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Enhancement in the Interface Shear Resistance Achieved by a Novel Geogrid with In-Plane Drainage

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ABSTRACT: The enhancement in the interface shear strength achieved in the short-term by a novel geogrid, which combines both reinforcement and in-plane drainage functions, was studied. Marginal fill material (wet gravelly clay) was standard Proctor compacted to achieve 92% of its maximum dry unit weight and tested in the large shearbox apparatus under consolidated-undrained conditions. Overall, the interface shear resistance (τ_{s-g}) values mobilized for the novel geogrid were similar to the undrained shear strength of the test soil itself. In contrast, the τ_{s-g} values mobilized for a conventional geogrid, which had similar physical and tensile strength properties, were between only 82 and 85% of the undrained shear strength of the test soil. The benefit is that the increase in the interface friction angle (achieved by in-place drainage along the novel geogrid) will facilitate the use of some marginal fills, which may already be readily available onsite, in the construction of earth structures.

1. INTRODUCTION

Earth structures (embankments and slopes) are traditionally constructed using suitable fill material that is placed and compacted to within 95% of its maximum dry unit weight achieved under standard Proctor compaction. Free-draining, high shear strength materials including gravel and sand are acceptable for use under any circumstances. Soils that contain less than 15% fines (percentage by weight passing the 0.063-mm sieve) are generally considered as suitable fill since these materials can be pre-drained if their water contents are too high. The fill material is generally placed and compacted in lifts, between 0.25 and 0.30 m in thickness, with at most 1.0-m depth of fill being placed in any given 24-hour period.

There is an economic benefit in using marginal fill (wet cohesive fill) that may already be readily available onsite rather than importing suitable fill material.

Marginal fill is defined as a predominantly granular material that includes high silt and/or clay fractions, and often has high water contents (typically only 90 to 95% of its maximum dry unit weight can be achieved under standard Proctor compaction).

The shear strength of the marginal fill may be reduced during the construction phase (short-term) due to the increase in the excess pore water pressure (generated during compaction of the poorly draining fill) and the increase in the total applied stress due to its self-weight. Geosynthetics can be included in the embankment core to provide reinforcement (geogrids) and/or preferential drainage channels (geosynthetics) thereby increasing its factor of safety against global and local instability. It is generally accepted that geogrids reinforce mainly through interlocking with adjacent soil. However, the angle of interface friction value mobilized by conventional geogrids is lower than the angle of shearing resistance value of the soil itself. Hence, in analyzing the factor of safety against slope instability where potential slip surfaces can align themselves along the soil-geosynthetic interface, the lower shear resistance value mobilized along the soil-geosynthetic interface must also be considered.

This paper studies the enhancement in the interface shear resistance that is achieved in the short-term (critical case in for example embankment construction) for a wet gravelly, clay fill material using a novel geogrid that combines both reinforcement and in-plane drainage functions. Kempton et al. (2000) and Zornberg and Kang (2005) reported the experimental results of dissipation and pullout tests on the novel geogrid in English China Clay (for $\sigma_v = 50$ kPa) and wet gravelly clay ($\sigma_v = 41$ kPa), respectively, which indicated that the mobilized interface shear resistance was about 30% greater than that mobilized for a conventional geogrid with similar physical and tensile strength properties. A series of consolidated-undrained large shearbox tests were conducted in the present study over the range of applied normal stresses typically associated with earth embankments between 5 and 10 m in height.

2. MATERIALS

2.1 Soil

The test soil was Brown Dublin Boulder Clay (Skipper et al., 2005); a gravelly clay of low plasticity (liquid limit of 31%, plastic limit of 16%, and plasticity index of 15%) that had a natural water content value of about 11%, and a specific gravity value of 2.70. Standard Proctor compaction tests, carried out on the material using a CBR compaction mould, indicated a maximum dry unit weight of 1.84 tonne/m³, which corresponded to an optimum water content for compaction of about 13.0% (Fig. 1).

