Earth Reinforcement Practice, Ochiai, Hayashi & Otani (eds) © 1993 Balkema, Rotterdam, ISBN 90 5410 093 1

Keynote lecture: A review of the behaviour of reinforced soil walls

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ABSTRACT: The results from published field and laboratory studies of the behaviour of reinforced soil walls are reviewed in the context of findings from recent numerical studies. It is shown that the behaviour of reinforced soil walls involves an interaction between the soil, the reinforcement, the facing and the foundation. A number of methods of analysis proposed for use in the design of these structures are reviewed with regard to the observed response of four instrumented walls.

I INTRODUCTION

In the past decades, empirical experience has shown that reinforced soil walls can be conveniently and economically constructed using extensible geosynthetic reinforcing elements within a soil mass, together with wall facings ranging from rigid full face concrete panels to a flexible geosynthetic "skin". Unfortunately, despite the fact that a great many reinforced soil walls have been safely constructed using extensible reinforcement and are performing well to date, the roles of the different components comprising these systems are still not well understood. To some extent, the use of geosynthetics as reinforcement preceded the development of suitable methods for analysis and design, and at present there is still no commonly accepted design standard.

The force in the reinforcement and the displacement at the face of reinforced soil walls after construction are usually much smaller than implied by current design methods. For example, even when a wall has been designed to fail (i.e. Factor of Safety = 1), failure typically occurs at a much higher load than anticipated based on current design methods (e.g. Billiard and Wu, 1991). Rimoldi (1988) examined eight case histories involving reinforced soil walls and steep slopes using extensible reinforcement and concluded that the factor of safety against reinforcement rupture was exceptionally high, suggesting that current design procedures are over conservative (see Table 1).

The conservatism present in conventional design arises from three main sources. Firstly, because of the uncertainties involved in construction control (e.g. non-uniformity in compaction and fill density), lower than actual shear strength parameters are usually specified for design.

Secondly, there are various degrees of strength loss in extensible reinforcement due to factors such as: ultraviolet (UV) light degradation, installation damage, creep, biological degradation, chemical aging (i.e. degradation mechanisms due to oxidation, environmental stress cracking, and hydrolysis). Strength losses due to installation damage and UV light degradation (unless the facing is left uncovered) occur at the initial stage of construction. Strength losses due to the other sources occur throughout the life of the wall. Because of the lack of long term test data on these degradation mechanisms, their effect on the strength loss in extensible reinforcement cannot be predicted precisely. Typically the

Table 1. Partial factors of safety for eight instrumented reinforced soil walls and steep slopes (modified from Rimoldi, 1988).

Factor of Safety	Kagoshima	Tucson	Lithonia	Gaspe	Cascade	Kingston*	Kingston[1]	Modena
FS1	0.9-2.3	0.9-3.2	-	1.05-2.4	0.75	2.16	1.93	2.9
FS2	2.7	1.2	11.6	1.21	1.14	-	-	3.6
FS3	2.5-6.1	1.2	-	1.28-2.9	0.85	-	-	10.9
FS4	11.3-27.0	11.5-32.0	-	9.1-21.0	3.0	32.0	11.4	27.2
FS5	4.5-10.8	4.2-11.6	-	2.4-5.6	2.4	11.6	4.1	13.6

FS1 = α_c/α_m FS2 = α_d/α_c FS3 = α_d/α_m FS4 = α_f/α_m FS5 = α_a/α_m
α_c = average tensile force calculated using in-situ parameters
α_m = measured tensile force or tensile force deduced from strain
α_d = average tensile force calculated using design parameters
α_f = peak tensile strength
α_a = allowable tensile strength
* full panel [1] discrete panel

allowable force in the reinforcement is obtained by dividing the short-term wide width tensile strength by a series of partial factors (see Allen, 1991 for a summary). Thus the specified allowable tensile strength to be used in the reinforcement usually amounts to a small fraction of the peak tensile strength determined by conventional tensile tests.

Finally, there are four main components of a reinforced soil wall: the fill, the reinforcement, the facing, and the foundation. Most analytical and design methods are either empirical in nature or based on limit equilibrium analysis calculations which do not consider deformations or interactions between the individual components of the wall system. As a consequence, these methods cannot adequately describe the real behaviour of reinforced soil walls. Hence, their application typically introduces an extra level of conservatism.

An additional problem in understanding the behaviour of reinforced soil walls is the limited amount of experimental or field data and the apparent inconsistencies evident in the interpretation of the data obtained from instrumented cases.

This Paper reviews some of the findings available from the literature relating to the behaviour of reinforced soil walls, with special consideration being given to horizontal deformation. A number of current design or analytical methods are reviewed. Results obtained from these methods are then compared with field performance of four case histories. The implications and findings from this study are discussed.

2 GENERAL CONSIDERATION

In the design of reinforced soil walls both external stability and internal stability are considered. In external stability analysis, the reinforced soil structure is treated as a conventional retaining structure and stability is checked against block sliding, overturning, tilting or bearing, and overall slope failure using existing methods of analysis for conventional retaining structures. Christopher et al. (1989) and Mitchell and Villet (1987) provided details regarding the evaluation of each of the external failure modes. Minimum factors of safety typically recommended for these modes of failure are as follows:

(a) sliding: 1.5
(b) overturning: 2.0
(c) bearing capacity: 2.0
(d) overall slope stability: 1.5

external stability analyses of reinforced soil walls are not considered in this paper.

Internal stability analyses consider two failure modes for the reinforcement:

(a) Pull-out failure of the reinforcement due to insufficient anchorage length behind the failure surface.

(b) Tensile failure of the reinforcement along the plane of maximum tensile force.

In addition to internal stability, horizontal deformation is also an important consideration for reinforced soil walls (especially when using extensible reinforcement).

The behaviour of reinforced soil walls may be considered to be understood if one understands:

(a) The state of stress within the reinforced soil mass (for the verification of the existence of a limit state).

(b) The state of strain in both the soil and the reinforcement (for the assessment of strain compatibility and the bond length required for internal stability).

(c) The force distribution in the reinforcement (for the evaluation of the factor of safety against reinforcement breakage, the location of the potential failure plane and the bonding force required).

(d) The horizontal soil pressure acting at the back of the reinforced soil mass and the vertical soil pressure at the base (for external stability assessment).

(e) The horizontal soil pressure acting at the face (for the design of facing).

(f) The vertical soil stress on each reinforcement layer (for the assessment of the available anchorage force against pull-out of the reinforcement from the soil).

(g) The horizontal and vertical forces transferred to the wall face (for the assessment of the adequacy of the facing foundation).

(h) the horizontal deformation of the reinforced soil mass (to ensure no excess deformation).

(i) the effect of varying the design parameters (i.e. reinforcement stiffness, soil properties, facing stiffness, foundation stiffness, surcharge condition, construction procedures, etc.) on the response of the system.

These issues will be discussed in the following sections.

3 VERTICAL STRESS DISTRIBUTION

In some design methods (e.g. Steward et al., 1977 - revised 1983; Bonaparte et al., 1987) it is necessary to make some assumptions regarding the vertical stress distribution within, and at the base of the reinforced soil mass for the calculation of: (a) the maximum force that can be carried by the reinforcement (i.e. for strength and anchorage considerations), and (b) the factors of safety against external stability. The validity of typical assumptions will be discussed below.

3.1 *Design assumptions concerning vertical stress distribution due to self weight and a uniform surcharge*

It is usual to assume that the vertical soil stress distributions, both within the reinforced soil mass and at its base, follow one of the three types of distribution as shown in Figure 1.

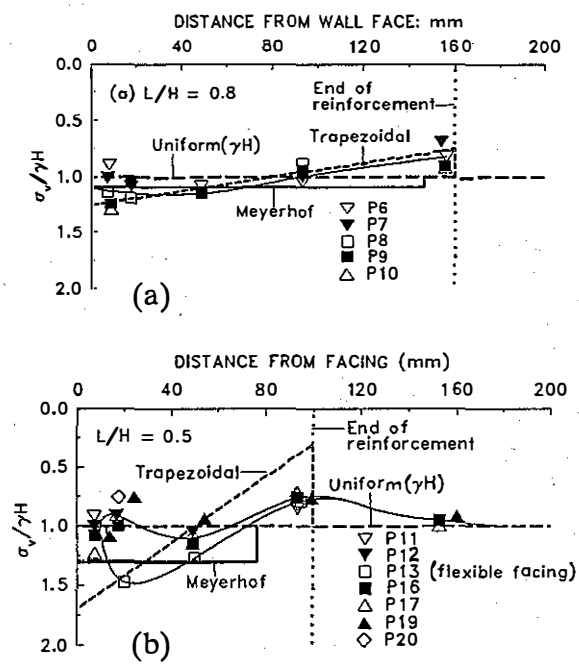

Figure 1. Pressure distribution - a comparison study: (a) L/H = 0.8, and (b) L/H = 0.5 (modified from Bolton and Pang, 1982).

The assumption of either a trapezoidal or Meyerhof distribution stems from conventional consideration of external stability where the reinforced soil mass is assumed to act as a rigid body. Static equilibrium of the reinforced

soil mass requires that under the action of the lateral thrust from the unreinforced retained fill the vertical stresses in the front of the reinforced soil mass are higher than those at the back and the effect increases towards the bottom. An implicit assumption behind both the trapezoidal distribution and Meyerhof type distribution is that there is a discontinuity in the vertical stress distribution close to the back of the reinforced soil mass.

3.2 *Measured vertical stress distribution due to self-weight and a uniform surcharge*

Observations of vertical soil stress distribution in laboratory models at or close to the base of the reinforced soil mass, subject to self-weight and a uniform surcharge, demonstrate evidence of an effect of the lateral thrust from the unreinforced retained fill but also deviate from the three common assumptions as shown in Figure 1 (Bolton and Pang, 1982). Similar observations can be made from results for full size walls (e.g. Wawrychuk, 1987).

In general, close to the facing, the measured vertical soil stress (σ_v) is less than the nominal overburden pressure (i.e. $\sigma_v = \gamma z + q$, where γ is the unit weight of fill, z is the depth below wall surface, and q is the intensity of uniform surcharge). Away from the facing the soil stress increases gradually and reaches a maximum exceeding the nominal value within the reinforced zone. Close to the end of reinforced zone the vertical soil stress reaches a minimum with a magnitude less than the nominal value. Further away into the unreinforced retained fill the vertical soil stress attains the nominal value.

The observed vertical soil stress distributions shown in Figure 1 appear to be related to: a) the ability of the reinforced soil mass to behave as a rigid body, b) the frictional resistances at both the fill/facing interface and the reinforced backfill/unreinforced retained fill interface, and c) the lateral thrust from the unreinforced retained fill.

The lateral thrust pushes the reinforced soil mass at the back producing a moment about the toe which tends to induce the highest stress at the front of the reinforced soil mass and the lowest stress at the back (in order to satisfy static equilibrium), provided that there are no interface frictional resistances at the two vertical boundaries. However, partial transfer of vertical stress from the fill to the facing through fill/facing friction (due to relative movement between the two) occurs at the front boundary and reduces the vertical stress locally such that the maximum vertical soil stress is forced to occur in the reinforced soil mass remote from the facing. Similarly, frictional resistance at the rear boundary between the reinforced backfill and unreinforced retained fill limits the decrease in vertical stress and forces the vertical soil stress to attain its nominal value at a location farther back into the unreinforced retained fill in order to satisfy continuity conditions. However, the magnitude of the interface shear resistance at the vertical boundaries, and the magnitude and location of the point of application of the lateral thrust are difficult to quantify.

3.3 *Factors affecting vertical stress distribution*

Small scale test results shown in Figure 1 indicate that as the reinforcement length to wall height ratio (L/H) increases and the vertical rigidity of the facing decreases (Test No.13) there is a tendency for the vertical stress to conform more to a trapezoidal distribution. The former appeared to result in the reinforced soil mass behaving more like a rigid body. The latter limited the transfer of vertical stress from the backfill to the facing (however the reverse will occur if the facing settles relative to the backfill).

