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Performance of a 37 m high geogrid reinforced soil structure constructed over a disused sewage treatment pond

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ABSTRACT: The development of a housing scheme on a hill site project in Cheras, Selangor, Malaysia necessitated the construction of a reinforced soil structure. The base of the structure was partially founded on a disused sewage treatment pond. The pond was reclaimed where soft unsuitable materials were excavated and replaced with rock compacted to form a firm base. A 37 m high geogrid reinforced soil structure was subsequently constructed on the rock fill foundation. The geogrid reinforced soil structure was constructed as a wrap-around system where the facing was hydroseeded with suitable vegetation. The backfill material used in the construction of the structure is composed of weathered residual soil and weathered rockfill of granite. Instrumentation in the form of inclinometers, pneumatic piezometers and settlement markers were installed at various locations in the structure to monitor its performance during and after construction, principally to measure the horizontal and vertical movements and pore pressure developments within the fill mass. Some significant movements were recorded and this was discussed in the paper. A finite element analysis was carried out to assess the displacements of the structure. This paper discusses the design of the structure and results of the field monitoring used to check and verify the important design assumptions and criteria.

1 INTRODUCTION

The development of housing on a hill site project in Cheras, Selangor, Malaysia necessitated the construction of an earth retaining structure to provide the necessary works platform for the construction of double storey terrace houses. The base of the structure was partially founded on a disused sewage treatment pond. The pond was reclaimed whereby soft unsuitable materials were excavated and replaced with rock fill compacted to form a firm and stable base.

A 17 m high geogrid reinforced soil structure was constructed on the compacted rock fill foundation. This structure, 17 m high and sloped at 4v:1h, supports another reinforced fill of 20 m high sloped at 1v:1.7h, above which are sited the double storey terrace houses. The total length of the structure is approximately 120 m. Figure 1 shows the layout of the structure and Figure 2 shows the typical cross-section detail of the reinforced earth structure. This paper discusses the design concept and performance monitoring during the construction of the 37 m high reinforced soil structure.

2 OVERVIEW OF DESIGN CONCEPT

The subsoil condition of the foundation generally was treated to ensure the bearing capacity is sufficient to sustain the loads from the structure. Figure 3 shows the mackintosh probe test results carried out in the pond prior to treatment. The figure shows that about 3 m depth of soft material was present at the base of the pond. The soft unsuitable materials in the pond were removed and replaced with selected compacted rockfill materials to form a stable rock toe.

The design criteria calls for a factor of safety against bearing failure, based on lower bound shear strength, to be not less than 2.0, and the factor of safety against local and global slope failure to be not less than 1.4.

The residual soils were derived from the weathering of a granitic profile and its composition comprise predominantly of clayey silty sand. The grain size distribution of the residual soil is as shown in Figure 4.

Reasonably conservative shear strength values were adopted for the design: For well compacted fill of residual soil of granite and retained soil layers, the effective cohesion (c′) adopted was 5 kPa and the

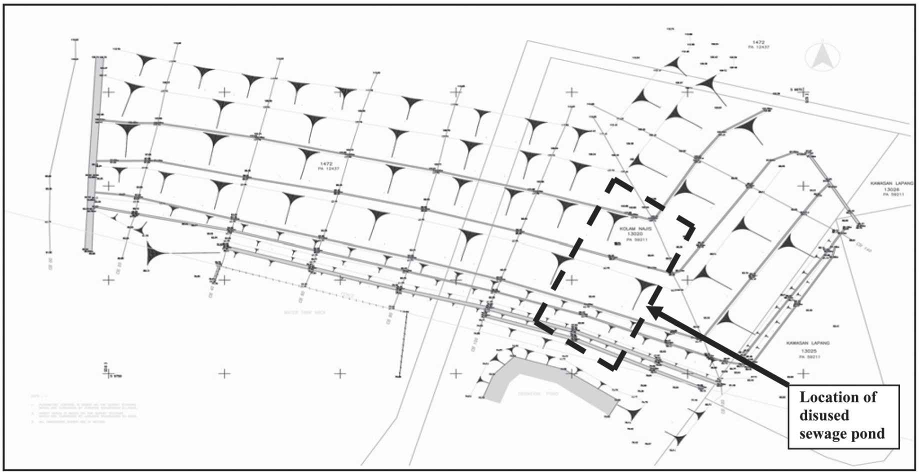

Figure 1. Layout of geogrid reinforced soil wall.

