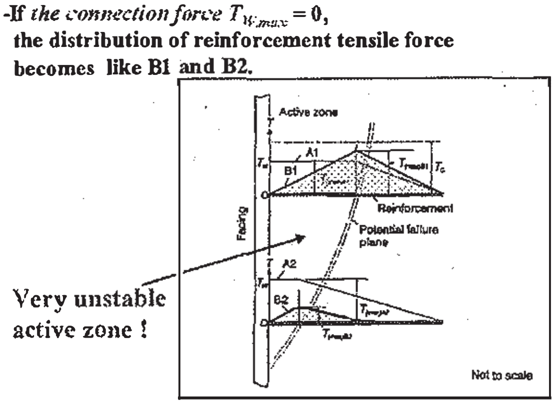


Figure 3 Distribution of tensile force in the reinforcement when no facing is used or when the reinforcement layers are not connected to the back of facing.

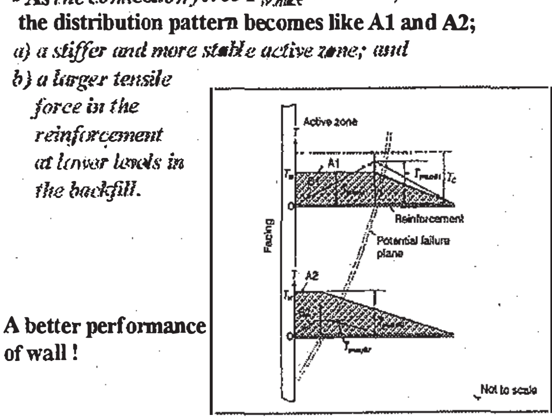


Figure 4 Distribution of tensile force in the reinforcement when there is a relatively stiff facing and the reinforcement is connected firmly to the back of facing.

bility of the facing structure is controlled by the stability against the lateral sliding force and overturning moment acting at the bottom of the facing structure. For this reason, the force acting inside the facing structure and at the bottom of facing structure is relatively large. The lateral sliding force and the overturning moment acting at the base of the facing structure increase approximately in proportional to respectively H^2 and H^3 (H: the wall height). Therefore, most of the conventional type RWs constructed on the subsoil that is not very stiff and strong is supported by a deep foundation (e.g., a pile foundation). It is particularly the case with conventional type RWs that are higher than say 5 m.

Reinforced soil RWs (**Figure 2**): The following two mechanisms are essential for the stability of reinforced soil RWs:

1) Tensile force developed in the reinforcement layers resist against earth pressure acting along all the potential failure planes having a safety factor that is lower than unity when the backfill is not reinforced. The force equilibrium along the failure surface having the smallest safety factor (i.e., the critical failure plane) for the concerned reinforced backfill controls most critically the stability of the reinforced soil RW. The reinforcement layers should have a sufficient capability to resist against the earth pressure acting along all the potential failure planes. By this mechanism, as long as the reinforcement layers are sufficiently long extending into the stationary zone located back the critical failure plane in the backfill, large lateral sliding force and overturning moment is not activated at the base of facing structure.
2) The active zone locating outside the critical failure plane (i.e., closer to the wall face) should be stable and stiff enough. This requirement cannot be satisfied only by means of reinforcement layers (even if they are very stiff and very long). The active zone could be somehow stable without using a rigid facing structure only when the reinforcement layers are planar while having a very small spacing between the vertically adjacent reinforcement layers and the backfill is well compacted to a high dry density. In that case, sufficiently large arching effects could be activated in each soil layer between vertically adjacent reinforcement layers immediately behind the wall face. However, it is the property of the soil not having a true cohesion that the compressive and shear strength and the stiffness decrease at a high rate with a decrease in the confining pressure approaching zero and become essentially zero when the confining pressure becomes zero. When the earth pressure acting at the wall face becomes zero by not using any rigid facing structure or without connection force between the back of a rigid facing and the reinforcement layers, the confining pressure in the backfill becomes very low and therefore the backfill immediately back the wall face, and eventually the whole active zone, would become weak and soft, as illustrated in **Figure 3**. On the other hand, **Figure 4** illustrates the distribution of tensile force in the reinforcement that could be activated when a relatively stiff facing is used and the reinforcement layers are connected firmly to the back of facing. In this case, it is possible and preferable to activate relatively large earth pressure by a good compaction of the backfill. In that case, the earth pressure at the back of facing may reach the earth pressure at rest in the unreinforced soil. In design, it would be relevant to assume that the active earth pressure when the backfill is not reinforced acts on the back of fac-

