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Pull-out Resistance Characteristics of Galvanised Steel Mesh Reinforcement Embedded in Silty Sand

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ABSTRACT

The dominant contribution to the pullout resistance of mesh type soil-reinforcement is provided by the bearing resistance on the transverse members. A range of equations has been proposed in the literature to predict this bearing resistance. However, these equations give highly divergent prediction. Furthermore, it is not clear whether the value of friction angle in these equations should be the peak value or at critical state. Some of the published experimental data suggest that the bearing resistance factor for the transverse bars is dependent on overburden stresses but the theoretical equations imply the contrary. In this paper, an attempt has been made to examine the pull-out resistance behaviour of steel mesh reinforcement embedded in fine grained crushed sandstone. Testing was conducted with a wide range of overburden pressures (ranging between 20 and 115 kPa). This enables us to examine the dependency of bearing resistance factor on test pressure. The soil was carefully characterised so that the different equations for predicting pullout resistance can be objectively evaluated.

Keywords: steel mesh reinforcement, pull-out test, pull-out resistance, bearing resistance factor.

1 INTRODUCTION

Reinforced soil walls (RSW) are widely used as an economical alternative for reinforced concrete or gravity type retaining structures. In general, these structures consist of three basic components: facing element, reinforcing element and select fill materials within which reinforcing elements are embedded. For the reinforcement to be able to develop its full tensile capacity in stabilising the select fill into a coherent mass, its pullout resistance has to be adequate. In general, pullout resistance depends on reinforcement type, properties of surrounding soil, overburden stress and embedded length in the passive zone (where the soil is anchoring the reinforcement).

For mesh type soil-reinforcements, the pull-out resistance, R_{pull} is affected by both frictional resistance (F_f) developed along the longitudinal members and the bearing resistance (F_b) of soil against transverse members of the mesh (Peterson and Anderson 1980; Bergado et al. 1993). That is,

$$R_{pull}=F_f+F_b \tag{1}$$

For inextensible mesh reinforcement, F_b contributes about 90% of the pull-out resistance. Thus the main focus of this paper will be on F_b. Assuming a bearing capacity type failure,

$$F_b=N_q\sigma_n A_b \tag{2}$$

where, N_q is the bearing resistance factor, σ_n is the stress relevant to the equation for N_q, and A_b is the projected bearing area. Equation (2) is commonly used to calculate F_b by assuming $\sigma_n=\sigma_{vo}$ where, σ_{vo} is the initial overburden stress at the reinforcement level. Thus equation (2) yields,

$$F_b=N_q\sigma_{vo}A_b \tag{3}$$

After Peterson and Anderson (1980) the expression of N_q associated with general shear failure mode can be expressed as

$$N_q = tan^2(\frac{\pi}{4}+\frac{\phi}{2})e^{\pi\, tan\phi} \tag{4}$$

where, ϕ is the friction angle of the soil. The above equation can be derived based on a horizontal bearing capacity equation applied in the horizontal direction and thus σ_n can be considered to be the horizontal stress (σ_h) in the vicinity of the transverse members.

Jewell et al. (1984) assumed a punching shear failure mode where, σ_n was considered to be equal to the vertical stress (σ_v) at the boundary of the punching zone and the formulation of N_q becomes as below,

$$N_q = tan(\frac{\pi}{4} + \frac{\phi}{2})e^{(\frac{\pi}{2}+\phi)tan\phi} \tag{5}$$

Matsui et al. (1996) proposed an alternative equation for N_q which can be expressed as follows,

$$N_q = e^{\pi\, tan\phi} tan(\frac{\pi}{4} + \frac{\phi}{2})\left[cos\left\{(\frac{\pi}{4} - \frac{\phi}{2})\right\} + (1 - sin\phi)sin\left\{(\frac{\pi}{4} - \frac{\phi}{2})\right\}\right] \tag{6}$$

In addition to the difference in assumed mechanisms, the above three equations also imply further difference in assumptions when used in conjunction with equation (3) intrinsically assume σ_n=σ_{vo}.

A number of published works compared the theoretical predictions with experimental results using sandy soil which suggested that most of the test data falls within the range of the theoretical predictions (Palmeira and Milligan 1989; Matsui et al. 1996). However, this is not particularly helpful for design because the calculated values of N_q from the three different equations differ significantly. For example, for ϕ=35^0, N_q varies from 8.85 to 33.3 which is a very large range. Furthermore, in equations (4), (5) and (6), it is not clear whether ϕ should be taken as the peak friction angle (ϕ_{peak}) or the friction angle at critical state (ϕ_{cv}), and this introduced approximately an extra 50% difference in the predicted N_q value.

