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Behaviour of geogrid-reinforced load transfer platforms for embankment on rammed aggregate piers

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ABSTRACT: A test embankment was constructed over soft ground to evaluate the performance of different types of load transfer platform (LTP) supported on rammed aggregate piers (called 'geopiers'). Three types of LTP were constructed in accordance with the recommended design for each: a geosynthetic-reinforced LTP with two layers of geogrid (catenary LTP), a geosynthetic-reinforced LTP with three or more layers of geogrid (beam LTP), and a reinforced concrete LTP. The results indicate that the differential settlement between the geopiers and the matrix soil is relatively small in all LTP sections, with the smallest in the beam LTP. The tensile strain in the geogrid was approximately 60% of the allowable design strain of 5% in the beam LTP. In the catenary LTP, the tensile strain in the geogrid was approximately 20% of the allowable design strain of 6%, although the differential settlement was larger than the beam LTP. The total and differential settlements observed indicate that the use of less stiff geopier columns together with LTPs provides an attractive option for supporting low embankments on soft ground with tolerable total and differential settlements. The presence of LTPs and the supporting columns also tend to reduce the lateral movement of the foundation soil. The cost analysis of the different LTPs indicates that construction costs for geosynthetic-reinforced LTPs are likely to vary with locality; however, it appears that beam LTPs offer a less costly approach, with enhanced performance.

KEYWORDS: Geosynthetics, Load transfer platform, Reinforcement, Geogrid, Rammed aggregate pier, Geopier, Column-supported embankment, Instrumentation

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1. INTRODUCTION

Column-supported embankments provide a rapid means of construction in areas where subsoil consists of fine-grained soft soils. A load transfer platform (LTP) is usually provided at the base of the embankment to even out differential settlement and minimise overall deformation of the embankments. The mechanism of load transfer in geosynthetic-reinforced LTPs supported on columns (columns include all columnar foundations, including various types of pile and pier) is, in general, poorly understood. In particular, there exist several formulations for soil arching from which the proportion of the embankment load that is transferred to the columns is calculated. The ratio of the average vertical stress transmitted to the subsoil at the base of the embankment due to soil arching and the average vertical stress at the base of the embankment if there is no soil arching is known as the arching ratio. Part of the function of the geosynthetic reinforcement is to retransmit the load apportioned to the subsoil to the columns, which can bear more load with less compression than the soft subsoil. The other part is to reduce lateral spreading of the embankment, thus rendering the use of inclined piles at the outer edges of the embankment unnecessary.

Different design concepts are currently being used by designers of geosynthetic-reinforced LTPs supported on columns (Collin 2004), but only two are considered in this study. The first is the catenary LTP, which utilises only a single layer or at most two layers of high-strength geosynthetic. The reinforcement in essence behaves as a structural element, and any benefits resulting from the creation of a composite reinforced soil mass are ignored (Collin 2004). The second is the beam LTP, where three or more layers of relatively low-strength geogrids (i.e. lower strength than that of the catenary LTP) are employed as reinforcement within a well-graded granular fill. It is assumed that there is interaction between multiple layers of the geosynthetic reinforcement and granular fill that results in a stiff LTP beam with less differential settlement between the columns and the matrix soil in between the columns. The thicker aggregate layer also contributes to the stiffness of the beam LTP. In the beam design, the aggregate gradation and the thickness of the LTP have to be accurately specified to ensure that arching region remains within the LTP. For the catenary LTP design, specific material gradation is not normally defined but the friction angle of the material is generally specified—usually 30° or higher (Collin 2004).

In most of the design methods, assumptions for the load distribution and the tension in the geosynthetic reinforcement are made to simplify the analysis:

(1) All the embankment loads that are not directly supported by the columns are assumed to be supported by the geosynthetic reinforcement that spans between adjacent columns.
(2) The load distribution between adjacent columns is the same in square and equal triangular column arrangements.
(3) The tensile force throughout the clear span between adjacent columns is constant, although field data and numerical analysis indicate that this is not the case.

Numerical studies presented by Tonks and Hillier (1998), Han and Gabr (2002), Pham *et al*. (2004) and others indicate that several factors affect the interactions between fill, geosynthetic reinforced LTP, column and subsoil. They include: the stiffness of the geosynthetic (or LTP); the stiffness of the column; the column–soil stiffness ratio; the clear spacing between the columns; the embankment height; the direct support provided by the subsoil to the LTP; and, to a lesser extent, the friction angle of the granular fill. Giroud *et al.* (1990) showed that a large variation in the friction angle of the aggregate does not affect the lateral earth pressure coefficient K in the arching zone. The design of the LTP is accomplished by calculating the tensile force in the geosynthetic and specifying the appropriate geosynthetic. The parameters that control the tensile force in the geosynthetic include the clear spacing between the columns, the column diameter, the distributed vertical load acting on the reinforcement between adjacent columns, and the consequent strain in the geosynthetic. Live loads and higher embankment fill unit weight increase the vertical load and therefore the tensile force in the geogrid.

Since 1995, when the British Standards Institution introduced the design of geosynthetic-reinforced piled embankments in BS 8006 (1995), several geotechnical bodies and researchers have initiated their own design methods. The Nordic Geotechnical Societies (2002) and Kempfert *et al.* (2004), a group of researchers from Germany, introduced new design methods. These methods are referred as the British, Nordic and German methods, respectively. These are based on the catenary LTP design. Collin (2004) recommended a different procedure for the design of LTPs, referred to herein as the Collin method: the beam LTP.

The main issue in determining the load on the geosynthetic reinforcement is choice of the appropriate arching theory/method to determine the proportion of the load that is transferred from the geosynthetic to the columns, and the proportion of the load supported by the subsoil. The strain in a geosynthetic depends very much on the magnitude of the differential settlement that causes the geosynthetic to elongate. In many reported case studies involving geosynthetic-reinforced LTPs supported on columns, the differential settlement between the column head and the subsoil is less than 50 mm. Only in one case study (Rogbeck *et al.* 1998) were the differential settlements higher (i.e. more than 170 mm), but in this case foam mattresses were placed between the columns to simulate extremely soft subsoil. The maximum tensile strain in the geogrid was almost six times higher in this case than in adjacent areas where mattresses were not used between the columns, which had mostly 20 mm of differential settlement. Several case studies (Brandl *et al*. 1997; Lin and Wong 1999; Wilson-Fahmy *et al*. 2005) indicate that larger differential settlement at the base of the embankment generates more tensile strain in the geogrids than smaller differential settlement. A numerical study by Jones *et al.* (1990) showed that the support provided by the subsoil can significantly reduce the tensile force in the geosynthetic. In most cases, tensile force in the geosynthetic is a fifth or less when the subsoil support is considered.

To obtain field evidence for the evaluation of the performance of the catenary and the beam LTPs, a heavily instrumented full-scale test embankment supported by rammed aggregate piers called ‘geopiers’ was constructed over soft ground in Gebeng, Pahang State, Malaysia, and the results are presented herein.