2.2 Geosynthetics

The novel geogrid (ParadrainTM) comprised two elements, namely: (i) flexible reinforcement straps that had been profiled to provide in-plane drainage channels; and (ii) drainage elements (non-woven geotextile that had been bonded to the shoulders of the drainage channels (Fig. 2). The reinforcement straps (33-mm wide and 2.5-mm thick in cross-section, and 75-mm pitch in the reinforcing direction) comprised polyester fibers that were enclosed in a smooth polyethylene sheath that had a

characteristic tensile strength of 150 kN/m (main reinforcement). It should be noted that there are other products available that simultaneously serve both reinforcement and drainage functions.

A conventional smooth biaxial geogrid (ParagridTM) that had the same physical and tensile strength properties was also tested in the shearbox apparatus as a basis for comparison. Further details on the properties of these geosynthetics are available from the manufacturer, Linear Composites Limited (2007), or in the literature (Kempton et al. 2000; Zornberg and Kang 2005).

FIG. 1. Standard Proctor compaction.

(a) Conventional geogrid (ParagridTM)

(b) Novel geogrid (ParadrainTM)

FIG. 2. Geogrid test materials (Linear Composites Limited, 2007).

3. EXPERIMENTAL METHOD

3.1 Test Program

A series of consolidated-undrained tests were conducted using the large shearbox (300 by 300-mm in plan and 150-mm in depth) in accordance with ASTM D5321 (2002) in order to measure the development of the shear resistance over a range of applied normal stresses ($\sigma_v = 111\text{--}222$ kPa), including:

- Five tests on the soil alone;
- Five tests shearing the soil over conventional geogrid attached to aluminum plate;
- Five tests shearing the soil over the novel geogrid attached to an aluminum plate;
- Five tests shearing the soil over an aluminum plate.

3.2 Specimen Preparation

The geogrid specimens were attached to a smooth aluminum plate (300 by 300-mm in plan) that was located flush with the shear plane at the mid-height of the shearbox, and with its main reinforcement straps aligned in the direction of shear (Fig. 3). Both the novel and conventional geogrids were essentially fully fixed (bonded to the aluminum plate using a water-proof adhesive and fixed at one end of the plate using three screw fasteners) which prevented elongation of the geogrid specimens. The novel geogrid specimens were secured such that their drainage (geotextile) element was in contact with the soil that was contained in the upper half of the shearbox (Fig. 3). The screw fasteners were necessary since some of the geogrid specimens became detached during trial shearing tests when only the adhesive had been used to form the bond.

The straps of the novel and conventional geogrid specimens covered about 56 and 58% of the plate surface area, respectively (greater than the 50% coverage recommended by Koerner (1998)). The soil was in direct contact with the aluminum plate over the remaining areas. New geogrid specimens were prepared and secured to the aluminum plate at the start of each test.

FIG. 3. Novel geogrid straps secured to aluminum plate.

The gravelly clay material was disaggregated to pass the 20-mm sieve size and the larger-sized solid particles were removed, which accounted for about 11% of its bulk mass. Distilled water was added to the soil, increasing its water content to about 16.5% (exceeding the optimum water content for compaction by 3.5%), and the mixture was allowed to equilibrate overnight. In this state, the test soil is categorized as a marginal fill material. All of the shearbox specimens had the same initial water content value and were imparted the same level of compaction during specimen preparation. A known mass of the wet soil was placed and compacted in the upper half of the shearbox in three layers of equal thickness, with each layer being imparted 25 blows of a steel tamper (75 by 100-mm in face area) to obtain a dry unit weight of 1.68 tonne/m³ (92% of the maximum dry unit weight achieved under standard Proctor compaction), and an air voids content of 8%.

3.3 Shearbox Tests

A normal stress was applied to the specimen via the serrated loading platen in

contact with the top of the soil specimen. The soil was allowed to consolidate under the applied stress over a period of between 18 and 24 hours before shearing commenced to simulate in the geotechnical laboratory the construction sequence whereby the fill material is placed and compacted in stages onsite, and allowed to consolidate during the intervening periods.

The consolidation properties of the soil could not be determined by applying curve-fitting techniques to the measured compression versus time responses since the soil was in a partially saturated state ($S_r \cong 78\%$). However, drainage and hence dissipation to the pore-air voids of the excess pore water pressure that had been generated under the applied stress would have been substantially complete for the novel geogrid by the end of the 18 to 24 hour consolidation period. This has been shown experimentally by Kempton et al. (2000) by continuous measurement of the excess pore water pressure response during dissipation and pullout tests on the novel geogrid in English China Clay.

