PULL OUT RESISTANCE OF GEOSYNTHETIC STRAPS FROM SHALE AND SILTY SANDS

Joel Lewis[1], Nathan Byrne[2], Russell Copeman[3], Tom Handel[4] and David Airey[5]
[1]AECOM, Sydney, [2]PB, Sydney, [3]Coffey Geotechnics, Sydney, [4]Leighton Contractors, Sydney
[5]School of Civil Engineering, University of Sydney

ABSTRACT

This paper presents results from a study of the pull out resistance of geosynthetic (Paraweb™) straps from two fill materials. The first is sand with variable silt contents and the second is Ashfield Shale. For both materials shear box tests of the fill material, shear box interface (fill material - geosynthetic) tests and pull out tests have been performed for a range of confining stresses.

There has been evidence of the poor performance of reinforced shale fills in Sydney, but the reasons for this are not well understood and many different opinions exist. These include weathering/breakdown of the shale, wetting/pore pressure development, creep and arching. Tests have been performed to evaluate which of the proposed mechanisms is responsible for the poor performance. It is shown that the tendency for even well compacted shale to compress when sheared contributes to arching, which leads to low pull out resistances. It has been found that when the shale is crushed to <75 μm the pull-out resistance is greater than when a uniform material with gravel sized lumps is used. This is explained by the arching mechanism and the impact of the grading on the stiffness, density and compressibility of the fill material.

It is widely accepted that reinforced earth walls (REWs) need to contain 'good quality' fill i.e. sands and gravels that are well graded. For example, the RTA QA specification R57 (2007) states that soil properties should be granular and contain less than 15% fines <75 μm. Tests have been performed to assess the effects of increasing non-plastic fines content by mixing silt with sand. Addition of silt affects maximum and minimum densities, frictional and dilatational characteristics and the pull-out resistance. It is shown that addition of non-plastic fines significantly improves the pull-out resistance for low fines contents and as further fines are added the pull-out resistance reduces. However, even with 26% fines the pull-out resistance is greater than for clean sand.

1 INTRODUCTION

It is a standard requirement that only high quality fill be used for backfill behind reinforced earth walls. However, for many projects the ability to use on-site materials not meeting the relevant specifications for 'good quality fill' would result in significant cost savings. This paper is concerned with the pull-out resistance of geosynthetic straps from two such materials. The first are silty sands that have non-plastic fines contents exceeding the 15% limit prescribed in many specifications (e.g. RTA, 2007) and the second are shale fills that meet the grading requirements, but have potential to degrade and are prohibited by the RTA specifications.

The ability to use marginal fills is limited by the lack of understanding of the factors that influence the pull-out resistance, as only a limited number of studies have specifically investigated poor quality fills (e.g. Wei, 1990; Bauer and Shang, 1993; Bergado *et al.*, 1993; Lee and Bobet, 2005) and available numerical tools to capture all these factors are limited.

Nevertheless, many studies have been conducted into the behaviour of reinforced earth walls and the factors governing the pull-out resistance are generally well understood. It is affected by the frictional properties of the soil and reinforcement, the degree of saturation and drainage of the fill, the stresses acting on the reinforcement, which are affected by burial depth and arching and the compaction/construction procedures (e.g. Shen *et al.*, 1979; Wei, 1990; McKelvey III, 1994; Jones, 1996). Many of these factors are inter-related and it is this inter-relation that is the cause of much of the difficulty when assessing the suitability of poor quality fills.

