DESIGN OF STEEP SLOPES USING FINE GRAINED FILLS AND NOVEL MULTIFUNCTIONAL GEOCOMPOSITES

J. Clancy[1] & P.J. Naughton[2]

[1] *Institute of Technology, Sligo, Ireland. (e-mail: clancy.jude@itsligo.ie)*
[2] *Institute of Technology, Sligo, Ireland. (e-mail: naughton.patrick@itsligo.ie)*

Abstract: Free draining granular fills have been the preferred choice for backfill materials for reinforced soil structures such as steep slopes owing to their good strength characteristics and ability to minimise the development of pore water pressures in the reinforced fill. The development of pore water pressures reduces both the shear resistance of the fill and the bond between the soil and reinforcement, resulting in possible stability and settlement problems.

Fine grained fills can be effectively used to construct reinforced soil structures if adequate drainage is provided in the reinforced zone. This can be achieved through the use of novel multifunctional geocomposites which combine reinforcement and drainage functions.

Results of shear box and pullout analysis reported by Naughton & Kempton (2004), Zornberg & Kang (2005) and O'Kelly & Naughton (2008) showed improved pullout resistance and shear strength at the soil-geosynthetic interface, where multifunctional geosynthetics have been used. Careful consideration of the properties of both the soil and geosynthetic needs to be given during the design process of reinforced structures. Naughton et al (2001) identified a design process for use in the design of reinforced soil structures using poor fill materials.

This paper highlights the problems of using poor quality fills in steep slopes and with the modifications required to the traditional design process of reinforced steep soil slopes when fine grained soils are used in conjunction with the novel geocomposite. The results of a parametric study are presented to demonstrate the effectiveness of reusing fine grained fill combined with multifunctional geosynthetics in steep slope construction. The optimal properties of the multifunctional geocomposite for this application are presented. Finally possible applications of this soil, using the modified design procedure, are identified.

Keywords: biaxial geogrids, cohesive soil, fine soil, geocomposite, limit equilibrium, reinforced slope

INTRODUCTION

Traditionally free draining granular fills are used to construct reinforced soil structures because of their high strength and ability to prevent the development of excess pore water pressures. Zeynep (1992) reported the use of granular fill to be the most expensive component of a reinforced soil retaining system, usually equating to about 40% of the total construction cost.

Potential build up of pore water pressure in the reinforced block is the main concern when using fine grained fills in reinforced soil structures as this results in lower shear strength than granular fill and reduced bond between the soil and the reinforcement which may result in deformation and settlement of the structure. Poor draining soils are also more difficult to compact when the moisture content is high, resulting in longer construction periods (Zornberg & Mitchell, 1994).

Large volumes of fine grained soils are disposed of each year as there are little uses for them on site. Annon. (2007a) reported that an estimated 500 million tonnes of construction and demolition waste is generated annually in the European Union, with over 11 million tonnes generated in Ireland in 2004. It was further estimated that in 2001, 38% of the total C&D waste produced that year was excavated soil (Annon, 2007a). Total construction and demolition waste for England was estimated at 89.6 million tonnes in 2005, with 46 million tonnes recycled and a further 15 million tonnes spread on exempt sites and the remaining 28 million tonnes sent to landfill as waste (Annon, 2007b).

Many studies have focused on the use of fine grained, cohesive fills in the construction of reinforced soil structures. The Transport Research Laboratory (TRL) investigated the use of cohesive soil in a soil wall with various forms of impermeable reinforcement such as plastic and steel strips. The wall itself consisted of upper and lower layer of sandy clay with a middle layer of granular fill. Its early performance was described by Murray and Boden (1979). High excess pore water pressures developed in the clay during construction, resulting in large deformations of the wall.

Liu et al (1994), reported on a 12m test embankment constructed of cohesive fill. The embankment was divided into four sections, one unreinforced, and the other three with differing geogrid reinforcements. The development of pore water pressures was linked to placement of the fill material. The pore pressures increased as fill was placed and dissipated when this activity ceased. It was also shown that as fill height increased, the rate of pore water dissipation decreased. Significant settlement of the embankment was only observed when it was fully constructed to its final height.

