Performance of Full Scale Embankment on Soft Bangkok Clay with Geogrid Reinforcement

D. T. Bergado & N. J. L. Menil
Asian Institute of Technology, Bangkok, Thailand

P. Rimoldi
Tenax SpA, Vigano, Italy

R. S. Douglas
RasWill Representative Pte Ltd, Singapore

ABSTRACT: A full scale and fully instrumented test embankment was built to 6 m high with Tenax TT 201 SAMP geogrid reinforcements. The performance evaluation of the full scale test embankment covers the period of nine months since the beginning of construction. Presently, the test embankment is stable despite large lateral deformation and settlements. The latest monitored value indicated maximum lateral ground movement at 3 to 4 m depth and surface settlement at the center of embankment to be 160 mm and 75 cm, respectively. The grid reinforcement was found to be loaded with 2.3 kN/m corresponding to 4.2% of the ultimate load of 55 kN/m. In addition, the location of the failure plane or the maximum tension line, neither follow the Coulomb/Rankine line of the Tie Back Wedge Method or the bilinear failure plane as proposed by Coherent Gravity Method. Compared to the previous performance of other reinforced embankment, the polymer Tenax TT 201 SAMP geogrid can be a viable reinforcement material.

1 INTRODUCTION

A full scale and fully instrumented test embankment with polymer Tenax TT 201 SAMP geogrid reinforcement was built to 6 m high on soft Bangkok clay at AIT Campus. Instrument readings were taken during the construction of the reinforced embankment/wall and at intervals throughout a period of nine (9) months after construction. Monitoring was carried out at regular intervals as soon as the instruments were installed. Thus, all the data relate to the commencement of loading of the embankment. The plan and section views of the test embankment together with the locations of the instrumentations are shown in Fig. 1ab. The pneumatic piezometers, inclinometers, and strains in the geogrids were monitored using standard portable readout boxes. Settlement plates and settlement markers were located and monitored using standard surveying techniques. Fluctuations in water table and dummy piezometers were measured using galvanometer electric wire system. Most of the results included in this paper were taken from the work of Menil (1994).

2 EXCESS PORE WATER PRESSURES

Figure 2 shows the typical relationship between increasing embankment height and the resulting excess pore water pressures measured by pneumatic piezometer at 6.0 m depth (P2) beneath the embankment. All the piezometers demonstrated an increase in the excess pore pressures during the full period of construction which were developed due to the increase in the surcharge load. Both the hydraulic and pneumatic piezometers showed the same response. Two of the pneumatic piezometers located at P1 and P3 ceased to function after construction probably due to the disconnection of the tubing. Consequently, hydraulic piezometers were relied upon for the subsequent readings after construction. It is also noted that both total and excess porewater pressures showed a tendency to decrease after construction indicating the subsoil consolidation. Vertical stress increase due to embankment loading in all locations are also incorporated on Fig. 2. The methods of Gray (1936) and Poulos and Davis (1974) was adopted to determine the stress increase. The former assumed a semi-infinite layer while the latter assumed a finite layer. The concept was based on a uniformly loaded rectangular area. Both methods also assumed that the subsoil is elastic, homogeneous and isotropic.