The effect of facing rigidity on the vertical soil stress distribution was also studied by Tatsuoka et al. (1989) with small scale models. It was demonstrated that under similar loading conditions the magnitude of the vertical soil stress in the vicinity of the toe decreases with increasing vertical rigidity of the facing, suggesting that the rigidity of the facing does provide some stabilizing effect. Similar observations were also reported by Jaber (1989) in centrifugal tests and by Bathurst et al. (1988) in large scale model tests.

In addition, small scale test results reported by Andrawes et al. (1990a) indicate that there is a larger increase in vertical soil stress above the nominal value as the number of layers of reinforcement is increased, suggesting that the reinforced soil mass tends towards rigid body behaviour when it is more heavily reinforced.

Higher vertical stress (20%) in the front of the reinforced soil mass relative to the middle and at the back was observed by Allen et al. (1992) in a 12.6 m high geosynthetic reinforced soil wall comprising some 34 layers of reinforcement. This is qualitatively consistent with the finding from laboratory models as discussed above.

There is insufficient data available for the assessment of the vertical soil stress distribution within the reinforced soil mass other than at or close to the base of the wall, although a substantial reduction in vertical stress from the nominal value has been observed close to the wall face at higher elevations in the wall (Murray and Farrah, 1990; Simac et al., 1990). However, results from numerical analysis by Ho and Rowe (1992) shown in Figure 2 indicate that the vertical soil stress distributions within the reinforced soil mass are similar to that at the base and confirm the general trend observed in field monitoring and in laboratory models. The increasing effect of the lateral thrust towards the bottom of the wall can be seen in Figure 2a (note the change in scale).

An increase in vertical stress will lead to a higher tensile force in the reinforcement and will require a higher bearing capacity in the foundation. In practice, an increase of this magnitude would not cause a failure of the wall because of the safety margin typically provided in the design. However, it is important for the designer to know the margin of safety involved if the possible increase in vertical soil stress is neglected in the analysis.

Reinforced soil walls are sometimes designed to support a strip load (e.g. a bridge abutment or loading platform). In such situations it is essential to examine the influence of the strip load on the vertical soil stress distribution. Most often the vertical soil stress increment within the reinforced soil mass due to external strip load is assumed to be distributed

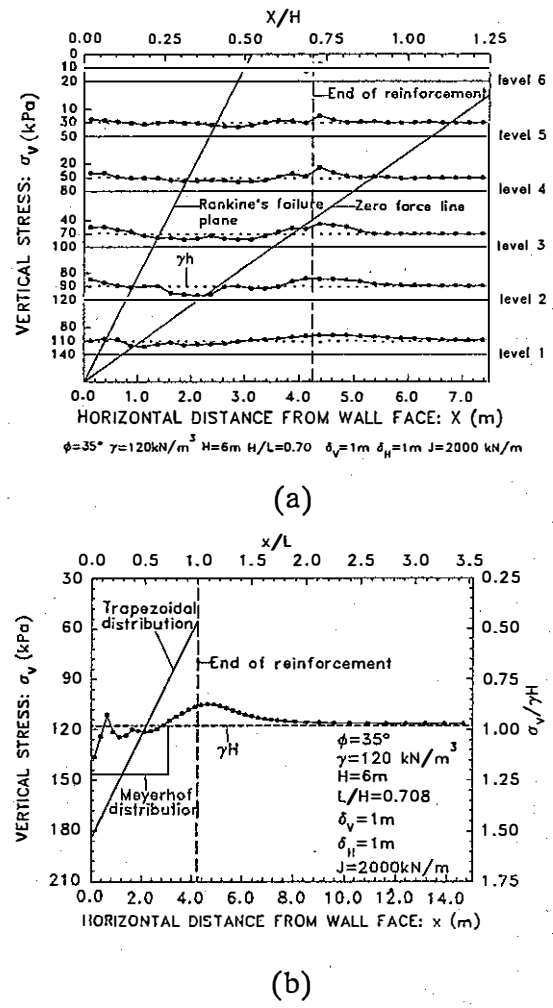

Figure 2. Horizontal soil stress from numerical analysis: (a) at wall face, and (b) at end of reinforced soil mass (Ho and Rowe, 1992).

according to the theoretical solution for a footing on an elastic half-space (i.e. Boussinesq Theory) or according to an approximate 2:1 stress dispersion method (e.g. Laba and Kennedy, 1986). Field observations on a steel strip reinforced soil wall (monitored by Bastick et al., 1989) where a strip load was applied adjacent to the rigid segment concrete facing showed some deviations from the elastic theory (assuming the facing acts as an axis of symmetry). At the face, the increment in vertical soil stress was almost zero. This is attributed to the fact that the facing is stiffer than the reinforced soil and hence carries most of the load. Near the centre of the strip load, the measured stress was up to 40 %

higher than that based on elastic theory. While further into the reinforced soil, the measured stress was lower.

Depending on the intensity and the location, the effect of the strip load on the vertical soil stress distribution may be significant. Firstly, the force in the reinforcement may be higher due to the higher vertical stress than anticipated based on elastic theory. Secondly, a higher bearing capacity in the foundation may be required in the vicinity of the toe and tension may develop at the heel of the reinforced soil mass.

In a reinforced soil wall constructed with extensible reinforcement and monitored by Balzer et al. (1990), it was observed that the stress from a strip load was dispersed at a very different rate than anticipated by a 2:1 spreading approximation or elastic theory (especially in the middle portion of the wall). Beneath the centre of the loaded area, the maximum vertical stress was similar to, or higher than, predicted by a 2:1 spread. At points outside the 2:1 zone there was significant stress increase. In conjunction with the vertical soil stress development the strain increment in the reinforcement layer at mid-height of the wall was observed to be the highest, indicating that the force in the reinforcement may be much higher than anticipated based on elastic theory or a 2:1 approximation. Hence, care should be taken in assessing the dispersion of vertical soil stress due to a strip load. Using conventional approaches to calculate the dispersion of strip load may be unsafe; the fact that failures generally do not occur may be regarded as reflecting the other levels of conservatism in the analysis/design of these walls. However, any "improved" design that reduces real factors of safety must also consider the potential lack of conservatism of this aspect of the problem.

4 HORIZONTAL SOIL STRESS DISTRIBUTION

Ideally one would assess the horizontal soil stress distribution by measuring the stress directly. However, due to practical difficulties, this is rarely done and the data that is available is generally restricted to the back of rigid type facings where installation of measuring devices is relatively easy. An alternate indirect approach more commonly used is to estimate the horizontal pressure based on the reinforcement force inferred from the observed strain at locations along the reinforcement. This approach implicitly assumes that only local horizontal force equilibrium and local interaction between the soil and the reinforcement need to be considered. The measurements are usually compared with the theoretical Rankine active K_a or the at rest K_o condition.

4.1 *Validity of the concept of equivalence in reinforcement force and horizontal soil stress*

The concept of the equivalence in reinforcement force and actual horizontal soil stress may be useful as a design assumption for calculating the maximum force required to be carried by the reinforcement and the anchorage length required to support such force. However, confusion may arise from using this approach to interpret the state of horizontal soil stress. Murray (1980a) has provided an example which illustrated the inconsistency that can arise in attempting to assume the equivalence of force in the reinforcement and horizontal soil stress. As shown in Figure 3, the equivalent soil force may not necessary correspond to the actual measured soil stress.

The force in the reinforcement depends solely on the strain and the stiffness of the reinforcement. Shearing of the soil adjacent to the reinforcement induces strain and hence force in the reinforcement. Care must be taken to distinguish between the equivalent horizontal soil stress and the actual horizontal soil stress.

This phenomenon has been demonstrated by results from large scale laboratory model walls monitored by Andrawes et al. (1990b). In these tests, the horizontal soil stress was measured at the back of the moveable steel wall facing at the end of wall construction (with negligible

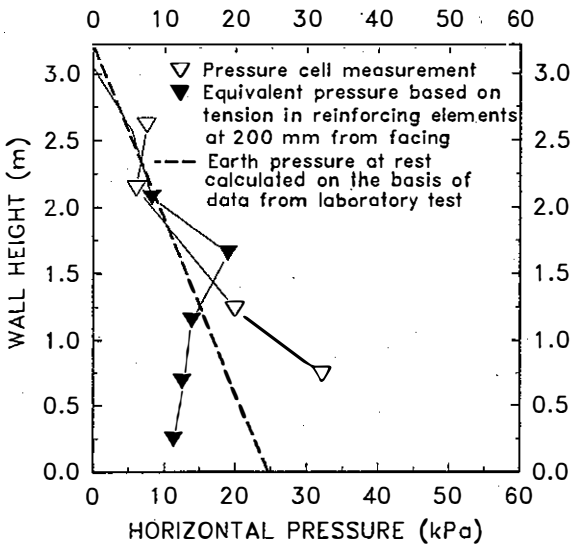

Figure 3. Pressures on the wall facing (modified from Murray, 1980a)

lateral movement) and after a rotation about the toe corresponding to 1.5 mm horizontal displacement at the top of the facing (i.e. 0.075 % of wall height). The wall face was supported by a jacking system to prevent movement during construction. The reinforcement was not attached to the wall face. After construction, very high horizontal soil stresses which exceeded the at rest K_o condition were locked in due to compaction, especially at the top of the wall. Rotation of the wall face about the toe resulted in substantial reduction in horizontal soil stress. In particular, the presence of the reinforcement reduced the horizontal soil stress further to values below the Rankine active K_a condition and approaching those corresponding to Coulomb's active wedge, suggesting that the compaction induced stress diminishes when sufficient wall yielding is allowed. If wall facing movement is constrained during construction (as in this case) relatively large horizontal soil stress could develop in association with small values of force in the reinforcement because soil/reinforcement interface friction and hence the strain in the reinforcement cannot be mobilized. It is noted that even after the constraint against facing movement has been reduced and the wall was permitted to rotate, a comparison of the equivalent soil stress inferred by the reinforcement force with the actual stress measurement showed a significant deviation.

In general, differences between inferred and measured horizontal stresses suggest that the assumption of local force equilibrium between the reinforcement and the horizontal soil stress does not adequately describe equilibrium conditions. Evidence which supports this contention has been reported by Murray and Farrah (1990) based on field observations and by Ho and Rowe (1992) based on numerical analysis. Both studies suggest that the same argument applies along the length of the reinforcement and not just adjacent to the wall facing. Thus, in general the force in the reinforcement is not equal to the horizontal soil stress.

4.2 *Comparison between "observed" and nominal horizontal soil stresses*

Attempts to compare "observed" horizontal soil stress to the "theoretical" Rankine active (K_a) condition or the at rest (K_o) condition implicitly assume that the vertical soil stress remains constant and equal to the overburden pressure, and that the principal stress directions remain vertical and horizontal. However, field observations indicate that the measured vertical stress differs from the nominal vertical (overburden) pressure. Furthermore, the assumption of no rotation in principal stress directions implies no shear stress at the soil/reinforcement interface. Rotation of principal stress directions in reinforced soil walls of up to 40° from vertical at the base and 10° to 20° at other locations have been measured by Murray and Farrar (1990). Jewel (1983) discussed the possibility of soil elements attaining an active state with an apparent lateral earth pressure coefficient greater than the K_a or even K_o condition due to rotation of principal stress directions. Assuming the vertical to be a plane of maximum stress obliquity, it can be shown that the apparent lateral pressure coefficient σ_h/σ_v is given by $(1-\sin^2\phi)/(1+\sin^2\phi)$. For typical values of sand friction angle the ratio σ_h/σ_v is approximately 1.2 times greater than K_o

calculated by Jaky's equation (i.e. $K_o = 1 - \sin\phi$) and 2 times greater than K_a. Therefore an apparent lateral pressure coefficient greater than K_o does not necessary mean that the soil is not in a plastic state. Thus, merely comparing the measured or inferred horizontal soil stress with the Rankine or at rest values will not generally provide a good indication of the stress state within the soil; plasticity in the soil can only be defined in terms of the principal stress ratio. Unfortunately, it is not feasible to implement the detailed instrumentation required to reveal the state of stress within the entire reinforced soil mass (theoretically each location would require 3 independent stress measurements oriented at different known directions). However, results from numerical analysis reported by Ho and Rowe (1992) have demonstrated the significance of this phenomenon and have shown that plasticity can exist even though the apparent lateral pressure coefficient is well above the K_a value. Correct assessment of the state of stress within reinforced soil walls requires consideration of the possibility of variation in vertical soil stress and rotation of principal stress directions.