1.1 FACTORS AFFECTING PULL-OUT RESISTANCE

- Soil friction – A function of particle shape, mineralogy and grading. Increasing angularity increases the ultimate frictional resistance and increases dilation and hence peak resistance. Well graded soils tend to have lower frictional resistance than uniform soils.
- Interface soil-reinforcement resistance. This is primarily a function of the roughness of the interface. This depends on the texture of the interface surface and the particle size. This can be expressed by a relative or normalised roughness, which describes the roughness of the interface relative to the mean particle size. Increasing fines content increases normalised roughness, and the friction that can be mobilised increases up to a limiting normalised roughness after which shearing occurs in, and is controlled by, the fill material.
- Reinforcement strength and stiffness - Pull-out capacity is limited by the capacity of the reinforcement straps, and by the need to limit their extension. Geosynthetic straps can develop large strains that can lead to large displacements of the REW, and the Poisson effect can lead to thinning of the straps which can reduce the normal stress acting due to arching (see below). Extensible reinforcement is generally found to have lower pull-out resistance than rigid.
- Burial depth – Because the response is primarily frictional increasing burial depth, which increases the normal stress acting on the reinforcement, should increase resistance to pull-out. Fill stiffness also increases with depth.
- Dilation – As reinforcement moves relative to the soil, volume changes occur in the soil adjacent to the interface. For a dense well compacted granular fill these volume changes will be expansive. As the stress increases (greater burial depth) the tendency for expansion reduces. The amount of expansion is also sensitive to the relative density. The tendency for a soil to expand when sheared leads to a peak resistance greater than its ultimate frictional resistance, but this effect is limited as the capacity for volume change is restricted.
- Confined expansion (dilation) also leads to the normal stress on the reinforcement increasing above that due to depth of burial, which in turn leads to increased frictional (pull-out) resistance. This effect continues even after the peak in shear resistance. If the soil around the reinforcement compresses then the reverse effect can occur, with the normal stress on the interface dropping below that due to depth of burial, and the stress "arches" around the reinforcement. The effects of the confined volume changes are a function of the stiffness of the surrounding soil. For a circular inclusion the relevant stiffness is given by 4G/d, where d is diameter and G is shear modulus of soil. The amount of expansion that occurs is a function of the relative density, the stress level, the normalised roughness and the extensibility of the reinforcement. Chevalier *et al*. (2009) show that the stress can drop very rapidly due to arching effects in loose sand.
- Relative density – depends on compaction/construction methods. High relative density is desirable to obtain maximum expansive strains in soil adjacent to interface, but heavy equipment cannot be used adjacent to an REW. Increasing relative density also increases stiffness.
- Grading – effects soil friction (mentioned above), volume change characteristics (more uniform can lead to more dilation), stiffness and drainage. Changes to the grading may occur due to crushing of particles during construction and during shearing, and due to the breakdown of aggregated particles.

The inter-related factors discussed above that influence the pull-out resistance are further complicated when using shale derived fills because of issues related to their durability and breakdown. It has been argued (Hopkins, 1988) that even high strength shales will eventually degrade into clays and silts, however, experience in NSW suggests that there are many instances of shales performing satisfactorily in roads over many years (Croft, 1966; Hatcher, 1962). For shale to perform satisfactorily it appears to be important to avoid excessive crushing and working of the particles during construction. Alternatively it has been suggested that shales should be highly worked to break down the particles as much as possible to prevent subsequent degradation and property changes in use (e.g. Barney *et al.*, 1981). As a result of experience in NSW the current RTA (2007) specifications (DCM R57, R58) state, "Material derived from argillaceous rock such as shales and claystones or other friable materials which are susceptible to breakdown shall not be used as reinforced fill material".

2 MATERIALS

2.1 SILTY SAND

A series of tests have been performed in which Sydney sand has been mixed with a silt to produce the gradings, shown in Figure 1, with fines contents varying from 0 to 30%. The sand particles are sub-angular, with d_{50} = 0.33 mm and no fines.

Soda Feldspar 200S was used for the non-plastic fines material. The material is a white inert powder with no odour. The particles are angular with d_{50} = 20 μm and 98.5% passing a 75 μm sieve. The feldspar contained 70% Silica with less than 1% silica fume. It contained approximately 18% Alumina and 11% Soda.

