Response of fiber reinforced polymer reinforced soil nails under pullout loads

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ABSTRACT

The use of fiber reinforced polymer (FRP) materials in civil engineering structures is relatively new. FRP reinforcing bars are starting to be used in lieu of conventional steel reinforcing bars where improved corrosion resistance is required in soil nailed reinforced earth retaining structures. Presently, there is only a relatively small body of knowledge on numerical modeling techniques and the response of FRP reinforcing nails under load. This paper provides an overview of research completed to date on both steel and FRP reinforced soil nails, and an outline of planned future research.

KEY WORDS: Soil nail; fiber reinforced polymer; glass fiber reinforced polymer; strain gauge; pullout capacity.

INTRODUCTION

Traditionally, soil nail earth retaining structures use steel reinforcing bars. Where steel is used, some standards and guidelines require up to three forms of corrosion protection. This typically involves hot dipped galvanizing the steel bar, an enveloping plastic sheath and a grouted annulus enveloping the bar. Advances in the development of fiber composites has resulted in the use of fiber reinforced polymer (FRP) reinforcing bars in lieu of steel bars for civil engineering structures.

FRP composites are a composition of fiber bound within a resin. FRP is an anisotropic material that is characterized by a high tensile strength in the direction of the reinforcing fibers. Fibers used in composites can be categorized based on their molecular structure using three groups: (1) polymeric; (2) carbon; and (3) inorganic. These fibers are typically set in an epoxy, polyester or vinyl-ester resin.

A key advantage of FRP materials is their high stiffness-to-weight ratio when compared to steel, being approximately 10 to 15 times higher. However, this advantage is offset by several shortcomings in comparison to steel. Firstly, the modulus of elasticity of FRP reinforcing bar is approximately four times less, resulting in larger displacements under pullout loads (Zhu, Yin et al., 2010). FRP also has a much lower shear modulus that can result in a lower ultimate shear resistance at a slip surface in a reinforced soil slope. Furthermore, when subjected to high tensile loads, FRP generally exhibits significant time-dependent elongation. Finally, FRP has a brittle failure mode.

SOIL NAIL STRUCTURES

Soil nailing became recognized as a support system for underground excavations in the 1960s where passive rock bolts together with a reinforced concrete facing provided a ground support system. Rabcewicz (1964) documented it as the New Austrian Tunneling Method. Since then, soil nailing has been commonly used as an earth retaining and slope stabilization technique. This technique is particularly suited to top-down excavations where vertical or near-vertical excavation faces are required.

Typically, soil nails are constructed using steel reinforcing bars up to 40mm in diameter, installed into a predrilled hole in the earth that is to be stabilized or retained. After the bars have been installed, the void around the annulus of the nail resulting from predrilling is filled with a cement-grout. The cement-grout transfers stress from the ground to the nail and inhibits corrosion of steel reinforcing bar.

Current Standards

Authorities worldwide have published various codes and guidelines dedicated to soil nailed retaining structures, including: British Standard BS-8006 (British Standards Institution, 2011); French National Research Project on Soil Nailing (Schlosser, 1993); U.S. Federal Highway Administration report on soil nail walls (Lazarte, Elias et al., 2003); and Geoguide 7 - Hong Kong Guide to Soil Nail Design and Construction (GEOGUIDE-7, 2008). Of these, only BS-8006 and Geoguide 7 make specific reference to the use of FRP soil nails.

Published Research Overview

Traditional soil nail research has typically used numerical modelling correlated with field observations. Numerical modelling methods such as the Finite Element Method (FEM) and Finite Difference Method (FDM) have been used by researchers over the past several decades to predict the behavior of soil nail structures. For example, researchers such as Sawicki, Lesniewska et al. (1988), Briaud and Lim (1997) and Zhang (1999) have all used numerical modelling to predict the behavior of soil nail structures and compared their results with field-scale experiments.

