Recent case histories of earth reinforcement

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The influence of backfill settlement or wall movement on the stability of reinforced soil structures

C.J.F.P. Jones & D. Gwede
Newcastle University, UK

ABSTRACT: In 1986 a number of reinforced soil walls constructed across steep ravines on a highway in eastern Tennessee, USA collapsed. It was observed that in most of the failed sections large deflections of the reinforcing strips had occurred. The subsequent investigation of the failures raised issues that were not adequately considered in design practice. Tests have been conducted to simulate differential settlement of the fill relative to the wall facing in reinforced soil structures using full scale reinforcements. The tests modeled the scale of the differential movements observed in Tennessee. The results of the tests showed that some forms of reinforcement slip when differential settlement occurs. This has serious implications, the structure is no longer coherent and the coherent gravity hypothesis used in some designs is no longer valid. It can be concluded that settlement should be included as part of the analysis of high reinforced soil structures carrying a heavy surcharge load.

1 INTRODUCTION

The construction of a scenic highway along the foothills of the Great Smokey Mountains in eastern Tennessee was started in 1984. Two sections of the highway are located in rugged terrain and are built in steep cuts and on high embankments. The design of these two sections incorporated 14 reinforced soil walls ranging in height up to 18 m which supported embankments up to 25 m high. In April 1986 one of the project walls collapsed while the embankment above it was being constructed. In May 1986, a second wall, which had been constructed eight months earlier, collapsed. Construction of the walls was suspended and a series of investigations was initiated, Lee *et al* (1994). The investigation revealed widespread deficiencies in a number of the other reinforced soil walls, two of which were in imminent danger of collapse. All the walls were constructed using reinforced concrete facing panels, steel strip reinforcement and frictional fill. The design of the walls was based upon the empirical coherent gravity method which is widely used for the design of reinforced soil structures using "inextensible" reinforcement.

A feature of the Great Smokey Mountains wall collapses was that the failure mechanism did not conform to the failure modes assumed in the design. Failure of the two walls which collapsed was initiated by part of the facing at ($^{1}/_{3}$–$^{1}/_{2}$) height exploding outward. Inspection of the collapsed walls immediately after the failures showed that the reinforcement had ruptured at the connections with the facing and that there was major differential settlement of the backfill with respect to the facing, which in both walls measured 490 mm below the reinforcement strip connection levels. Similar differential settlements of the reinforcement relative to the facing have been have been observed in collapsed walls in Japan, as with the Tennessee walls rupture of the connections also occurred, (Tatsuoka, 2006).

The failure mechanism of the walls was identified as being complex, involving forward translation of the structures, loss of adhesion of the reinforcement, mechanical instability of the facing leading to rupture of the reinforcement/facing connections. The investigation identified a number of answered questions including:

(i) The transfer of tension in the reinforcement strips which preceded collapse and in particular why the failures occurred in the reinforcement/facing connections when current theoretical models assume that maximum reinforcement tension occurs at a position remote from the facing.
(ii) The seriousness of stress induced by deflection of the reinforcement.

This paper considers the influence of backfill settlement or wall movement on the stability of reinforced soil structures and reports on laboratory studies undertaken using full scale reinforcements into the fill/reinforcement mechanisms which develop.

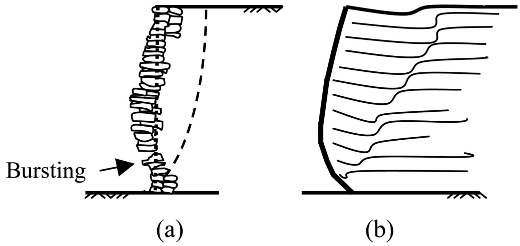

Figure 1. Failure mechanisms for (a) Victorian masonry walls (after Jones, 1979), (b) model reinforced soil walls (after John, 1983).