2. TEST EMBANKMENT SITE CONDITIONS

A full-scale test embankment was constructed at Gebeng, Pahang, in the east coast of peninsular Malaysia. The test embankment site was located adjacent to a bridge that crossed a creek called Baluk River. The bridge is part of a highway that is being constructed to bypass and connect an industrial area in Gebeng to a toll highway that connects Kuantan, the state capital of Pahang, to Kuala Lumpur, the capital of Malaysia. The test embankment site is located in a small flat floodplain some 8 km from the coastline. Figure 1 shows the subsoil profile under the test embankment, and its vicinity. Site investigation indicated a soft silty clay/clayey silt layer as deep as 15 m at some locations. However, this layer generally extended approximately 5 to 6 m below the original ground level. The soils in this layer are composed of highly plastic clay and silt with natural water contents between 35% and 61%.

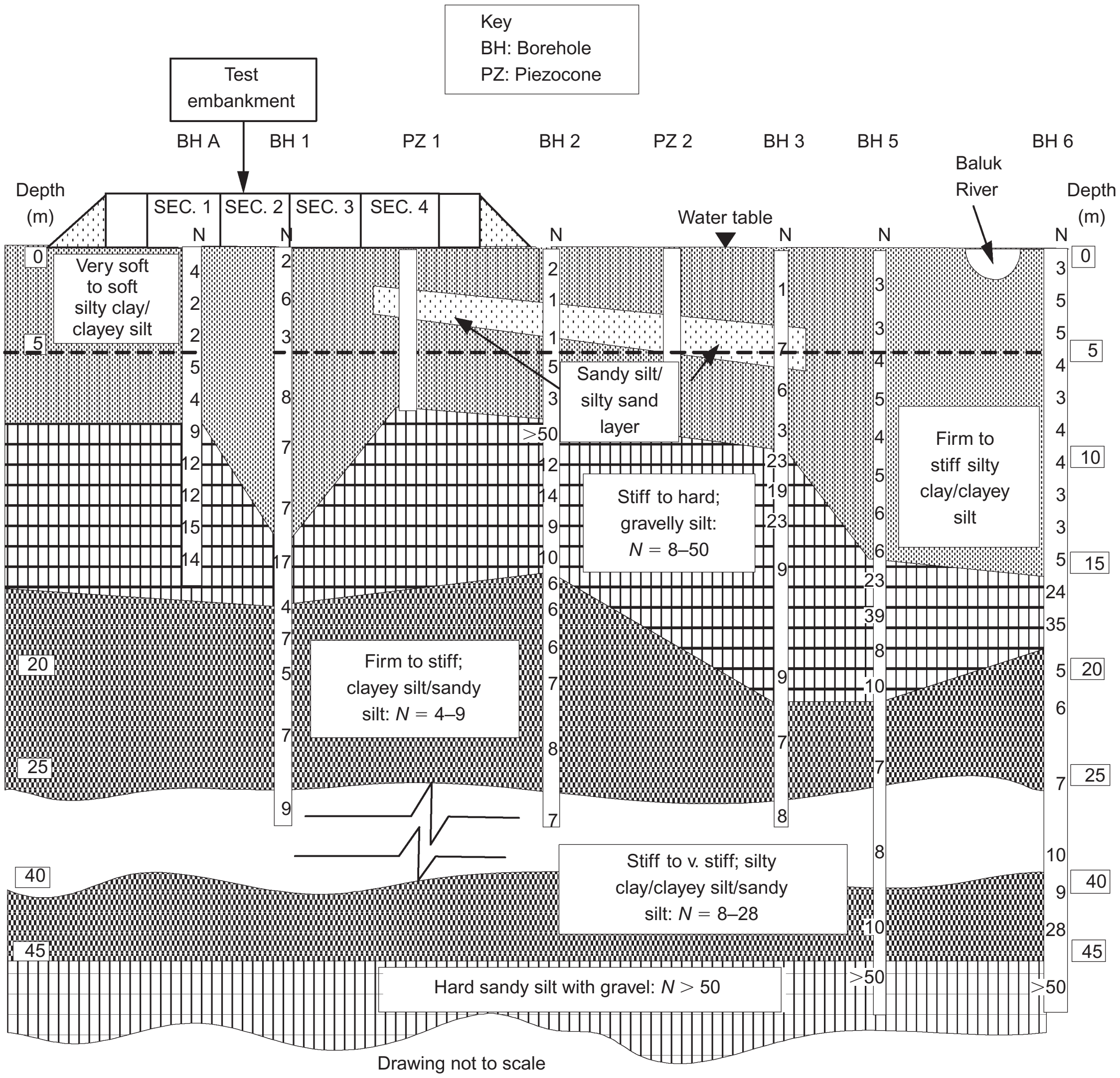


Figure 1. Subsoil profile under test embankment

Field vane tests indicated that the shear strength in this layer ranges from 14 to 60 kPa, with most of the values less than 25 kPa, and the sensitivity varies from 3 to 11. The top clayey/silty soil is interspersed with thin organic and sandy layers that were sometimes detected during drilling for the installation of the geopiers. One of the advantages of using geopiers is that the drilled-out subsoil can be identified and logged, and simple in situ shear strength tests can be carried out at the site if necessary. Detail information on the subsoil can be correlated with the existing bore logs to ascertain the design assumptions used for the geopiers. The top layer is underlain by stiff to hard gravelly silt, which, in turn, is underlain by firm to stiff silt. The water table in this area is very high, at or just below the original ground level.

3. LTP SECTIONS IN TEST EMBANKMENT

The test embankment was constructed adjacent to a recently constructed highway bridge. The bridge approach embankment was supported by a reinforced concrete LTP supported on concrete piles. The length of the test embankment was approximately 90 m, the width was 14.5 m, and the height was 3.5 m. The side slopes of the embankment were 1V:1.5H. The test embankment was divided into four major sections (Sections 1–4), and two control sections (C1 and C2) at the two ends of the embankment.

The major sections were supported by different types of LTP supported on geopiers. All the geopiers were installed in a square arrangement with a specific centre-to-centre spacing (hereafter referred to as the spacing). The dimensions, the types of LTP and the geopier square-arrangement spacing for each section are shown schematically in Figure 2.

The geosynthetic-reinforced LTPs consist of an aggregate layer with geogrid reinforcement within the aggregate. Section 1 has a 1.5 m thick beam LTP with four layers of a biaxial extruded polypropylene geogrid (Tensar SS20) reinforcement spaced at 0.3 m apart vertically within the LTP, and supported on geopiers spaced in a 3.25 m centre-to-centre square pattern. Section 2 has a 1.0 m thick beam LTP, but with three layers of the same geogrid as in Section 1 spaced at 0.3 m apart vertically, and supported