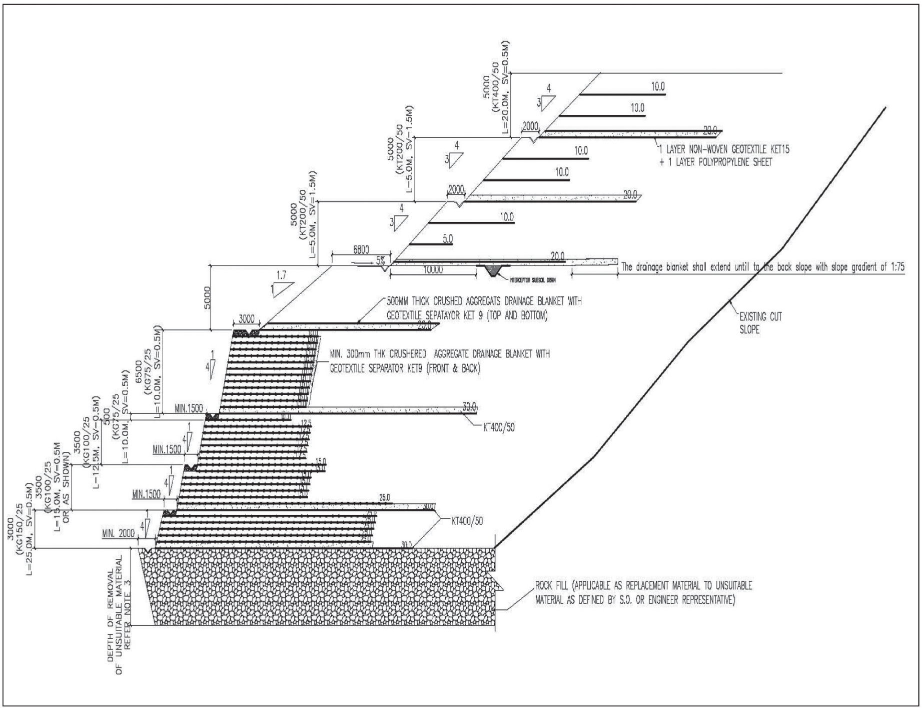

Figure 2. Typical cross-section detail of structure.

effective friction angle (ϕ′) was 32°. For the foundation layer, the c′ is 0 kPa and the ϕ′ is 30°. Table 1 provides a summary of the design parameters adopted.

The shear strength parameters of the fill material were verified by carrying out large shear box tests. The dimension of the shear box is 300 mm × 300 mm. This size of shear box was used to take account of the maximum grains size of the soil particles of 10 mm.

High strength polyester woven geotextile and high strength knitted geogrid were used to reinforce the structure. KiaraTex high strength woven geotextile KT 400/50 was laid in the founding layer of the structure. The ultimate strength of this material is

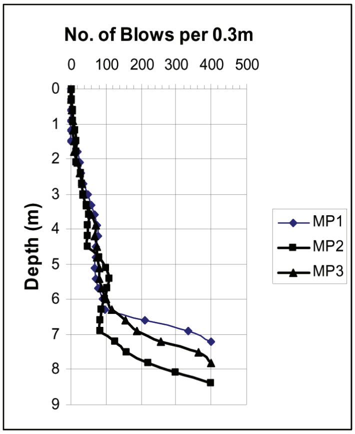

Figure 3. Results of Mackintosh probe tests carried out in Pond.

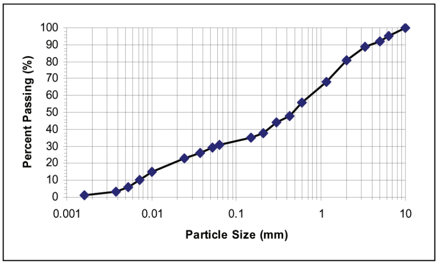

Figure 4. Particle size distribution of fill material.

Table 1. Design parameters.

Stratum	Design Parameters		
	Effective Cohesion (kN/m^2)	Effective Friction Angle (deg)	Bulk Unit Weight (kN/m^3)
Well Compacted fill	5	32	18
Retained layer	5	32	18
Foundation layer	0	30	18

400 kN/m, and a factor of safety of 3.0 was adopted. This value of safety factor was chosen in anticipation of the heavy rainfall, which coincides with the monsoon season, during the construction period.

For the remaining layers, KiaraGrid high strength knitted geogrid were used. Grades of the geogrids included KG 75/25, KG 100/25 and KG 150/25. SIM creep test was carried out on KG 400/200 which was manufactured with the similar polyester yarns to verify the long term deformation characteristics of the material. The results indicated that the retained tensile strength after 114 years exceeded 81% of its specified ultimate value.