The effectiveness of non-woven geotextiles in the reinforcement of steep clay slopes was assessed by Tatsuoka & Yamouchi (1986). Two large scale embankments were constructed using Kanto loam, volcanic ash silty clay, which is common in Japan. Embankment I used the geotextile reinforcements at different vertical spacing on each side while Embankment II used the same vertical spacings but different reinforcement lengths. In Embankment I, larger horizontal and vertical deflection was noted on the slope with the bigger vertical spacings. The same result was noted for the slope with shorter reinforcement at Embankment II. The performance of the geotextile as reinforcement was assessed when the embankments were demolished and a cross-section of the structure was visible. Large cracks were evident only in the unreinforced sections while only minor hair cracks were observed in the top soil layers of the reinforced zones.

The objective of this paper is to identify the required properties and layout of a geocomposite for use with fine grained fill. The study analysed the dissipation of excess pore pressures and stability of a 10m high slope constructed using three fine grained fills.

APPROACH TO FINE GRAINED FILLS ADOPTED BY CODES OF PRACTICE

Design codes treat the coarse and fine grained soils used in backfills differently. Both U.K. codes, BS 8006 (1995) & HA 68/94 (1994), offer no gradation limits for the material used, with BS 8006 (1995) stating that cohesive fills are permitted providing adequate reinforcement is used. The Federal Highway Administrations (FHWA, 2001) design code does provide a gradation limit for a maximum proportion of fines, as does Geoguide 6 (2002) although its limit is not as stringent.

Table 1. Treatment of cohesive fills by various codes of practice

Code of Practice	Requirements
BS 8006 (1996) UK	Cohesive fills may be used in new or reinstated slopes in combination with the appropriate reinforcement
HA 68/94 (1994) UK	Does not prohibit the use of cohesive fills
FHWA (2001) USA	Permits the use of soils with up to 15% passing the No.200 sieve (0.075mm)
Geoguide 6 (2002) Hong Kong	Permits the use of soils with up to 30% passing the No.200 sieve (0.075mm)

RESEARCH ON THE USE OF A DRAINAGE COMPONENT WITH POORER SOILS

Much research has been undertaken into the inclusion of a drainage component in reinforced soil structures (Kempton et al, 2000, Lopez et al, 2005, Zornberg & Kang, 2005, O'Kelly & Naughton, 2008, Boardman, 1998, Naughton and Kempton, 2004, Heshmati, 1993). These components usually take the form of a novel geocomposite which has the dual functions of reinforcement and drainage. The geocomposite is designed with in-plane drainage which dissipates excess pore water pressure resulting in improved strength, bond and reduced settlement of the structure, making it ideal for use with marginal, poorly draining materials.

Kempton et al (2000) reported on dissipation and pull-out testing on English China Clay, this material being chosen due to its low permeability. Included in the soil mass was the geocomposite with combined drainage and reinforcement capabilities. That study revealed the following:

- The new combined reinforcement drainage geogrid dissipated the excess pore water pressure in the fill to 20% of its initial value in 32 hours,
- Initial excess pore water pressure in the immediate vicinity of the new geocomposite only reached 40% of the applied stress,
- Even though the drainage channel was only on one side, dissipation of excess pore pressures occurred on both sides of the geocomposite,
- Pullout resistance was increased after both full and partial dissipation of excess pore water pressure,
- Adequate transmissivity is provided to remove water from soil even at low hydraulic gradient,
- There was no evidence of clogging or washing through of fines during the test period.

Lopez et al (2005) also compared the performance of a geocomposite, a geogrid with in-plane drainage (Paradrain) to a geogrid without (Paragrid). The efficiency of this geocomposite, a combination of high tenacity polyester yarns encased in a polyethylene sheet and a thermally bonded nonwoven fabric, was defined as the difference in pullout strength achieved by the former at the same initial pore pressure. The results showed that the geocomposite was more efficient at higher initial pore pressure values.

Zornberg & Kang (2005) studied the improvement in pullout resistance brought about by the inclusion of a geocomposite. The geocomposite consisted of a geogrid with polyester filament core with polyethylene sheath and drainage channels involving a polypropylene and polyethylene nonwoven geotextile. They showed that the use of in-plane drainage with a geogrid increased the pull-out resistance by approximately 30% compared to a geogrid without drainage capacity for the soil placement and loading conditions used in the testing program.