The specimens were then sheared quickly in order to determine the shear resistance values mobilized over the different interfaces under undrained conditions. The soil contained in the upper half of the shearbox was displaced relative to the geogrid specimen that had been fixed to the aluminum plate, which was located flush with shear plane) in the bottom half of the shearbox. Although the soil specimens were in a partially saturated state, the displacement rate of 0.25 mm/minute effectively produced undrained shear conditions (Koerner (1998), Koutsourais et al. (1991), amongst others). Reinforced earth embankments are typically 5 to 10 m in height so the different interfaces were tested in the shearbox apparatus under applied normal stresses in the range $\sigma_v = 111\text{--}222$ kPa.

4. EXPERIMENTAL RESULTS AND ANALYSIS

The shearbox results were analyzed in terms of the total stress condition since the pore pressure response was unknown during the shearing stage. However, drainage and hence the dissipation of the excess pore pressures to the pore-air voids resulted in increasing shear resistance with increasing applied normal stress. The Mohr-Coulomb failure line of best fit for the soil itself gave $c = 35$ kPa and $\phi = 26^\circ$ (peak values mobilized at between 7 and 9% strain); where c is the apparent cohesion and ϕ is the angle of shearing resistance (total stress condition). Similar analysis for the soil-aluminum plate interface, and using the shear resistance values mobilized at 8% strain gave $a = 5$ kPa, and $\delta = 20^\circ$; where a is the apparent adhesion and δ is the angle of interface friction (total stress condition).

Figure 4 shows the plots of the shear resistance mobilized against the horizontal displacement for the combined soil-geogrid-aluminum plate interface. The tests were terminated at the limiting horizontal displacement of 45 mm (15% strain) of the shearbox apparatus. The plots have a characteristic shape with most of the interface shear resistance was mobilized by about 9 mm horizontal displacement (3% strain), although the shear resistance continued to increase steadily without reaching a peak value. Zornberg and Kang (2005) reported similar constitutive behavior from pullout tests on the novel geogrid in a wet gravelly clay.

FIG. 4. Shear stress versus horizontal displacement for soil-plate-geogrid interface.

Figure 5 shows the undrained interface shear resistance (τ_{s-g}) that would have been mobilized for the soil-geogrid combination (more representative of the in-service field conditions). The interface resistance plots in Fig. 4 were adjusted using Eq. (1) which takes into account that over the shear plane, the soil specimen had been in contact with the aluminum plate on 42 or 44% of the shear area instead of the soil shearing on itself, as occurs in the in-service field condition. The contribution of interface interlock (gravelly clay particles and geogrid straps) to the mobilized shear resistance was not significant since the shearing occurred in the direction of the main reinforcement straps.

$$\tau_{s-g} = \tau_{s-p-g} + [(\tau_{s-s} - \tau_{s-p})(1 - A_g)] \quad (1)$$

where τ_{s-p-g} is the shear resistance mobilized along the combined soil-plate-geogrid interface; τ_{s-s} is the shear resistance of the soil itself; τ_{s-p} is the shear resistance mobilized along the soil-aluminum plate; and A_g is the fraction of the plate surface area that had been covered by the geogrid reinforcement straps.

FIG. 5. Interface shear resistance versus horizontal displacement for soil-geogrid.

Figure 6 shows the τ_{s-g} interface resistance values calculated for the novel and conventional geogrids at 5, 7.5 and 10% strains (relative horizontal displacements of 15, 22.5 and 30 mm). The τ_{s-g} values have been normalized by the corresponding shear resistance values of the soil itself (τ_{s-s}). The 5 to 10% strain range was chosen since

the undrained shear strength of the soil had been fully mobilized by about 8 to 9% strain.

The τ_{s-g} interface shear resistance for the novel geogrid was dependent both on the stress and strain levels, reducing slightly with increasing applied normal stress (Fig. 6). Overall, the τ_{s-g} values mobilized at 10% strain were for practical purposes the same as the undrained shear strength of the soil. However, the ratio of the τ_{s-g} to τ_{s-s} values mobilized for the conventional geogrid at 10% strain was only about 82%. This indicates that for the in-service field condition, slippage along the interface between the soil and conventional geogrid is more likely to occur rather than shear failure of the soil itself, and this failure mechanism must be considered in analyzing the factor of safety against slope instability where potential slip surfaces can align themselves along the soil-geosynthetic interface. Figure 6 also indicates that the ratio of the τ_{s-g} to τ_{s-s} values is somewhat strain level dependent, with the ratio reducing slightly with increasing strain.