The geogrids were laid in layers vertically spaced at 500 mm. The ultimate strength of these materials were 75 kN/m. 100 kN/m and 150 kN/m, respectively, with a cumulative factor of safety (for construction damage, creep, etc.) of 3.0 as discussed above for the woven geotextiles.

A typical cross-section detail of the structure is shown in Figure 2. The upper reinforced soil slopes were reinforced with high strength woven geotextile, KT200/50. Table 2 provides a summary of the design stresses of the geosynthetic materials used.

Table 2. Design Stresses in Geotextiles and Geogrids.

Geosynthetics Grade	Ultimate Strength (kN/m)	Actual Design Strength (kN/m)
Geogrid		
KG150/25	150	49
KG100/25	100	32
KG75/25	75	24
Woven Geotextile		
KT400/50	400	122
KT200/50	200	61

The stability analysis was performed considering internal stability and overall stability. A minimum factor of safety of 1.40 was adopted for the design. Both the circular and translational modes of failure were investigated for the internal and global stability. The stability analysis was based on Bishop's method of slices. Figures 5, 6, 7 and 8 show the construction of the reinforced earth structure.

Figure 5. Rock-fill Base of Structure.

Figure 6. Base Layer of Woven Geotextile.

Figure 7. Completed Section of wall.

Figure 8. Panoramic view of wall.

3 MONITORING

The monitoring system installed included inclinometers, settlement plates and pneumatic piezoneters. The inclinometers and pneumatic piezometers were installed from the fourth, fifth and sixth berms of the structure. The piezometric levels recorded indicate a stable piezometric level, hovering at about the base of the structure. Figure 9 shows the position of these instruments on the structure.

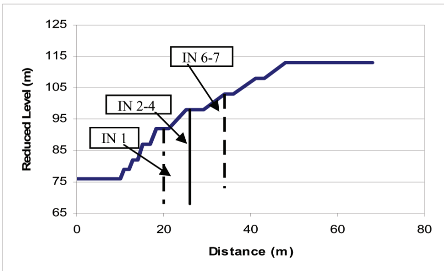

Figure 9. Location of inclinometers on structure.

The maximum downslope (lateral) movements of the inclinometer readings recorded near the upper levels of instruments are shown in Figure 10: The inclinometer readings show that the structure moved a significant movement in the initial stages but stabilised towards the end of the monitoring period.

Settlement markers were installed at various levels, as shown in Figure 11. The records from the settlement markers on Berm No. 6 are shown in Figure 12. The results show that the structure has reached stability with respect to the vertical movements.

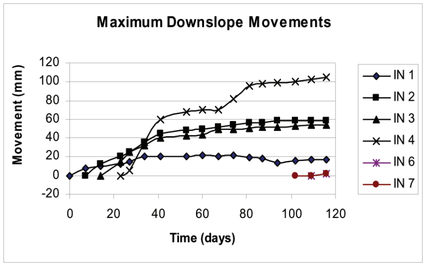

Figure 10. Maximum downslope move Figure 8: Panoramic view of wall.

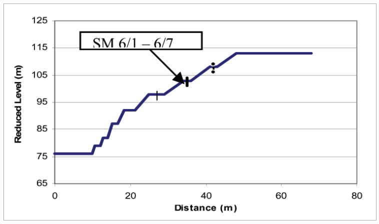

Figure 11. Location of Settlement Markers.

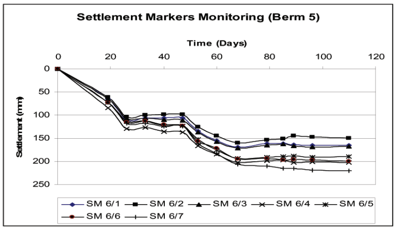

Figure 12. Records from the Settlement Markers.

4 FINITE ELEMENT ANALYSIS

The reinforced soil structure was modelled in finite element using the PLAXIS Finite Element software (version 8). The geogrid and woven geotextile were modelled as tension elements. The analysis was performed with the assumption of a drained condition, adopting the Mohr-Coulomb model. Parameters relating to the stiffness of the geogrid, woven geotextile and retained backfill soil are shown in Table 3. Figures 13 and 14 show the results of the analysis.

Table 3. Parameters adopted in Finite Element Analysis.