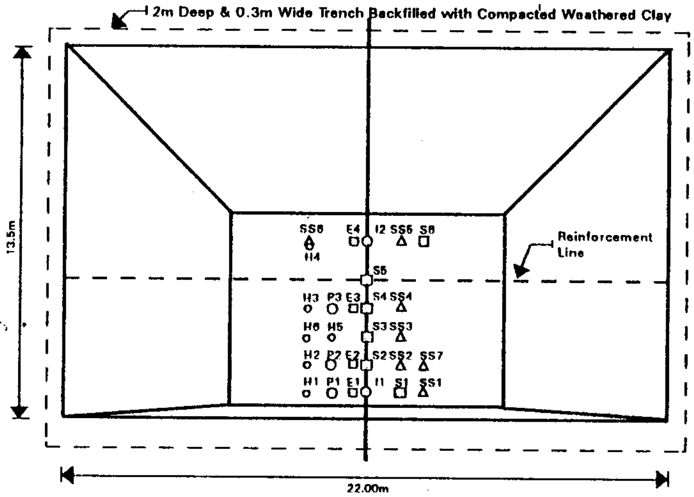

Fig. 1a Instrumentation Layout Plan of Test Embankment

3 SETTLEMENTS

Surface and subsurface settlement curves followed the classical behavior, with an initially high rate of settlement during construction and slowing down thereafter. About 200 days after the end of construction, the rate of settlement considerably decreased. Correspondingly, about 70% degree of consolidation was attained. The surface settlements indicated by S_1 to S_6 were observed to be almost identical at about 25 cm at the end of construction and continue to increase thereafter. Figure 3 showed that at the end of 270 days, the magnitude of surface settlements in all locations is almost the same, signifying near uniform distribution of vertical stress. The last monitored reading showed maximum value at S_3 and S_4, both at the center followed by S_5, S_2 and S_1 at near center and at front with lowest value at S_6 near the back slope.