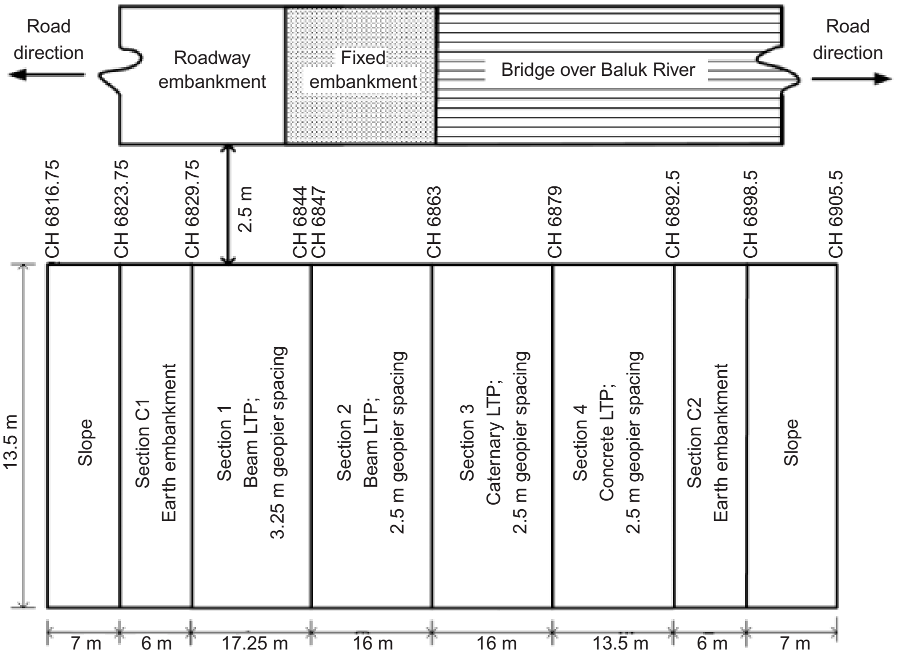


Figure 2. Layout of major LTP sections and control sections

on geopiers spaced in a 2.5 m centre-to-centre square pattern. These beam LTPs were designed in accordance with the Collin method. Section 3 has a catenary LTP with two layers of a uniaxial woven polyester high-strength geogrid coated with polyvinyl chloride (Miragrid 24XT) spaced at 75 mm apart vertically. One layer is placed in the transverse direction and the other in the longitudinal direction to the embankment. The section was supported on geopier elements spaced at 2.5 m apart centre-to-centre in a square pattern. The catenary LTP was designed in accordance with the British method. The properties of the two types of geogrid used are given in Table 1. The bottom-layer geogrids in Sections 1, 2 and 3 were placed directly over the geopiers. The final major section, Section 4, was supported on 0.3 m thick continuous steel-reinforced concrete LTP and geopier elements spaced in a 2.5 m centre-to-centre square pattern. The detailed layout of the LTPs for the various sections is presented in Figure 3.

The concrete slab was constructed directly over the geopiers without any connection between the concrete slab and the geopiers. The control sections were not supported by the geopiers, and were built directly over the ground. The function of the control sections is to provide a reference to the improvement provided in the major sections and to provide a buffer to even out the shear stresses at the end sections. No load factors were used in the design of the LTPs.

3.1. Geopier foundation

The rammed aggregate pier, called a 'geopier,' is a relatively new intermediate-depth columnar foundation introduced in the construction industry (Fox and Cowell 1998). Geopiers are installed by drilling holes 0.6 to 0.9 m in diameter and filling them with select aggregate in thin layers. Each layer of the aggregate is rammed with a bevelled shape rammer for about 20 s. Typically, the drilled holes extend between 2 and 8 m below grade (White and Suleiman 2004). The installation process of a geopier is illustrated in Figure 4. In collapsible soils, casings have to be used to support the subsoil from collapsing into the drilled hole: if this occurred, the installation process would be slower. In the test embankment, the initial drilled diameter of the geopiers was 0.75 m, and the initial depth of the drilled hole for the

Table 1. Properties of the geogrids used

Property	Biaxial extruded geogrid		High-strength uniaxial geogrid	
	Machine direction	Cross-machine direction	Machine direction	Cross-machine direction
Tensile strength (at ultimate) (kN/m)	20	20	370.3	43.6
Tensile strength (at 5% strain) (kN/m)	14.0	14.0	93.3	17.5
Tensile modulus (stiffness) (kN/m)	280	280	1870	350
Grid aperture size (mm)	39	39	101	17.8
Mass/unit area (kg/m^2)	0.220		1.289	

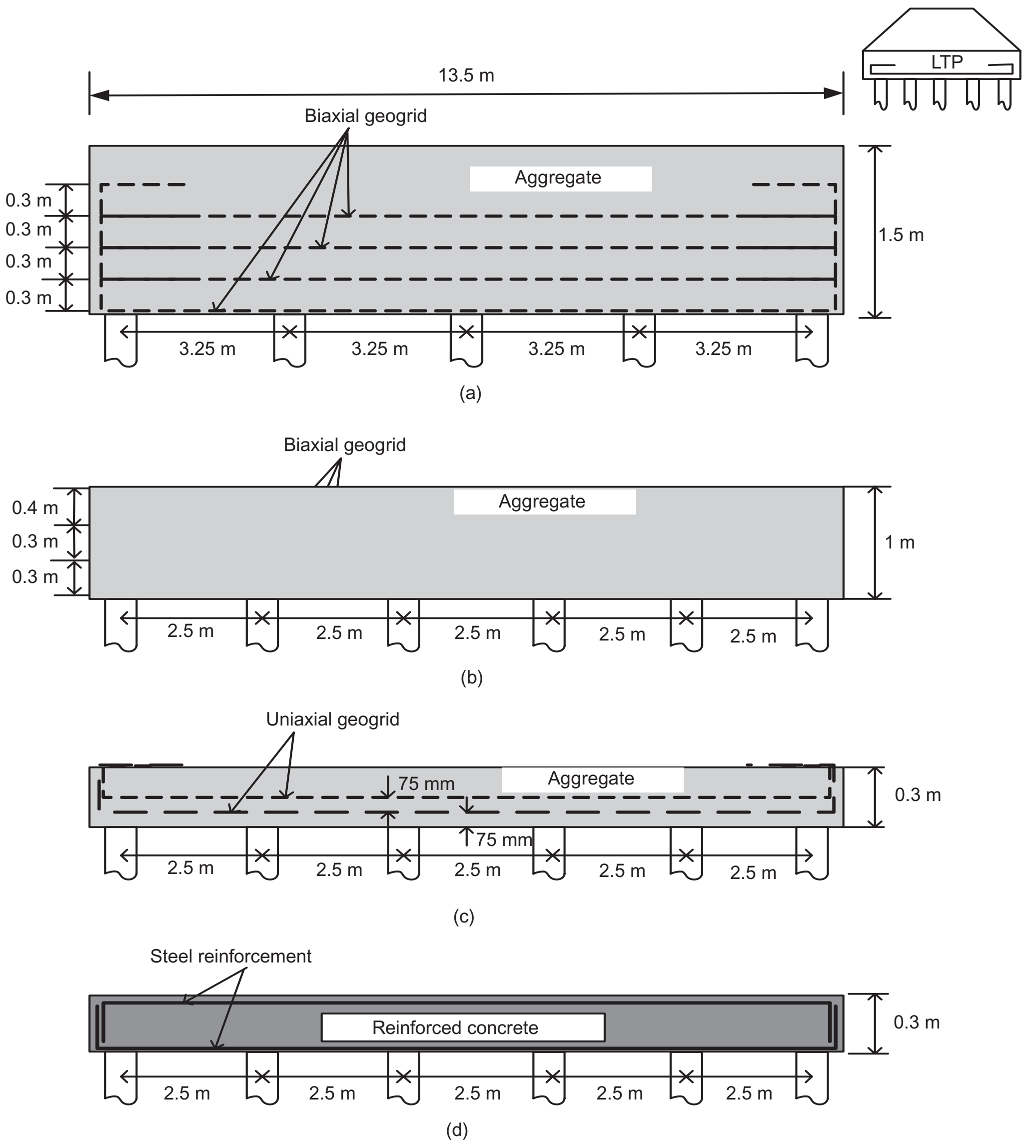
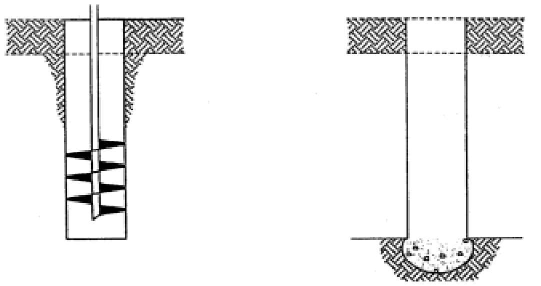