FRP soil nail research into its behavior under load appears to have only commenced in mid-2000. An overview of notable FRP soil nail research that compares numerical modelling with test results derived from laboratory and full-scale field tests follows.
A pilot study on hollow FRP nails was undertaken by Cheng, Choi et al. (2009), incorporating extensive laboratory and field tests combined with numerical modeling. This early research identified the feasibility of using hollow FRP soil nails in lieu of traditional steel reinforcing bar. The ultimate pullout capacity of FRP nails (four in total) was assessed with strain under load and measured by strain gauges. The researchers also demonstrated that FDM could be used to create a three-dimensional numerical model that was useful in predicting the behavior of FRP soil nails.

Zhou, Yin et al. (2011) developed a three-dimensional finite element model to simulate the behavior of a fiber reinforced plastic soil nail in a pullout box under different overburden and grouting pressures. The FEM results were compared to soil-nails embedded in decomposed granite modelled using the modified Drucker-Prager/Cap model. The soil-nail interface was represented by the Coulomb friction model. This research found soil stress variations surrounding the soil nail during drilling, grouting, saturation, and pullout were simulated by the finite element modelling when compared with available field test data.

Zhu, Yin et al. (2010) used optic fiber with Bragg Grating sensors and strain gauges to measure tensile strains in both hollow FRP and steel soil nails at different locations. A load cell and linear displacement transformers were used to measure pullout force and displacements at the nail head. The research identified certain differences between steel and FRP reinforced nails in relation to load transfer mechanisms. It also developed a hyperbolic shear stress-strain relationship that can be used to describe the behavior of hollow FRP nails.

**PROPOSED RESEARCH**

The use of FRP in soil nail structures in Australia has been limited. Government authorities generally exercise caution before broadly adopting new technologies, such as the use of FRP in soil nails. Therefore, this new research will supplement the relatively small body of existing research on FRP soil nail behavior under load.

This proposed research is unique in that it assesses the response under load of both FRP and steel nails side-by-side under exactly the same field conditions. The research will also include numerical modelling of the soil nail field tests using FEM to predict their response under pullout loads.

**Test Site**

*The site* for the proposed field tests is located in Yalata, Gold Coast City, Queensland, Australia. The natural subsurface conditions encountered during the site investigation generally comprised meta-sandstone or meta-siltstone rock. This is consistent with the published regional geological mapping of the area by Whitaker and Green (1980).

*Soil nail pullout testing* is proposed to be undertaken as per the general arrangement shown in Fig. 1. Soil nail pullout tests comprise nails installed vertically that are loaded under tension until failure. The nails will only be installed in the residual clay layer in order to reduce the number of variables. Strain gauges will be located on each reinforcing bar and within the grout annulus of the nail.

Ground water was not encountered during drilling operations, which is consistent with the surrounding topography and elevation of the site. A total of four boreholes were sunk from which undisturbed samples were recovered and laboratory testing undertaken.

**Geotechnical Characterization of the Test Site**

*Preliminary geotechnical investigation* works on the site have been undertaken utilizing undisturbed sampling and Cone Penetrometer Tests (CPTs). Subsurface conditions were found to comprise three distinct layers, described as: engineered fill; residual clay (approx. 2.0 meters deep); and meta-siltstone rock.

*Laboratory testing* of undisturbed samples from two boreholes (denoted as BH5 and BH6) was undertaken to determine the engineering characteristics of the residual clay in which the soil nails will be located. Tests comprised: Atterberg Limit; particle size distribution; oedometer; and triaxial tests.

Atterberg Limit tests indicate that the clay layer in which the soil nails will be located is a Silty Sandy Clay material with medium to high plasticity (Fig. 1). The average liquid limit (LL) and plasticity index (PI) is 45 percent and 29 percent respectively, indicating medium plasticity.