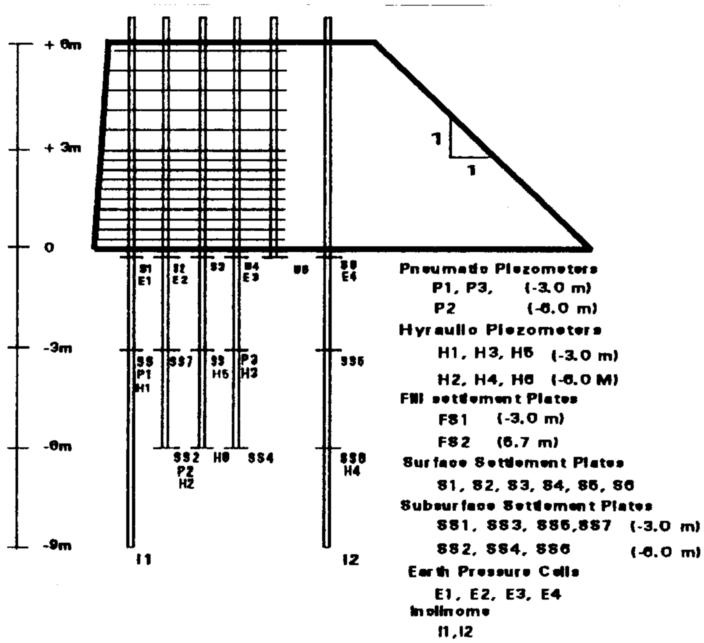

Fig. 1b Cross Section of Instrumented Test Embankment

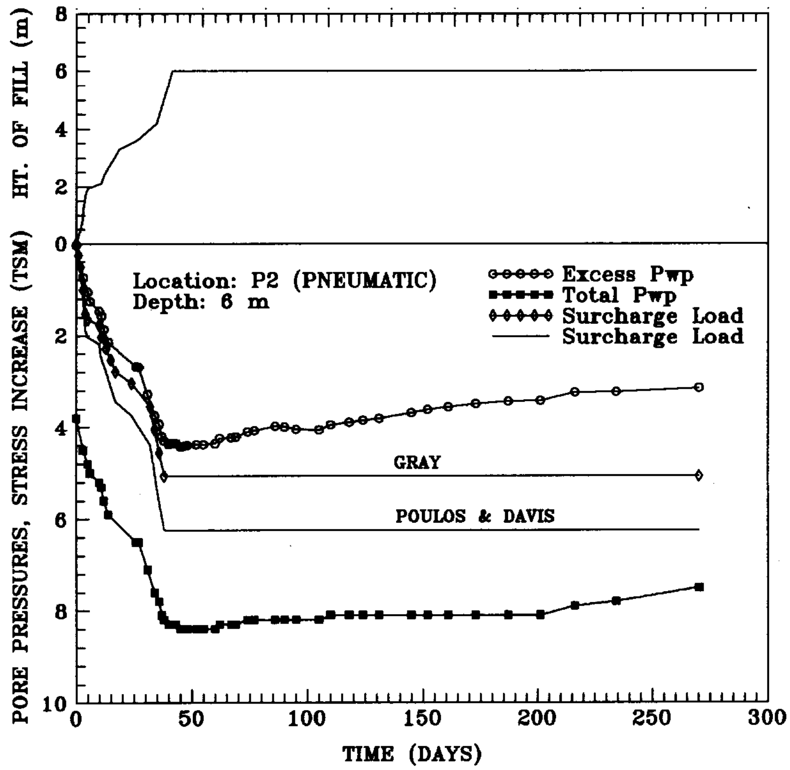

Fig. 2 Relationship between increasing embankment height and the resulting excess pore water pressure

4 LATERAL DISPLACEMENT

The distribution of the horizontal movements at face and at the back, both in the embankment and the subsoil were monitored by means of a biaxial inclinometer. The inclinometer showed that the lateral movement in the subsoil occurred mainly between 3 to 4 meters below the ground surface. On the other hand, the maximum lateral movement at the face wall occurred at the top of test embankment. The recorded lateral movement of the soft clay subsoil at the end of construction is about 50 mm which is approximately 35% of the maximum recorded value of 160 mm after 223 days. Inclinometer 2 which is located at the back slope, showed outward movement with smaller magnitude than the front. The direction of the lateral movement of the embankment and subsoil at the back slope is opposite to that of the front wall.

5 VERTICAL SOIL PRESSURE

To determine the vertical pressure distribution at the base of the embankment, four pneumatic pressure cells (SINCO) were installed. The measured distribution of the vertical stress at the base and therefore on the geogrid laid at the base is slightly nonlinear. This variations of base pressure is due to the subsoil compression beneath the embankment. The maximum value recorded at the end of construction occurred at the front and decreases farther from the wall face with the lowest pressure measured at a distance of about 6.5 m away from the wall face.

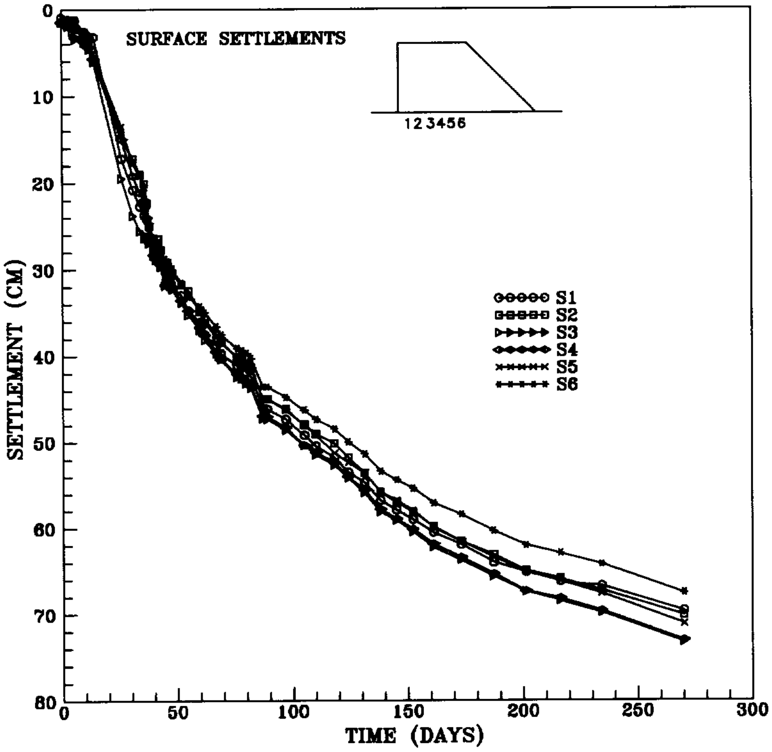

Fig. 3 Surface settlements – time relationship

6 GEOGRID STRAIN

Strains in the geogrid reinforcement were measured at seven different elevations, each instrumented fully with 10 resistance strain gages (Type YFLA-5). The tensile forces were obtained using the calibration curve derived from laboratory pullout tests at different applied normal pressures. The distribution of strains along the geogrid length after construction at different elevations are presented in Fig. 4 together with measured settlements. The values plotted are chosen from different periods of time with noticeable change in magnitude, including the maximum recorded value. What is worth noting is that all the observed tensile strains are in the range of 0.4% to 1.1%, corresponding to 0.84 to 2.3 kN/m load in the geogrids. Comparing this load to the maximum tensile strength of geogrid, which is 55 kN/m, the grid are only loaded to between 1.5% to 4.2% of the ultimate strength.