FIG. 6. Normalized undrained interface shear resistance.

5. DISCUSSION

The novel geogrid performed considerably better than the conventional geogrids (which had similar tensile strength properties) for the wet marginal fill material tested, in agreement with other studies by Zornberg and Kang (2005) and Kempton et al. (2000). The excess pore pressures generated in the vicinity of the geogrid reinforcement straps dissipate rapidly along its in-plane drainage channels (Kempton et al. 2000). A higher state of effective stress, and hence a higher shear resistance, is achieved in the immediate vicinity of the shear zone, thereby enhancing geotechnical stability particularly in the short term. Furthermore, the construction of earth structures (embankments and slopes) using wet marginal fill material becomes feasible with the inclusion of the novel geogrid. The novel geogrid significantly reduces the time required for primary consolidation of the wet fill material to occur, thereby reducing the construction time. In contrast, the interface shear resistance of the conventional geogrid reduces to only about 82 to 85% of the undrained shear strength of the soil itself due to the build up in the excess pore pressure that occurs during shearing, and the smoother geogrid surface.

6. SUMMARY AND CONCLUSIONS

The enhancement in the short-term interface shear strength achieved by a novel geogrid, which combines both reinforcement and in-plane drainage functions, was studied. The test soil (marginal fill) was a wet gravelly clay ($c = 35$ kPa and $\phi = 26^\circ$ from large shearbox tests) that had been compacted to obtain 92% of its maximum dry unit weight achieved under standard Proctor compaction, and its water content value exceeded the optimum water content for compaction by 3.5%. The soil specimens were consolidated over a period of between 18 and 24 hours in the large shearbox apparatus and sheared quickly (undrained condition).

Overall, the shear resistances values (τ_{s-g}) mobilized along the novel geogrid-soil interface were similar to the undrained shear strength of the soil itself. The excess pore pressures (generated on applying the normal stress and subsequent shearing) dissipated rapidly along the in-plane drainage channels in the geogrid reinforcement straps, thereby achieving a high shear resistance in the immediate vicinity of the shear zone. In contrast, the τ_{s-g} values mobilized for the conventional geogrid (with similar physical and tensile strength properties) were only between 82 and 85% of the undrained shear strength of the soil. The benefit of the novel geogrid is that the increase in the interface friction angle achieved by its in-place drainage capability facilitates the use of some marginal fills in the construction of earth structures.

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8. REFERENCES

- ASTM D5321 (2002). Standard test method for determining the coefficient of soil and geosynthetic or geosynthetic and geosynthetic friction by the direct shear method. ASTM International, Pennsylvania, USA.
- Kempton, G.T., Jones, C.J.F.P., Jewell, R.A. and Naughton, P.J. (2000). "Construction of slopes using cohesive fills and a new innovative geosynthetic material." *Proc. 2nd European Geosynthetics Conference (EuroGeo 2)*, 15–18 October, Bologna: 825–828.
- Koerner, R.M. (1998). *Designing with Geosynthetics*, Prentice Hall.
- Koutsourais, M.M., Sprague, C.J., and Pucetas, R.C., (1991). "Interfacial friction study of cap and liner components for landfill design." *Geotextiles and Geomembranes* 10 (5–6): 531–548.
- Linear Composites Limited (2007). Product Data sheets. Available at www.linearcomposites.co.uk
- Skipper, J., Follett, B., Menkiti, C.O., Long, M., and Clark-Hughes, J. (2005). "The engineering geology and characterization of Dublin Boulder Clay." *Quarterly J. Engineering Geology and Hydrogeology* 38 (2): 171–188.
- Zornberg, J.G. and Kang, Y. (2005). "Pullout of geosynthetic reinforcement with in-plane drainage capability." *Proc. ASCE Geo-Frontiers*, 24–26 January, Austin, Texas (available on CD).