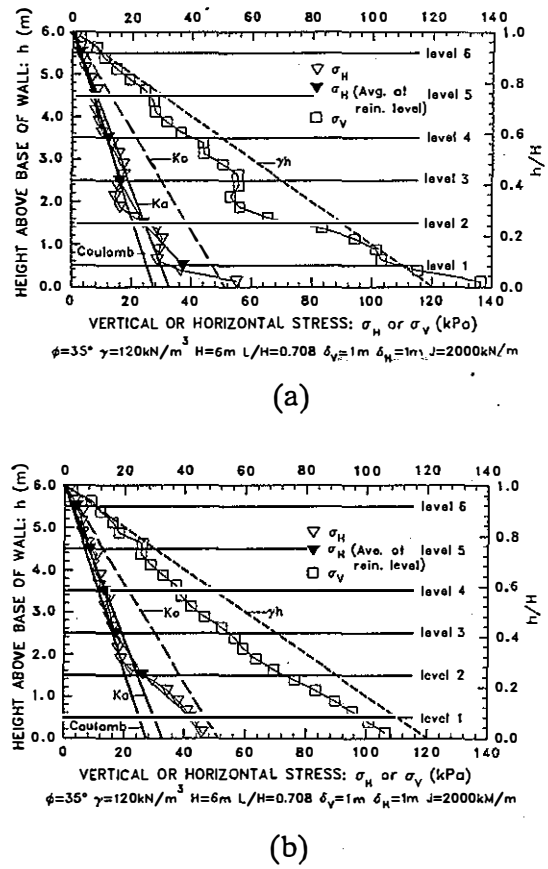

Figure 4. Horizontal stress distribution from numerical analysis. (a) at wall face, (b) at back of reinforced soil block (Ho and Rowe, 1992).

4.3 *Field observations and numerical analysis*

Results from laboratory model tests by Andrawes et al. (1990b) indicate that the horizontal soil stress at the wall face is slightly less than that predicted by Rankine active theory and approaches that based on the Coulomb failure wedge, except at the bottom of the wall where the influence of the foundation is significant. A similar conclusion can also be reached from the results of numerical analysis shown in Figure 4 at the wall face and at the back of the reinforced soil mass. In this case the deviation of horizontal soil stress from the Rankine active conditions is mainly due to the variation in the overburden stress.

Very few direct measurements of horizontal soil stress in geosynthetic reinforced soil walls have been reported in the literature. Figure 5 shows the distribution of measured horizontal soil stress $\sigma_{h(mea)}$ down the face of the wall for four monitored full scale structures. The measured horizontal soil stress is normalized with respect to the nominal Rankine active condition (i.e. $\sigma_{h(theo)} = K_a \gamma h$, $K_a = (1-\sin\phi)/(1+\sin\phi)$). This data suggests that the equivalent lateral pressure coefficient K is much less than the nominal active lateral pressure coefficient K_a. Similarly, in studies reported by Thamm and Lesniewska (1990) the horizontal soil stress induced from a strip surcharge loading was less than that predicted by elastic theory, with the discrepancy increasing with increasing surcharge intensity. Apart from errors in measurement (e.g malfunction of measuring instrument or loose contact between the measuring device and the soil due to lesser compaction in the vicinity of the wall face), the relatively small measured horizontal soil stress compared to the assumed Rankine active state may be due to the

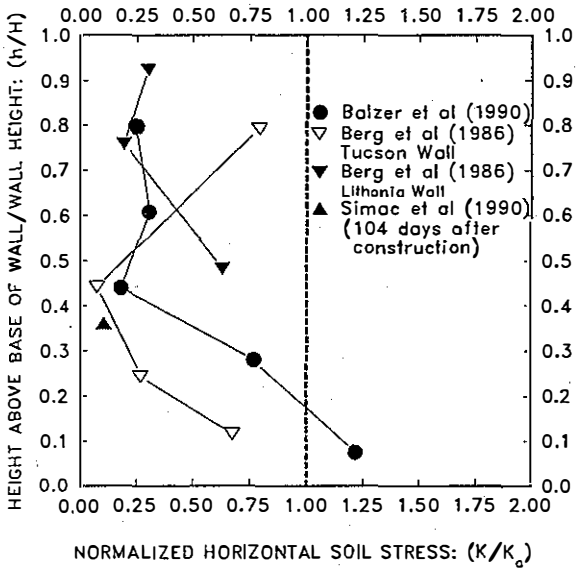

Figure 5. Field observations of horizontal soil stress at wall face.

following factors:

(a) The shear strength parameter ϕ of the backfill is usually based on the peak friction angle measured in laboratory triaxial compression tests. However, in reinforced soil walls plane strain conditions prevail and it is well known that the plane strain friction angle may be a few degrees higher than that determined from triaxial tests (Lambe and Whitman, 1969) . Use of the friction angle obtained from triaxial tests would overestimate the actual horizontal soil stress.

(b) A significant decrease in vertical soil stresses, relative to the nominal overburden pressure, at the wall face have been measured in the field. Thus the nominal Rankine active pressure based on the overburden stress would overestimate that which should be observed.

(c) The existence of an apparent cohesion due to the unsaturated nature of the backfill. In the field, water is often added to facilitate compaction. If sufficient fine soil particles are present there would be an apparent cohesion due to capillary action resulting in a decrease in the horizontal soil stress required for plastic equilibrium.

(d) The concept of "integration effect" as discussed by Jenner (1990) and Fukuda et al. (1986), whereby the inclusion of reinforcement within the soil may impart additional benefits other than just its tensile strength, especially for geogrid structures. By introducing reinforcement throughout the structure the composite material (i.e. the soil and the reinforcement) may exhibit properties that are better than the sum of the two individual material properties due to the lateral restraint imparted by the reinforcement to the soil particles. The interaction between the reinforcement and the soil particle has been shown to increase the load spreading ability of soils in road pavement and foundation situations (Guido et al., 1987; Milligan and Love, 1984), and the shear strength of soil in laboratory triaxial tests. Thus, the possibility of an apparent higher friction angle and hence lower horizontal soil stress for equilibrium warrants consideration.

The small scale laboratory studies carried out by Andrawes et al. (1990a) to examine the effect of a yielding boundary on horizontal soil stress development provides some additional insight regarding these observations. Their results indicated that the horizontal soil stress at the wall face depends on the number of layers of reinforcement and the degree of yielding of the wall face as indicated in Figure 6. Allowing the wall face and the soil to deform relative to the reinforcement results in stress transfer to the reinforcement and less horizontal stress being transferred to the facing. Increasing the number of layers of reinforcement increases the effect. This might partly explain the exceptionally low horizontal soil stress observed in real structures. Unfortunately there is no information on the

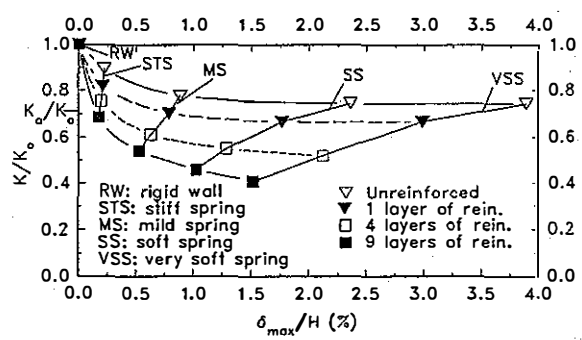

Figure 6. Variation of measured lateral earth pressure coefficient with wall movement and the number of reinforcement layers (after Andrawes et al., 1990a).

horizontal soil stress distribution farther back into the reinforced soil; additional field data is necessary to better understand this behaviour.

While the effect of the large reduction in horizontal soil stress at the wall face may not be important for soil walls used mainly as soil retaining structures, the effect is beneficial for situations where it is desirable to minimize the loading on adjacent structural elements (e.g. basement walls, bridge abutments, etc.) provided that provision is made to allow sufficient deformation before the soil/wall comes into contact with the structural elements.

5 FORCE IN REINFORCEMENT

5.1 *Basic considerations*

The magnitude of force in the reinforcement depends on the shear strength mobilized in the backfill, the horizontal soil strain at which the shear strength is mobilized, and the stiffness of the reinforced system. The shear strength determines the active force required for equilibrium, the system stiffness determines the strain and hence the force in the system for equilibrium.

There are two important points to be considered. Firstly, for the right choice of system stiffness it is possible to attain equilibrium state with the lowest force in the reinforcement. Secondly, beyond the limit state of the soil, further deformation due to reinforcement creep, for example, results in no change in the total system force for equilibrium. This is clearly demonstrated in a geogrid reinforced slope monitored by Fannin (1991) where direct measurement of reinforcement force and reinforcement strain at comparable locations indicated that reinforcement strain increased with time while reinforcement force showed only slight initial increase and remained virtually unchanged with time. It is expected that the total force in the system will remain unchanged in situations where the system stiffness required for equilibrium is beyond the limit state.

The concept of system compatibility (e.g. Jones, 1990) is attractive in principle, however this approach may not be entirely suitable for describing local compatibility between the soil and a single reinforcement layer. In practice the system stiffness also depends on the stiffness of the facing, the stiffness of the foundation and the stiffness of the soil when the soil is not in a plastic state. Interactions must exist between all the components of the system and there is the question of how the total required force is distributed to the various components and between layers of reinforcement. For example, in the case of a wall with a rigid full panel facing, force equilibrium is only satisfied when one considers the toe forces transferred to the bottom of the facing (Bathurst et al., 1989). Hence, the correct prediction of force in the reinforcement requires consideration of the interactions between all the components of the system.

5.2 *Field measurement of force in reinforcement*

Most often, only the strain in the reinforcement is measured and the force is deduced from the strain. For extensible reinforcement there are difficulties in interpreting the force in the reinforcement based on the measured reinforcement strain. Firstly, almost all extensible reinforcement is subject to some creep. The amount of creep depends on temperature, load level, the polymer, and the method of manufacture of the reinforcement. The strain in the reinforcement will increase with time under constant load. Without proper calibration, the force in the reinforcement will be overestimated. Secondly, the stiffness of some extensible reinforcement is dependent on the confining stress (the stiffness is higher if the confining stress is larger). Therefore the same reinforcement embedded at different depths might exhibit different stress-strain characteristics. Neglecting this effect may result in underestimate of the force in the reinforcement.

In general, the maximum tensile force deduced from measured reinforcement strain is less than that predicted by the Rankine

active condition especially close to the base of the wall, suggesting that the foundation has a significant effect on the development of force in the reinforcement, especially towards the bottom of the wall. This has been observed in numerous field cases (Thamm and Lesniewska, 1990; Simac et al., 1990; Bathurst et al., 1988; Wawrychuk, 1987). Similar trends of maximum tensile stress distribution have also been report by Ho and Rowe (1992) based on numerical analysis (Figure 7) which showed that the sum of the reinforcement forces together with the toe force is equal to the Rankine active force, suggesting that the foundation does have a significant effect on the force developed in the reinforcement.