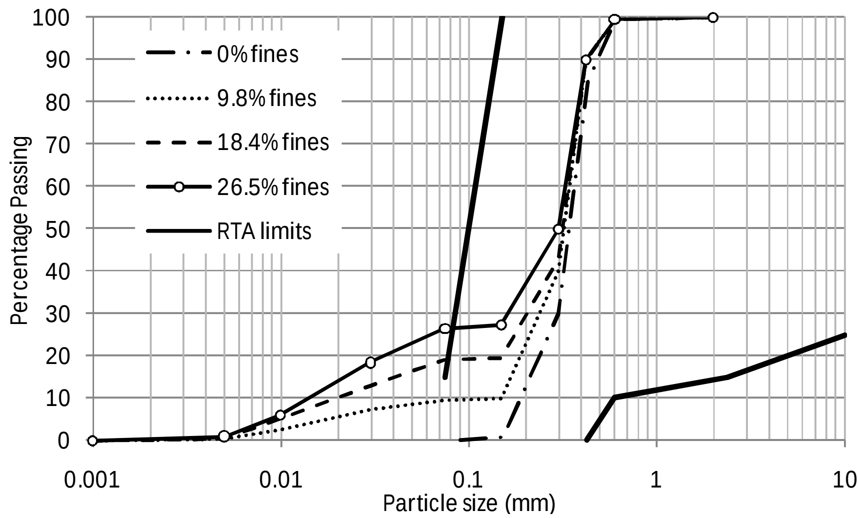

Figure 1: Soil gradings for sand-silt mixtures,

One of the consequences of adding fines is that the minimum and maximum void ratios change. Soil compacted to the same density would thus have different relative densities and this could contribute to differences in response. Minimum and maximum void ratios were measured using the standard procedures in AS 1289. The results shown in Table 1 indicate a trend for the limiting void ratios to decrease for silt contents up to 20% and then to increase as silt content increases further. This pattern has been observed in other studies (Salgado *et al.*, 2000; Lade and Yamamuro, 1997) with different sands when non-plastic fines have been added. The difficulty of reliably measuring the limiting void ratios with silt contents greater than 15% has been widely discussed in the literature and may partly explain the relative densities greater than 1 achieved in the tests discussed below.

Table 1: Minimum and maximum void ratios of silty sand mixtures

Silt Fines (%)	e_{max}	e_{min}
0	0.76	0.55
9.75	0.78	0.55
18.4	0.76	0.47
26.5	0.86	0.48
100	2.31	1.08

2.2 SHALE

Fragments of Ashfield Shale were obtained from a deep excavation adjacent to Chatswood train station. The *in situ* material has been reported as Class III and Class IV Ashfield Shale. Approximately 400 kg of broken down shale was collected from the excavation, the majority of which was classified as gravelly silty sand. Some coarse gravel and cobbles were also included. The shale pieces were highly angular and platey (sphericity 0.1-0.7 and roundness 0.1). Slake durability tests were performed on ten pieces each weighing approximately 50 g according to the standard ISRM test procedure. The 2-cycle slake durability was I_d = 91.1% suggesting a hard and durable material that should be suitable for select fill.

To investigate the effects of possible degradation of the shale three gradings were selected. The first was a uniform gravel comprised of highly angular intact gravel sized lumps. This grading would not meet the RTA specifications for good quality fill. The sandy gravel grading was intended to produce a material that would satisfy the grading requirements for a well graded fill with a small fines component. The silty sand grading was produced by mechanically breaking down the large pieces with a hammer to give a large fines content representative of a weathered shale. It should be noted that mechanical breakdown of the Ashfield shale was difficult in practice and is another indication of the durability of the material.

Additional durability tests involving 5 wet-dry cycles and 3 compaction cycles as specified by RTA (2007) were performed to assess possible changes to the grading of the medium gravel as a result of construction processes. The resulting weathered gravel grading is also shown on Figure 2. Most of the breakdown occurred during the compaction cycles, and was mostly the result of particles splitting along bedding planes. Some flaking of small fragments was observed but very little fines were generated. It is believed that the 'well graded' sandy gravel would be more resistant to particle breakdown. Thus, even allowing for possible heavier compaction loads and more wetting and drying cycles, it appears that only limited breakdown of the Ashfield shale derived fill should occur during construction and this should not result in significantly altered engineering properties.

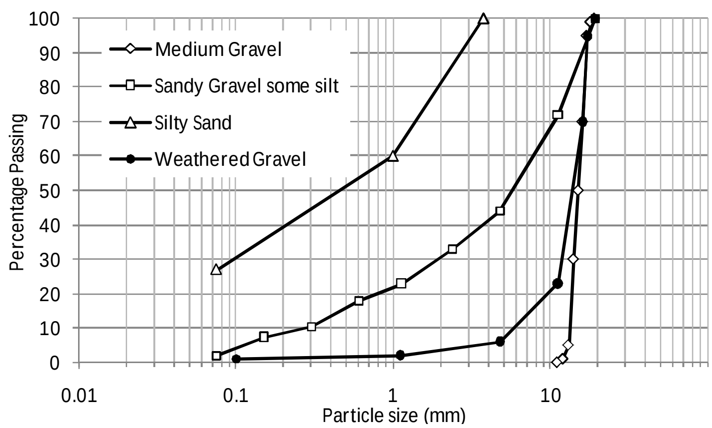

Figure 2: Particle gradings of materials derived from shale.