O'Kelly & Naughton (2008) investigated the improvement in interface shear strength associated with a geocomposite when compared to a traditional geogrid with the same physical and tensile strength properties, Paragrid. A marginal fill (liquid limit 31%, plastic limit 16%, plasticity index 15%) was compacted to 92% of its maximum dry density and tested under consolidated-undrained conditions in a large shearbox apparatus. The use of the geocomposite resulted in mobilization of interface shear resistance similar to that of the surrounding soil. The presence of the drainage channels rapidly dissipated the excess pore water pressures achieving a high shear resistance in the immediate vicinity of the shear plane. The shear resistance for the conventional geogrid were only between 82% and 85% of the undrained shear strength of the surrounding soil.

Boardman (1998) studied the change in the rate of consolidation associated with the geocomposite using a modified Rowe cell apparatus. It was shown that the inclusion of the composite geotextile resulted in reducing the drainage path by half, in turn increasing the rate of consolidation. For a soil sample consolidated at $50kN/m^2$ the time required for consolidation was reduced from approximately 100 to just 40 hours. A smaller reduction was noted for $100kN/m^2$ consolidation pressure. A small improvement in pullout resistance was recorded for the smaller consolidation pressure with larger improvements after consolidation at $100kN/m^2$.

Naughton and Kempton (2004) reported on the use of the same geocomposite in the reinstatement of a failed slope in Taiwan. The original slope had failed due to a combination of pore water pressure build-up during typhoon season, poor drainage and soil conditions. For the reinstatement the silty clay from the failed slope was reused in combination with the geocomposite meaning considerable cost saving, as expensive granular fill did not have to be imported. The geocomposite allowed rapid dissipation of the pore water pressures and permitted the work to be carried out in only three weeks during the typhoon season.

Heshmati (1993) reported on how the geocomposite improves the physical properties of fine grained cohesive soils. The study concentrated on the shear strength properties of kaolin provided by the drainage function and separately by the reinforcing function of a number of geotextiles. The interaction between the cohesive soil and the geosynthetics were monitored using a scanning electron microscope. The use of geosynthetics improved both the cohesion and shear strength of the clay, with cohesion being improved five-fold. Unexpectedly it was found that the use of a composite geosynthetic did not improve the shear strength which was possibly due to water being maintained between the drainage and reinforcing layers with failure probably along this plane. It was proposed that a geocomposite in which the reinforcement is embedded within the drainage geotextile would produce a positive result instead.

DESIGN CONSIDERATIONS FOR REINFORCED SOIL STRUCTURES USING FINE GRAINED SOIL

Designers of reinforced soil structures generally specify granular material as backfill due to its excellent strength and drainage characteristics. However when fine grained fills are to be used, particular consideration needs to be paid to the drainage conditions in both the short and long term. Christopher et al (1998) recognised that there were different design criteria when considering reinforcement-drainage geocomposites in marginal fills. Three adverse conditions were identified in the design of reinforced soil structures using poorly draining material.

- Condition 1: *Generation of pore water pressures within the reinforced fill.* Excess pore water pressures can build up in poorly draining soils during construction, particularly during the placement of load on the soil, e.g. compaction. A permeable reinforcement can dissipate these pressures owing to their secondary function as a lateral drain.
- Condition 2: *Wetting front advancing into the reinforced fill.* Post-construction infiltration into the backfill soil may result in the loss of shear strength of the soil. This infiltration is possible due to the formation of tension cracks on the surface of the soil. A geocomposite with in-plane drainage capability can drain these cracks when they reach down to the first reinforcement layer.
- Condition 3: *Seepage Configuration within the reinforced soil.* Seepage into the reinforced fill can come from adjacent ground or from fluctuations in the water table. Again this adverse condition could be countered by the lateral drainage ability of the geosynthetic.

The design philosophy suggested by Christopher et al (1998) was for a two-phase analysis. Firstly, a total stress analysis was performed for each of the three conditions ignoring the drainage contribution given by the geocomposite, and secondly, an effective stress analysis was performed for each condition taking the contribution to drainage into account. Conditions 2 and 3 can be addressed by the inclusion of adequately designed drainage system underneath and behind the reinforced soil block.

