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Pullout Resistance of Ribbed Steel Strips in a Crushed Sandstone with High Fines Content

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Summary: The pullout resistance of a ribbed steel strip embedded in a crushed sandstone with high fines content was investigated by large scale pullout testing. The pullout apparatus has compartments on the sides and rear of the box to allow wetting of the soil. Overburden stresses used in the testing were in the range of 20 to 110 kPa. In addition to measuring the pullout resistance under monotonic loading, the ability of the reinforcement to hold a pullout load, ie. creep characteristics, was also studied. The test results showed that the apparent coefficient of interface friction may still be higher than tan$ at low overburden stresses. This can be explained by the concept of constrained dilatancy. The highly dilatant behaviour of this soil is supported by independent triaxial testing.

INTRODUCTION

Traditional materials used in New South Wales for reinforced earth structures are granular materials, often with fines contents limited to below 15% passing the 75 ym sieve size (AASHTO 1997, RTA R57 2002) in order to satisfy the design requirements for mechanical strength, drainage and corrosion. However, higher fines content is allowed in some specifications provided that appropriate measures are made in the design and specification (Geoguide 6 2002).

One beneficial "by-product" of using granular soil as select fill is the high interface strength between the strip and the surrounding soil. The apparent coefficient of interface friction recommended in most design documents is higher than $\tan\phi$ at low overburden stress. An apparent coefficient of interface friction may be as high as $2\tan\phi$ near the surface. However, the data to justify such a design recommendation is largely based on soils with low fines content. If soils with high fines content are used as select fill, quantification of this interface parameter requires investigation.

Crushed sandstone material from the upper Blue Mountains, approximately 100km west of Sydney, can have fines content up to 40% (and with typically 15% and 5% passing 10µm and 2pm respectively). This is significantly higher than the Hawkesbury Sandstone material found in the Sydney Basin, which is generally used as select fill for reinforced earth wall construction in Sydney. Materials with this high fines content in steel-strip reinforced earth walls are uncommon due to concern over strength and durability. The objective of this paper has investigated the pullout resistance of a proprietary ribbed steel-strip in a crushed sandstone with high fines content by large scale pullout testing. The tests results are explained in terms of constrained dilatancy.

MATERIALS

The reinforcement is a galvanized proprietary ribbed steel strip 50mm width by 4mm thick. The soil is a crushed sandstone from the upper Blue Mountains which is a silty sand. The plasticity index of the materials is between 5 and non-plastic. The soil, as ripped from the site in-situ, had about 20-30% fines. The soil grains showed significant disintegration with compaction leading to the high fines content. The compacted soil retrieved from the pullout box after completion of testing had a fines content close to 40%. The typical grading curves before and after a pullout test are presented in Fig. 1.

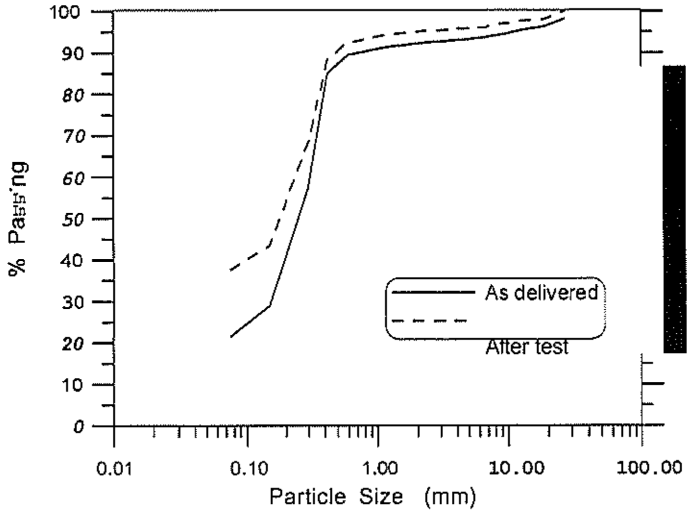


Figure 1. Grading curves.