Figure 3. Schematic layout of the different LTPs in: (a) Section 1, beam LTP; (b) Section 2, beam LTP; (c) Section 3, catenary LTP; (d) Section 4, concrete LTP

geopiers was 5.5 m. Two load tests based on ASTM D 1143 were carried on two of the geopiers to determine their moduli.

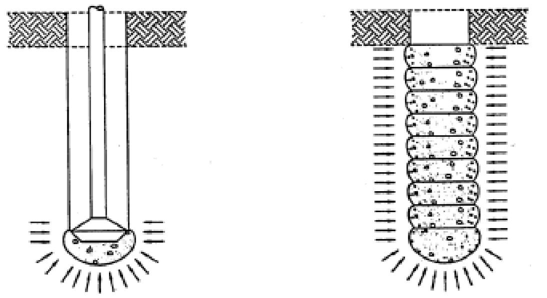
3.2. Instrumentation

Resistance wire strain gauges were attached to each layer of the geogrids to measure the strain of the geogrids at three locations: the edge of the geopier, half-way between the geopiers, and at the centre of the square pattern of geopiers. The strain gauges were positioned in the transverse and longitudinal directions to the embankment. Settlement plates in Sections 1 and 2 were set directly above the geopiers and at the centre of the square pattern of geopiers directly on every layer of the geogrids to measure settlement at each layer. The differential settlement at each layer of geogrid is the difference between the settlement measured at the top of the geopier and the settlement of the layer at the centre of the square pattern of geopiers. In Section 3, only one pair of settlement plates was placed directly on the second layer of the geogrid on the geopier, and also at the centre of the square pattern of geopiers, because the spacing between the top and bottom layer geogrids was small. In Section 4, the settlement plates were placed below the concrete slab: one on the geopier and another on the soil. Figure 5 shows the locations of the settlement plates and the other instruments installed. For the control sections C1 and C2 the settlement plates were placed on the soil. Earth pressure cells were positioned on the geopiers and in between the geopier. Piezometers and extensometers were placed at different depths under all the sections. Most of the measuring points for the piezometer and the extensometer were located within the soft subsoil layer. The deepest measuring point for the piezometer and the extensometer was close to 22 m below the working platform grade. Vertical inclinometers were placed at the toe of the embankment in between the geopiers, and horizontal inclinometers immediately under and across the embankment in between the geopiers. All of these instruments were placed in every major section, but the control



1. Excavate cavity

2. Place single lift well-graded stone



3. Form bottom bulb

4. Place thin lifts of aggregate and ram with bevelled tamper

Figure 4. Installation process for a geopier element (after Fox and Cowell 1998)

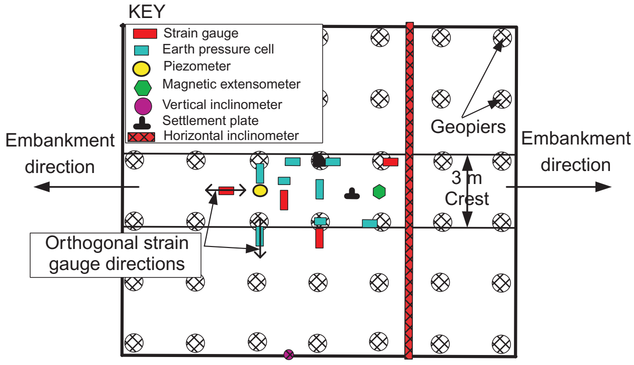


Figure 5. Typical locations of strain gauges and other instruments

sections were not instrumented except for the settlement plates.

The geogrids were instrumented with strain gauges. The major concern with regards to strain gauges attached to geogrids is to ensure that the gauges are still functional after the placement of aggregate and fill over them. Hayden *et al.* (1999) reported that approximately 50% of the strain gauges were broken after placement of the pavement, and the gauges continued to fail as time progressed. For this reason special protection was accorded to the strain gauges against moisture and the abrasive aggregate in this case. In addition to protection of the strain gauges, their number was doubled, so that even if half of them failed there still would be an adequate number of working gauges.

As there were two different geogrids, two different methods were employed for installation of the strain gauges. The method of placement of strain gauges on the biaxial geogrid (Sections 1 and 2) was modified from the work of Hayden *et al.* (1999). The gauges were glued to a geomembrane because the ribs of the biaxial geogrid were too small for them to be fixed directly to the geogrid. The geomembrane, cut in a dog-bone shape, was made up of the same polymer as the geogrid and had approximately the same cross-sectional area as the removed geogrid ribs, as shown schematically in Figure 6. The geomembrane was attached to the geogrid using a pair of thicker geomembranes at each end of the dog-bone geomembrane, which were secured together using steel bolts and nuts. Over the strain gauges, a thin layer (approximately 1 mm) of neoprene rubber waterproof coating was placed. This was followed by a layer of bentonite powder approximately 20 mm thick. The bentonite powder was overlaid with a layer of geotextile before finally placement of some sand to cushion against the aggregate. As strain gauges were glued to both sides of the geomembrane, similar protection was provided for the bottom strain gauge, as shown in Figure 6. In Section 3, where a uniaxial woven geogrid was used, the strain gauges were glued directly on top (only) of the geogrid, because the rib was wide enough to accommodate the strain gauge. The glue used was a single-component room-temperature-curing cyanoacrylate glue for both the biaxial extruded and the uniaxial woven geogrids. Similar protection layers were provided for the strain gauges in Section 3 as in Sections 1 and 2.

Laboratory tensile tests were carried out on both types of geogrid with strain gauges attached to them to establish whether the measured strains in the geogrids were the same as the strains obtained from the strain gauges. The results showed that for both types of geogrid there was a linear relationship between the measured strains in the geogrid and those from the strain gauge. After more than six months, approximately 70% of the 128 strain gauges installed were still in working condition.

All the strain gauges in the test embankment were placed in the middle area of each section whenever possible. Strain gauges measure strain in one direction: therefore half of the strain gauges were aligned in the transverse direction and the other half were aligned in the longitudinal direction to the embankment. The selected aggregate used in the LTPs consisted of well-graded crushed granitic rock with fine material that was less than 3%. The aggregate is normally used as a sub-base layer for road pavements. Over the aggregate blanket, the embankment was constructed using gravelly sandy clay.

4. RESULTS AND DISCUSSION

Monitoring of most of the data began before the construction of the LTPs, and continued six months after completion of the embankment in February 2005. The results presented for this study are based on the data collected over 6 months following the construction of the test embankment.