Materials	Units	Values
Bulk density of backfill soil	kN/m^3	18
Stiffness of backfill soil (E)	kN/m^2	25,000
Stiffness of KG150/25 (EA)	kN/m	1500
Stiffness of KG100/25 (EA)	kN/m	1000
Stiffness of KG75/25 (EA)	kN/m	750
Stiffness of KT200/25 (EA)	kN/m	2000

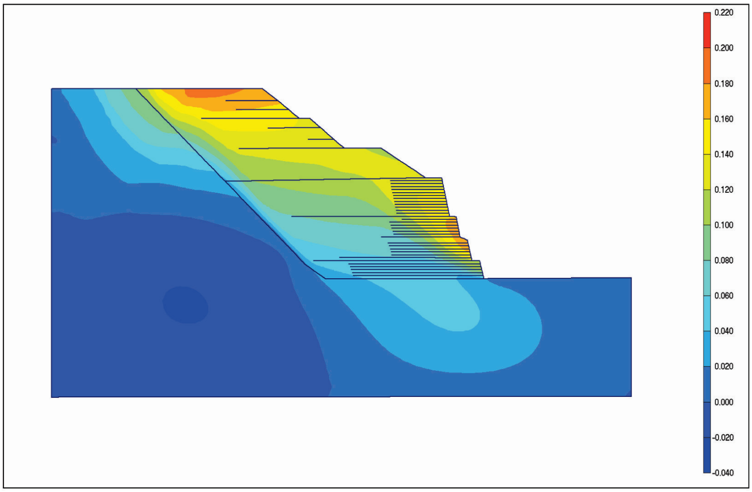

Figure 13. Plot of horizontal displacements.

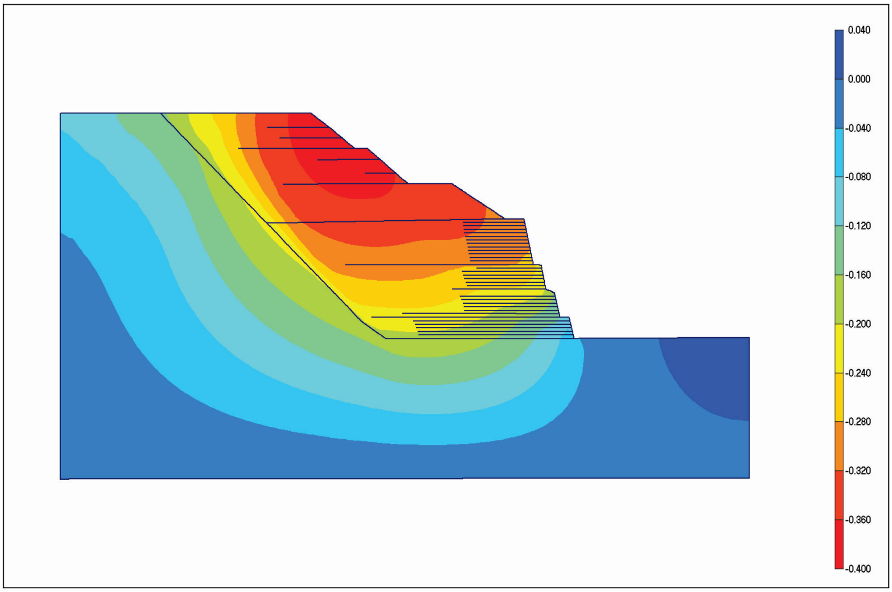

Figure 14. Plot of vertical displacements.

At Berm No. 5 which corresponds to the location of the inclinometer measurements, the calculated horizontal displacement corresponds to a value of between 120 mm and 140 mm. At the crest of the structure the total horizontal displacement was calculated as between 180 mm and 200 mm.

At Berm No. 6, the calculated vertical displacement corresponds to a value of between 320 mm and 360 mm. At the crest of the structure, the vertical displacement was calculated as between 360 mm and 400 mm.

5 DISCUSSION

The large lateral movements recorded in the early stages of the construction were thought to be due to the straightening and tensioning of the geogrid, based on observations during geogrid installation. The maximum horizontal movement recorded was 110 mm, occurring near the upper levels (Berm 5). This relates to a maximum strain in the geogrid of 1.1%. The maximum settlement recorded was 220 mm, corresponding to a compression of 1.3% of the height of the structure.

Relating the results of the finite element analysis to the field monitoring results suggest that the finite element analysis had predicted the displacement within a reasonable accuracy. Generally, the measured displacements are smaller than the predicted values.

6 CONCLUSION

A case study describing the design concept and performance monitoring of a 37 m high geogrid reinforced soil wall has been described. The monitoring results indicate that the wall has reached a stable state with small residual horizontal and vertical movements recorded. The case study showed that high reinforced soil walls can be constructed and yield a safe and stable structure.

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