PULLOUT TESTING

Pullout Box

The pullout box is schematically illustrated in Figs. 2a-b. The box design is similar to that presented in Lo (1990) except that the current design has a larger overall dimension of 2m in length and is fitted with perforated partitions. The locations of these partitions are adjustable and hence the effective size of the pullout box can be reduced to match the type of reinforcement and test conditions. The partitions create side and rear compartments which, for this study, stored water for wetting the soil. The partitions were lined with a non-woven geotextile in order to prevent loss of soil through the perforations and to promote wetting.

The pullout box consists of two halves and this enables good control of the location and alignment of the reinforcement to avoid jamming at the exit slit. The rationale for the provision of a flexible sleeve was discussed in Raju et al (1998). In essence, the stress state in the vicinity of the front wall is complicated and the flexible sleeve isolates the reinforcement from shear stress transfer in this problematic zone. A flexible sleeve was formed by machining off the ribs and encapsulating this length of steel in a triaxial membrane. High vacuum silicone grease applied to the interface between the strip and the membrane. Thus shear stress transfer was reduced to a minimum following the principles of free ends in triaxial testing. This system was protected from puncturing by a pair of greased rubber sheets. The pullout force was applied with a hydraulic actuator that can operate in either displacement or force controlled mode. The connection of the steel strip to the actuator was via a self-aligning articulation system. Reinforcement pullout was measured with a displacement transducer magnetic-mounted on top of the steel strip.

Testing Methodology

The soil from site was ripped in-situ and pretreated on site. Pretreatment involved spreading of the materials to a thickness of about 300mm with six passes of a 15-ton excavator. After mixing, this process was repeated and the materials delivered to the laboratory in sealed drums. In the laboratory, the moisture content of the soil was checked, and adjusted where necessary, prior to placement in the pullout box. The soil was compacted to a target dry density of 95% MDD (maximum dry density) with a percussion compactor in layers of approximately 33mm. The top half of the pullout box was removed while placing and compacting the lower half of the soil. Once the soil was compacted to mid-height, the reinforcement strip with the flexible sleeve was placed and aligned before placing the top half of the pullout box. The interface between the two halves of the pullout box was lined with a compressible material. Two aluminium packing strips were placed next to the reinforcement to prevent clamping of the reinforcement. Upon completion of the assembly, a preload pressure of 150 kPa was applied prior to reducing to the test pressures. This was to simulate compaction stress in wall construction. The compartments were then filled with water. A reticulation system was incorporated to ensure the compartments were always full of water. Pullout testing was conducted after completion of wetting.

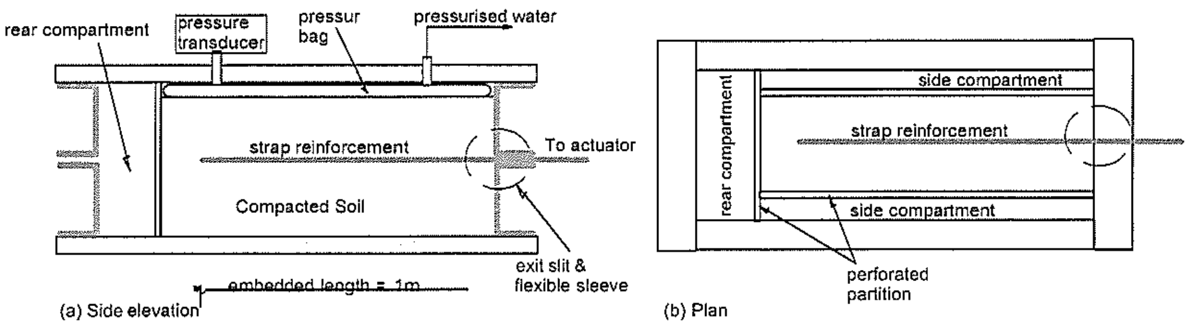


Figure 2. Pullout apparatus

Two series of pullout tests were conducted. The test in the main series, referred to as test P1 to P5a. were conducted after 24 hours of wetting. Each test in this series consisted of four stages as explained below:

1. Reinforcement pullout in a displacement controlled mode to a maximum actuator displacement of lOmm at a displacement speed of –0.05 mm/min.
2. The actuator control was then changed to a constant load that was approximately 85% of the maximum load recorded in the previous stage. This load was maintained overnight.
3. The actuator control was reset to displacement control and pulling continued until pullout was recorded or displacement limit was reached.
4. The system was unloaded and the test pressure was reduced to zero and pullout was continued in a displacement controlled mode. This last stage of pulling is referred to as the dummy pull in this paper.