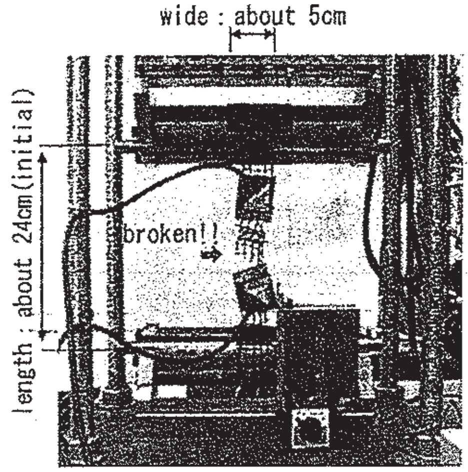
ing. In this case, the tensile force in the reinforcement layers becomes essentially constant in the active zone and the lateral confining pressure in the active zone becomes sufficiently large, at least to the level in the unreinforced backfill that is retained with a stable conventional type RW structure (Jewell, 1990). In the author's opinion, it is not wise to make the backfill of reinforced soil RWs weaker and softer than the unreinforced backfill retained by a conventional type RW.

It is ready seen from the above that the essential reinforcing mechanism of reinforced soil RWs is ***not*** "to reduce the earth pressure acting on the back of facing by means of tensile-reinforcing". Rather, it is to prevent by using tensile reinforcement layers the decrease in the strength and stiffness of backfill that is not retained with a stable cantilever wall structure. So, the role of facing is not simply "to prevent the spilling out of the backfill", but it is one of the essential structural components that should be able to ensure a high integrity of the whole wall system (e.g., Tatsuoka, 1992; 1997c).

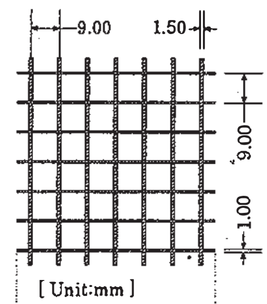
3 STRUCTURAL COMPONENTS OF REINFORCED SOIL RWS

3.1 Reinforcement (material and geometry)

Geosynthetic sheets or grids and metal strips or meshes are the two major tensile reinforcement types that are currently used in practice. On the other hand, the tensile reinforcement is often classified into two groups depending on the material stiffness, not on the performance of reinforced soil structures with the backfill that is reinforced with the concerned type of reinforcement. That is, the geosynthetic reinforcement is often classified as *extensible* reinforcement while the metal reinforcement as *inextensible* reinforcement despite that geometries and arrangements when used to reinforce the backfill could be largely different in different projects. This classification may give such a misleading notion that the geosynthetic reinforcement cannot function as good as the metal reinforcement and the deformation of geosynthetic-reinforced soil retaining walls could become too large. The above notion is however not based on facts, as discussed below.



Geogrid tested



A polymer geotextile

Figure 5 Setup of the tensile tests of geogrid (Hirakawa et al., 2001).

Technical issues related to geosynthetic reinforcement: It is true that the geosynthetic reinforcement has generally a much low material tensile stiffness and strength than the metal reinforcement. It should be noted however that most of the geosynthetic reinforcement that is used to tensile-reinforce the backfill of RW is planar with a much smaller vertical spacing than those employed with the metal reinforcement. Therefore, geosynthetic reinforcement layers usually have a much larger contact area with the backfill than the metal reinforcement. For this reason, it is usual that the deformation during construction of geosynthetic-reinforced RWs does not become too large because of a low stiffness of reinforcement layers, while the pull-out failure of reinforcement layers is not likely to occur. More importantly, some amount of wall deformation during construction usually does not become a serious problem unless the connection of the reinforcement and the facing is damaged due to too large relative settlement between them or the final wall face alignment becomes unacceptable or both.

Rather, it is a more serious problem if vertical or lateral residual deformation of the wall in service becomes too large, exceeding the allowable value. In relation to this problem, the geosynthetic reinforcement is often considered to be inferior to the metal reinforcement. However, this is a wrong notion and the following two points are important.

First, the backfill could exhibit much more large creep deformation than usual geosynthetic reinforcement. It is particularly the case when the safety factor for ultimate failure of the backfill is relatively low, which could result from:

1) inadequate arrangements of reinforcement layers (i.e., a too small total contact area with the backfill or a too small length or a too large vertical spacing between reinforcement layers); or
2) a relatively low degree of compaction of backfill (in particular, when using backfill containing a large amount of fines); or
3) the use of a too flexible facing structure (such as wrapped-around wall face).