A design method for steep slopes constructed from cohesive fills and an innovative geocomposite was proposed by Naughton et al (2001). This design method aims to dissipate any excess pore water pressure present in the slope during the construction stage that would result in an increase in shear strength of the fill and enhanced bond between the reinforcement and fill. By dissipating the pore water pressures during construction required adjustments to the slope due to vertical and horizontal displacements can be made as construction proceeds. This approach also results in a one stage stability analysis, an effective stress analysis, as the excess pore pressure is dissipated fully before construction of subsequent layers. The authors proposed a simple flow chart which set out the steps to be taken in designing the slope, illustrated in Figure 1.

Naughton et al (2001) proposed a limit of 0.5m of the height of each lift to control short term stability of the slope face. The authors calculated the dissipation time based on the coefficient of consolidation and applied an appropriate factor of safety to account for unforeseen events. The settlement of each lift was shown to be related to the initial height of the lift, the coefficient of volume compressibility and the change in the vertical effective stress. The volume of water to be dissipated could then be determined from the magnitude of settlement assuming a saturated soil. The slope was then designed using an effective stress analysis for the ultimate limit state. The required transmissivity of the geogrid could be calculated once the time for consolidation and volume of water leaving the soil were known. This can then be compared to the available transmissivity in the geocomposite. If the transmissivity provided by the geogrid is insufficient, the height of the lift should be reduced and the design procedure repeated.

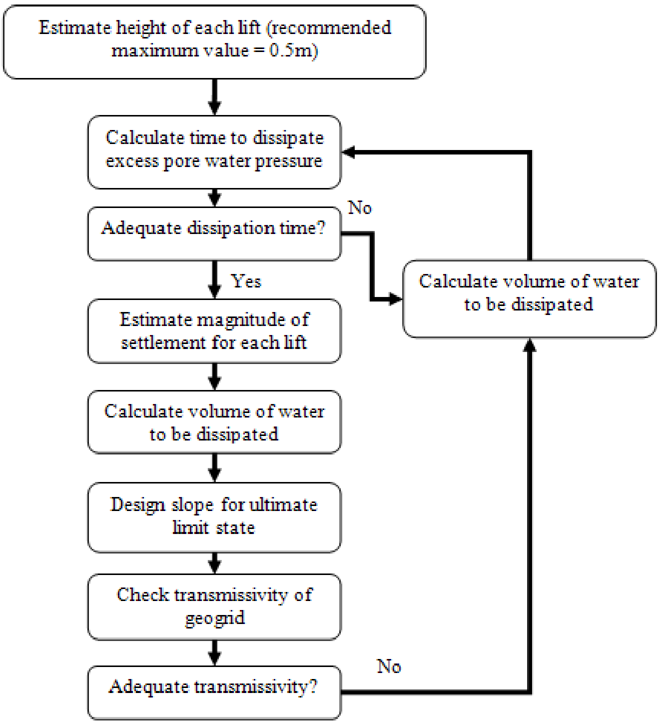

Figure 1. Flow chart for design of steep slopes using fine grained backfill, after Naughton et al (2001).

PROPERTIES OF FINE GRAINED SOILS EXAMINED IN THIS STUDY

Three fine grained soils were examined in this study. Their classification, shear strength and consolidation characteristics are presented in Table 2. All testing was carried out in accordance with BS 1377 (1990). The strength and consolidation characteristics were determined from incremental consolidated direct shear and oedometer testing on recompacted samples prepared at optimum moisture content using standard compaction. The soils were selected randomly from construction sites in the Northwest of Ireland.

Soil A was taken from a site, which was being infilled by surplus excavation material and is classified as a clay of intermediate plasticity. Soil B, a silt of high plasticity, was extracted from approximately 2m below ground level from a site on which a local authority machinery yard is being constructed. Finally, Soil C was excavated from an area where development was taking place and the soil was not suitable for reuse there. Its classification is a clay of high plasticity. The three soils selected are considered representative of fine grained excavated soil waste available on construction sites.

Table 2. Classification, shear strength and consolidation properties of soils included in testing program

	Soil A	**Soil B**	**Soil C**
Plastic Limit (%)	21.3	48.1	25.7
Liquid Limit (%)	43.2	69.2	52.5
Plasticity Index (%)	22.1	21.1	26.8
Unit Weight ((kN/ m^3)*	17.505	13.190	15.495
Percentage passing 63μm sieve	31	50	58
Angle of friction ϕ'	33.0	26.3	18.6
Cohesion c′	9.7	35.8	21.5
Coefficient of consolidation, C_v (m^2/year)	2.124-25.635	0.967-30.187	1.367-28.831
Coefficient of compressibility, m_v (m^2/MN)	0.016-2.247	0.018-0.738	0.018-1.884

* Corresponding to 95 % of maximum dry density.