2 FAILURE MODES

Current design methods for reinforced soil structures are based upon limit states which may be defined in terms of limit modes covering external, internal and combined failure mechanisms. Analytical procedures adopted for reinforced soil structures often consider the various limit modes separately, although this is explicitly warned against in some codes such as Hong Kong Geoguide 6, (GEO, 2002). Critically, the implication of the development of one limit mode on another may not always be considered or appreciated. Consideration of the Tennessee wall failures indicate that combined limit modes can lead to unexpected failure mechanisms. In particular, the development of the Limit Modes of Sliding, Bearing, Reinforcement Adhesion and Deformation could have a major influence on the Limit Modes associated with Reinforcement Rupture and Rupture of the Facing.

Field observations of reinforced soil structures indicate that all backfill materials settle but that the settlement is less than 1% of wall height, (Findlay, 1978; Jones *et al*, 1990; Jones and Hassan, 1992). The settlement of the backfill in the Tennessee structures exceeded 3 per cent of the height.

Settlement of the backfill can occur with outward movement of the wall face produced as a result of sliding, lack of reinforcement adhesion or deformation. Figure 1 shows the deformation of dry stone retaining walls which collapse in a manner similar to the Tennessee walls in that they burst outwards at ($\frac{1}{3}$–$\frac{1}{2}$) height. Figure 2 shows a reinforced soil wall of height H and reinforcement length L that has translated a distance dL forwards. If the total volume of the backfill remains constant, the translation would result in a backfill settlement of dH where:

$$\mathrm{d}H = \frac{\mathrm{dL} + H\tan\theta - \sqrt{[(\mathrm{d}L + H\tan\theta) - 2H\mathrm{d}L\tan\theta]}}{\tan\theta} \quad (1)$$

In Tennessee a translation of 300 mm would have resulted in the observed deflection of 490 mm of the reinforcement relative to the facing. In addition to translation, isolated bulging of the facing could also induce strip deflections. Bulging of the facing of the walls in Tennessee was observed.

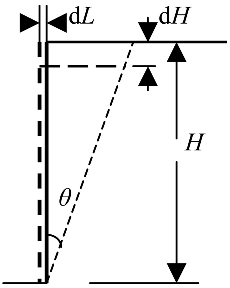

Figure 2. Wall-induced settlement (see equation (1)).

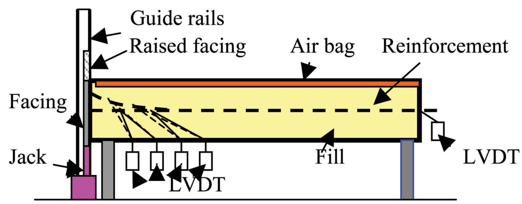

Figure 3. Settlement box.

3 LABORATORY STUDIES

In order to study the effects of differential settlement of the reinforced fill relative to the facing a special settlement box was developed. The settlement box was constructed as a conventional reinforcement pull out box having a front wall mounted between two vertical steel columns which allowed the face to move up or down in a vertical plane. Two hydraulic jacks were attached to the front wall to power the vertical movement, Figure 3. The overall dimensions of the box were (3.0 × 0.7 × 0.6 m) for the length, breadth and depth respectively permitting the use of full scale reinforcement.

Overburden pressure was simulated by the application of a uniform normal stress applied to the top of the fill by an airbag. The airbag was capable of developing a normal stress of 140 kN/m^2 equivalent to a fill height of approximately 7 m. Thus the stress conditions associated with the Tennessee failures could be replicated in the laboratory studies.

3.1 *Fill*

The fill used in the tests was Leighton Buzzard Sand. This was placed in the settlement box in layers and compacted using a hand-held vibrating plate compactor. An average density of 16.0 kN/m^3 was achieved. The specific gravity of the fill was 2.65 and the angle of internal friction was; ($\phi_{cv} = 32°$; $\phi_{peak} = 42°$).