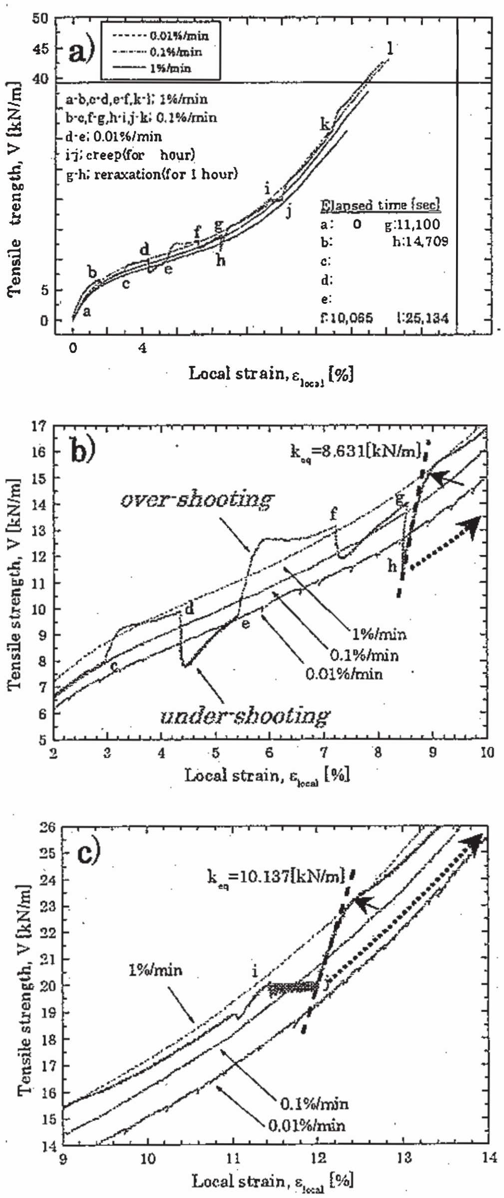


Figure 6 Relationships between tensile force and axial strain from three tests at different constant strain rates and a test in which the strain rate was changed stepwise and a stress relaxation test and a creep test were performed (Hirakawa et al., 2001).

As far as the author knows, most cases of prototype geosynthetic-reinforced soil RWs in which the post-construction residual deformation became an actual problem were associated with either or two or all of the three factors listed above.

Secondly, in the current design practice, the design tensile strength is considered to decrease with the increase in the design life, implicitly assuming that creep is a deteriorating phenomenon for geosynthetics. However this concept is utterly different from the real behaviour of geosynthetics. For example, the relationships between the tensile force and the axial strain obtained from tensile tests on a polymer geogrid (**Figure 5**) are presented in **Figure 6**. Among these tests, three monotonic loading tests were performed at three different constant strain rates. In the other test, the strain rate was changed stepwise and a stress relaxation test and a creep test were performed during otherwise monotonic loading at a constant strain. The axial strain was measured locally near the center of the specimen (Figure 5). The following trends of behaviour may be noted from Figure 6.

1) In the constant strain rate monotonic loading tests, the geogrid exhibited larger stiffness and strength at a higher strain rate. This behaviour is due to the viscous property of the test material, which is a phenomenon different from "ageing". "Ageing" is defined as changes in the material property with time, such as deteriorating changes in the property of geosynthetic by exposing to the UV light.
2) Although the period of creep test was short in one of these tests (Figure 6c), the following trend is evident. That is, the stress-strain behaviour immediately after loading was restarted at the original strain rate was very stiff (nearly the same as the elastic behaviour) and then it joined the original stress-strain curve that would have been obtained when the loading had continued at that strain rate without an intermission of creep test. This result indicates that creep is **not** a deteriorating phenomenon (unlike the negative ageing effect). Similar results from tests with much longer creep periods (up to several years) have been reported in the literature (e.g., Paulson and Bernardi, 1997).
3) The behaviour after loading was restarted at a constant strain rate following a stress relaxation test was essentially the same as the one after loading was restarted at the same constant strain rate following a creep stage (Figure 6b).
4) These two facts above indicate that the time that has elapsed since the start of loading until the same stress-strain state, for example the states indicated with an arrow in Figures 6b and 6c, could be totally different for different stress histories (e.g., between a pair of monotonic loading tests

performed at the same constant strain rate with and without a creep or stress relaxation stage).