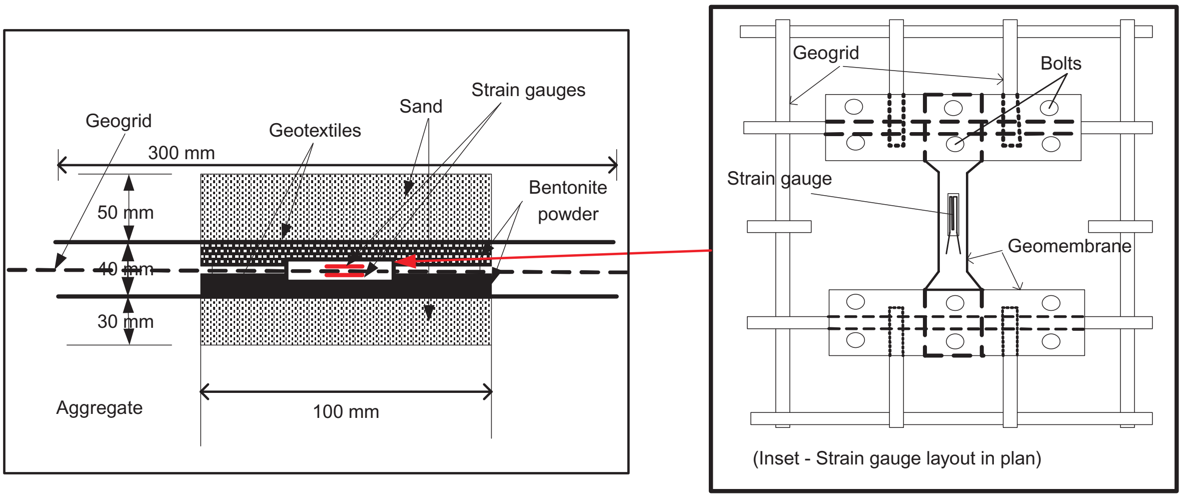
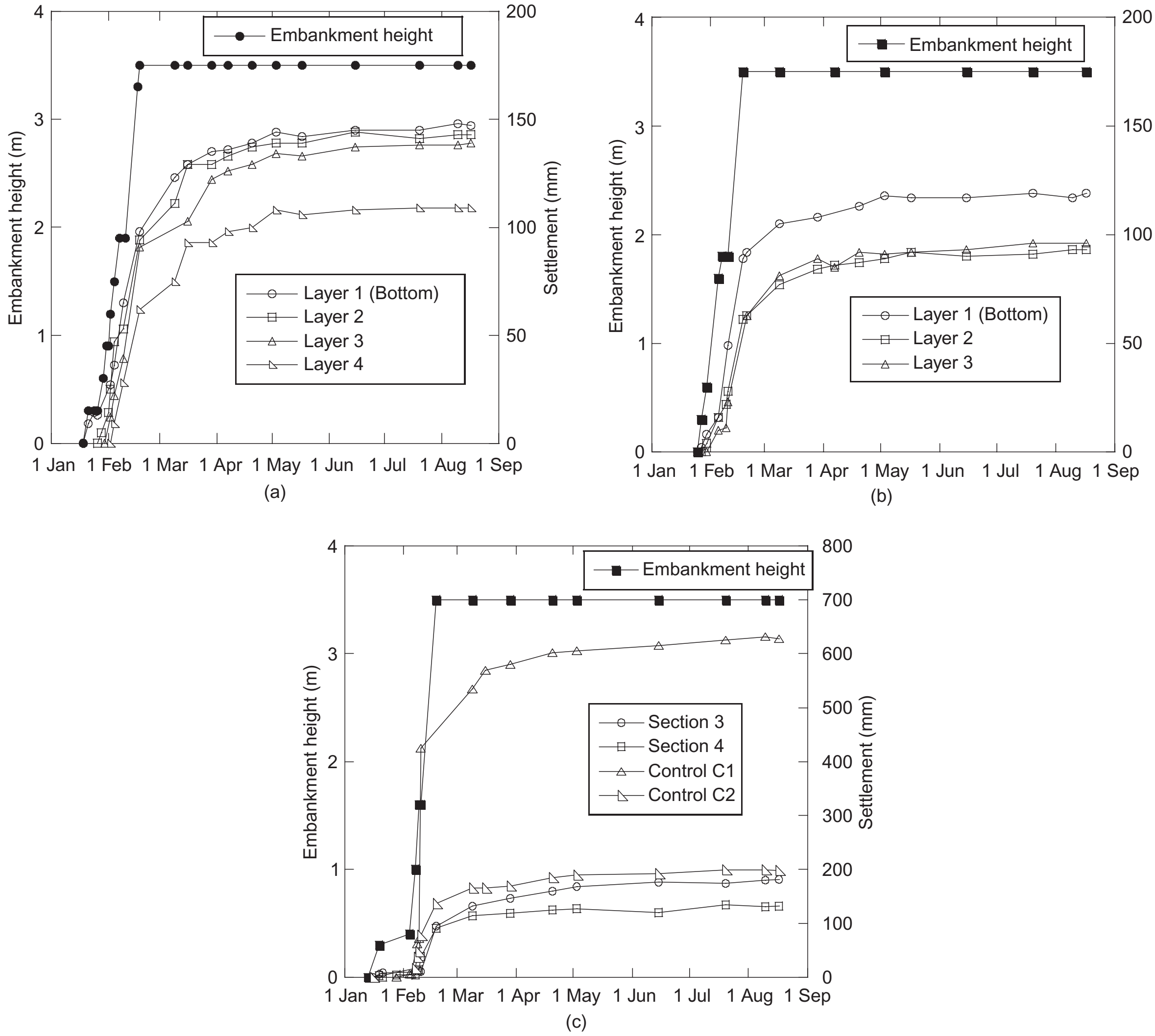


Figure 6. Schematic diagram of strain gauge installation and protection

4.1. Settlement of the embankment

Figure 7 shows the total settlement at different layers of the geogrid within the LTPs in Sections 1 and 2, and the total settlement at the base of embankment in Sections 3 and 4 in the centre of the square pattern of geopiers. The plots also indicate that generally more than 90% of the total settlement had taken place within 2 months after completion of the test embankment. During and after the



(a)

(b)

(c)

Figure 7. Total settlement of major LTP sections (at centre of square pattern of geopiers) and control sections: (a) Section 1; (b) Section 2; (c) Sections 3, 4, C1 and C2

construction of the embankment the porewater in the subsoil under all the major sections did not show any significant increase, benefiting from the geopiers acting as vertical drains. The increase of porewater pressure in the subsoil was less than 10 kPa.

The measured settlement at various locations on the embankment surface was greater than at the base of the embankment. The large surface settlements are attributed to embankment fill that was not well compacted because of the large number of instruments and instrument casings, which made proper compaction difficult. The total settlement of the bottom-layer geogrids is higher than that of at the layers above it in Sections 1 and 2. In Section 1, where the vertical distance between the bottom and the top layer geogrid is 900 mm, the difference in total settlement is approximately 40 mm. For Section 2, where the vertical distance between the bottom and the top layer geogrid is 600 mm, the total settlement difference is approximately 20 mm.