Matsui et al. (1996) suggested that the test result agree best with prediction from equation (6) but it was not clarified whether ϕ_{peak} or ϕ_{cv} was used in the comparison. Furthermore Matsui's comparison implies N_q is independent of test pressure but some published literature suggested that N_q depends on overburden stress (Christopher et al. 1989).

Moreover, the pullout resistance measured under low overburden stress is of small value and can be significantly affected by the friction at the exit slit (Lo 1998). Thus one may also question to what extent this will "artificially" contribute to higher measured N_q factor at low overburden stress.

The objective of this paper is to investigate whether the friction angle relevant to bearing resistance factor should be taken as the peak value or critical state friction angle and then compare the test results with theoretical predictions. The influence of friction at the exit slit was allowed for using special dummy pullout tests.

2 TESTED SOIL

The soil used in this study was sourced from a construction site in Blue Mountains, Sydney. The soil is classified as well graded silty sand and contains about 17% of non-plastic fines. The grain size distribution curve for the soil is presented in Figure 1. The maximum dry density (MDD) and the optimum moisture content (OMC) were determined from standard proctor compaction test and were found to be 1.98 ton/m^3 and 9.58% respectively. Consolidated drained triaxial tests were conducted to measure the friction angle at both peak and critical state, and also to find out the dilatancy characteristics of the soil. The triaxial test specimens were prepared at the same target dry density as for both large scale pull-out tests, i.e. 95% of MDD. The measured ϕ_{cv} was found to be 36.5^0. The measured ϕ_{peak} was in the range of 41.8^0 to 43.2^0 for the range of confining stress 50 to 200 kPa, showing a trend where ϕ_{peak} slightly decreased with the increase of confining stress. Such a detailed characterization of the soil will ensure that the F_b can be calculated objectively using independently measured soil parameters.

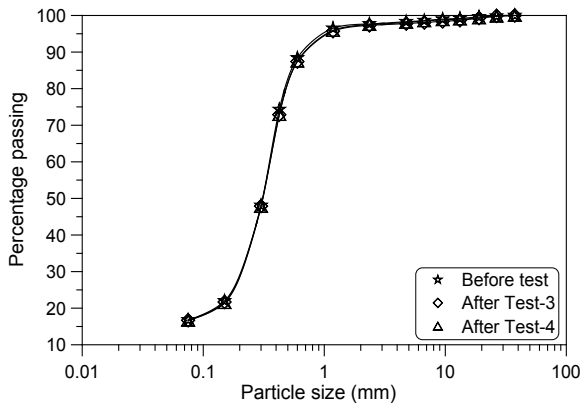

Figure 1. Grain size distribution curve of tested soil

3 TEST CONFIGURATION

3.1 Pull-out testing arrangement

A pull-out box 2 m long, 1.1 m wide and 0.4 m height was used for this study. Perforated internal partitions were used to adjust the effective size (1700 mm X 700 mm X 400 mm) of the box depending on the type and size of the reinforcing elements and test conditions. To measure horizontal stresses of the soil mass at reinforcement level, the pull-out box was instrumented with four earth pressure cells at the mid height and was placed horizontally at the side walls.

Partitions were used to investigate and to store water for wetting of the soil mass. Geotextile liner was used to prevent loss of soil through the perforations. The pull-out box consisted of two separate halves which was helpful for the placement and alignment of the reinforcement embedded in the soil mass. The plan and elevation of the pull-out box with reinforcement are shown in Figure 2a and 2b.

To prevent jamming and development of shear stress at the pulling end of the reinforcement, soft 180 mm PVC tubes were sleeved over the longitudinal bars. Lubricants were used at the inside surface and a latex membrane with silicon grease was wrapped on the outside surface of each PVC tube. About 100 mm of the 180 mm sleeves were inside the box. A pair of rubber sheets lubricated with grease were used between the front wall and the sleeve to prevent puncturing of the sleeves by the soil which followed the principle of flexible sleeve as discussed by Raju et al. (1998). The remaining 80mm of the sleeves were passed through the exit hole to prevent the jamming force. Lubricated rubber sheets were used to prevent soil loss at the pulling end. To prevent jamming of the top half of the box onto the reinforcement, 13 mm spacers were used in between the top and bottom half of the pull-out box.