A complete history of the above four stages of testing is presented in Fig. 3a.

A second test series were conducted with prolong soaking prior to the commencement of pulling. These tests, however, skipped stage 2 and stage 3.

TEST RESULTS

A typical load displacement curve is presented in Fig. 3b. In the first stage of pulling, the maximum load achieved is already close to pullout failure. Note that the pullout appeared to be initially negative. This is an aberration of measurement due to slight initial rotation of the steel strip in the vertical plane at the displacement transducer. The second stage is to verify that the creep characteristic of the system at high pullout force. As evident in Fig. 3b. the actuator could maintain the load at the prescribed value and the creep displacement recorded was minimal. The test results indicate negligible concern over creep failure. The maximum pullout force registered in the third stage is close to or slightly higher than that recorded in the first stage. Thus the test results do not indicate any deterioration due to one additional day of wetting plus previous loading history at high applied load.

The last stage, the dummy pull, is a measure of the correction (or errors) in the maximum pullout force. If the test condition is ideal, then the pullout force recorded in a dummy pull, F_{dum}, should be zero. A non-zero force can be attributed to resistance in the exit slit and apparent adhesion between the soil and the strip (as the soil is not 100% saturated). Hence, in the subsequent interpretation of pullout resistance, a corrected force F_{corr} given by the following equation should be used.

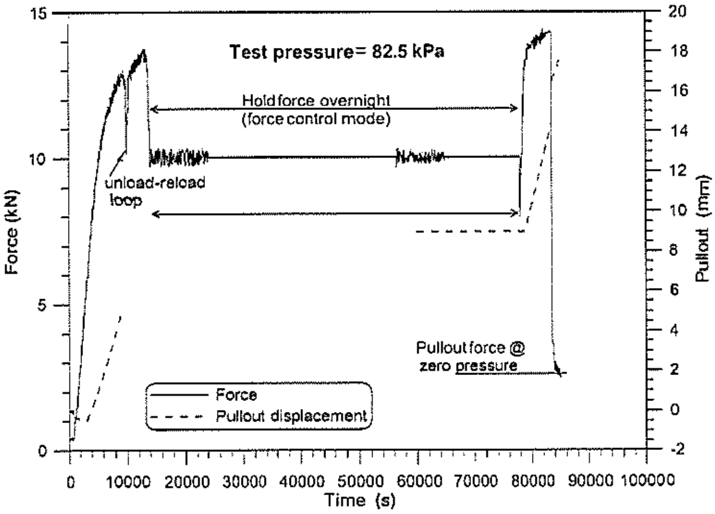
$$F_{corr} = F_{max} - F_{dum} \tag{1}$$

where F_{max} is the maximum pullout force recorded.

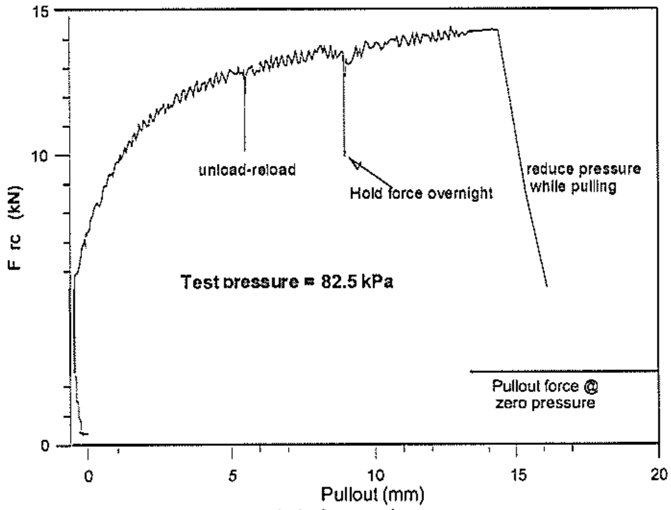
In simple terms, the pullout resistance, R_p. could be related to the normal stress at reinforcement level, σ_{vo}, by the following equation:

$$R_p = f.\sigma_{vo}.A_s \tag{2}$$

where f is the friction factor, often referred to as the apparent coefficient of interface friction; A_s is the surface area of the reinforcement in contact with soil.



(a) History plot



(b) Load deformation curve

Figure 3. Typical pullout test results.

The fiiction factor, f, is plotted against test pressure in Figure 4. Data points for the main test series were shown as open squares, whereas the two tests with prolong soaking were shown as filled squares. Test data for the main series, tests P1 to P5a, followed a clearly defined trend, with f increasing with reduction in test pressure.

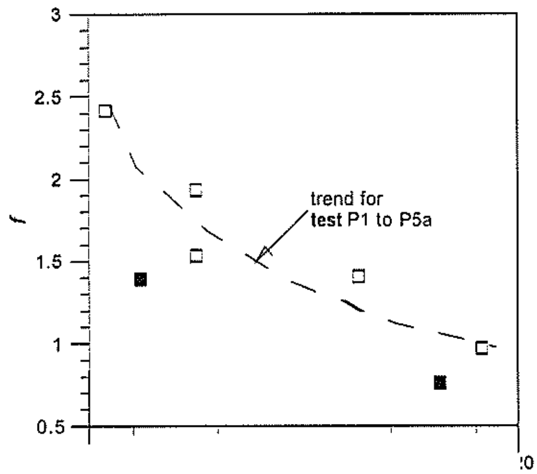


Figure 4. Friction factor versus test pressure.

However, the two data points corresponding to prolong wetting gave significantly lower friction factor. The lower friction factors achieved by these two tests were likely to be due to higher moisture content induced by prolong wetting. The higher moisture content led to lowering of soil suction. This, in turn, led to two consequences: reduction of apparent adhesion along the soil – reinforcement interface, and reduction in stiffness of surrounding soil. The effect of the former is likely to be partially compensated by the F_{dum} of Eqn (1). The later will reduce the effect of constrained dilatancy. However, the trend of higher friction factor at lower test pressure was still followed by the data points corresponding to prolong wetting. This implies constrained dilatancy was still playing a significant role. It is also pertinent to note that the test conducted after a week's prolong wetting with σ_{vo} = 32 kPa still gave a friction factor of 1.4, which was significantly and considerably higher than tanϕ = 0.965, where ϕ is the independently measured peak friction angle as discussed in a later section. The lowest friction factor was 0.76 for the test conducted at σ_{vo} = 100 kPa with prolong wetting, and this value was still higher than tan+,,, even with ϕ_{cv} in the range of 31 to 36'.

CONSTRAINED DILATANCY

The increase of *f* with reduction in test pressure cannot be explained simply by increase of friction angle. The relationship is better explained by the constrained dilatancy theory (Milligan and Tei 1998; Lo 1998, 2003) illustrated in Fig. 5 and as briefly described below.