In the current geosynthetic engineering practice, the isochronous theory is often used to predict time-dependent stress-strain behaviour of a given type of geosynthetic reinforcement. According to this theory, the current stress is controlled uniquely by the current strain and the time that has elapsed since the start of loading. The test results presented in Figure 6 (and others; e.g., Hirakawa et al., 2001) show that the isochronous theory is not relevant at all. For example, in the test results presented in Figure 6, when loading was restarted from point ***h*** (after a stress relaxation stage) and from point ***j*** (after a creep stage), according to the isochronous theory, the stress-strain behaviour should become like the ones denoted by the broken curves. The prediction of such behaviour as above leads to such a wrong notion that the ultimate strength decreases due to a history of stress relaxation or creep ! It has also been proven that the isochronous model is not able to explain the effects of viscous property observed with soils and rocks (Tatsuoka et al., 2000, 2001). The test results presented in Figure 6 show that the current stress is controlled basically by the current strain and its rate (more rigorously by the current irreversible strain, its rate and their history; Di Benedetto et al. 2002; Tatsuoka et al. 2002a). The simulation of the test results presented in Figure 6 by a constitutive model developed based on such a framework as above is reported by Hirakawa et al. (2002) and Tatsuoka et al. (2002b).

<u>Technical issues related to metal reinforcement:</u> Corrosion is one of the most important potential problems with this reinforcement type. So, the speed and amount of corrosion should be predicted when using this type of reinforcement. Galvanisation is the most widely used measures to alleviate this problem. In addition, because of a material high stiffness and strength, the geometry of metal reinforcement is usually strip or bar. For this reason, the contact area of metal reinforcement with the surrounding backfill is relatively small and therefore pull-out failure becomes more likely to occur than the tensile rupture failure of reinforcement. These two notions are relevant particularly when metal strip reinforcement, which does not have a function of drainage, is arranged in a high water content backfill including a large amount of fines, say more than 30 %.

3.2 Facing

It is argued in the precedent section that the function of facing structure is simply to prevent the spilling out of backfill, but it is one of the essential structural components of reinforced soil RWs. For this reason,

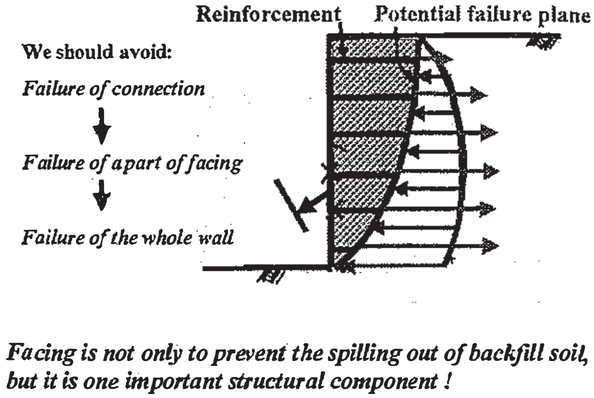


Figure 7 Schematic diagram illustrating the failure of a reinforced soil RW due to the failure of facing.

a high integrity of facing structure is usually very important. Referring to **Figure** 7, suppose that due to an inadequate arrangement of reinforcement layers (e.g., use of strip reinforcement with a too short length), the reinforcement layers at the low level of the wall are pull-out to some extent from the stationary zone located back of the potential failure plane (having the smallest safety factor). Suppose that the active zone outside the potential failure plane then slides down along the potential failure plane relatively to the facing structure. This relative movement damages the connection between the reinforcement layers and the back of the facing. When the facing is of discrete panel type, if a connection failure takes place at the back of a certain panel, it is quite possible that due to a lack of high integrity with discrete panel type facing, dropping off of a single panel from the facing system results into a chain-reaction failure of the whole facing system and then the whole of the wall system. Such a type of failure as above is unlikely to take place when the facing is of full-height rigid type because of a much higher degree of integrity.

The design philosophy of facing structure depends on the prediction or assumption with respect to the magnitude of earth pressure that would act on the back of facing. When it is predicted or assumed that essentially zero or very small earth pressure acts on the back of facing, due attentions to the stability of facing structure and the connection strength may not be paid in the design of reinforced soil wall. In such a case as above, even when the pull-out failure of reinforcement is not likely to take place, for example by using planar reinforcement (such as a geogrid), reinforced soil RWs may fail due to the failure of facing system or connection failure or both.

A typical example of the above is the failure of several geogrid-reinforced RWs with a facing structure consisting of concrete blocks during the 1999 ChiChi Earthquake, Taiwan (**Figure 8**). In the au-