2.3 GEOSYNTHETIC REINFORCING STRAPS

Paraweb^{TM} grade 50 was used for the reinforcing straps. The straps are discrete bundles of high tenacity polyester fibres lying parallel to each other and encased in a tough durable polymeric sheath (Linear Composites, 2007). The straps have an ultimate tensile strength of 50 kN.

3 EQUIPMENT AND PROCEDURE

3.1 SHEAR BOX TESTS

To measure the internal friction angle of the fill materials, tests have been performed in a 100 mm square shear box for silty sand specimens and a large 300 mm, square in plan, shear box for shale specimens. For the silty sands, material was placed dry into the shear box and compacted in layers to produce dense specimens with relative densities that ranged from 1 to 1.06. For the shale specimens a similar procedure was used except that water was added to the two finer gradings at approximately the optimum moisture content and compaction energy was equivalent to standard compaction. The resulting dry densities for the shale varied with fines content with a similar pattern to the silty sand. The shale dry density increased from a mean of 1.4 t/m^3 for the gravel specimens to 1.69 t/m^3 for the sandy gravel and then reduced slightly to 1.62 t/m^3 for the silty sand.

3.2 INTERFACE TESTS

Tests have been performed in the large 300 mm square shear box. The bottom half of the box has been filled by a steel plate and the geosynthetic straps screwed down to the plate so that the upper surface of the geosynthetic was flush with the split between the two halves of the shear box. New straps were used in each test to avoid changes to the interface roughness caused by sand indentations in the geosynthetic surface. Material was compacted into the shear box in the same way as for the soil-soil tests.

3.3 PULL-OUT TESTS

The pull-out tests have been performed in a specially manufactured apparatus with internal dimensions 262 mm by 297 mm in plan by 290 mm high. A slit in the box 100 mm wide by 6 mm high is machined at the centre (mid-height) of one of the sides of the box through which the geosynthetic strap (90 mm wide by 3 mm thick) protrudes. The reinforcing strap is clamped to an actuator which is able to pull out the straps at a rate of 1 mm/min. During testing the pullout load and displacement are monitored continuously. A normal stress is applied using a hydraulic jack to the soil through a steel plate on the surface of the fill. The various materials were compacted into the pull-out apparatus in the same way as for the shear box and interface tests.

4 RESULTS AND DISCUSSION

4.1 EFFECTS OF NON-PLASTIC FINES CONTENT

Figure 3 shows a comparison of the shear stress mobilised in the three apparatuses for the mixture with 18.4% fines. The shear stress in the pull-out tests is estimated from the total pull-out force divided by the surface area of the strap contained within the soil container. All the tests shown in Figure 3 were conducted with an applied normal stress of 30 kPa. It may be seen that the ultimate resistance is similar in the soil-soil and soil-interface tests, although much higher peak shear stresses were mobilised in the interface tests. This pattern is unusual as one would expect greater dilation in the soil-soil tests and hence greater peak resistance. It is believed that this result is related to the use of different shear boxes, a 100 mm box was used for the soil-soil tests and a 300 mm box for the soil-interface tests. The pull-out test mobilises significantly higher shear stress. As the response is entirely frictional this is an indication that the normal stress acting on the geosynthetic straps must be significantly greater than the average applied stress. Similar trends were observed for all fines contents.