**Fig. 1: Soil nail pullout test general arrangement**

A total of three Consolidated-Undrained (CU) single stage triaxial tests at different confining pressures were undertaken on undisturbed clay samples recovered from boreholes BH5 and BH6. Oedometer tests were also completed on samples from the same boreholes. Undrained soil strength parameters were calculated (Table 1) from these tests that can be used in the Mohr-Coulomb constitutive model for FEM modelling.
Table 1: Calculated Mohr-Coulomb model parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of internal friction, $\varphi'$</td>
<td>25°</td>
</tr>
<tr>
<td>Cohesion, $c'$</td>
<td>12.5 kPa</td>
</tr>
<tr>
<td>Modulus of elasticity, $E_{50}$</td>
<td>10.9 MPa</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.5</td>
</tr>
<tr>
<td>Angle of dilatancy, $\psi$</td>
<td>0°</td>
</tr>
<tr>
<td>Permeability, $k$</td>
<td>$7.0 \times 10^{-10}$ m/s</td>
</tr>
</tbody>
</table>

Oedometer tests also provide insight into the stress history of samples taken from the site. They allow for the consolidation ratio ($OCR$) to be calculated, expressed as (Lambe and Whitman, 1979):

$$OCR = \frac{\sigma_{vm}}{\sigma_{vo}}$$  \hspace{1cm} (1)

where,

\(\sigma_{vm}\) = Maximum past consolidation stress

\(\sigma_{vo}\) = Current in-situ vertical stress.

The maximum past consolidation stress was estimated using a plot of void ratio, and log of effective stress. From that plot, the maximum past stress condition was estimated using the graphical method proposed by Casagrande (1936). The $OCR$ for samples from boreholes BH5 and BH6 were calculated as 2.5 and 2.0 respectively, indicating a lightly consolidated clay.

Approximations for undrained shear strength $C_u$ can made using relationship proposed by Vermeer (1982):

$$C_u = c' \cos \varphi' + \frac{1}{2}(K_o + 1) \sigma_{vo} \sin \varphi'$$ \hspace{1cm} (2)

Ladd (1991) also proposes another relationship:

$$C_u = 0.22\sigma_{vo} OCR^{0.8}$$ \hspace{1cm} (3)

With reference to the lateral earth coefficient $K_o$ in Eq. 2, Mayne and Kulhawy (1982) proposed the following relationship for an over consolidated clay:

$$K_o = (1 - \sin \varphi') OCR \sin \varphi'$$ \hspace{1cm} (4)

The shear strength ratio (Eq. 5) proposed by Jamiolkowski (1985) provides a useful reference value for clays with a plasticity index $PI < 60$ percent.

$$\frac{C_u}{\sigma_{vo}} = 0.23 \pm 0.04$$ \hspace{1cm} (5)

From Eqs 2-4, undrained shear strength values were calculated and are presented in Table 2. The shear strength ratio corresponding with estimated values for $C_u$ are also provided in Table 2. These values generally fall within the range prescribed by Eq. 5.

Table 2: Estimate of undrained shear strength $C_u$ from laboratory tests

<table>
<thead>
<tr>
<th>Borehole</th>
<th>$C_u$ (Eq. 2)</th>
<th>Shear Ratio (Eq. 5)</th>
<th>$C_u$ (Eq. 3)</th>
<th>Shear Ratio (Eq. 5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH5</td>
<td>33 kPa</td>
<td>0.28</td>
<td>26 kPa</td>
<td>0.22</td>
</tr>
<tr>
<td>BH6</td>
<td>38 kPa</td>
<td>0.22</td>
<td>35 kPa</td>
<td>0.21</td>
</tr>
</tbody>
</table>

**In situ testing** on the site using CPTs were used to determine the subsurface soil profile and corresponding strength characteristics. As documented by many researchers, such as Robertson and Cabal (2012), Eid and Stark (1998) and (Kim, Prezzi et al., 2006), the relationship between undrained shear strength $C_u$ and cone resistance $q_c$ is based on the relationship for bearing capacity theory originally proposed by Terzaghi (1943), expressed as:

$$C_u = q_c - \sigma_v$$ \hspace{1cm} (6)

where,

$C_u$ = Undrained shear strength

$q_c$ = Total cone resistance (tip resistance corrected for pore pressure effects)

$\sigma_v$ = Total mean, horizontal or vertical in-situ stress (depending on theory being considered) at the depth of penetration

$N_{kt}$ = Empirical cone factor.