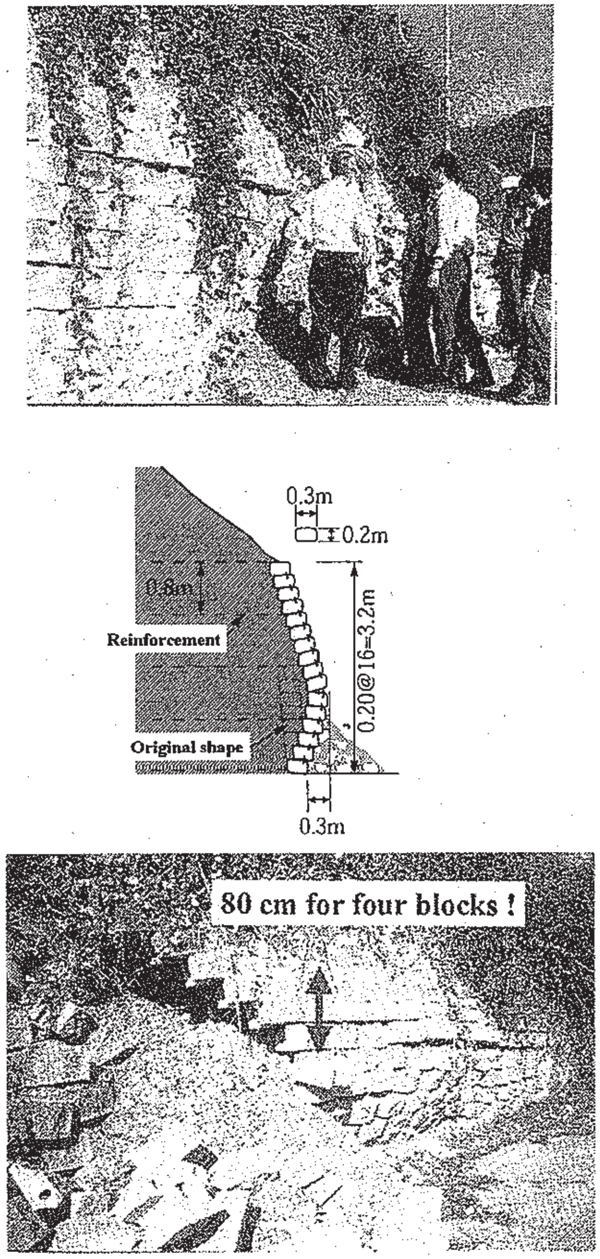


Figure 8 Failure of a geogrid-reinforced woil wall having a discrete block facing during the 1999 ChiChi Earthquake, Taiwan

thor's opinion, the main causes for the failure of these walls are firstly a too large vertical spacing of reinforcement layers (i.e., 60 cm) and secondly a very small connection strength between the reinforcement layers and the facing. It seems to the author that such arrangements as above resulted from such a design concept that nearly no or very small earth pressure would act at the back of the facing.

One may say that reinforced soil retaining walls could be stable enough even without using a rigid facing or without ensuring a high connection strength between the rigid facing and the reinforcement layers, because arching effects could be activated immediately back the wall face in each soil layer between vertically adjacent reinforcement layers, making the earth pressure acting at the wall face very small. However, the following remarks are important:

1) The arching effects become less reliable as the ratio of the particle size of backfill to the vertical spacing of reinforcement layers becomes smaller, as the backfill becomes less compacted, and as the contact area between the backfill and the reinforcement becomes smaller.
2) The arching effects may be largely lost by severe shaking during a major earthquake and by a significant drop in the suction in the backfill due to the percolation of rainwater.

It also seems to the author that it is not relevant to rely on such arching effects as above when the allowable deformation of wall is relatively small.

3.3 Backfill

The currently important discussion points with the backfill include the following:

Whether on-site soil including a large amount of high water content clay can be safely used as the backfill of a reinforced soil RW: The answer depends on many factors including the following:

1) Whether a relevant drainage system is provided in the backfill.
2) Whether relevant measures is taken to prevent the percolation of rainwater into the backfill.
3) Whether the reinforcement has a sufficiently high capacity for the pull-out failure.
4) Whether the backfill can be compacted reasonably well.

The use of reinforcement having a function of drainage as well as a function of tensile reinforcement, such as a composite consisting of non-woven and woven geotextiles, together with a relatively small vertical spacing (say 30 cm) is one of the possible solutions to alleviate the problems 3) and 4) (Tatsuoka et al., 1997c).

The relationship between the design strength of backfill and the peak and residual strengths obtained from laboratory stress-strain tests (such as triaxial compression tests or plane strain compression tests): This issue is also relevant to the unreinforced backfill of retaining wall structures and unreinforced embankments. The design shear strength employed in the usual geotechnical engineering practice is close to the residual strength of a given type of soil. For example, an angle of internal friction of 35 degrees is often used for sand backfill. It is true however that this conventional design methodology cannot take into account the effects of compaction level and backfill type, because the residual strength does not reflect these effects. For this reason, this design methodology could be too conservative when the backfill is a well-compacted well-graded soil with a relatively large particle size, having a peak shear strength that is much higher than the residual strength.