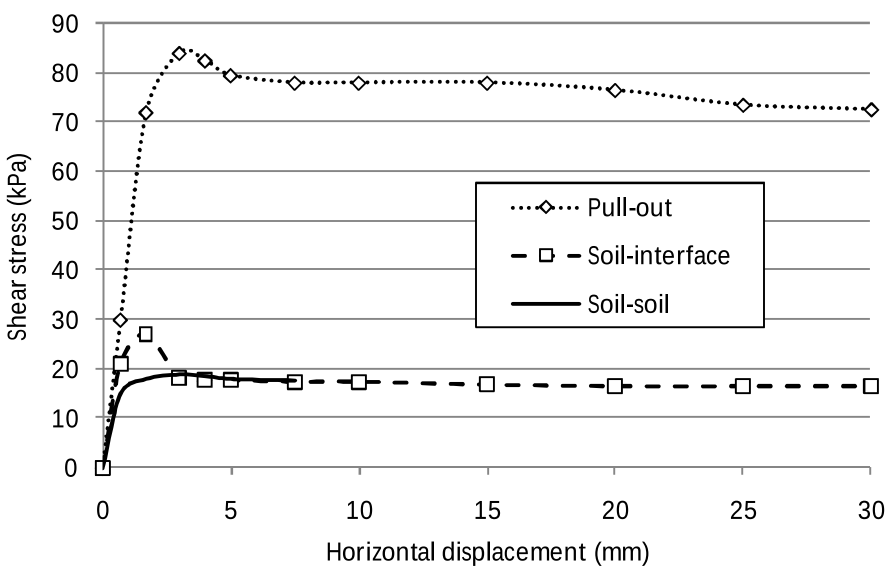

Figure 3: Comparison of stress-displacement responses from the three apparatus (18% fines, σ_n=30 kPa).

Figure 4 shows a comparison of the peak and ultimate friction angles inferred from the three apparatus (calculated from $\emptyset = tan^{-1}(\tau/\sigma)$) as a function of fines content. The soil-soil shear box tests show little effect of fines content and a practically constant ultimate friction angle of 31.5^o. The soil-interface tests show a trend for the ultimate frictional resistance to increase as fines are added, but no change for fines contents between 10% and 26%. The interface tests show a marked increase in peak resistance, indicative of greater dilation, at 10% fines and this effect continues at higher fines contents. The pattern in the interface tests is consistent with other studies of silty sands (e.g. Salgado *et al.*, 2000), and suggests that the relative density in the small soil-soil shear box tests was less than in the other apparatus even though similar methods of compaction were employed. It is clear that the pull-out resistance initially increases as fines are added to the sand, and with further fines the resistance drops back to that of the sand. Similar trends have been noted in other studies (e.g. Lee and Bobet, 2005). Pull-out tests on loose sand also showed increases in resistance as fines percentage increases before dropping below that of the sand at fines content of 26%. It was also observed that the amount of dilation was similar up to 18% fines, and tended to drop off beyond this fines content.

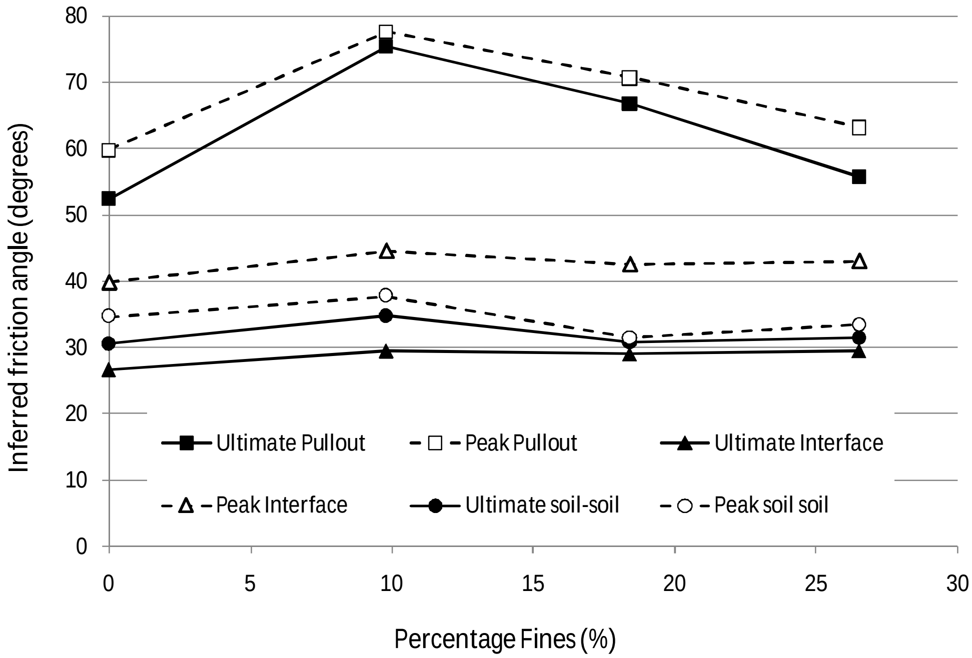

Figure 4: Effect of fines content on resistance in different tests.