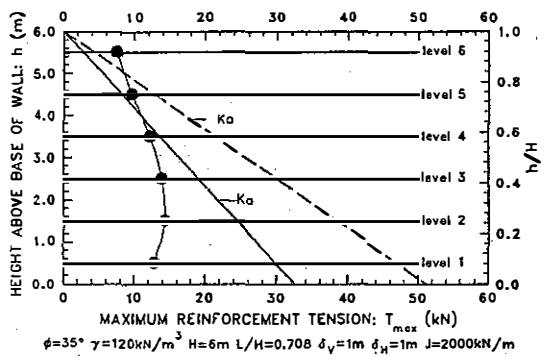

Figure 7. Variation of maximum reinforcement tension with height of wall from numerical analysis (Ho and Rowe, 1992).

In cases where the vertical spacing of reinforcement decreases proportionally with depth, field observations (Fannin, 1991) indicate that the maximum tensile force in each layer of reinforcement is more or less uniform except near the bottom where the force is substantially lower, which is again due to the influence of the foundation. This is consistent with the analytical findings by Jewel (1987).

Another interesting phenomenon is that the reinforcement force measured in extensible reinforcement near the top of the wall can be comparable to that measured in inextensible reinforcement at similar locations as shown in Figure 8 (Adib, 1988; field observations) and as observed by Jaber (1989, laboratory model results where no compaction was simulated). The only parameter varied in these tests was the reinforcement stiffness. This suggests that structural interaction between the wall components is also responsible for the distribution of force between layers of reinforcement. Hence, the high reinforcement force relative to the K_a or K_o condition close to the top of the wall cannot be solely attributed to the effect of compaction or the restraint of deformation due to the high reinforcement stiffness as assumed in "Coherent Gravity" method of analysis. Similar numerical observations have been made by the authors who also found that maximum force in the reinforcement becomes more uniform with decreasing reinforcement stiffness (with equal vertical spacing of reinforcement).

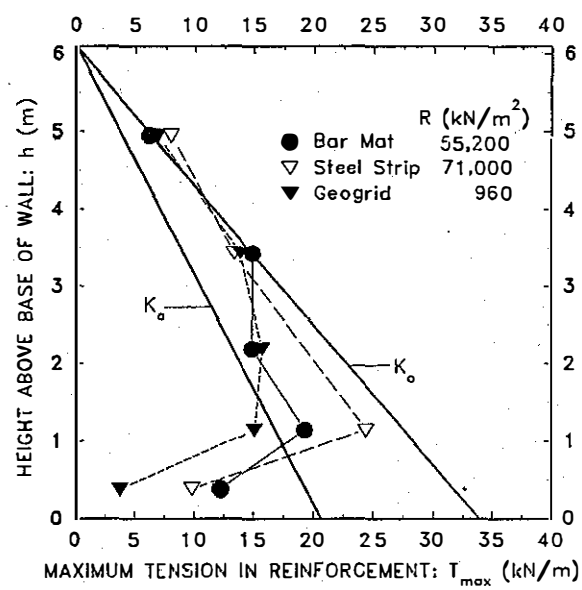

Figure 8. Effect of reinforcement stiffness on tension in reinforcement (data from Adib, 1988).

The foregoing observations imply that the magnitude of the force in each layer of reinforcement is dependent on the interaction between the wall components (i.e. the reinforcement, the facing and the foundation) and the spatial arrangement of the reinforcement.

The effect of compaction on the development of tensile force in walls using extensible

reinforcement has not been adequately addressed in the literature. The magnitude will depend on the compaction effort and is expected to be variable. Finley (1978) demonstrated that with increasing overburden height the compaction induced force in the reinforcement lower down the wall was gradually overcome by that required to support the overburden stress, suggesting that the force induced by compaction may be significant only for reinforcement near the top of the wall. In practice, the reinforcement distribution is typically uniform and the maximum force in the reinforcement would occur in the lower part of the wall; hence compaction may not be the controlling factor in determining the maximum force to be carried by the reinforcement, especially for high walls.

5.3 *Force distribution in a reinforcement layer*

The force distribution in a reinforcement layer is most influenced by the construction method, the lateral restraint provided at the facing during construction, and the facing-reinforcement connection details. Observations based on field and laboratory measurements indicate that there are three general types of distribution.

Type (a) corresponds to the situation where the reinforcement is not attached to the facing during backfilling as in the yielding wall concept of construction (Andrawes et al., 1990b), or in the situation where wrapped back type facing is used but the facing provides little or no lateral restraint against deformation. This situation is similar to an idealized direct shear test. The facing and the backfill are allowed to move relative to the reinforcement. Direct sliding of soil towards the front relative to the reinforcement induces strain and hence force in the reinforcement. However, force development in the reinforcement occurs over some length and the maximum force/strain in the reinforcement occurs at a location in the reinforced zone remote from the facing. Beyond that the force/strain reduces to zero towards the free end at the back. This situation was best illustrated in a laboratory model walls monitored by Fukuda et al. (1986). The reinforcement was not attached to the facing during backfilling, a rotation of the facing in the order of 1/300 radian resulted in the maximum strain occurring at the potential failure plane, but the strain at the front end of the reinforcement was zero. The maximum force in the reinforcement (deduced from strain) was calculated to be approximately equivalent to the reduction in horizontal soil stress measured at the wall face. Similar behaviour was observed by Andrawes et al. (1990b) in large scale model tests.

Type (b) corresponds to the situation similar to a idealized pull-out test with the reinforcement fixed to the facing. This situation may occur when little or no deformation is allowed during wall construction (i.e. by providing lateral restraint against facing movement) but without any connection between the facing and the reinforcement. Upon construction, attaching the reinforcement to the facing and removal of the lateral support will allow the wall to deform in a fashion similar to pulling the reinforcement away from the soil. In this situation the force/strain induced in the reinforcement is largest at the facing and decreases to zero towards the free end at the back. This behaviour is evident in the results from large scale model test results reported by Andrawes et al. (1990b). This type of construction is rarely used in practice since in the field it is not feasible to provide full lateral restraint during construction, especially with high walls; but the test results served to demonstrate the influence of construction method on the development of tensile force in the reinforcement.

Type (c) is most commonly seen in structures constructed with discrete panel facing with little or no lateral restraint against deformation during construction or in structures constructed with full panel facing where lateral restraint is limited. Translation or rotation of the facing tends to pull the reinforcement away from the soil, but at the same time the soil is able to slide relative to the reinforcement. Hence both types of stress transfer described in (a) and (b) occur

simultaneously and the resulting force distribution is a combination of the two. This was again demonstrated by Fukuda et al. (1986) in a laboratory model wall where the reinforcement was attached to the facing during construction. Rotation of the facing results in the reinforcement force/strain remaining more or less constant up to the potential failure plane and decreasing to zero towards the free end of the reinforcement. Force in the reinforcement (deduced from strain) was calculated to be approximately equivalent to the horizontal soil stress acting on the wall face.

There may be situations where the conditions are similar to those described in type (c) distribution but with an additional peak force/strain occurring at the facing with a magnitude at least comparable to that within the reinforced zone as has been noted by Bathurst et al. (1988). The trend towards high connection strains appears to be a common phenomenon with rigid panel structures and is attributed to the relative downward movement of the backfill with respect to the less compliant panel structure. It was suggested that good quality control of fill placement and compaction adjacent to the facing is required to avoid loose soil or a void below the connection that can subsequently lead to overstressing the reinforcement at the facing.

5.4 *Locus of maximum reinforcement force*

The locus of maximum reinforcement force is usually assumed to coincide with the failure plane. Reinforced Earth is a patented construction system where there is little variation in the material used and in the construction method. The locus of maximum tensile force is rather unique and approaches a logarithmic spiral which intersects the ground at approximately 0.3H from the facing. In contrast, reinforced soil walls have a large range of variation in material properties and construction methods. The locus of maximum tensile force is not unique under working stress conditions which is demonstrated by observations on real structures reported in the literature. As discussed in section 5.3, the construction method has a significant effect on the development of reinforcement force. Hence, the locus of the maximum tensile force is expected to be dependent on the construction method.

Jewell (1985) discussed the effect of bond between the fill and the reinforcement on the locus of maximum tensile force. It was stated that the locus of maximum tensile force will always be inclined to $45+\phi/2$ to the horizontal if there is sufficient bond between the fill and the reinforcement. Only when there is insufficient bond, does the locus of maximum tensile force move towards the facing. The bond between the reinforcement and the fill depends on the fill/reinforcement interface friction angle and the contact area (i.e. width x length). For soil walls using extensible reinforcement, the interface shear strength is usually compatible with the fill and the reinforcement is in the form of a sheet or grid spanning along the full width of the wall. Hence the bond is mainly controlled by the length of the reinforcement. In this case the locus of maximum tensile force will shift towards the facing if the length of the reinforcement is too short. There is no direct field data to show the effect of reinforcement length on the locus of maximum tensile force for a wide range of reinforcement lengths. However numerical analysis by the authors indicated that there is a tendency for the locus of maximum tensile force to shift towards the facing in the upper part of the wall when the reinforcement length to height ratio is reduced from 0.7 to 0.4.

Based on finite element analysis results, Adib (1988) demonstrated that the stiffness of reinforcement also affects the locus of maximum tensile force. With very high reinforcement stiffness (such as steel strips) the locus of maximum tensile force was also observed to shift towards the facing even under the condition of no slip between the soil and the reinforcement (i.e. $\phi_{fill/rein} >> \phi$).

An interesting phenomenon observed in walls constructed with wrapped back facing is illustrated in Figure 9. The locus of maximum tensile force appears to be parallel to the Rankine failure plane with a shifted origin.

Part of the fill column residing between the wrapped back portion of the reinforcement layer acted as if to form a physical wall facing of some finite thickness, shifting the origin of the back of the "physical wall facing" farther into the reinforced zone. Whether this is an actual behaviour or just a coincidence is not yet certain. However, there are a number of cases where a similar phenomenon appears evident (e.g. Thamm et al. (1990), Wichter et al. (1986); Fukuda et al. (1986); and Jaber, 1989).

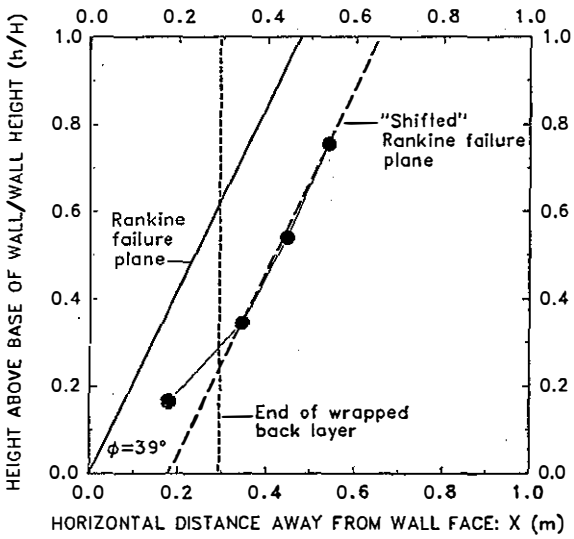

Figure 9. Locus of maximum tensile force in a reinforced soil wall with wrapped back facing (data from Billiard and Wu, 1991).

The location of a strip load and the type of facing also affect the locus of the potential failure plane. For example, in a geotextile reinforced soil wall with wrapped back facing monitored by Krieger and Thamm (1991), the location of the failure plane at the top of the wall was observed to shift towards the back of the load strip. Failure appeared to have been initiated by slippage between two layers of reinforcement higher up the wall. As a result the origin of the failure surface was shifted upward away from the toe.

The foregoing discussions suggest that the locus of maximum tensile force (or failure plane) in soil walls using extensible reinforcement is dependent on various factors and will not in general be unique. However, except for occasions where the wall is subject to strip loading, there is evidence that when the system is approaching failure the locus of maximum tensile force tends towards the Rankine failure plane (as observed by Bathurst et al., 1988; Simac et al., 1990; Juran and Christopher, 1989; and Jaber, 1989).