To alleviate the above-mentioned drawback of using the residual strength in the design, although it is only partly, the use of both peak and residual strengths in the limit equilibrium stability analysis has been proposed by Koseki et al. (1997) for the estimation of dynamic earth pressure (i.e., a modification of the Mononobe-Okabe method); and by Leshchinsky (2000) for the stability analysis of reinforced soil slopes. In their methods, the location of the critical failure plane is determined by using the peak strength of soil and the limit equilibrium along the critical failure plane is examined by using the residual strength of soil, assuming that the strength along the failure planes (i.e., inside the shear bands) has dropped quickly from the peak value to the residual value until the ultimate failure of the concerned soil mass is reached. In reality, the rate of stress drop with the increase in the shear displacement along the failure plane (i.e., the shear deformation of shear band) becomes slower with the increase in the particle size (Yoshida et al., 1995; Yoshida and Tatsuoka, 1997). For this reason, it is quite possible that, as the particle size of backfill becomes larger under otherwise the same conditions, the deformation of soil structures, including reinforced soil RWs, that are undergoing progressive failure becomes smaller, showing more ductile behaviour.

3.4 Subsoil conditions

Failure of the ground supporting a reinforced soil RW should be avoided. Reinforced soil RWs (and also soil structures in general) are generally flexible and can accommodate the deformation of ground to a much larger extent than the value allowed with conventional type RWs (i.e., masonry or reinforced concrete structures). For this reason, most reinforced soil RWs are not supported with a deep foundation, such as a pile foundation, unless the ground is very weak and soft.

On the other hand, when a somehow axially rigid facing structure (such as discrete panel facing, modular block facing and full-height rigid facing) is constructed before or during constructing the backfill, too large deformation of the supporting ground may damage the connection between the facing and the reinforcement layers. Too large deformation of the supporting ground may also make the final alignment of wall face unacceptable. To alleviate these problems, a staged construction procedure has been proposed (Tatsuoka et al., 1997c). In that method, after most of the potential deformation of the backfill and the supporting ground has taken place, a full-height rigid facing is cast-in-place on the face of the wall that has been completed by placing gravel-filled bags at the shoulder of each soil layer.

4 ROLES OF MODEL TESTS IN THE LATBORATORY AND IN THE FIELD

4.1 General

Basically, there are the following two categories of model tests of reinforced soil RWs:

1) relatively small scale ones that are performed in the laboratory; and
2) relatively large scale, or full scale, ones that are performed in the field.

In the former category, there are two types of tests, which are so-called 1g model tests and centrifuge model tests. Although there is a strong criticism on geotechnical 1g model tests, it should be noted that either of these two types of laboratory model tests could not be perfect nor ideal, having respective inherent limitations, as discussed below.

4.2 Scale effect

First of all, it is known that results from geotechnical 1g model tests using the same type of soil as the prototype are subjected to the effects of the size of a structure model, such as a strip footing. Tatsuoka et al. (1991), Tatsuoka (2001) and Siddiquee et al. (1999, 2000) showed that the so-called scale effect involved in the bearing capacity of sand ground subjected to footing loading consists of;

a) pressure level effect; and
b) particle size effect.

The pressure level effect can be observed by changing the pressure level while keeping the ratio of the particle size to the footing size constant in model tests. So, the pressure level can be observed typically by changing the acceleration level in a centrifuge test using the same structural model and the same type of sand. On the other hand, the particle size effect can be observed by changing the ratio of the particle size to the model size (e.g., the footing width) while keeping the pressure level in the model constant in model tests. So, the particle size effect can be typically noted by comparing results from a set of model tests using the same type of sand and keeping the same pressure level, but, in these tests, the ratio of the particle size to the footing size is different using different model sizes (e.g., different footing widths) while employing different acceleration levels (e.g., Siddiquee et al. 1999). The particle size can also be noted by comparing results from a set of model tests performed at the same pressure level using the same model size (e.g., the same footing width), but, in these tests, different types of sand having different particle sizes but having the same pre-peak strength and deformation characteristics and peak strength are used (e.g., Tatsuoka et al. 1997a; Tatsuoka 2001).

The importance of reproducing the prototype pressure level in geotechnical model tests is not due simply to the fact that the shear strength and tangent stiffness of soil changes with a change in the pressure level. That is, if the shear strength and the tangent stiffness for all stress paths of soil were perfectly proportional to the pressure level, it would enough to perform relevant 1g model tests to predict the behaviour of a given prototype structure. This is because both force and displacements in that model test are perfectly proportional to those of the prototype. In actuality, however, the stress-strain properties of soil are not like the above. For model tests on reinforced soil RWs, not only dimensions but also strength and stiffness of prototype reinforcement and facing structures should be properly scaled down, which is sometimes very difficult, particularly with structural members having a locally complicated three-dimensional structure (such as a grid reinforcement).