From Eq. 6 it is clear that an accurate estimation of $C_u$ is limited by the adopted empirical value $N_{kt}$. Significant empirical research (Aas, Lacasse et al. (1986), Baligh, Ladd et al. (1980), Stark and Juhrend (1989), O’Riordan, Davies et al. (1982), Lunne and Kleven (1981), and La Rochelle, Zebdi et al. (1988)) indicates a range for possible values of $N_{kt}$ of between 10 and 20 in clays.

Calculated undrained shear values from CPT data at boreholes BH5 and BH6 is presented in Table 3 using an upper $N_{kt}$ value of 19. These values correspond to the same depth of values presented in Table 2. The shear ratio corresponding to the estimated $C_u$ is also provided in Table 3, which fall(s) well outside the range prescribed by Eq. 5. It is unclear why there is such a large discrepancy between the estimated values of $C_u$ derived from laboratory (Table 2) and in situ (Table 3) tests. Therefore, it is proposed to undertake additional laboratory and in situ testing to clarify this discrepancy.

Table 3: Estimate of undrained shear strength $C_u$ from CPTs

<table>
<thead>
<tr>
<th>Borehole</th>
<th>$C_u$ (Eq. 6)</th>
<th>Shear Ratio (Eq. 5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH5</td>
<td>222 kPa</td>
<td>1.30</td>
</tr>
<tr>
<td>BH6</td>
<td>168 kPa</td>
<td>1.43</td>
</tr>
</tbody>
</table>

**Monitoring Instrumentation**

Before commencing full-scale field tests, laboratory testing was undertaken to verify that the strain gauges – attached to the FRP and steel reinforcing bars and set in the grout – accurately and reliably measured strains when the soil nail was subjected to a tensile load (pullout test). The general arrangement for the laboratory specimen used to test the strain gauges is shown in Fig. 2. The form of the specimens was designed to replicate the load experienced by a soil nail tested under a uniaxial tensile load to validate strain gauge measurements.

Six specimens were constructed in total: three with steel reinforcing and three with FRP reinforcing. One of the FRP reinforced specimens was found to be defective and was therefore not tested.

**Testing of specimens** was undertaken using a universal testing machine (MTS) until failure (Fig. 3). Strains were measured from the four strain gauges: two on the reinforcing bar and two in the grout. Data from the strain gauges was acquired using LabVIEW software combined with a load cell on the MTS.
Due to the unique nature of this test, there is no standard loading rate. Therefore, a loading rate based on pullout failure of the reinforcing bar was adopted using ASTM A944-10 (2015): Standard Test Method for Comparing Bond Strength of Steel Reinforcing Bars to Concrete Using Beam-End Specimens (ASTM International, 2015). Based on this standard, a loading rate of 0.350 mm per min. was adopted.

The cement grout response to a uniaxial tensile loading was determined using grout strain gauges cast into the neat cement grout. The grout specimens had a water-cement ratio of approximately 0.25. Representative samples of grout were taken for every test specimen and tested for compressive strength after curing. The average compressive strength was 34.8 MPa, and a modulus of elasticity $E_c$ of 34.6 GPa was calculated from the stress-strain curve of the compressive tests.

Using strain data from the strain gauges embedded in the grout (Fig. 4) and the stress derived from the load cell, stress-strain curves have been plotted for individual strain gauges and are presented in Fig. 4.

The steel reinforcing bar response to a uniaxial tensile loading was determined using diametrically opposed electrical strain gauges. These were attached to the reinforcing bar using a Cyanoacrylate adhesive. Before strain gauges were attached, the surface was prepared by grinding back the reinforcing bar to a flat surface.