7 MAXIMUM TENSION LINE

Previous research on reinforced embankment supported on good foundation showed that maximum tension line can either be defined by Coulomb/Rankine type linear failure plane (Juran and Christopher, 1989) or a bilinear/logarithmic spiral failure plane (Schlosser and Buhan, 1990) depending on the stiffness of the reinforcement. At this stage of development whereby subsoil and embankment's movement is still going on, and thereby the stresses in the geogrid would still change, it is premature to evaluate the potential failure plane. Nevertheless, within the considered time frame, it was demonstrated that neither of the aforementioned criteria is observed as shown in Fig. 4.

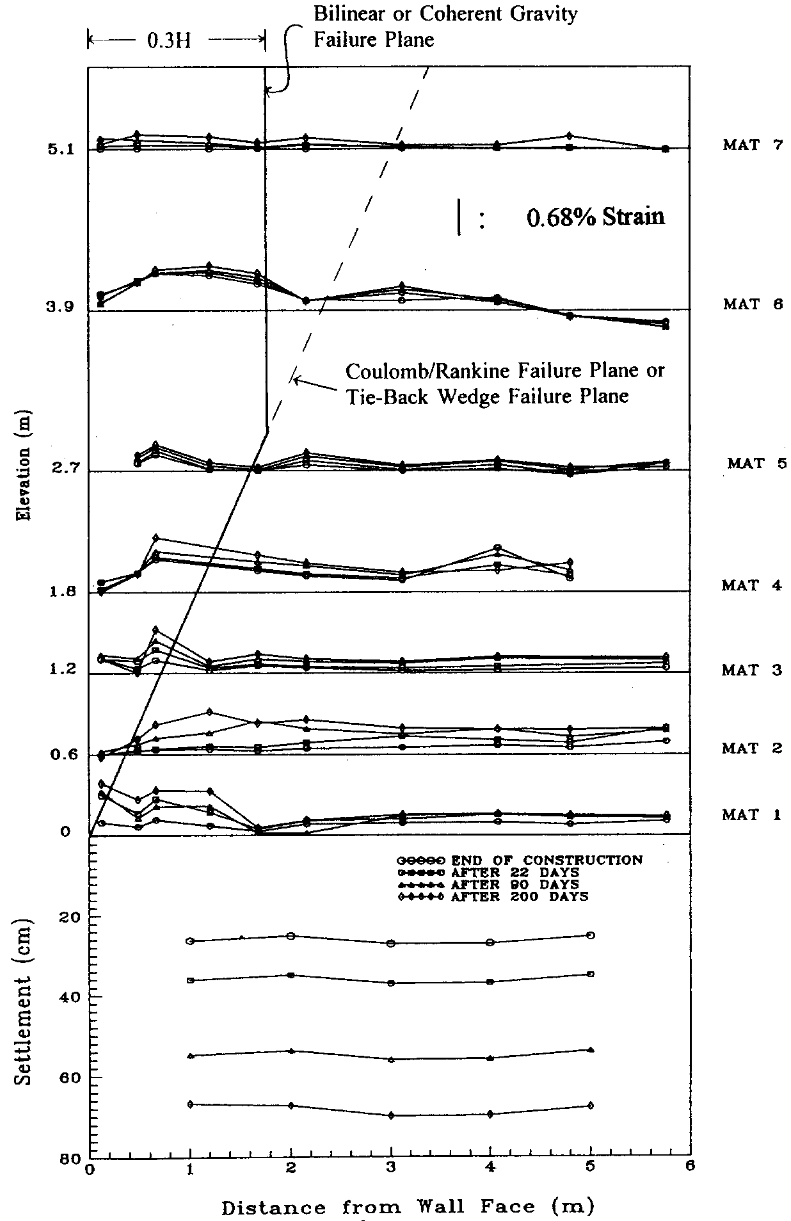

Fig. 4 Variation of Strain with Distance from Wall Face at Different Elevations with Surface Settlement Profile