As the response between the geosynthetic strap and the soil is purely frictional the high inferred friction angles can only be explained by the normal stress on the straps exceeding that from the applied normal stress. This arises from the dilation that occurs as the soil is sheared adjacent to the strap and the restraint to dilation provided by the surrounding soil leads to increased normal stress acting on the strap. Considering the effects of the fines content it is suggested that the enhanced pull-out resistance at 10% fines is a consequence of an increase in the soil-interface friction, due to increased normalised roughness and possibly greater stiffness of the surrounding soil. As the fines content increases further there is no change to the frictional characteristics, but a reduced stiffness leads to less effect from dilation. Further increase of fines leads to reduction of dilation and reduction of stiffness leading to lower resistance.

This interpretation of the pull-out tests is consistent with data reported by Salgado *et al.* (2000) from triaxial tests on silty sand. They showed that the shear stiffness at small strain decreased as fines content increased. However, the ultimate frictional resistance increased with fines content and the dilation increased to a maximum at about 5% fines, then reduced with increasing fines but was still greater than for clean sand at 20% fines content.

4.2 ASHFIELD SHALE

4.2.1 Results

A comparison of the shear stress versus displacement from the three test types is shown in Figure 5 for the sandy gravel grading. The specimens were compacted in the same way as the silty sand, however it is evident that the shale fill is more compressible. There was no dilation in any of the tests and this is evident from the gradual rise of shear stress to the ultimate value in both soil-soil and soil-interface tests. Similar trends were observed for all three gradings. Tests were performed at three normal stress levels and in each apparatus the peak resistance was linearly related to the normal stress with no apparent cohesion. A summary of the results is presented in Figure 6. This shows the mean friction angle inferred from each set of tests for the three gradings and for the three apparatuses.

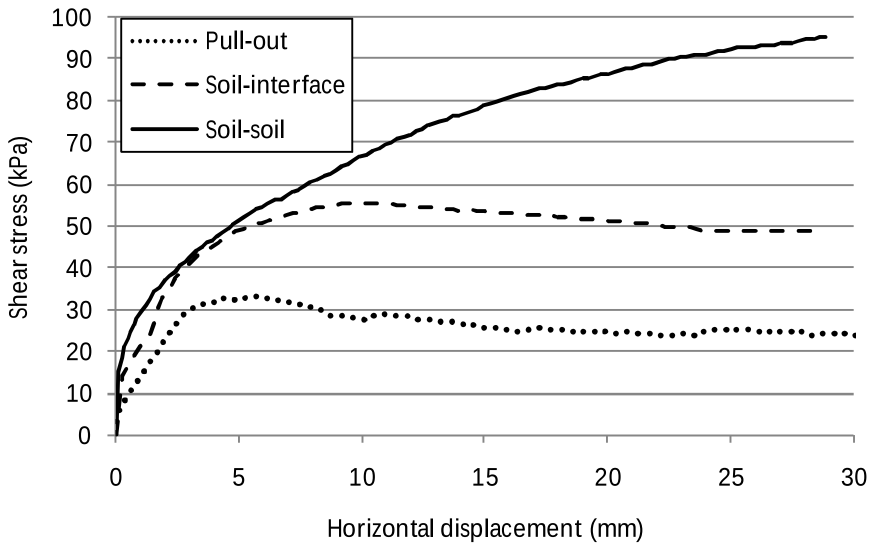

Figure 5: Comparison of stress-displacement responses from the three apparatuses (σ_n = 100 kPa).

It can be seen from Figure 6 that the friction angle from the soil-soil tests reduces as the fines content increases. The high friction angle is a consequence of the angularity of the shale lumps. The slight decrease in friction angle is believed to be a consequence of the decreasing uniformity of the grading, as the breakdown of the shale does not affect the angularity. In the soil-interface tests the opposite trend of increasing friction angle with decreasing uniformity of the grading is observed. This is believed to be a result of increases in the normalised roughness as the amount of fines increases. The pull-out resistance increases in the same way as the soil-interface tests, however, the 27% increase in resistance in the pull-out tests is significantly greater than the 18% increase in the interface friction.