6 HORIZONTAL DISPLACEMENT

6.1 *Assessment of horizontal deformation*

The magnitude of horizontal movement depends on factors such as: compaction effort, reinforcement extensibility (taking account of the stress-strain-time-temperature dependencies of the reinforcement), reinforcement density (i.e. horizontal spacing and vertical spacing), reinforcement length, facing type, reinforcement-facing connection details, strength and deformation characteristics of both the backfill and the foundation soil, surcharge intensity, location of strip surcharge loading, and construction practice.

Unfortunately, there is no simple method for predicting the lateral deformation which would occur in reinforced soil walls. Although several methods have been proposed to deal with lateral wall deformation (e.g. Jewel, 1987; Murray, 1980b; Jaber, 1988), none appear to have adequately taken account of the interaction between the various components of the reinforced soil wall system and more research is needed in this area.

An early approach aimed at avoiding excessive horizontal deformation in the design of reinforced soil walls was to impose a limit on the strain (usually 5 %) to be developed in the reinforcement as a basis for the choice of the reinforcement type to be used. However, this approach may force the designer either to the use relatively high stiffness reinforcement or excessive number of layers of reinforcement. In fact, this procedure appears to be unwarranted based on field observations in which measured reinforcement strains typically have been less than 1 % (e.g. Bell and Steward, 1977; Berg et al., 1986; and

Simac et al., 1990).

An alternate approach is to estimate the probable horizontal displacement based on past performance data from similar structures (e.g. Christopher et al., 1989). This empirical method (based on results from finite element analysis, small scale tests, and limited field evidence on walls up to 6 m) considers the effect of reinforcement extensibility, reinforcement length to wall height ratio, and surcharge intensity on the magnitude of horizontal movement. This method does not explicitly provide a quantitative distinction between inextensible or extensible system, but suggests that the maximum face deformation in an extensible system may be 3 times as large as that in an inextensible system for two otherwise identical walls. However, irrespective of the reinforcement stiffness, a reinforcement length to wall height ratio (L/H) less than 0.3 would result in unacceptable lateral face movement. The movement is relatively insensitive to reinforcement length for a ratio greater than 1.2. Similar observations based on numerical investigations have been made by the authors. Simple consideration of the geometry of the problem may hint that the lower L/H ratio is controlled by external stability against overturning, while the upper limit is controlled by the location of the theoretical stable slope (i.e. the zone beyond which no reinforcement is required for equilibrium). Assuming the reinforced soil mass acts as a rigid body with no surcharge and the backfill is in a state of active equilibrium, it may be shown that the minimum L/H ratio required for a factor of safety of unity against overturning is given by $(K_a/3)^{1/2}$. Typical value of ϕ for fill used is in the range of 35° to 40°, this represents a L/H ratio in the range of 0.3 to 0.27. On the other hand, the upper limit of the L/H ratio is controlled by the inverse of the angle of stable slope (i.e. cot ϕ). This would give a value of 1.43 to 1.19 for ϕ of 35° to 40°.

6.2 *Sources of horizontal deformation*

In order to understand the relative importance of the factors affecting horizontal deformation in reinforced soil walls it is essential to be able to identify the sources of the deformation. Unfortunately, field monitoring of horizontal deformations is usually carried out at the wall face where the measurement only indicates total displacement without providing any information concerning the different sources of deformation.

The horizontal deformation in reinforced soil walls may be expected to arise from:

1. deformation within the reinforced zone
2. deformation of unreinforced zone
3. movement due to construction
4. foundation movement

6.2.1 *Horizontal deformation in reinforced zone*

For reinforced soil walls constructed on a competent foundation, very limited horizontal deformation is expected within the self-stable zone below plane OD originating from the toe inclined at an angle ϕ (internal friction angle of fill) to the horizontal (see Figure 10).

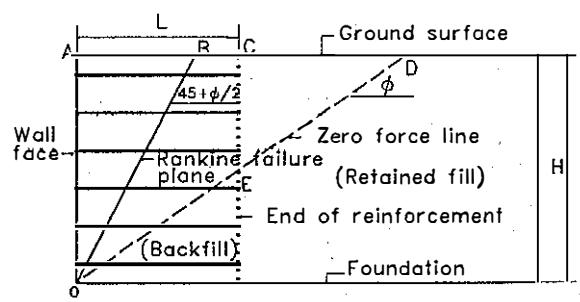

Figure 10. Sources of horizontal deformation in reinforced soil walls.

Figure 11 shows the horizontal soil movement within the reinforced soil mass revealed by an inclinometer installed at some distance away from the facing as reported by Simac et al. (1990). The horizontal soil movement relative to the base is insignificant below the self-stable slope, suggesting that most of the deformation in the reinforced zone comes from straining of the soil above the self-stable slope (area OABD in Figure 10). This phenomenon has been confirmed in a numerical analysis by Ho and Rowe (1992) as will be discussed in more details in Sect. 6.2.2.

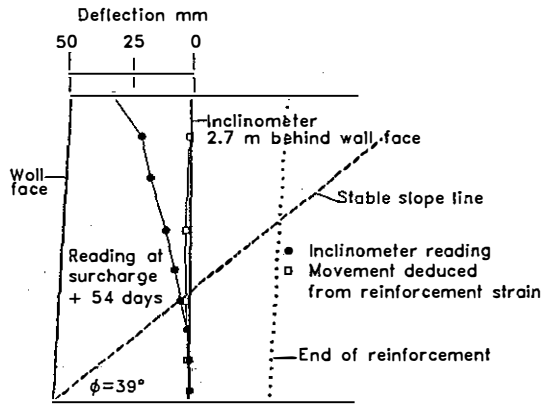

Figure 11. Horizontal deformation within a reinforced soil wall (data from Simac et al., 1990).

If strain compatibility between the soil and the reinforcement is maintained (which is likely the case for geosynthetic reinforcement) and the reinforcement is attached to the facing, integration of the strains along the reinforcement layers might be expected to be a good measure of the horizontal movement of the soil within the reinforced zone. Most of the horizontal movements would be expected to come from the active zone (area OAB in Figure 10). Less horizontal deformation is expected from the transition zone (area OBCE in Figure 10). The primary factors governing the soil movement within the reinforced zone would then be the stiffness and density of the reinforcement, and the shear strength of the fill. Higher reinforcement stiffness and higher reinforcement density reduce the strain in the soil, and larger shear strength of fill (i.e. larger friction angle ϕ) results in less force in the reinforcement being required to maintain equilibrium and hence less deformation (all other factors being equal).

6.2.2 *Horizontal deformation in unreinforced zone*

Practical design considerations normally result in a truncated reinforcement layout (i.e. uniform reinforcement length along the height of the wall) as opposed to ideal reinforcement layout (i.e. increasing reinforcement length from bottom to top, with the reinforcement length extending beyond the stable slope at each reinforcement level). In most instances, there is usually a portion of unreinforced retained fill at the top of the wall behind the reinforcement (area CDE in Figure 10) where the reinforcement length to wall height ratio L/H is less than cot ϕ. The magnitude of the horizontal movement in the unreinforced zone depends primary on the magnitude of the soil strain and the area of soil to be strained since deformation is an integration of strain over the area. The former is governed by the amount of horizontal movement in the reinforced zone which determines the strain required in the unreinforced zone for equilibrium. The latter is governed by the horizontal position of the free end of the reinforcement relative to the self-stable slope which in turn is dependent on the reinforcement length and the friction angle of the backfill. For a fixed reinforcement length, a lower backfill friction angle will result in a larger area of unreinforced retained fill above the self-stable slope. The importance of the unreinforced zone in the development of overall horizontal movement in reinforced soil walls can be best illustrated by considering results from three monitored cases as described below.

In case 1 (Bathurst et al., 1988), the reinforcement length to wall height ratio (L/H) was 1 which is much greater than the inverse of the slope of the fill friction angle (cot ϕ) of 0.75 for ϕ = 53°. There was no unreinforced zone above the stable slope. Extensometers attached to the end of the reinforcement showed negligible movements suggesting that no movement arose from the back of the reinforced zone.

In Case 2 (Allen et al., 1992), the L/H ratio was 0.766 which is slightly smaller than cot ϕ of 0.933 to 1.072 (ϕ = 47° to 43°) resulting in a relatively small unreinforced zone above the stable slope. An inclinometer installed at the end of the reinforcement indicated a maximum horizontal movement of the order of 4 mm, indicating the movement at the back of the reinforced soil mass was relatively small.

In case 3 (Simac et al., 1990), the L/H ratio was 0.75 and cot ϕ was 1.235 for ϕ = 39°, resulting in a large region of unreinforced soil mass above the stable slope. Figure 11 shows the movements observed from an inclinometer installed at 2.7 m behind the wall face and the elongation (deduced by integration of the reported strains) of the reinforcement between the inclinometer position to the end of the reinforcement. The inclinometer reading shows the absolute soil movement behind the inclinometer position while the elongation in the reinforcement indicates the horizontal soil movement between the inclinometer and the end of the reinforcement (assuming no slip occurs between the reinforcement and the surrounding soil). The difference between the two would then represent the soil movements at the back of the reinforced soil mass. The large difference observed clearly suggests that the soil movement behind the reinforced soil mass is significant due to straining of the unreinforced soil zone above the stable slope. Although part of this difference might be attributed to the slippage between the soil and the reinforcement, it would require a slippage in the order of 30 mm (e.g. at the top) to explain the discrepancy and this is very unlikely. In fact, no slippage was reported for this wall.

A similar phenomenon has been reported by Ho and Rowe (1992) based on results from numerical analysis which indicated that the horizontal soil movement in the unreinforced zone is responsible for a significant portion of overall horizontal movement at the wall face when L/H is less than cot ϕ. Hence, it appears that the soil movements in the unreinforced zone above the stable slope requires consideration when assessing the overall movements at the wall face.

The effect of the reinforcement stiffness on the deformation of reinforced soil walls is apparent from observations of full scale structures (Adib, 1988) and laboratory models (Jaber, 1989) where identical walls constructed with reinforcement of different stiffnesses showed that the horizontal movement at the wall face decreases with increasing reinforcement stiffness. As yet there has been no field studies of the effect of fill shear strength on horizontal deformation. However, the results from numerical analysis by the authors indicate that increasing the fill shear strength decreases the horizontal deformation of reinforced soil walls. The decrease comes from both a reduction in the internal deformation of the reinforced soil mass and a reduction in the deformation at the back of the reinforced soil mass as shown in Figure 12.

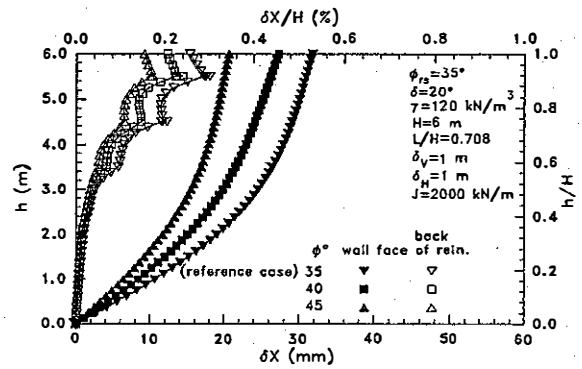

Figure 12. Influence of soil shear strength on horizontal deformation in reinforced soil walls (numerical analysis).

6.2.3 *Horizontal deformation due to construction*

For reinforced soil walls constructed with full height panel facings, the facing panels are erected and supported by props during construction, backfilling and compaction. In this case, horizontal deformation is negligible during construction and occurs after wall construction and removal of props. However, for reinforced soil walls using segmented type facings (e.g. discrete facing panel or wrap back facing) the facings and the backfill are constructed together incrementally. Large local horizontal deformation will occur at the time of construction when the soil lift is built. This creates a moving datum for the next lift to be constructed because the next facing is aligned over the already deformed lower facing. For each additional fill lift, it is then the outward movement developed in the layer immediately below, over which the facing is aligned, that causes the move in the datum.