3.2 *Reinforcement*

Two forms of reinforcement were used in the tests. The main study focused on the use of "high adherence" steel strip reinforcement as this was the reinforcement used in the Tennessee walls. This reinforcement develops greater resistance to pull out than plane strip, by having raised ribs 3 mm high cast into the surface. The reinforcement used was 50 mm wide, 6 mm thick and extended beyond the end of the test box. Tensile tests on the reinforcement showed the yield strength to be 490 MN/m^2 and the ultimate tensile strength to be 650 MN/m^2. Attachment of the reinforcement to the wall face was provided using a bolted connection similar to that used in field applications.

The second form of reinforcement which was studied for comparative purposes was a polymeric geogrid formed from high density polyethylene (HDPE). The reinforcement had a characteristic tensile strength of 80 kN/m. A strip of 0.5 m width was used in the tests. The polymeric grid reinforcement was connected to the moving wall face using two methods. In the first method a complete length of reinforcement was cast into the concrete facing panel. In the second a short starter piece of reinforcement was cast into the facing to which the main length of reinforcement was connected using a flat HDPE bodkin. Both of these connection systems are used in field applications.

3.3 *Displacement and force measurements*

Displacements of the steel strip reinforcement tested were measured at the wall face connection, the free end of the reinforcement and at two locations along the embedded length close to the facing. The displacements were measured using linear variable differential transformers (LVDT) attached to a computer. Based upon the measured displacements, the deflection profile of the reinforcement associated with any vertical location of the facing could be determined.

When extensible polymeric grid reinforcement was tested, the deflection profiles were determined using a water level formed from polymeric tubing attached to the reinforcement passing the full length of the cell and through the facing and rear of the pullout box. (This procedure could not be used with the steel strip reinforcement as it produced too much interference with the strip/fill adhesion characteristics.)

Tensile loads generated in the reinforcement during the tests were determined in two ways:

(a) By the use of strain gauges attached to the steel reinforcement.
(b) By the use of BISON gauges attached to the geogrid reinforcement.

Table 1. Settlement tests.

Test	Overburden pressure (kN/m^2)	Facing deflection (mm)	Reinforcement type	Connection method
1a	0	N/A	nil	–
2b	28	N/A	nil	–
3	21	200	geogrid	fixed
4	28	150	geogrid	fixed
5	28	150	geogrid	bodkin
6	28	200	geogrid	fixed
7	28	150	geogrid	fixed
8	85	180	steel strip	bolt
9	105	250	steel strip	bolt
10	116.5	160	steel strip	bolt
11	140	200	steel strip	bolt

Note: a. test with no backfill or reinforcement.
b. test with no reinforcement

3.4 *Test programme*

A total of 11 settlement tests were undertaken using overburden pressures ranging from (21–140 kN/m^2), Table 1. The lower overburden pressures were used with the polymeric reinforcement as this material had a very high soil/reinforcement adhesion which was fully developed at low pressures. The tests in the steel strip reinforcement used the maximum overburden capacity of the test box.

To simulate differential settlement of the fill relative to the facing the front face of the pullout box was displaced vertically. The rate of movement was 5 mm/minute and the maximum vertical displacement was 250 mm. Tests 1 and 2 were undertaken to determine the forced needed to move the wall facing alone and when fill but no reinforcement was present.

In addition to the settlement tests, five pull out tests were undertaken using the steel strip reinforcement. These were conducted at two overburden pressures of 60 and 100 kN/m^2, to determine the apparent friction coefficient (μ^*) of this form of reinforcement.

4 TEST RESULTS

The forces required to move the facing vertically under different conditions is shown in Figure 4. The deflection profiles of the steel reinforcement are shown in Figure 5 and those of the grid reinforcement in Figure 6.

Figure 7 shows the strain recorded in the geogrid reinforcement in Tests 3 to 7. Many of the strain gauges attached to the steel reinforcement failed during the test and little data was recorded, at no time did recorded strain exceed 0.08%.