The linear portion of the theoretical stress-strain curve has been plotted using a value ($E_c$) of 34.6 GPa (the mean of all samples). The mean absolute percentage error (MAPE) was used to determine the accuracy of test results and corresponding theoretical values. The MAPE provides a simple description the accuracy of measured ($A_t$) and forecast ($F_t$) values defined as:

$$\text{MAPE} = \frac{100}{n} \sum_{t=1}^{n} \left| \frac{A_t - F_t}{A_t} \right|$$

The calculated mean absolute percentage error (MAPE) relative to the theoretical value for all strain gauges ranged from 6 to 59 percent, which is considered unacceptable. The lowest MAPE values were recorded on the specimens with the steel reinforcing bar, but for only one of the two strain gauges in each grout specimen.
Using strain data acquired from the strain gauges attached to the reinforcing bar, the load response of the bar is presented graphically in Fig. 5. Stress values have been derived from data acquired from the MTS load cell and the reinforcing bar’s nominal cross-sectional area provided by the manufacturer (Reid, 2008). Using the steel bar’s nominal modulus of elasticity \( (E_s = 200 \text{ GPa}) \), the theoretical stress-strain load response is plotted on the graph provided in Fig. 5. The calculated MAPE relative to the theoretical value for all strain gauges ranged from 5 to 42 percent, noting that gauges on specimens 2 and 3 (2-SG 2, 3-SG1 and 3-SG 2) all recorded MAPE values between 5 and 8 percent, which is considered acceptable. One of the strain gauges on specimen 2 (2-SG 1) was found to be defective and was therefore not presented in the results.

Of the three specimens constructed with the FRP reinforcing bar, one specimen was found to be defective and could not be tested. In addition, one of the strain gauges on specimen 9 (strain gauge 9-SG 1) was also defective. This resulted in only three strain gauge readings.

The FRP reinforcing bar response to a uniaxial tensile loading was also determined using diametrically opposed electrical strain gauges adhered to the bar with a Cyanoacrylate adhesive. Before strain gauges were attached, the surface was prepared by grinding back the reinforcing bar to a flat surface. Similar to the steel reinforcing bar, during tensile loading of the specimen, data was acquired from the strain gauges attached to the specimens. Stress-strain results are provided graphically as a stress-strain function in Fig. 6.

The theoretical stress-strain response for the FRP bar has been calculated based on the manufacturer’s specified modulus of elasticity of 50 GPa and nominal cross-sectional area (Bluey Technologies, 2016). The calculated MAPE for each strain gauge, relative to the corresponding theoretical value of strain for a given stress, ranged between 20 and 44 percent. This magnitude of error is considered unacceptable.

CONCLUSIONS

Published research on the use of FRP in soil nails appears to be limited. New research on FRP soil nails is proposed that will add to the current body of research. The research will provide a direct comparison of the response of FRP and steel reinforced soil nails when subjected to pullout loads.

A test site for this new research has been identified in Yatula, and preliminary geotechnical site investigations have been completed. These investigations included undisturbed sampling and in situ testing using CPTs. Atterberg limits, triaxial and oedometer laboratory testing was performed on recovered clay samples to determine the engineering parameters. The results from these tests appear to correspond with generally accepted values; however, undrained shear strength results from CPTs do not correlate with those derived from laboratory tests. Therefore, additional laboratory and in situ testing is proposed to clarify this discrepancy.

Laboratory testing has commenced on specimens that replicate soil nails under uniaxial tensile loads. This testing has shown that strain gauge readings show significant variance to theoretical values for grout, steel and FRP materials. At this point, it is not clear what the cause of this variance is. One reason could be the misalignment of gauges, which can result in significant errors. For FRP materials, Sharpe (2008) indicates that for a misalignment as small as 4 degrees an error of more than 15 percent is possible. Therefore, further laboratory testing of strain gauges is required to establish a reliable and repeatable arrangement for strain gauges.

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REFERENCES