4.2.2 Discussion

Unlike the silty sand which shows greater pull-out resistances than expected from the interface tests, the shale materials show lower pull-out resistances. This is primarily a consequence of the absence of dilation in the shale specimens tested. It is believed that this is a consequence of the highly angular particles. As is well known increasing angularity in granular materials leads to increases in void ratio, decreases in density. Thus although the material is well compacted, the ability of the material to dilate is limited.

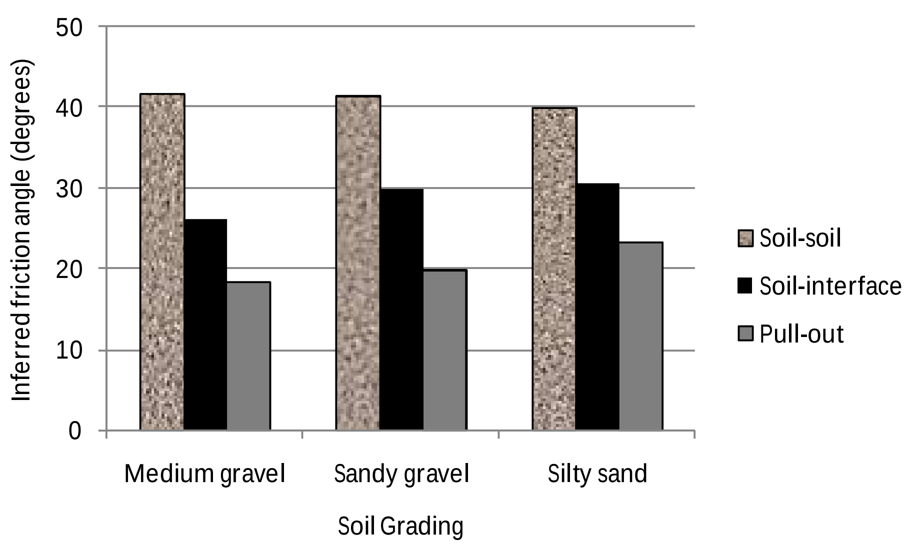

Figure 6: Comparison of peak friction angles inferred from different apparatus.

The ultimate pull-out resistances from specimens which had been saturated by submerging them under water were essentially identical to those tested at the as compacted moisture content. The responses thus appear to be essentially frictional. As for the silty sand the changes in the frictional resistance can be explained by the confined dilation (here it is compression) which results in lower normal stresses acting on the reinforcement straps than expected from the normal stress applied to the apparatus. As the fines content increases the stiffness of the material can be expected to reduce and thus for the grading with the highest fines content we would expect the lowest change of stress for a given amount of compression. However, we would also expect the material with the highest fines content to be more compressible. In these tests the same compaction energy was used for all three shale gradings and the results of these conflicting factors were to result in an increased pull-out resistance with increasing fines. Given the interplay of these factors it may be that other gradings and shales may lead to different trends in pull-out resistance.

The effects of grading on shale derived from Bringelly Shale were also investigated by Saadi (2000). In these tests the shale was crushed so that the material was predominantly fine grained and results from pull-out tests in the same apparatus as described above were compared with the uncrushed uniform gravel. It was observed that the frictional pull-out resistance with the crushed shale was slightly greater than for the gravel sized shale pieces. This is consistent with the trend observed in this study on shale from Ashfield Shale showing greater pull-out resistance with increasing fines. Another interesting observation from the tests performed by Saadi (2000) was that after pulling out the reinforcing strap the hole did not close up and was clearly visible when dismantling the apparatus. This occurred despite the Paraweb strap being 90 mm wide and only 3 mm deep, and is further evidence of the importance of arching.

5 CONCLUSIONS

The tests on sands with non-plastic silty fines have shown that the pull-out resistance of geosynthetic straps initially increases as silt is added until the silt content is somewhere between 5% and 10% and then decreases gradually as further silt is added. At a silt content of 26% the pull-out resistance was still greater than for the uniform sand. The increase in resistance is believed to come from the greater dilation of the sand after silt is added, which is partly a consequence of the increased density due to the change in grading. The reducing stiffness that occurs as silt content increases acts to limit the effects of the dilation and the dilation also reduces as silt content increases beyond about 20%.