STABILITY OF REINFORCED SLOPES USING FINE GRAINED FILLS

The method proposed by Naughton et al (2001) was used to design steep reinforced slopes using each of the three soil types investigated in this study. The critical element in the design was to select a lift height that would facilitate construction of a single lift in a period of approximately 24 hours. A construction time of 24 hours per layer is considered appropriate for steep slopes of short to medium length. This time period would allow one lift to be constructed each day on site, making the entire slope construction very efficient.

Figure 2 illustrates the slope dimensions used in the designs, a 10m high steep slope inclined at 70° to the horizontal. The bulk unit weight of the foundation fill was taken as 18 kN/m^3.

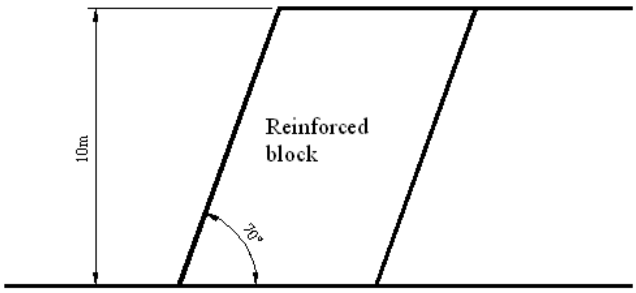

Figure 2. Slope used in design examples.

Once the height of lift was selected to give the required dissipation time the stability was checked in accordance with BS 8006 (1995) based on a limit equilibrium approach using Oasys Slope 18.2 (Oasys, 2007). The reinforcement used for all slopes was a polyethylene/polyester geogrid, with initial short term strength of 100 kN/m. The partial material factors, corresponding to a design life of 120 years, for the geogrid are presented in Table 4. The analysis assumed that adequate drainage was provided at each reinforcement lift to facilitate double drainage during dissipation of excess pore pressures. The investigation varied the values of r_u used in the design to simulate different levels of consolidation. Analysis was first carried out with a uniform value for the pore water pressure parameter, r_u, of 0.5. This would be the value of r_u of the layer immediately under the one constructed. It simulates only partial dissipation of excess pore water pressure and more rapid slope construction. This revealed that the slopes would fail during construction at heights below 10m, i.e. the Factor of Safety (FOS) being less than 1. The slope failure heights for this analyse are shown in Table 5.

The analysis was repeated with varying values of r_u to simulate the dissipation of pore water pressure in each layer, with the value decreasing as slope height increased. This dissipation may be brought about by the inclusion of a drainage element in the reinforcement. The analysis resulted in all three slopes maintaining a F.O.S. above unity for the full slope height of 10m. The r_u value is determined from an average value of the each of the layers constructed. The stability of the slope was checked with a r_u value of 0.2, corresponding to a degree of consolidation of 80% as recommended by Naughton et al (2001) for all designs. Using this value, the slopes constructed with Soils B & C remain stable to the full height of 10m, whereas the slope composed of Soil A had a FOS of close to but slight less than unity. Table 5 shows the final design of the slope for each soil type.

Table 3. Load partial factors used in analysis

	Partial Factor
Dead Load	1.2
Live Load	1.3
Unit Weight	1.5
Cohesion c′	1.6
Angle of friction ϕ'	1.0
Sliding along reinforcement	1.3
Reinforcement pullout	1.3

Table 4. Material partial factors used in analysis

	Partial Factor
Friction Interaction	0.4
Adhesion Interaction	0.4
Creep Reduction	0.27
Manufacture	1.0
Damage	1.05
Environmental	1.05
Extrapolation of Test Data	1.3