The values of the apparent friction coefficient, μ^*, of the high adherence steel strip reinforcement

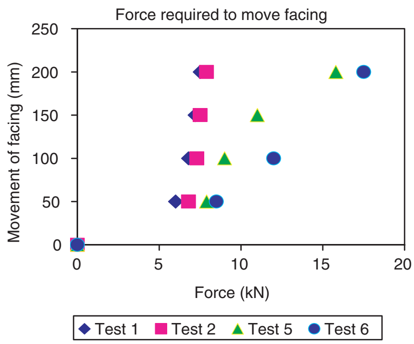

Figure 4. Force required to move facing.

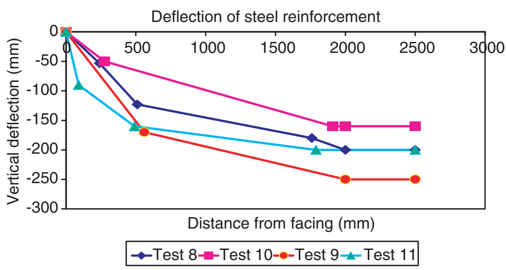

Figure 5. Deflection of the steel reinforcement with vertical movement of the facing.

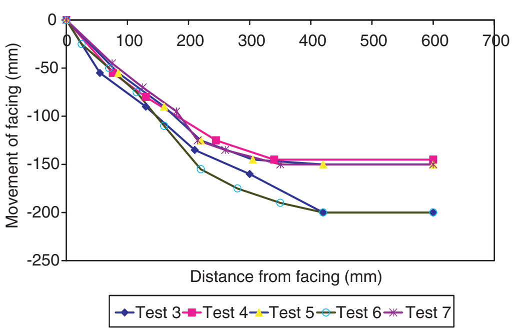

Figure 6. Deflection of the grid reinforcement with movement of the facing.

obtained from the pullout tests and calculated from strain gauge readings are shown in Tables 2 and 3.

5 DISCUSSION

5.1 *Force required to move facing*

The force required to move the facing, equivalent to the back wall friction of fill settling relative to the facing, ranged from 6–8 kN/m^2. This increased substantially when reinforcement was present, Figure 4.

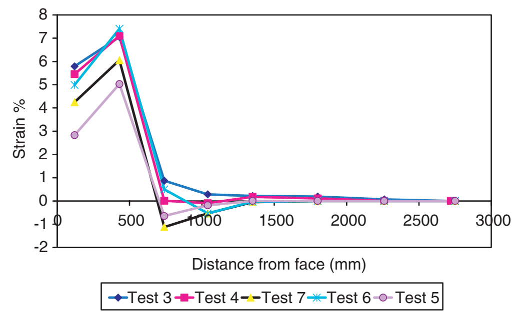

Figure 7. Strain in geogrid reinforcemt.

Table 2. Apparent friction coefficient, μ^*, from pullout tests.

Overburden pressure (kN/m^2)	Displacement at peak load (mm)	Apparent friction coefficient, μ^*
60	33	1.69
60	32	1.50
100	32	0.80
100	35	1.01
100	35	1.06

Table 3. Values of μ^* calculated from strain gauge readings.

Test No.	Pressure (kN/m^2)	Max. Tension in reinforcement	Apparent friction coefficient, μ^*
9	105	63.5–77.4	1.8–2.2
10	116.5	40–53.8	1.0–1.4
11	140	81.2–95.0	1.7–2.1

5.2 *Deflection profile of the reinforcement*

In the Tennessee wall failures a uniform deflection profile of the steel reinforcement was observed and this was replicated in the laboratory trials. In the failing walls, reinforcement frictional force was mobilized and the reinforcement slipped, becoming part of the failure mechanism. The steel reinforcement in the laboratory trials also slipped. It was observed that the steel reinforcement – facing connections had undergone significant strain with cracking of the galvanizing coating clearly visible. This implies that bending stresses were generated at the connections.

The deflection profile of the steel reinforcement in Tennessee prior to and following failure and in the laboratory trails can be expressed by the polynomial, (Lee *et al*, 1994):

$$S = A\,[1 - e\,(-K^2\,\chi^2)] \qquad (2)$$

Where, S = deflection of the reinforcement; A and K = constants; χ = horizontal distance measured from the facing.