The ends of the longitudinal bars were attached to the loading arrangement using nuts to ensure good alignment between the reinforcement and the loading train. Two linear variable differential transformers (LVDTs) were mounted directly on the loading clamp to measure displacements of the reinforcement during the test. This will ensure that the measured pullout displacement will not be affected by compliance in the loading train. The force-displacement curve indicates the displacement at failure which is an indication to identify whether ϕ_{peak} or ϕ_{cv} should be used for the equations. A data acquisition system was used to collect and store data from the measuring devices.

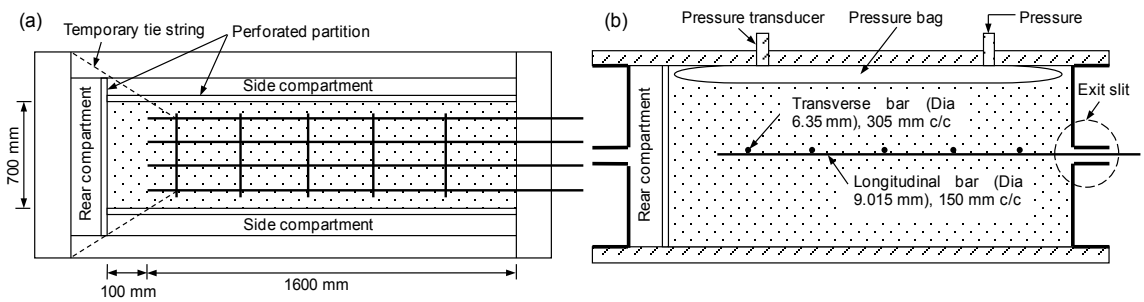

Figure 2. Testing arrangement of pull-out box; (a) plan view and (b) elevation

3.2 Pull-out testing procedure

The test material was collected from the field and was stored in big air tight containers. Required amount of water was added during specimen preparation to achieve the desired moisture content. After wrapping the sides and bottom of the box with geotextiles, the pre-weighted soil sample was placed and compacted in 11 layers with a percussion compactor while maintaining the compacted thickness in each layer so that targeted dry density (95% of MDD) could be achieved. After filling the bottom half of the box by compacted soil, the reinforcement was placed. The reinforcement was tied temporarily by two strings to keep it in place (Figure 2a) and then the top half of the box was filled with compacted soil and a geotextile sheet was placed at the top. To apply vertical pressure to the soil mass, a pressure bag was placed at the top surface and the top lid of the pull-out box was put back (Figure 2b). To apply pressure, water was pumped into the pressure bag and an initial magnitude of 150 kPa was applied and kept for about 60 minutes. This was done to replicate the effect of compaction performed in a construction site using a large roller. The pressure was then reduced to the desired test pressure. The compacted soil sample was then soaked for two days prior to pull-out test to replicate the weakened state of soil due to prolonged wetting. This wetting process increased soil moisture content by about 2% over the optimum level. Pull-out force in a displacement control mode was applied by a hydraulic actuator with a displacement rate of 0.065 mm/minute. Upon reaching 20 mm displacement of the reinforcement, the pull-out force was then reduced to about 75% of the maximum value attained. This force, denoted as F_{hold}, was maintained overnight by the actuator in load control mode. The purpose of the hold force was to confirm the slow rate of the actuator and to observe the occurrence of any creep when the reinforcement is subjected to high pull-out force.

4 DUMMY PULL-OUT TESTING

Although the soil mass was soaked prior to the application of pull-out force, some apparent cohesion may still existed. There could be some resistance present at the exit slit. These extra forces had to be deducted from the measured maximum force (F_{max}) when calculating R_{pull}. It is also noted that these forces may also vary between tests. Therefore, a dummy pull-out at zero overburden pressure was conducted as the last phase of every test at the same moisture content and displacement rate of the respective pull-out tests. The measured resistance in a dummy test was the extra force denoted as P_{dum}. For an ideal test condition, P_{dum} is supposed be zero (Lo 2009). Therefore, any measured non-zero P_{dum} was the correction to be applied to the F_{max} to calculate R_{pull} by using the following equation,

$$R_{pull}=F_{max}-P_{dum} \quad (7)$$

5 TEST RESULTS

A total of seven pull-out tests were conducted with a wide range of overburden pressure ranging between 20 and 115 kPa. The summary results of all pull-out tests are presented in Table 1. The force-displacement responses obtained from all the tests are presented in Figure 3a. The pullout displacement is based on the LVDT readings mounted at the clamp and thus compliance of the loading train (if based on actuator travel) was automatically excluded. As stated earlier, a hold force was applied and was maintained overnight using the load control mode of the actuator after reaching the peak strength for each test. The measured displacement throughout the hold period of each pull-out test was negligible. This suggests that the rate of pull-out was slow enough and the creep amount was insignificant.