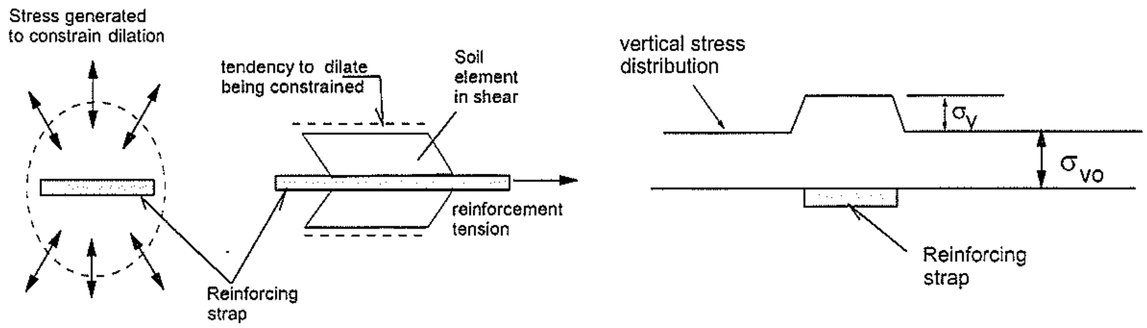


Figure 5. Constrained dilatancy

Soil in the vicinity of the reinforcement will be subject to intense shearing during reinforcement pullout, and shear band will form due to the rough ribbed surface. However, the soil particles at the interface are constrained from dilation by the surrounding soil. This generates additional local normal stress acting on the reinforcement surface. Thus the failure shear stress τ_f could be given by:

$$\tau_f = \sigma_n.\tan\phi = (\sigma_{vo} + \sigma_y).\tan\phi = \sigma_{vo}(1 + \sigma_y/\sigma_{vo}).\tan\phi \qquad (4)$$

where σ_{vo} = overburden pressure or test pressure; σ_y = increased normal stress due to shearing

Since σ_y is not reflected by a two dimensional stress analysis, its effect has to be reflected by an increased in *f* for ribbed steel strip. Therefore we have:

$$f = (1 + \sigma_y/\sigma_{vo}).\tan\phi \qquad (5)$$

It is clear from Eqn *(5)* that f will increase at low test pressure (or overburden stress). A simplified theory for predicting the increase of *f* with reduction in σ_{vo} is reported by Lo (2003).

To investigate the dilatant potential of this soil with a high fines content, a separate consolidated drained triaxial test was carried out. The result of a sample compacted to 1.95 g/cm^3 is presented in Figure 6. Both axial force and strain were measured using transducers placed inside the pressure chamber. A GDS digital controller was connected to the pore pressure line to achieve control of back pressure and reliable measurement of volume strain. The test was conducted at a relatively low effective confining stress or effective minor principal stress, σ'_3, of 77 kPa. For reinforcement pullout to be critical, the stress is usually low. Hence the results of this test is considered to be representative.

The q - ε_1 curves is presented in Fig. 6a, where $q = \sigma_1 - \sigma_3$ is the deviator stress, σ_1 is the major principal stress and ε_1 is the axial strain. A distinct peak failure was observed and the failure stress state gave a secant peak friction angle of 44^0. Although the sample was manifesting essentially uniform deformation at peak failure which occurred at ε_1 of about 2.5%, significant non-uniformity developed at ε_1 of about 7%, which was before the critical state was attained. The volume response, presented in Fig. 6b, clearly indicates a diltatant behaviour.