Equation (2) can also be used to describe the deflection profile of the polymeric grid reinforcement but a better fit is obtained using a cubic polynomial:

$$S = H_{max} - 0.75\chi + 0.014\chi^2 / \sqrt{H_{max}} - 9x10^{-7}\chi^3 \quad (3)$$

Where H_{max} = maximum movement of the face.

5.3 *Strain and slip in the reinforcement*

Observations in the laboratory trials showed that the steel reinforcement slipped in all the tests. Slip started to occur at vertical movements of the facing of 20 to 40 mm. This confirms the stiffness of the reinforcement, that there was little extension of the materials and that it would be best to identify the material as effectively being inextensible. Slipping was initiated at strain levels well below that required to bring the reinforcement to yield even allowing for a length of 5m. Assuming no elongation Lee *et al* (1994) deduced that slip, S, can be defined by:

$$S = \int_{x_1}^{x_2} [1 + f'(x^2)]dx \quad (4)$$

Where $f'(x)$ is the equation describing the deflected profile of the reinforcement.

Applying Equation (4) the measured slip in the laboratory tests can be compared with the calculated slip, this is shown in Table 4.

The tests also produced evidence that differential settlement of the fill relative to the facing can introduce significant bending stresses in the reinforcement, particularly if the connections are rigid. A deflection of 175 mm at a distance of 1000 mm from the wall face is theoretically able to induce a stress, $\sigma_y = 350$ MPa. In reality some slackness will reduce this significantly.

The deflected profile of the polymeric reinforcement is a function of the total differential movement between the fill and the facing. The maximum strain following a 200 mm differential movement of the facing relative to the fill was 7 per cent which occurred at a distance of approximately 500 mm behind the facing, Figure 7. The reduction in strain at the facing could be a result of fixity of the reinforcement to the concrete facing. In none of the tests was geogrid reinforcement located 2000 mm behind the facing subjected to strain. This observation leads to the conclusion that grid reinforcement does not slip. When a bodkin joint was introduced the maximum strain in the geogrid reinforcement reduced to approximately 5 per cent, Test 5 Figure 7. The forces needed to move the facing also reduced, Figure 4.

Table 4. Measured slip compared with calculated slip.

Test No.	Measured slip (mm)	Calculated slip (mm)
8	55	33–68
9	40	30–50
10	27	22–32
11	30	25

5.4 *Apparent friction coefficient* μ*

The value of the apparent friction, μ*, in the pullout tests changed with the level of stress on the reinforcement, Table 2. The peak shear was mobilized at about 35 mm displacement. The values of μ* calculated from the strain gauges were higher than those obtained from the pullout tests, Table 3. The results compare with values obtained by other researchers and from pullout tests. Schlosser and Elias (1978) give a range for μ* of 0.33 to 2.5 for ribbed steel reinforcement in fine sand.

6 CONCLUSIONS

The reinforcement deflection profiles obtained in the laboratory study agreed closely with the observed deflections associated with the Tennessee failures. Once the fill settles >120 mm the steel reinforcement will slip. Failure to slip would result in rupture of the reinforcement. As soon as the reinforcement slips the concept of a *coherent* structure is lost and the analytical model using this concept is invalid. When reinforcement slip occurs forward movement of the structure is likely, leading to additional differential settlement of the fill relative to the facing. Settlements of the fill in the range of 120 mm could occur at the base of high walls, particularly those supporting embankments and this should be a consideration in the design.

Differential settlement of the fill relative to the facing induces bending stresses in the connections which are not usually considered during design; this could be part of the reason why the observed failures involved rupture of the connections.

Polymeric geogrid reinforcement was able to accommodate the differential settlements imposed during the tests and no slipping occurred. However the reinforcement was subjected to strains greater than those usually accepted. The use of bodkin connections appeared to have a positive effect in reducing reinforcement strain.

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