4.4 Particle size effect

The origin of the particle size effect in geotechnical physical model tests is such material behaviour of soil that each shear band has an intrinsic thickness (i.e., a characteristics length) that is basically proportional to the particle size (Yoshida et al., 1995; Yoshida and Tatsuoka, 1997). The changing rate of stress state in the post-peak regime in a shear band is controlled by the shear strain of shear band. Therefore, for the same amount of shear displacement of shear band, the post-peak change in the stress state in the shear band becomes different for different particle sizes under otherwise the same conditions. Consequently, for the same amount of deformation of a soil mass in a model test, the change in the stress state in the soil mass including a shear band (or bands) is different for different particle sizes of soil.

In addition, the local failure mechanism in the backfill adjacent to each reinforcement layer may also change with a change in the ratio between the local geometric dimension of the reinforcement (e.g., the aperture and thickness of grid) and the particle size of the backfill soil. The ratio between the vertical spacing of reinforcement layers and the particle size would also affect the arching effects in each soil layer between vertically adjacent reinforcement layers. If the same sand type as the prototype is used in a small-scaled model, the arching effects in the soil layers could be more easily activated in smaller model tests.

It is readily seen from the above that the use of the prototype sand type in small-scaled models could be equivalent to the use of gravel in the prototype, which could result into an overestimation of the stiffness and stability of the prototype structure by the model tests.

4.5 1g versus centrifuge model tests

If the failure mechanism in the 1g model tests is principally different from the prototype structure, results from the 1g model tests could be misleading to predict the failure of the concerned prototype structure. The author considers that it is possible to make the failure mechanism in the 1g model tests principally the same with that of prototype structures. In that case, even when results from 1g tests are quantitatively different from the behaviour of the concerned prototype structure, effects of influencing factors (such as reinforcement arrangement, facing rigidity and connection strength between the facing and the reinforcement) could be adequately evaluated by 1g model tests. In addition, when a numerical analysis procedure, such as FEM analysis, can be validated by results from relevant 1g model tests, we can expect that this numerical analysis procedure can also be applied to prototype structures.

There are a number of advantages with 1g model tests, including the following:

1) relatively low cost for the construction and maintenance of model test facilities and for test running;
2) relatively easy model preparation, in particular for reinforced soil RW models having complicated structural components; and
3) relatively easy loading procedure and relatively easy observation of model behaviour.

In summary, in the author's opinion, it is not relevant to conclude that 1g model tests are always inferior to centrifuge model tests as a tool to investigate into the deformation and failure of reinforced soil RWs. Rather, it is important to understand that either of the two model testing methods has inherent advantages and limitations. Therefore, we should take advantage of both types of model tests.

5 NUMERICAL ANALYSIS AND DESIGN PROCEDURE

5.1 General

Basically the following two types of numerical methods are currently used in the design and research of reinforced soil RWs:

a) limit equilibrium-based stability analysis, which is the primary tool for stability analysis of reinforced soil RWs in routine design practice; and
b) FEM analysis, which is used to predict the deformation and sometimes the failure of reinforced soil RWs primarily for research purposes.

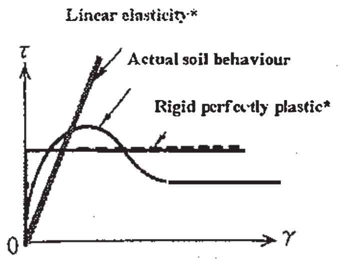


Figure 9. Schematic diagram illustrating the simplifications of actual stress-strain property of soil in numerical analysis.

5.2 Limit equilibrium analysis

The limit equilibrium analysis is full of mathematically inconsistencies when compared with more mathematically rigorous methods (such as the stress characteristics method and the limit analysis). It should be noted however that the most serious limitation of the limit equilibrium analysis is not such mathematical drawbacks, but it results from the use of a largely simplified (or over-simplified) stress-strain properties of soil used in the analysis (**Figure 9**). That is, in the usual limit equilibrium-based stability analysis of unreinforced and reinforced soil structures, the stress-strain behaviour of soil is assumed to be ***isotropic rigid-perfectly plastic with failure planes having zero thickness***. This assumption is equivalent to assume that the failure in a soil mass is not progressive in the sense that the peak shear strength of soil is mobilized simultaneously all along the failure planes (i.e., the failure is not progressive). This assumption becomes less realistic as the rotation of the direction of failure plane in a soil mass becomes larger. Therefore, we should be careful with results from this type of analysis because of this inherent limitation. One method to alleviate this problem (despite that it is not totally) is to use both the peak and residual strengths of soil as mentioned earlier in this report.