The lift height suggested by Naughton et al (2001) of 0.5m was greater than that calculated for any of the soil types examined. Soil A had the longest dissipation time, 28.5 hours, on the lower layers, which overall increased the total dissipation time for the slope, 668 hours, although this could have been shortened by using shallower lifts. Soil B had the shortest length of reinforcement, 6 m, and also the shortest dissipation time, 462 hours. This is due to relatively high C_v values established the materials low unit weight. Soil C proved to be the most unsuitable soil for use in the slope. A much greater length of geotextile was required to stabilise the structure and a reduced vertical spacing. Analysis showed that achieving adequate stability was the critical design element for this slope, not the dissipation time. This was possibly due to the soil properties, with the material having a low angle of shearing resistance and the largest proportion of fines, 58 % passing 63μm. The properties of this soil would be the most typical for fine grained fills. The large quantity of reinforcement used highlights the problems associated with using fine grained fills in reinforced soil structures. The maximum transmissivity of the reinforcement geogrid was also determined. The highest values were found to be required at the top of the slope where the vertical settlement was at its greatest owing to the compressibility characteristics of the soils. These levels of transmissivity could be provided by the use of the aforementioned geocomposite which has a transmissivity of 0.9l/m.hr at a hydraulic gradient of 0.1 (Linear Composites, 2008) which is in line with the hydraulic gradient calculated for the slopes.

Table 5. Dimensions and results from design procedure

	Soil A	Soil B	Soil C
Suitable Lift Height (m)	0.4	0.4	0.25
Reinforcement Length (m)	6.0	6.0	10.0
Longest lift dissipation time (hours)	28.5	20.2	16.0
Total dissipation time (hours)	668	462	483
Maximum required transmissivity (l/m.hr)	0.403	0.118	0.328
Factor of Safety at 10m (r_u = 0.5)	0.555	0.671	0.691
Height of Failure at r_u = 0.5 (m)	4.95	6.138	6.6
Factor of Safety at 10m (varying r_u)	1.042	1.090	1.092
Factor of Safety at 10m (r_u = 0.2)	0.974	1.038	1.010

APPLICATIONS OF FINE GRAINED FILLS IN THE CONSTRUCTION SECTOR

Based on the current available knowledge on the use of fine grained soil it is not possible to recommend their use in all steep reinforced slope applications. Care needs to be taken in design to adequately predict the magnitude of vertical settlement and deformation at the face of the slope to meet serviceability limit state requirements (BS 8006, 1995).

Possible applications include non critical structures, structures which can tolerate some deformation without affecting their performance. These types of structures would include noise bunds, landscaping features and other non trafficked structures. The use of a novel multifunctional geocomposite increases the opportunities where fine grained soil could be used, owing to its ability to dissipate excess pore water pressures, increase the strength and bond of the backfill material and decreasing the time for differential settlement of the structure to occur.

CONCLUSIONS

Large volumes of fine grained soils are produced on construction sites each year (Annon, 2007a & b). Most of this fill is considered waste and is not routinely reused for the construction of reinforced steep slopes and other soil reinforcement applications.

Research (Kempton et al, 2000, Lopez et al, 2005, Zornberg & Kang, 2005, Boardman, 1998 & Heshmati, 1993) has shown that fine grained soil can be successfully used as backfill material provided adequate drainage is provided in the body of the structure. Excess pore water pressures, generated using construction, can be rapidly dissipated resulting in increased strength and deformations occurring during the construction period (Naughton et al, 2001). Dual function geosynthetics, combining reinforcement and drainage components offer a practical means of utilising fine grained soils as backfill materials. Existing design method needs to be modified to take account of the properties and problems associated with fine grained fills, especially the need to dissipate excess pore pressure during construction.

The properties of three fine grained soils, typical of the waste materials generated on many construction sites, were presented and were shown to have a wide range of strength and consolidation characteristics.

A steep slope was designed using each of the fine grained soil examined as backfill material. The design presented combines a method for determine the maximum height of each lift to allow dissipation of excess pore pressures in a 24 hour period. The stability of the slope was checked using effective stress analysis, as excess pore pressures generated during construction have been dissipated, using conventional slope stability software. Varying the values of the pore pressure parameter r_u and hence the degree of consolidation with slope construction as dissipation of layers occurred resulted in the stability of the slope increasing. This dissipation may be induced by incorporating a drainage element into the reinforcement. The required transmissivity of the geogrid was also determined for each of the soil types. Fine grained fills with high percentage of fines and low angles of shearing resistance require longer lengths of reinforcement placed at closer vertical spacing's than that expected from the use of granular free draining fills.

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