From the point of view of the pull-out resistance the limit of 15% fines which appears in most specifications for reinforced earth walls appears to be conservative. However, it should be noted that only one sand and silt have been investigated and considering the interacting factors it is recommended that further study of different materials should be considered before accepting higher fines contents. It may also be noted that the pull-out capacity is only one component of the reinforced earth wall system and the impact of reductions in soil stiffness and soil permeability that occur from adding the fines also need to be considered.

The tests on the various gradings of Ashfield shale have suggested that this material is relatively durable and is unlikely to break down significantly during construction, or after construction when it is buried below the surface. Even if it were to break down there is no evidence to suggest that this would deleteriously affect the pull-out resistance of geosynthetic straps, indeed all the evidence suggests that the pull-out resistance would increase if finer material is present at the interface. The primary problem with the shale fill appears to be its low density, a consequence of its angularity, which leads to little or no dilation as it shears. The tendency to compression combined with a relatively high stiffness lead to low pull-out resistances. A consequence of the low pull-out resistance is that more reinforcing straps will be required and the economics may dictate that reinforced soil walls using shale are not viable.

It may be noted that for both shale and silty sand the soil-interface and soil-soil tests give a poor indication of the likely pull-out resistance and it is recommended that pull-out tests should be conducted before any marginal fill is used in a reinforced earth wall.

6 REFERENCES

Barney, C.H., Lovell, C.W. and Wood, L.E. (1981) Development of a laboratory compaction-degradation test for shales, Transportation Research Record 790, Washington Transportation Research Board

Bauer, G.E. and Shang, Q. (1993) Pullout resistance of large geogrid specimens in site specific soil, Geotechnical Engineering, Vol 24, No. 1, June, pp 17-37

Bergado, D.T., Shivashankar, R., Alfaro, M.C., Chai, J-C. and Balasubramaniam, A.S. (1993) Interaction behaviour of steel grid reinforcements in a clayey sand, Geotechnique, 43, 4, 589-603

Chevalier B., Combe G., and Villard P., (2009) Experimental and numerical study of the response of granular layer in the trap-door problem, Powders and Grains 2009, Golden - CO (USA)

Croft, J.B. (1966) An investigation into the methods of selecting shales for pavement construction, ARRB Proc. 3, 2, 1261-1285

Hatcher, N.F. (1963) Shale road pavements, Main Roads Journal, 28, 4, 110-117

Hopkins, T.C. (1988) Shear strengths of compacted shales, Kentucky Transportation Research Program, University of Kentucky, Lexington

Jones C J F P (1996) Earth Reinforcement and Soil Structures, Thomas Telford edition, Butterworth & Co

Lade, P. And Yamamoura, J. (1997) Effects of non-plastic fines on static liquefaction of sands, Canadian Geotechnical Journal, 34, 6, 918-928

Lee, H-S and Bobet, A (2005) Laboratory Evaluation of Pullout Capacity of Reinforced Silty Sands in Drained and Undrained Conditions, Geotechnical Testing Journal, 28, 4, 370-379

Linear Composites (2007) Products-Paraweb™ accessed from http://linearcomposites.com/prod_paraweb. htm

McKelvey III, J.A. (1994) The anatomy of soil arching, Geotextiles and Geomembranes, 13, 317-329

Roads and Traffic Authority. (2007). RTA QA Specification R57, Design of Reinforced Soil Walls. Accessed from http://www.rta.nsw.gov.au/doingbusinesswithus/specifications/roadworks.html

Saadi, G. (2000) The frictional behaviour of reinforced shale, Undergraduate Thesis, University of Sydney

Salgado, R., Bandini, P. And Karim, A. (2000) Shear strength and stiffness of silty sand, Journal of Geotechnical and Geoenvironmental Engineering, 126, 5, 451-462

Schlosser, F. and Elias, V. (1978) Friction in reinforced earth, Proc. Symp. on Earth Reinforcement, American Society of Civil Engineers, 735-763

Shen, C.K., Mitchell, J.F., DeNatale, J.S. and Romstad, K.M. (1979) Laboratory testing and model studies of friction in reinforced earth walls, Proc. Int. Conf. on Soil Reinforcement, Paris, 1, 169-174.

Wei, M.J. (1990) Frictional behaviour of reinforcement in reinforced earth fill materials, PhD thesis, University of Glasgow