5.3 FEM

The FEM numerical analysis is much more advanced than the limit equilibrium method. It is therefore necessary to use a more soil-like constitutive model to balance with a more sophisticated analysis procedure. For example, for the analysis of the failure of sand specimens reinforced with a grid and other types of reinforcement in plane strain compression tests and the failure of level sand ground reinforced with metal strips loaded with a strip footing, Kotake et al. (1999, 2001, 2002) and Peng et al. (2000) used a realistic constitutive model of sand. That is, the constitutive model takes into account the fact that the strength and deformation characteristics of sand are inherently ***anisotropic***, strongly ***non-linear*** by the effects of strain and pressure in the pre-peak regime with a ***large peak dilatancy angle***, exhibiting a strong trend of post peak ***strain-softening*** by ***strain localization into a shear band(s) having a thickness in proportion to the mean diameter of particle***. They concluded that the use of a realistic constitutive model for sand is imperative for realistic numerical analysis (particularly in the failure analysis). For this reason, we should be careful with results from FEM analysis using a simplified soil model (such as isotropic elasto-perfectly plastic with failure planes having zero thickness, as used in usual limit equilibrium-based stability analysis).

The following issues are also currently important for the FEM analysis of reinforced soil RWs:

1) For 2D plane strain FEM analysis (or other types of numerical analysis), prototype reinforcement layers having a locally 3D structure (e.g., strip and grid) should be properly modelled into 2D model reinforcement layers (i.e., planar reinforcement layers). For example, two different types of prototype reinforcement layers, strip and planar reinforcement layers, having the same stiffness per unit width of reinforcement layer should be modelled into different model planar reinforcement layers for 2D plane strain FEM analysis. Peng et al. (2000) proposed the use of model planar reinforcement layers having an equivalent angle of friction at the interface between the reinforcement layer and the backfill while having the same stiffness with given prototype geogrid reinforcement. The equivalent angle of friction is a function of covering ratio and stiffness of the geogrid. They showed that the equivalent angle of friction increases particularly with the increase in the covering ratio.
2) Both backfill and geosynthetic reinforcement are elasto-viscoplastic materials. Therefore, creep deformation of reinforced soil RWs during service can be properly analyzed only by numerical analysis based on proper elasto-viscoplastic constitutive models of both backfill and geosynthetic reinforcement. It is also the case with the creep deformation of metal strip-reinforced soil RWs although the viscous property of metal reinforcement could be insignificant. In such numerical analysis, the stress path and history in the backfill and reinforcement are not as simple as in those based on elasto-plastic material properties. For example, even under constant boundary load conditions, local stress states may not be constant due to the viscous properties of backfill soil and reinforcement. The development of relevant constitutive models for the backfill and geosynthetic reinforcement having elasto-

viscoplastic properties as well as relevant numerical analysis procedures is one of the research topics that will have to be studied. In this respect, at least we can say that the isochronous theory is not relevant for constitutive modelling of both backfill and geosynthetic reinforcement.

6. SEISMIC DESIGN

Experiences from recent major earthquakes, including the 1995 Hyogo-ken Nambu (Kobe) Earthquake (Tatsuoka et al. 1996, 1997b & c, 1998) and the 1995 Northridge Earthquake (White and Holtz 1997), have shown that the seismic performance of reinforced soil RWs could be generally much better than that of conventional-type RWs, in particular gravity-type RWs. Results from a set of laboratory model shaking table tests supports the above (e.g., Murata et al. 1994; Koseki et al. 1997; Bathurst and Alfaro 1998; and Tatsuoka et al. 1998). In this respect, the following remarks are important:

1) As stated in the precedent sections, several geogrid-reinforced RWs with a facing structure consisting of concrete blocks failed during the 1999 ChiChi Earthquake, because of a too large vertical spacing between the reinforcement layers and a very small connection strength between the facing and the reinforcement layers. Perhaps, this configuration was due to design without paying due attentions to the stability of block facing and the connection strength, assuming nearly no earth pressure acting at the back of the facing.
2) A number of steel-reinforced soil RWs having a discrete panel facing survived high seismic load during the 1995 Hyogo-ken Nambu Earthquake. However, some of them exhibited relatively large relative displacements between adjacent panels as well as relatively large deformation at the wall face (Tatsuoka et al., 1997b). They were rebuilt because of the above.

7. SUMMARY

In this report, several discussion points that are considered relevant to this session on reinforced soil retaining wall structures are summarized, together with the personal views of the author on them. It may be seen from the above that despite that the reinforced soil RW is now a rather established technology, there are still a number of uncertain and poorly understood aspects that warrant more study in the future.

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