Table 2 shows the end point (6 months after completion of the embankment) total settlement at the base of the embankment on the geopiers and on the subsoil in the centre of the square pattern of geopiers, as well as the differential settlement in all sections. The total settlements for Sections 2 and 4 are almost the same: the difference can be explained by the minor local variations of the subsoil underneath each section. The total settlement in Section 1 could be expected to be higher than all the other sections because the geopiers spacing is larger, that is, fewer columns. However, the highest total settlement of the embankment is recorded in Section 3. Closer inspection of the subsoil profile indicates that the soft and firm layers are approximately 16.5 m under this section as opposed to 8.5–10 m under the other sections. Furthermore, as will be shown later, the lateral displacements are also higher in Section 3, potentially contributing to the observed larger vertical settlements.

The differential settlement is small, with the largest being about 30 mm in Section 3 (i.e. the catenary LTP). In the other sections the differential settlement ranges from 4 mm to 16 mm. It can be expected that the use of less stiff columns to support embankments generally will result in higher total settlement but less differential settlement at the base of the embankment compared with stiffer columns. This is supported by numerical studies (Han and Gabr 2002). The differential settlement above the bottom layers in Sections 1 and 2 was 5 mm or less: in other words, the differential settlement from the base up either decreases or remains the same as the differential settlement at the base of the embankment in the beam LTPs.

The large differential settlement at the base of the embankment between columns may be reflected at the surface of the embankment and cause unwarranted maintenance problem, especially if the embankment height is low. Large differential settlement, if not attended to in the design of the geosynthetic reinforcement, can cause excessive strain (including creep strain) in the geosynthetic, which will further exacerbate the differential settlement. The Nordic method suggests that the differential settlement should not exceed 150 mm, whereas the other methods are silent on the allowable differential settlement. In this context, the differential settlement is considered large when it exceeds 150 mm.

The current design methods for geosynthetic-reinforced LTPs supported on columns assume that the columns yield very slightly under the weight of the embankment and the surcharge loads. The German and Collin methods allow for support by the subsoil; the British and Nordic methods disregard it.

In embankment support applications the geopier elements may be allowed to settle more than 25 mm, the maximum settlement for which they are normally designed for as foundations for structures. In highway embankments a high total settlement of the embankment may in itself not be detrimental to the roadway if it is not accompanied by high differential settlement at the surface of the embankment, provided it is not abutted to an unyielding structure or an existing embankment that has ceased to settle. Some highway specifications allow for high post-construction total settlement of embankments (Lin and Wong 1999) but are very stringent in specifying the embankment surface differential settlement, usually with an angular rotation of less than 1%. The Public Works Department of Malaysia (1985) allows for 250 mm post-construction settlement and a surface angular rotation of less than 1% in the geotechnical design criteria for roadwork. If the columns are properly designed, the total settlement may not be difficult to control, but the differential settlement that causes angular rotation can be critical in the case of low embankments because the arching in the fill may not be fully developed. Therefore it is extremely important to control the differential settlement between the columns and the subsoil in order to avoid continual roadway surface maintenance. For high embankments, the surface differential settlement may not pose any serious problem if the embankment fill is well compacted (i.e. the differential settlement at subgrade may be reduced when it reaches the surface for high embankments). Therefore the use of more compressible columns such as geopiers, stone columns and deep cement mixed

Table 2. Total and differential settlements at base of embankment in different sections

	Section 1	Section 2	Section 3	Section 4	Section C1	Section C2
Total settlement on geopier (mm)	143	108	152	116	NA	NA
Total settlement on soil (mm)	147	119	181	132	628	198
Differential settlement (mm)	4	11	29	16	NA	NA

NA = Not applicable

columns together with geosynthetic-reinforced LTPs can be effective in reducing the total and differential settlement, especially for low embankments.

4.2. Strains in the geogrids

The tensile strain in the geogrid is usually the main controlling parameter in the design of LTPs supported on columns. The strain also indirectly measures the force in the geogrid. A review of geogrid strains in the test embankment indicates that the strains measured at the edge of the geopiers are consistently higher than those measured midway between the columns. Numerical analyses by Pham *et al*. (2004) and Han and Gabr (2002) show that maximum tensile strain in the geogrids occurs at the edge of the columns and minimum tensile strain occurs at the centre between the columns, supporting the field observation. The strains also vary with the geogrid layer as well as with the LTP section. The magnitude of strain in a geogrid depends upon the initial localised stress induced during the placement of the geogrid, the additional stresses applied by the loads, and the interaction between the geogrid and the aggregate. Because of the above factors, it is unlikely that the strain measured at different locations, under what is assumed to be the same conditions, will be the same. A case study reported by Rogbeck *et al*. (1998) showed that the measured strain of geogrid placed at different locations under similar conditions ranged between 0.5% and 4.5%. They also reported that, in the same area under similar conditions as in the first case, but where differential settlements were smaller (because of soil support), the resulting geogrid strains were −0.2% to 0.8%. Jenner *et al*. (1998) reported a similar finding: the strain in the geogrid under the same conditions varied from 0.5% to 2.8%.

The measured transverse geogrid strains at the edge of the geopiers are shown in Figure 8 as the embankment was built and subsequently. The strain for each layer is measured at a point directly above the geopier edge. The strains continued to increase after the completion of embankment construction before they essentially stabilised at nearly constant values. In the test embankment, approximately 60% of the strain gauges were placed at the