In principle, a cumulative adjustment should

be made at each incremental level to allow for the already moved starting point of each undeformed lift. This acts to increase the outward movements higher in the wall face. The magnitude of this kind of movement is difficult to estimate. It depends on the compaction effort (which is likely to be quite variable from layer to layer), the connection between the facing panels (which determines whether the facing rotates or translates), and whether local temporary support is provided during construction of the layer under consideration. Wawrychuk (1987) has shown that both translational and especially rotational movement of incremental facings occur both during wall construction and surcharge loading. Methods that have been purposed to deal with this problem (e.g. Jewel, 1987; and Adib, 1988) assume that the horizontal deformation due to incremental construction of a particular lift is considered as a single independent layer subject to some equivalent lateral pressure due to soil load and compaction load with no interaction from the lift below or above. The calculated movements are in addition to those due to internal deformation of the reinforced soil wall itself. Hence, in their assumptions there is some overlapping between the horizontal movement due to internal deformation and due to construction. Both assumed the facings are rigid and can either rotate or translate. Possible slip between the soil and reinforcement was also considered by Adib (1988).

In the absence of an appropriate analytical method to deal with this type of construction movement, it would be appropriate to provide batter for incremental facings during erection to compensate for the possible change in datum during construction.

6.2.4 *Horizontal deformation from foundation*

Movement caused by local overstressing of the foundation beneath the toe, overall settlement/yielding of the foundation, or due to sliding between the reinforced soil mass and the foundation will cause additional movements at the wall face. These movements are related to external stability of the entire reinforced soil system and are dependant on the strength and stiffness of the foundation. Most of the reinforced soil walls constructed to date are built on foundation which could be considered to be competent although face movement due to foundation movement has been noted (e.g. Jones and Edwards, 1980).

Current design practices specify a relatively large margin of safety against foundation failure and do not explicitly consider external movements of the reinforced soil walls in addition to those resulting from internal deformation within the wall (i.e. the foundation is generally considered to be stiff and non-yielding). In fact, there is no simple method of investigating the influence of the foundation on the deformation characteristics of reinforced soil wall since this would require consideration of the interaction between the foundation and the reinforced soil wall itself. For the time being, it would appear to be appropriate to provide denser reinforcement or higher stiffness reinforcement close to the base for situations where the competence of the foundation is less than desired. A finite element analysis would be required where there is reason for concern regarding the effect of foundation deformations.

6.3 Post construction horizontal deformation

One important consideration when using extensible reinforcement is their creep behaviour (i.e. increase in strain with time under constant loading). The post construction movements in reinforced soil walls constructed with extensible reinforcement under no change in loading conditions are attributed to creep in the reinforcement. Creep strains accumulate once the reinforcement is subject to load. Thus, since it takes time to construct a wall, the movement measured in the wall face up to the end of construction should have included some movements due to creep in the reinforcement (especially for reinforcement near the bottom of the wall) unless the wall face is fully restricted against horizontal movement and no force is induced in the reinforcement until wall construction is

completed. It is generally impossible to isolate the movement due to creep from the total displacement. Post construction monitoring of horizontal deformations in reinforced soil walls with extensible reinforcement has shown evidence of continuous movements with time but with a decreasing rate (e.g. Bathurst et al., 1988; Fannin, 1991; Allen et al., 1992). The magnitude of creep deformation varied in individual cases and was dependent on the creep characteristics of the reinforcement. The deformations were generally much less than those expected based on in-isolation creep tests, signifying that soil confinement has the effect of reducing the magnitude of creep in extensible reinforcement. These observations are consistent with the results of laboratory tests on extensible reinforcement confined in soil where the total strains are much less than those when tested in-isolation under the same loading (e.g. McGown et al., 1982). Isochronous curves have been developed and used in the prediction of long-term horizontal movements (e.g. Bathurst et al., 1992). However, when making use of such laboratory data in predicting actual field behaviour of the reinforcement it is important to recognize that the pattern of loading imposed in the field and in the laboratory are not directly comparable. The field structure is incrementally loaded over a period of time during construction and any surcharge is also usually loaded incrementally. In contrast, laboratory specimens in isochronous load-strain tests are loaded instantaneously. In addition, the boundary conditions and the interaction between the components in real structures cannot be fully simulated in laboratory tests. For these reasons it is not possible to make a rigorous comparison of laboratory data with field creep measurement.

6.4 *Other factors affecting horizontal deformation*

One factor which significantly affects the magnitude of horizontal movements at the wall face is the rigidity provided by the facing. Facings in the form of concrete full height panels exhibit the largest axial and flexural rigidity, while wrapped back type facings will provide the least. Facings with high axial rigidity settle less than the fill and help to support part of the overburden load through fill/facing shear transfer and thereby reduce the force and hence the strain in the reinforcement. Facings with high flexural rigidity provide lateral restraint to deformation and reduce the horizontal movement at the wall face. For example, Wawrychuk (1987) reported the responses of incremental horizontal movement at the wall face due to various uniform surcharge loadings for two identical walls with different facings, one with full timber panels and one with discrete timber panels. Under the same loading conditions, the wall with discrete facings deformed much more (by a factor of 2) than one with full panel facing due to the lack of flexural rigidity. However, it is interesting to note that although the total horizontal movement in the wall face for the wall with discrete panel facing was larger, the creep deformation was less at each loading stage compared to that in the wall with full panel facing. This phenomenon, as yet unexplained, reveals the complexity of the response of reinforced structure to creep.

The location of strip surcharge can have a large effect on the response of horizontal movement (based on results of a numerical analysis reported by Chew et al., 1990). Higher horizontal movement resulted from closer proximity of the strip load to the wall face, but even much higher horizontal movement may occur when the strip load is applied directly on top of the unreinforced retained zone. The absence of reinforcement in such region could not suppress the development of plastic soil strains resulting in a large horizontal movement at the wall face. Similar findings have been observed by the authors.

Construction details also have a large effect on horizontal movement at the wall face. For example, Andrawes et al. (1990b) examined two identical large scale laboratory model walls with propped full rigid panel facing. In one case, the fill was directly in contact with the facing. In the second case a compressible layer was placed between the fill and the back

of the facing. During backfilling significant horizontal soil stresses built up behind the wall and the reinforcement strains were relatively small for the wall without the compressible layer. For the wall with the compressible layer, the horizontal soil stresses on the facing were considerably reduced but the strains in the reinforcement were correspondingly increased. Attaching the reinforcement to the facing and removing the props resulted in large horizontal soil stress release and a large increase in reinforcement strain in the wall without compressible layer. However, the stresses in the wall with the compressible layer remained virtually unchanged with only small increases in reinforcement strain and a relatively small deformation was observed. Thus, accurate prediction of horizontal movement requires consideration of the actual construction conditions.

The foregoing discussion indicates that there are a number of factors which need to be considered when assessing the probable horizontal movement that could occur in reinforced soil walls. Current analytical methods which only consider the elongation in the reinforcement (induced by force and reinforcement stiffness) as a measure of the horizontal movement at the wall face may not be adequate in addressing the problem.

7 COMPARISON OF CURRENT DESIGN METHODS

Numerous approaches have been developed for the analysis and design of reinforced soil walls using extensible reinforcement. All methods that are applicable for routine design use limit equilibrium calculations to determine the factors of safety against wall failure. These methods generally fall into two main categories. Methods in the first category use simple force equilibrium analysis whereby the destabilizing horizontal force from the soil is balanced by the stabilizing horizontal force provided by the reinforcement. The second category involves methods which evaluate the force and/or moment equilibrium on an assumed failure surface similar to conventional slope stability analysis but with the inclusion of the balancing force/moment provided by the reinforcement. These equilibrium methods do not consider either the stress-deformation characteristics of the structure or the interactions which occur between the wall components (i.e. the soil, the reinforcement, the facing, and the foundation).

A study was conducted to compare the results obtained from some design methods published in the literature with the observed performance for four case histories. All together twelve methods were selected for this purpose. These methods have been widely used for the design of numerous reinforced soil walls. They are:

(1) Jewel method (1987);
(2) Coherent Gravity method (Schlosser, 1990);
(3) Bonaparte et al. method (1987);
(4) Forest Service method (Steward, Williamson and Mohney, 1977 - revised 1983);
(5) Broms method (1978);
(6) Collin method (1986);
(7) TC method (1986);
(8) Murray method (1980b);
(9) Simac et al. method (1990);
(10) Leschinsky and Perry method (1989);
(11) Schmertmann et al. method (1987); and,
(12) Ruegger method (1986).

A number of these analyses represent special cases of the tie-back design introduced into the British Codes in the late 1970's (e.g. BE3/78).

7.1 *Description of methods*

In this paper only a brief description of the methods used is presented. A detailed description of each method can be found in the references cited.

Methods 1 through 9 may be considered as stress methods which are based on lateral earth pressure considerations. Limit equilibrium analysis is used to equate the force tending to cause instability to the stabilizing force provided by the horizontal reinforcement. The stresses considered in these methods are: (a) the vertical soil stress, (b) the horizontal soil stress, (c) the stress in

the reinforcement, and (d) the horizontal resistance to pull-out of the reinforcement behind the potential failure plane. Two independent factors of safety are calculated for each layer of reinforcement. The factor of safety for reinforcement rupture is the ratio of reinforcement strength to the force required to maintain equilibrium with the lateral earth pressure. The factor of safety for reinforcement pull-out is the ratio of the pull-out resistance to the lateral earth pressure required to be supported by the layer of reinforcement. Pull-out resistance is provided by the soil/reinforcement interface friction under the overburden stress on the portion of the reinforcement layer behind the potential failure plane. The remaining three methods (i.e. methods 10 to 12) employ the common approaches used in conventional slope stability analysis, which involves the analysis of stresses on a failure surface. They are referred to as "Slope Stability" methods subsequently. There are three noticeable differences among these methods as follow: (a) the shape of the failure surface, (b) the distribution of force in the reinforcement, and (c) the means by which a surcharge is considered. The Murray method and the TC method also require an independent check on overall and multiple wedge stability respectively. These independent calculations are included in the "Slope Stability" method.

There are also significant difference in the definition of safety factors among both the "Tie-Back Wedge" methods (Method 1, 3 to 9) and the "Slope Stability " methods (Method 10 to 12). Some methods explicitly prescribed allowable strengths and safety factors while others provide general guidelines only. However, in general, the design methods use allowable strengths which are significantly lower than ultimate strengths and further safety factors are applied to account for the uncertainties in the behaviour of the reinforcement and soil/reinforcement interaction mechanisms.

7.2 *Description of case histories*

The twelve design methods were used to predict the performance of four reinforced soil walls tested under controlled conditions. The purpose was to compare the design methods without the influence of the criteria used for defining the factor of safety and selection of allowable reinforcement strength. Reported ultimate tensile strengths were used with all factors of safety (or partial factors) set equal to 1.0. It should be noted that the solution provided by the Ruegger method has a built-in safety factor of 1.3, therefore the results from this method should be viewed accordingly and are not directly comparable to the other results.

The first three walls are large-scale model geogrid-reinforced soil walls constructed as part of a long-term research project at the Royal Military College of Canada (RMC). Details of these test walls have been presented by Bathurst, Benjamin and Jarret (1988). These cases are referred to as Wall 1, Wall 2, and Wall 3.

The major component of the RMC model test facility consisted of a reinforced concrete form in which a reinforced soil wall can be built up to 3.6 m high, 2.4 m wide, and 6.0 m deep. A vertical surcharge could be applied at the top of the wall by pressurizing air bags up to an equivalent fill height of 6 m. All three walls were constructed to 3 m (not including a 250 mm binding layer) with four layers of 3 m long reinforcement at depths of 0.5, 1.25, 2.0, and 2.75 m respectively. The walls were constructed by slightly pretensioning the reinforcement and then carefully compacting the backfill with a reported friction angle of 53°. The major difference between the three walls are as follows. Wall 1 and Wall 2 were the same except for the timber facing used. The facing in Wall 1 was a full timber panel while the facing in Wall 2 was a segmented timber panel. Lateral support was provided in Wall 1 during construction while only temporary local support was provided in Wall 2. The reinforcement used in Wall 1 and Wall 2 was a high strength polyethylene geogrid with an ultimate wide width strength of 170 kN/m and a stiffness of 750 kN/m at 2 % strain. Wall 3 was similar to Wall 2 except for the reinforcement used. The reinforcement in Wall 3 was a relatively weak polyethylene

geogrid with a wide width tensile strength of 14 kN/m and a stiffness of 170 kN/m at 2 % strain.