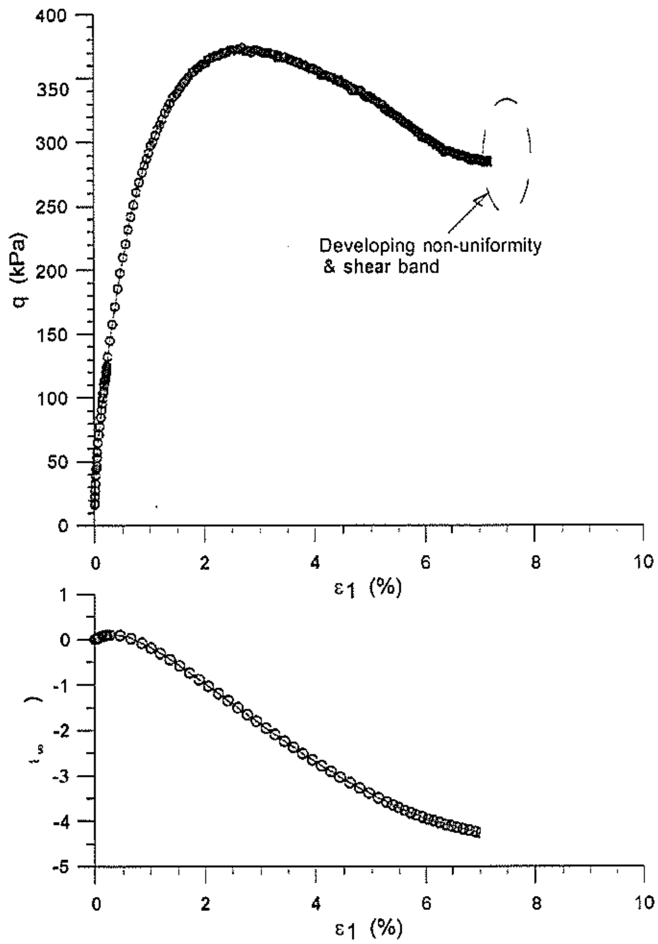


Figure 6. Triaxial test results. (a above; b below)

The dilatant behaviour during all stages of shearing in a triaxial test can be seen more clearly with an R-D plot presented in Fig. 7, where $R = \sigma'_1/\sigma'_3$, $D = 1 - d\varepsilon_{vol}/d\varepsilon_1$, ε_{vol} = volumetric strain, and prime indicates effective stress. D, also referred to as the dilatancy factor, was computed by central difference approximation using data sets recorded at close time interval. The data points followed closely Rowe's stress dilatancy equation (Rowe et al 1964) and this indicates essentially a "sand behaviour". The characteristic state, defined by the first occurrence of D = 1, occurs at a principal effective stress ratio of 3.10, and this stress ratio can be used to calculate a lower bound estimate of ϕ_{cv}. This leads to $\phi_{cv} > 31°$, which is considered to be reasonable. A maximum dilatancy factor of about 1.9 was attained at failure and this corresponds to a highly dilatant behaviour.

CONCLUSIONS

A large scale pullout test of a proprietary ribbed steel reinforcement embedded in a crushed sandstone with a high fines content was carried out. The test procedure by using a large scale pullout box is described and the results have been examined in detail. Despite the observation that the interface friction factor was affected by prolong soaking, for the material under examination, a significant inverse relationship between the apparent coefficient of interface friction and overburden pressure was unambiguously identified. This behaviour pattern could be explained by the constrained dilatancy theory, and that the dilatant behaviour of the material is verified by independent triaxial testing.

The pullout test results of the sandstone/reinforcement system is consistent with the well known behaviour of granular materials. The limited test results suggest that the amount of fines in the material may not necessarily be a critical factor in determining the pullout behaviour of the material, as commonly conceived. Other factors, such as the nature of the fines, may have more over-riding effects on the pullout characteristics. More research is required to establish this relationship.

Triaxial testing, when conducted with reliable measurements of both axial and volumetric strain, gives the dilatancy characteristics of the soil. This serves as an indicator for assessing the likely relative pullout resistance of various reinforcement soil combinations prior to commissioning expensive large scale pullout testing.

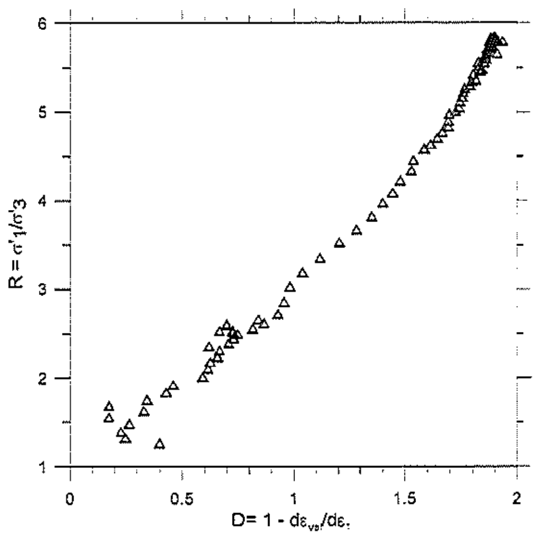


Figure 7. Stress dilatancy plot

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