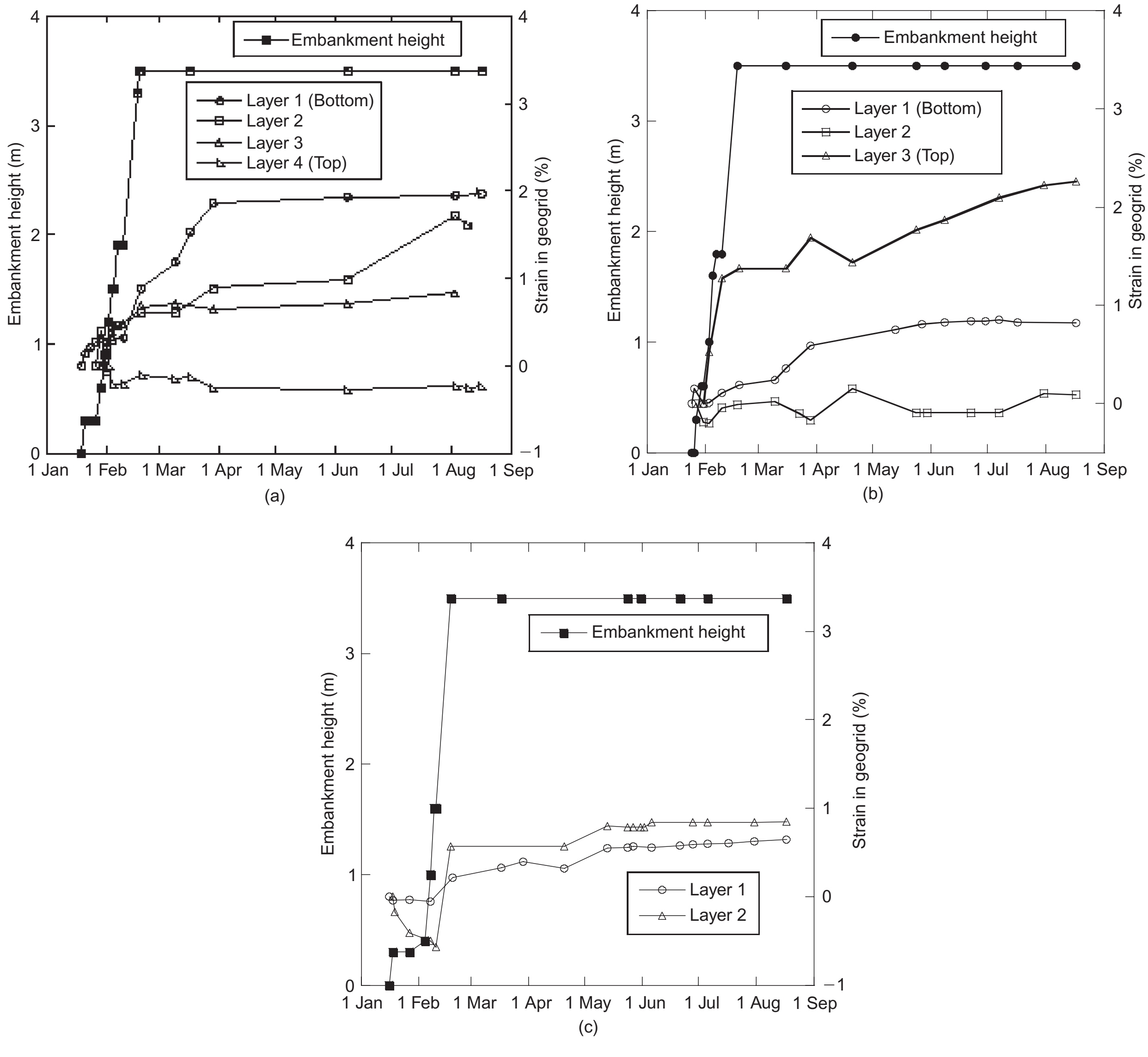


Figure 8. Strain in geogrids (at edge of geopier) in transverse direction at same geopier position but in different layers: (a) Section 1; (b) Section 2; (c) Section 3

edge of the geopiers, and the remaining 40% were placed midway between adjacent geopiers, in either the transverse or longitudinal direction.

The strains measured in Section 3 indicate that those in the top layer (Layer 2) are higher than those in the lower layer (Layer 1) (both measured in the transverse direction). These layers were separated by only by a 75 mm-thick aggregate; the top layer had its stronger axis (i.e. the machine direction of the uniaxial geogrid) in the longitudinal direction, and the lower layer in the transverse direction. Therefore the lower layer was intended to be the main reinforcement. It appears that the top geogrid contributes to reinforcement in the transverse direction. In this case, the tensile force is estimated to be 12.1 kN/m in the Layer 1 geogrid and 3.0 kN/m in the Layer 2 geogrid, based on the modulus of the uniaxial geogrid in the two axes. The tensile force resistance provided by the geogrid in the cross-machine direction (the low-strength direction) is approximately 25% of that provided in the machine direction (the higher-strength direction) of the geogrid. However, it is possible that at higher stress levels the contribution by the geogrid in the cross-machine direction will diminish.

The final strains in the transverse direction in the geogrids at the edge of the geopiers, and between the geopiers for all layers in Sections 1, 2 and 3, are summarised in Table 3. The strains in the geogrids in the longitudinal direction are not tabulated. There was insignificant difference between the tensile strain in the transverse and the longitudinal directions, but the strains in the transverse direction were mostly higher.

In Section 1, the general trend points to a reduction in the average tensile strain in the geogrids at the edge of the geopiers from the bottom layer to the top. Occasionally there are compressive strains recorded at the topmost geogrid layer. The measured strains between the geopiers do not show any specific pattern. The tensile strain seems to vary in a small range with the geogrid layer, except at the top layer, where the tensile strain magnitude is lower than for the rest of the layers. The maximum strain recorded in the geogrids (3.16%) is still smaller than that assumed in the beam LTP design (i.e. 5%). The reduction in strain in the consecutive geogrid layers from bottom to top indicates that correct assumptions are made in the design of the beam LTP: that is, the tensile force in the geogrids gets smaller in the upper layers. The strains in the geogrids at the edge of the geopiers are higher than the strains between the geopiers.

In Section 2, the strain in the geogrids at the edge of the geopiers is higher at the bottom and the top geogrid layer than at the middle layer. The strains in the geogrids between the geopiers are lower than the strain at the edge of the geopiers. Large compressive strains were recorded between the geopiers for geogrid Layers 2 and 3. The reason for this is unknown, but it is likely to be due to the initial conditions during placement of the geogrids.

The data for Section 3 also demonstrate that the tensile strains in the geogrid are higher at the edge of the geopiers than in between the geopiers. However, the tensile strain in the stronger machine direction of the uniaxial geogrid (transverse in Layer 1; longitudinal in Layer 2) is higher in the bottom layer than the top layer, indicating a slightly higher lateral force acting in the transverse direction. Theoretically, the tensile strain in the transverse direction should be much higher than in the longitudinal direction, because in the transverse direction the lateral thrust of the embankment contributes to higher tensile strain in that direction.

The design and field tensile forces in each layer of the geogrid are compared in Tables 4 and 5. The field tensile forces given in Tables 4 and 5 were calculated from the longitudinal tensile strains observed at the edge of the geopiers and the geogrid modulus in the same direction. The longitudinal strains were used because in this direction the geogrid strain is not significantly influenced by the lateral spreading of the embankment that occurs in the transverse direction. The design tensile forces were also calculated from the specified tensile strains in each design method for load transfer and the relevant geogrid modulus. In all of the methods, except for the German method, the maximum design strain is specified in the calculation, i.e., 6% for the British and Nordic design methods and 5% for the Collin design method. The German method also limits the strain to 6%; however, the limiting strain value is not explicitly used in the calculation of the tensile force. Table 4 shows that all the design methods for the catenary LTP

Table 3. Strain in transverse direction in geogrids at edge of geopiers and between geopiers

Geogrid layer	Location of strain gauges	Range and average of strains in each layer of geogrid (%)					
		Section 1		Section 2		Section 3	
		Range	Avg	Range	Avg	Range	Avg
Layer 1 (bottom)	Edge of geopier	1.82–3.16	2.32	0.82–1.67	1.26	0.25–1.09	0.66
	Between geopiers	−0.57–1.15	0.59	−0.7–2.31	0.96	0.19–0.83	0.55
Layer 2	Edge of geopier	1.06–2.66	2.07	0.09–1.16	0.61	0.85	0.85
	Between geopiers	−0.26–1.15	0.68	(−2.47)–(−0.01)	−1.26	−0.06–0.49	0.22
Layer 3	Edge of geopier	1.23	1.23	0.83–2.26	1.40	NA	NA
	Between geopiers	0.04–1.25	0.82	−3.93–1.24	−0.96	NA	NA
Layer 4	Edge of geopier	−1.88–2.22	0.37	NA	NA	NA	NA
	Between geopiers	−0.43–0.15	-0.14	NA	NA	NA	NA

NA = Not applicable

Table 4. Comparison of design and field tensile forces for catenary design (Section 3)

Basis for tensile force	Tensile force (kN/m)
Field (maximum value)	15
British method	141
Nordic method	77
German method[a]	47–112

[a] Range of tensile forces varies depending on Young's modulus of subsoil used.

overestimate the tensile force in the geogrids. The small differential settlements given in Table 2 are also indicative of the low tensile strains measured in the geogrids. In the German design method the tensile force in the geogrid depends on the modulus of the subsoil. The values of tensile force from the German method are based on the minimum and maximum values of the modulus of the subsoil obtained from laboratory and field tests (i.e. a lower subsoil modulus results in a higher tensile strain in the geogrid, and vice versa). The British and Nordic methods do not consider the support provided by the subsoil: therefore these methods are expected to be conservative. The tensile forces in the geogrid from the British and Nordic methods are approximately five and nine times the field tensile force, respectively. For the German method, the calculated tensile force is between three and 7.5 times the field value. For the beam LTPs, the calculated tensile force in the geogrid layers is generally within the range of values observed in the field (Table 5).