Wall 1 and Wall 2 were surcharged in increments to 12, 30, and 50 kPa. The 50 kPa surcharge was applied up to 1000 hrs. No failure occurred in Wall 1 and Wall 2. Wall 3 was surcharged by similar loading sequence. The 50 kPa surcharge was sustained for 162 hrs, during which soil failure occurred. Additional surcharge up to 100 kPa was applied, at which time collapse of the wall occurred. The failure mode reported was due to creep rather than reinforcement rupture or pull-out.

Wall 4 in this study involved one of the two full-scale, geogrid reinforced soil walls tested to failure reported by Minami et al. (1987). This wall is referred to as "Case 1" in the cited reference. The purpose of this test wall was to observe the failure mode and to determine the minimum reinforcement length required for equilibrium. The wall was constructed up to 4 m high with four layers of reinforcement of varying length as shown in Figure 13. It was constructed with granular materials with a reported friction angle of 31°.

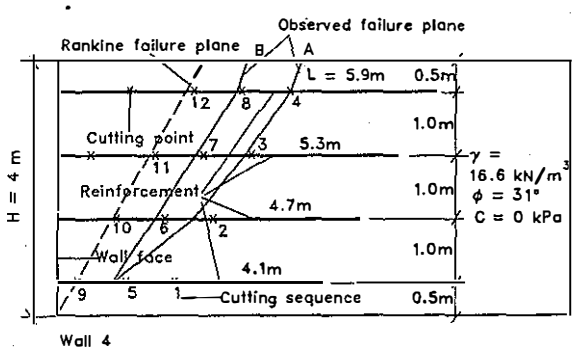

Figure 13. Geometry of Wall 4 and locations of observed failure plane (after Minami et al., 1987).

The reinforcement used was a relatively extensible polypropylene geogrid with a reported ultimate tensile strength of 18 kN/m. However, it should be noted that the reported tensile strength may be high because of the relatively fast testing rate (50mm/min). The wall was constructed with no compaction and gaps (10 mm) were provided between the horizontal junction of each timber facing and the junction between the facing and the foundation, thus limiting the transfer of vertical stress to the foundation. No surcharge was applied to this wall. To induce wall failure, successive cutting of reinforcement at prescribed location on the reinforcement layer was made by an electrical process (see Figure 13). Failure was initiated at the time of cutting point (5) with a crack forming at the surface at point (A). The crack at point (A) propagated thereafter through the preceding cutting points followed by another crack forming at point (B) after a cutting was made at point (6). Collapse of the wall occurred when a final cut was made at point (8). Examination of the wall after the test showed evidence of two failure planes within the reinforced soil passing almost through the cutting points as shown in Figure 13. It was stated that the failure was caused by : (a) pull-out of the upmost layer of reinforcement and (b) reinforcement breakage (by cutting) of the lowest reinforcement layer. Simple external stability analyses indicate that the minimum length of reinforcement (i.e. Factor of Safety = 1, $\phi_{fill/foundation} = \phi$) required for stability against sliding and stability against overturning is 1.07 m and 1.67 m respectively, indicating that failure due to sliding along the foundation is also a possibility since the length of the lowest layer of reinforcement at failure was approximately equal to 1.05 m. However, due to the nonuniform nature of the reinforcement length of each layer, the "equivalent" reinforcement length is difficult to quantify.

7.3 *Conditions of comparison*

Since actual conditions in the case histories do not satisfy the requirements of some of the design methods, some adjustments are necessary as stated below:

(1) The Schmertmann et al. method was developed for steep slopes (up to 80°) and friction angles up to 35°. Therefore extrapolation is required for values beyond these limits. In addition, the method specifies a distribution of reinforcement decreasing

proportionally with depth with equal force in the reinforcement.

(2) Extrapolation procedures similar to those in the Schmertmann et al. method are also used in the Ruegger method since the solutions are only provided for friction angles up to 35°.

(3) In the Murray, Collin, and Forest Service methods the lowest layer of reinforcement is assumed to be at the foundation level when calculating the reinforcement force.

(4) In the Broms method equal vertical spacing of reinforcement is assumed.

(5) When calculating the minimum reinforcement length required in Wall 4, the fill/reinforcement interface coefficient μ is assumed to be equal to tan (0.67ϕ) except in the Jewel method and the Schmertmann et al. method where μ is equal to 1.0 and 0.9 respectively. This procedure does not have any significant effect on the calculated minimum length of reinforcement required in most methods except in the Murray method.

7.4 *Results of comparison*

For all walls, comparison was made between calculated maximum tensile forces with those actually measured in the tests. Comparison in Wall 1, Wall 2, and Wall 3 is made at a surcharge intensity of 50 kPa. The reinforcement forces in Wall 1 and Wall 2 were cited by Wawrychuk (1987). The reinforcement forces in Wall 3 were cited by Leschinsky and Perry (1989).

The reinforcement forces in Wall 4 were deduced from measured strains and the corresponding stress-strain curve. However, this calculation did not account for the creep strain in the reinforcement which was evident in the test. Therefore, to minimize the effect of creep strain, the strains during the period when no construction activities occurred were neglected in obtaining a lower bound estimate of the force. Wall 4 involved an actual failure, the minimum reinforcement lengths required are calculated for each method and compared with those measured in the test.

Due to the large amount of data the comparison is divided into two groups. The first group largely corresponds to those belong to the "Tie-Back Wedge" methods and the second group corresponds to those belong to the "Slope Stability" methods.

Figure 14 to Figure 17 show the reinforcement tension calculated by each method for each layer of reinforcement along with the ultimate reinforcement strength. In the RMC walls (Wall 1, Wall 2, and Wall 3) the calculated forces in each wall are the same in each method since the only parameters required in these methods are the fill density, the friction angle, the surcharge intensity and the geometry of the wall, which are the same in all three walls.

For both Wall 1 and Wall 2 almost all methods overpredicted the tensile forces in the reinforcement, especially the Forest Service, Collin, Schmertmann et al., Ruegger and Coherent Gravity methods. Coherent gravity method (developed for inextensible reinforcement) assumes the lateral pressure coefficient varies from K_o at the top to K_a at depth (i.e. 6 m). Thus the horizontal pressure is overestimated for soil walls with extensible reinforcement. The Forest Service method assumes the magnitude of horizontal pressure corresponding to K_o condition, again overestimating the horizontal pressure in this case. The Collin method is purely empirical; the horizontal pressure is dependent on the intensity of the surcharge and the fill height but independent on the fill friction angle (except for surcharge loading) and the fill density. The Schmertmann et al. method assumes the surcharge to be equal to some equivalent fill height and uses the adjusted fill height in the analysis. The force required to satisfied equilibrium in the slope stability analysis is very dependent on the fill height used. Therefore the concept of incorporating the surcharge as some equivalent fill may overestimate the force in the reinforcement, especially for high surcharge intensity. The Ruegger method specifies a limit of the surcharge to be 20% of the wall height. In these walls, the surcharge was about 100 % of the wall height. In addition, a factor of safety equal to 1.3 has been included in the solution, therefore the results may not be comparable.

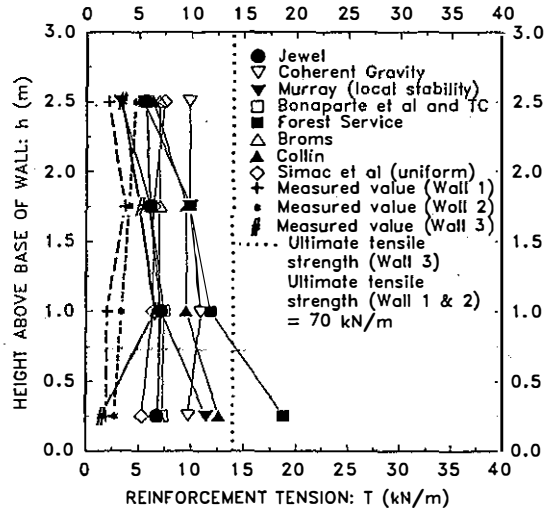

Figure 14. Comparison between measured and calculated reinforcement tension in Wall 1, Wall 2, and Wall 3 ("Tie-Back-Wedge" method).

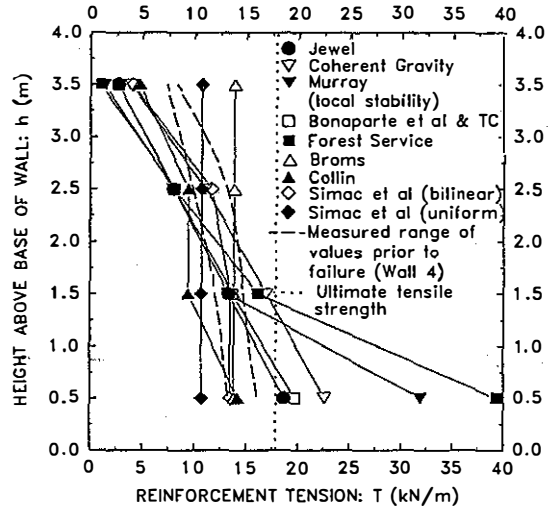

Figure 16. Comparison between measured and calculated reinforcement tension in Wall 4 ("Tie-Back-Wedge" method).

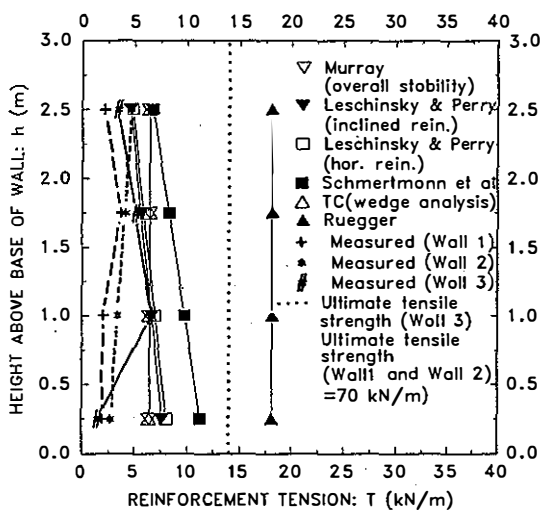

Figure 15. Comparison between measured and calculated reinforcement tension in Wall 1, Wall 2, and Wall 3 ("Slope Stability" method).

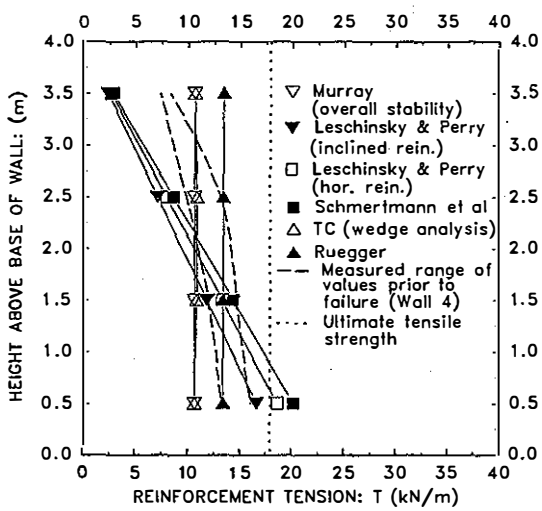

Figure 17. Comparison between measured and calculated reinforcement tension in Wall 4 ("Slope Stability" method).

In Wall 3, the predictions are closer except for those methods mentioned earlier in the predictions for Wall 1 and Wall 2. Again all methods overpredicted the force at the lowest layer of reinforcement. Both the Forest Service method and the Rugger method predicted reinforcement failure.