It is evident from Figure 3a that, F_{max} was mobilised at a displacement of the same magnitude of bearing dimension (6.35 mm) of transverse bars for the overburden pressure below 80 kPa and the pull-out displacements were higher at higher overburden pressures (above 80 kPa). Thus, the ratio of pull-out displacement to the bearing dimension of transverse bar at failure was very high which indicate very high amount of strain in soil mass. Thus the friction angle relevant to calculating F_b should be taken as the critical state friction angle of soil, which is independent of confining stress.

The recorded values of P_{dum} from all dummy pull-out tests are also summarised in Table 1. The average value of P_{dum} was 6.88 kN with a standard deviation of 0.97 kN. The values of R_{pull} for all the tests were obtained by using equation (7) which are also summarised in table 1.

Table 1: Summary of pull-out tests result and bearing resistance factors

Test	Applied pressure kPa	F_{max} kN	P_{dum} kN	R_{pull} kN	σ_{vo} kPa	F_f kN	F_b kN	N_q (without correction)	N_q (with correction)
1	56	41.19	7.50	33.69	59.1	3.90	29.79	42.21	30.53
2	73	46.80	5.96	40.84	76.1	5.02	35.82	37.25	28.51
3	86	47.80	6.12	41.68	89.1	5.88	35.80	32.49	24.34
4	20	21.78	5.65	16.13	23.1	1.52	14.61	57.11	38.30
5	36	29.20	8.30	20.9	39.1	2.58	18.32	45.23	28.38
6	115	59.72	7.41	52.31	118.1	7.79	44.52	30.63	22.83
7	23	21.50	7.24	14.26	26.1	1.72	12.54	49.89	29.10

A number of sieve analyses were conducted before and after tests to investigate whether particle crushing during the pull-out test. The grading curves of the tested materials around the reinforcement are also plotted in Figure 1. Evidently, particle crushing due to pull-out test was not significant, and thus the soil used is not anything unusual.

Four independent pressure cells were used to measure the horizontal stresses at reinforcement level, σ_h, as this is an intrinsically noisy parameter to measure. Three values (maximum, minimum and average) were taken to represent the horizontal stress for each test. The scatter of the measured σ_h, defined as half the difference between the maximum and minimum value for each test, is in the range of 6.07 to 14.37 kPa. Despite this scatter and that σ_h changes during pullout testing, it is clear that $\sigma_h<\sigma_{vo}$, and $\sigma_{ho}<\sigma_{vo}$, where σ_{ho} is the initial horizontal stress at reinforcement level. The earth pressure coefficient at reinforcement level, *K,* defined *as* σ_h over σ_{vo} was also calculated. This gives three values of *K* (corresponding to maximum, minimum and average) for each test. Due to page limitation, detail plots of *K* cannot be provided. However, it is evident that *K* decrease from 0.90 at σ_{vo} of 20 kPa to 0.31 at σ_{vo} of 115 kPa.

6 BEARING RESISTANCE FACTOR

The frictional resistance of mesh reinforcement along longitudinal bars (F_f) can be obtained as

$$F_f=A_s\sigma_{vo}\tan\delta \tag{8}$$

where, A_s is the frictional area of the reinforcement and δ is the friction angle between soil-reinforcement interface. For sand and silt with smooth metallic surfaces the value of δ/ϕ typically ranges from 0.4 to 0.79 (Potyondy 1961). An average value of δ/ϕ=0.55 (i.e. $\delta=20^0$) was taken for this study. Thus the value of F_b can be obtained as

$$F_b=R_{pull}-F_f \tag{9}$$

The value of σ_{vo} is taken as 3.1 kPa higher than the applied test pressure at the top of the soil mass to account for the weight of soil in the upper half of the test box. The values of N_q at different test pressure condition calculated using equation (3) are also presented in Table 1. The plot of N_q versus σ_{vo} is presented in Figure 3b. To assess the effect of correcting for P_{dum}, two calculations for N_q are presented in Figure 3b, with and without correction. It is evident from Figure 3b that, while considering F_{max} (without correction), N_q decreases from about 57 to 30 with the increase of overburden pressure from 20 to 115 kPa. However, when correction was considered, N_q drops from 38 to 23 at the same range of overburden pressure. It is not clear whether the reduction in N_q at higher pressure (beyond 80 kPa) was due to the observed bending of the transverse bars.