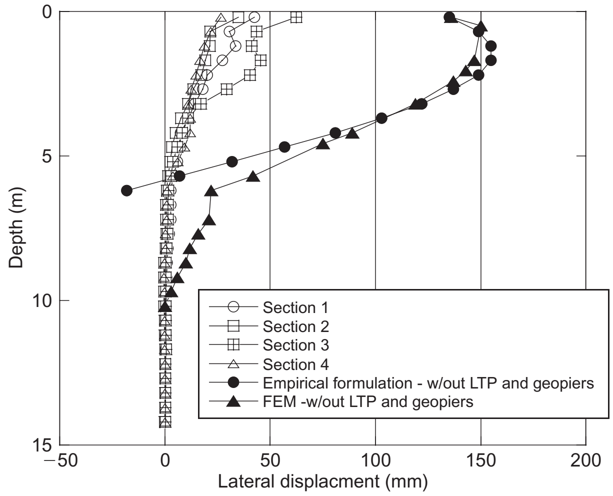


Figure 9. Lateral displacement of subsoil at different sections

4.3. Subsoil lateral movement

The lateral shear displacement of the foundation at the edge of the embankment is shown in Figure 9 as obtained from the vertical inclinometer data. The data indicate that the use of LTPs supported on columns limits the lateral shear displacement of the subsoil at the edge of the embankment. The beam LTP appears to perform slightly better in reducing lateral displacement than the catenary LTP when the spacing of the geopiers is the same (Section 2 compared with Section 3). Even when the geopier spacing is higher (Section 1), the magnitude of lateral displacement of the beam LTP is still smaller than for the catenary LTP (Section 3). Also shown in Figure 9 are the lateral shear displacement without a load transfer platform from an empirical relationship given by Bourges and Mieussens (1979) and the finite element analysis performed by Plaxis 7.2 based on the subsoil properties for the embankment not supported by an LTP and geopiers. The lateral subsoil shear displacements without the LTP and the geopier could be significantly (three to four times) higher. Li *et al.* (2002) stated that no procedure is currently available to predict the effect of column-supported LTPs on lateral displacement of the subsoil. This is probably a result of the common assumption that lateral displacement is no longer a critical factor. The use of geosynthetic-reinforced LTPs with softer columns also results in relatively small lateral displacements, and they are not expected to exert significant lateral forces on the foundation of existing adjacent structures.

4.4. Construction cost

The actual cost of construction of the beam LTP (Sections 1 and 2) per m^2 is less than that of the catenary LTP (Section 3), largely because of the high cost of the high-strength geogrid used in the catenary LTP, almost five times that of the low-strength geogrid. Although the high-strength uniaxial geogrid is easier to place and spread, it requires more personnel for handling because it is heavier. Based on the Malaysian construction costs, the embankment supported on the catenary LTP is approximately 8% more expensive than the one constructed over the beam LTP that has the same geopier spacing. In increasing order of cost, the cheapest is the reinforced concrete LTP,

Table 5. Comparison of design and field tensile forces for beam design (Sections 1 and 2)

Layer number	Section 1		Section 2	
	Field tensile force (kN/m)	Design tensile force (kN/m)	Field tensile force (kN/m)	Design tensile force (kN/m)
Layer 1	2.9–6.2	4.9	0.7–6	3
Layer 2	3.1–6.8	3.4	2.3–3.2	1.6
Layer 3	0.6–3.0	2.1	1.3–5.5	0.5
Layer 4	0–3.1	0.7	NA	NA

NA = Not applicable

followed by the beam LTP in Section 1, the beam LTP in Section 2, and (the most expensive) the catenary LTP in Section 3. The cost includes labour and materials for the geopier foundation, the aggregate blanket, the embankment fill above the LTP, and the geogrids. The cost of the concrete LTP is low because material and labour costs in Malaysia for concrete work are lower than in other countries such as the USA. However, in tropical countries like Malaysia, where thunderstorms are unpredictable and occur almost daily, quality control of the concrete slabs during the rain is extremely difficult. Section 1 is cheaper than the other geosynthetic-reinforced LTP sections because the spacing of the geopiers is larger, thus offsetting the cost of the additional geosynthetic and the thicker aggregate LTP. A factor that needs to be considered in the Collin method is the requirement of select aggregate in the LTP, which can be very costly, especially if the distance to the source is far. The cost of the geogrid (the material cost only) in Section 1 is 13% of the total cost of this section, for Section 2 it is 10%, and for Section 3 it is 27.5%. In the USA the geogrid cost would be lower but the labour costs would be higher. The cost is also a function of column length. If columns are longer the LTP cost is a smaller percentage of the overall cost.

5. CONCLUSIONS

Evaluation of the performance of different embankment load transfer platforms supported on geopiers is presented in terms of total and differential settlements, geogrid strains and forces, subsoil lateral displacements, and cost.

- The use of LTPs together with rammed aggregate piers (geopiers) as the columnar elements in supporting an embankment reduces the total settlement of the embankment; more importantly, the differential settlement between the columns and the foundation subsoil is very small. The measured settlements within the beam LTPs are reduced or remain the same from the base up within the LTP. The use of more compressible columns (e.g. geopiers) together with LTP would be suitable for low embankments, as this system results in low differential settlements.
- The tensile strains measured in the geogrids indicate that they are significantly higher at the edge of the geopier elements than midway between the geopiers. The strains in the different layers of the geogrid vary. Comparison of the field tensile forces estimated from the maximum strain in the geogrids and the design tensile forces shows that the various design methods overestimate the expected tensile forces by a very large margin (more than three times) in the case of the catenary LTP. For the beam LTP, the field and design tensile forces are more comparable.
- The lateral shear displacement of the subsoil with the LTP system is small. Finite element analysis and empirical calculations show that without the LTP and the geopier subsoil the lateral shear displacements could be significantly (three to four times) higher.
- Construction costs for geosynthetic-reinforced LTPs are likely to vary with locality; however, it appears that beam LTPs offer a less costly approach with enhanced performance.

All the current design methods for geosynthetic-reinforced LTPs have provided a safe design, but they can be refined so that the construction cost can be further reduced; in particular, a less conservative approach for catenary LTPs (British, Nordic and German design methods) seems to be warranted.

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