Most of the methods predicted an increase of reinforcement force with depth which was not evident for Wall 1 and Wall 2 although there was evidence of an increase with depth for Wall 3 (except at the lowest layer).

All the RMC walls involved the same wall geometry, same fill properties, and same surcharge intensity; but different reinforcement strength and stiffness, facing type and construction method. Variations in the measured forces among the three RMC

walls may be largely explained by these differences. In contrast, each method of analysis predicts the same reinforcement force for each of the three walls because this depends only on the strength, density and surcharge intensity. None of the methods can distinguish the differences that exist between the walls.

The situations in Wall 4 was simpler and approached the more idealized conditions assumed in the design methods (i.e. no compaction and limited influence from the wall face and the foundation). Most methods predicted a similar trend of increasing reinforcement force with depth. An increase with depth was also evident from measurements, but the measured rate of increase was much slower than generally predicted. Some methods predicted reinforcement rupture at the lowest layer of reinforcement.

In contrast to the predictions in Wall 1, Wall 2 and Wall 3, where the measured force in the lowest layer was the minimum, the measured force in the lowest layer in Wall 4 was the maximum. This is due to the limited influence from the wall face and foundation in Wall 4. These conditions approached those assumed in the design methods. Hence, most design methods also predicted the maximum reinforcement force in the lowest layer. In addition, most design methods underestimated the force in the uppermost layer. Wall 4 was built without any compaction, therefore the relatively high reinforcement force (compared to the rest) must be due to the interaction between the layers of reinforcement and redistribution of forces during the cutting process. Since these were not modelled in the design methods, these characteristics could not be captured. In fact, design methods that result in uniform force distribution appear to have provided a better prediction. This suggests that global equilibrium may be more important at the verge of failure of this wall. This finding may be more clearly seen in Table 2 where the sum of the predicted forces in the reinforcement are compared with those measured for each wall. In Wall 1, Wall 2 and Wall 3, the sum of forces predicted in each wall is much larger than those measured due to the influence of the foundation. In contrast, the sum of reinforcement forces predicted for Wall 4 by most design methods are in generally good agreement with those measured, especially when compared to the lower bound which is considered a better estimate of the actual total force. Although the sum of forces between predicted and measured were close, the distribution of force varied. It appears that there is the potential of force redistribution between the layers of the reinforcement in the real wall (redistribution of force was evident in the test). The ability to redistribute force among layers of reinforcement (e.g. due to local overstressing) is beneficial and may provide additional safety against wall failure, but this would only be significant for walls consisting of a large number of layers of reinforcement. The Ruegger method and the Schmertmann et al. method overpredicted the reinforcement force significantly in the RMC walls which had a 50 kPa surcharge. However, in Wall 4 where no surcharge was applied, their predictions were close to those measured. This suggests that using the concept of equivalent fill height to account for the surcharge in "Slope Stability" analysis may not be appropriate.

Predictions of the minimum required length for equilibrium in Wall 4 are shown in Figure 18 and Figure 19. Most predictions underestimated the required minimum reinforcement length and lay close to the lower bound based on the measured failure planes. The Broms method consistently overpredicted and the Leschinsky and Perry method underpredicted near the top of the wall and overpredicted at depth. Eventhough a factor of safety equal 1.3 has been incorporated in the Ruegger method, the prediction still underestimated the minimum required reinforcement length near the top of the wall.

One important feature of this wall is that the observed failure plane was more or less parallel to the Rankine failure plane. All the "Tie-Back Wedge" methods (except the Murray method) predicted the similar inclination of failure plane. Thus under idealized conditions such as those existed in Wall 4, the "Tie-Back Wedge" methods

Table 2. Comparison between measured and calculated total tension (kN/m) in reinforcement.

Method	Wall 1	Wall 2	Wall 3	Wall 4
Jewel	26.51	25.61	25.61	42.6
Coherent Gravity	40.33	40.33	40.33	54.64
Murray (local)	27.95	27.95	27.95	54.45
Murray (overall)	25.6	25.6	25.6	42.6
Bonaparte et al.	26.63	26.63	26.63	44.06
Forest Service	46.06	46.06	46.06	64.56
Broms	27.84	27.84	27.84	55.38
Collin	38.01	38.01	38.01	37.73
TC (local)	26.63	26.63	26.63	44.06
TC (wedge)	30.13	30.13	30.13	43.26
Simac et al. (uniform)	25.6	25.6	25.6	42.6
Simac et al. (bi-linear)	-	-	-	42.6
Leschinsky and Perry (inclined)	24.4	24.4	24.4	37.94
Leschinsky and Perry (hori.)	25.6	25.6	25.6	42.6
Schmertmann et al.	35.89	35.98	35.98	46.06
Ruegger	72.48	72.48	72.48	53.76
Measured	9.85	14.93	16.6	52.36-42.56

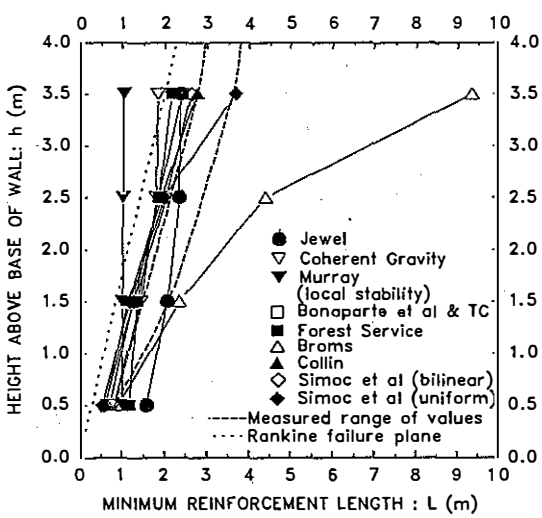

Figure 18. Comparison between observed and calculated minimum reinforcement length in Wall 4 ("Tie-Back-Wedge" method).

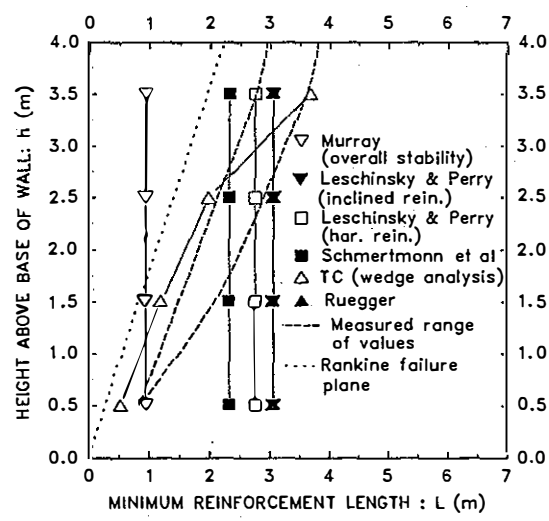

Figure 19. Comparison of observed and calculated minimum reinforcement length in Wall 4 ("Slope Stability" method).

provide a reasonable prediction of the shape of the failure plane. However, this conclusion should not be generalized for walls under more complex conditions. Lastly, the interpretations from the comparison are also subject to the assumptions made in 7.3. Under different assumptions, the results of some of the comparisons may be different. In

examining the result it is important to recognized that discrepancies between "predicted" and "observed" behaviour can occur due to factors such as uncertainties regarding the actual strength of the backfill (e.g. moisture in the fill can result in a significant apparent cohesion) and the conditions underwhich the test is performed (e.g. friction along the sidewalls for the RMC tests) and that these discrepancies are not related to limitations of the method of analysis. Keeping these factors in mind and considering all aspects of the comparison for the four walls, it is the authors' conclusion that in the method proposed by Jewel (Method No. 1), and Bonaparte et al. (Method No. 3), both generally provided a reasonable prediction of force to be resisted by the reinforcement and the location of the failure surface for the cases examined, although there was a tendency to underestimate the force in the upper part of the wall for Wall 4. The Broms method was conservative (in some instances unduely) for the cases examined. Not surprisingly, the Coherent Gravity method (developed for inextensible reinforcement) provides a conservative prediction of the forces likely to be developed in the reinforcement but was no better or worse than most other methods in terms of predicting the location of the failure surface for Wall 4. The Forest Service method and the Ruegger method tended to substantially overestimate the force that was to be resisted by the reinforcement.

8 CONCLUSION

The following conclusions summarize the findings of this paper:

(1) The vertical stress distribution at the base of reinforced soil walls is influenced by (a) the reinforcement length to wall height ratio, (b) the amount of reinforcement, (c) the rigidity of the facing, (d) the interface friction at the front boundary and the back boundary of the reinforced soil mass, and (e) strip loading. Under conditions of self-weight and a uniform surcharge, the vertical soil stress in the vicinity of the toe may be up to 25 % higher than the nominal value. In practice, this increase would not be significant due to the margin of safety provided by current design methods. However, strip loading may have a significant influence on the distribution of vertical stress. The increment in vertical stress due to a strip load may be substantially higher than that anticipated by common approaches. Care is required under such conditions.

(2) Inconsistencies in data interpretation have been discussed. The common approach of assuming equivalence between reinforcement force and horizontal soil stress has been shown to have important limitations. Similarly, comparison of the horizontal soil stress implied from Rankine active or at rest conditions with the measured or inferred value will not generally provide a good indication of the state of stress within the reinforced soil mass.

(3) The exceptionally low field measurements of horizontal soil stress at the wall face have been discussed. The main reasons for the low magnitude are: (a) the prevailing plane strain condition in the field results in a higher effective friction angle than generally assumed in analysis and design, (b) increased strength due to partial transfer of vertical stress to the facing, (c) the possibility of capillary action in the field, (d) additional shear strength provided by the interaction between the fill and the reinforcement, and (e) transfer of stress to reinforcement through a shearing process.

(4) Compaction appears to have relatively insignificant effect on the development of horizontal soil stress in reinforced soil walls using extensible reinforcement due to the relatively large magnitude of wall yielding.

(5) The force development in reinforcement depends on the stress-strain response of the entire system, which in turns depends on the strength of the soil; the quantity of the reinforcement, the stiffness and the spatial arrangement of the reinforcement, the stiffness of the facing and the foundation, the facing/reinforcement connection details, and the construction method.

(6) Under working stress conditions, the locus of maximum tensile force is not unique in reinforced soil walls constructed using

extensible reinforcement. It depends on the construction method, the facing/reinforcement connection details, the facing type, the length and stiffness of the reinforcement, and the location of the strip load. However, when the wall system approaches a state of failure, it appears that the locus of maximum tensile force tends towards the Rankine failure plane.

(7) Horizontal deformation in reinforced soil walls depends on a large number of factors. The most important controlling factors appear to be (a) the reinforcement stiffness, (b) the reinforcement length to wall height ratio and the shear strength of the soil, (c) the flexibility of the facing, (d) the construction methods and sequence, and (e) the intensity of surcharge and location of strip load in relation to the reinforced zone.

(8) Common design methods which are based on semi-empirical approaches or limit equilibrium analyses are inadequate for modelling reinforced soil walls under working stress conditions. On the basis of comparison of observed and calculated reinforcement forces it is concluded that the "Tie-Back Wedge" methods of analyses provide a reasonable prediction of the forces in the reinforcement for simple walls (i.e. limited influence from the facing and the foundation) in terms of total force required for equilibrium when the wall system is approaching failure. In particular, the methods proposed by Jewel (1987), Bonaparte et al. (1987), and Simac et al. (1990) appear to be among the better methods of analysis. However, none of the commonly used methods provides good prediction of force distribution among the layers of reinforcement because they are not capable of modelling the interaction between the wall components.

(9) There are significant variations among individual methods, and some are excessively conservative.

(10) There is insufficient field data available to advance the understanding of the behaviour of reinforced soil wall. It is recommended that more intensive instrumentation should be incorporated in future monitoring of real structures. Care should be taken in proper interpretation of field data.

ACKNOWLEDGEMENTS

The work presented in this paper is funded by the Natural Science and Engineering Research Council of Canada under grant A1007.

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