8 COMPARISON OF PERFORMANCES OF WELDED WIRE AND POLYMER TENAX GEOGRID TEST EMBANKMENT

The welded wire reinforced test embankment was previously constructed in AIT Campus (Bergado et al, 1990). Both embankments have similar dimensions and similar backfill soil consisting of weathered Bangkok clay. The main points of comparison are focused on the different behavior of the test embankment such as settlement, lateral movements, vertical base pressures, and tensile forces in the reinforcements. Table 1 shows the comparison of embankment behavior at 270 days after the beginning of construction. Similarity of behavior was observed. Differences were observed in the lateral movements and vertical pressures due to slight differences in the construction procedures and techniques.

Table 1 Comparative Results on Surface Settlements, Lateral Deformation and Vertical Pressure of Welded Wire Wall and Polymer Geogrid Reinforced Embankments

PARAMETERS	LOCATION	WELDED WIRE	POLYMER GEOGRID
Lateral Deformation (mm)	Top of Embank.	435	360
	3 m below g.s.	135	160
Surface Settlements (cm)	Front	85	65
	Center	70	75
Vertical Pressure (kPa)	Front	1	50
	Center	82	12

Notes: Results are taken at the end of about 270 days since beginning of construction.

9 CONCLUSIONS

Performance evaluations based from the results of fully instrumented full scale test embankment have been made. The results were compared to the previous studies of welded wire reinforced test embankments. The following conclusions can be made:

1) The location of maximum tensile forces along the geogrid reinforcement does not coincide with either Coulomb/Rankine Tieback Wedge or the bilinear Coherent Gravity Method due to compression of underlying soft subsoils.

2) The maximum tensile stress in the reinforcement correspond to only 4.2% of the ultimate strength.

3) The degree of consolidation of 70% was achieved after 200 days from the start of construction.

4) Similar behavior was observed in comparing with previous test embankment performance. Thus, the utilization of Tenax TT 201 SAMP geogrid with weathered clay backfill is viable alternative for construction of reinforced embankment/wall.

REFERENCES

Bergado, D.T, Sampaco, C.L., Shivashankar, R., Alfaro, M.C., Anderson, L.R. and Balasubramaniam, A.S. (1991) Performance of welded wire wall with poor quality backfill on soft clay, *ASCE Geotech. Eng. Congress*, Boulder, Colorado, U.S.A., pp. 909-922.

Cancelli, A., Rimoldi, P. and Togni, S. (1992) Frictional characteristics of geogrids by means of direct shear and pullout tests, *Proc. of the Intl. Sym. on Earth Reinforcement Practice*, Fokuoka, Japan, pp. 51-56.

Gray, H. (1936) Stress distribution in elastic solids, *Proc. 1st Intl. Conf. Soil Mech. and Foundation Eng'g.* Vol. 2, p. 157.

Juran I. and Christopher, B. (1989) Laboratory model study on geosynthetic reinforced soil walls, *J. of Geotech. Eng. Div.*, ASCE Vol. 115, no. 7, pp. 92-111.

Menil, N.J.L. (1994) Performance and Interaction of Polymer geogrid based on laboratory tests and full scale test embankment, *AIT M. Eng. Thesis*.

Poulos H.G. and Davis, E.H. (1974) Elastic solutions for soil and rocks mechanics, *John Wiley and Sons Inc*.

Schlosser, F. and De Buhan, P. (1990) Theory and design related to the performance of reinforced soil, *Proc. of Symp. on Performance of Reinforced Soil Structures*, Thomas Telford, pp. 1-14.