For $\phi=\phi_{cv}=36.5^0$, the values of N_q of the tested material obtained using equations (4), (5) and (6) are 40.24, 10.16 and 21.81 respectively which are shown in Figure 3b using dotted lines. It is also evident from Figure 3b that, equation (4) over-predicts significantly whereas the equation (5) under-predicts considerably. Among those, the value of N_q obtained from the equation (6) proposed by Matsui et al. (1996) shows the best overall match with test results considering corrected bearing resistance. However, the influence of σ_{vo} on N_q cannot be reflected as ϕ_{cv} is independent of stress level.

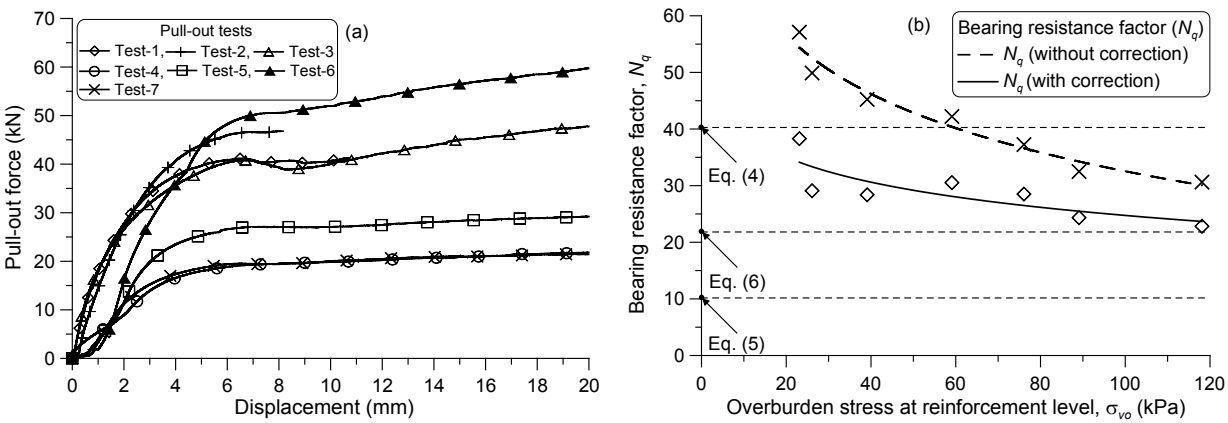

Figure 3. Pull-out test results; (a) force-displacement responses (b) N_q versus σ_{vo} plot

The equation for N_q proposed by Peterson and Anderson (1980) is essentially a bearing capacity equation for shallow footing applied in horizontal direction where σ_n is assumed to be equal to σ_h. Thus, replacing of σ_n by σ_{vo} in equation (2) is theoretically problematic. To achieve more realistic values of N_q, equation (4) can be re-written based on horizontal footing with $\sigma_n=\sigma_h=K\sigma_{vo}$, where K is roughly determined from test results which depends on σ_{vo}. Therefore N_q of equation (4) is modified by an extra term as follows

$$N_q = K\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)e^{\pi\tan\phi} \tag{10}$$

7 CONCLUSION

The pull-out resistance behaviour of galvanised steel mesh reinforcement embedded in silty sand was investigated within a wide range of overburden pressure. From this study, the following main conclusions can be drawn:

1. The ratio of pull-out displacement to the bearing dimension of transverse bar at failure was very high. Thus, the friction angle relevant to the equations of N_q should be taken as the friction angle of soil at critical state.
2. Considering corrected bearing resistance the influence of test pressure on N_q was less than that obtained while considering maximum recorded force without correction.
3. Equation (4) or equation (5) or equation (6) cannot adequately predict the pull-out test results.
4. The horizontal stress was found to be significantly less than the vertical stress at reinforcement level. Thus, replacing σ_n by σ_{vo} in the bearing capacity equation is theoretically problematic and may induce error in calculations.

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