A63 SELBY BYPASS - DESIGN AND CONSTRUCTION OF A 1.6 KM GEOSYNTHETIC REINFORCED PILED EMBANKMENT

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ABSTRACT: The A63 Selby bypass comprises 10km of new road around the town of Selby in North Yorkshire, UK. High strength, high modulus geosynthetics with initial ultimate tensile strengths of up to 1600 kN/m were incorporated over a 1.6 km long section of piled embankment construction on very poor ground across the River Ouse flood plain to cope with embankment heights of up to 9.5m. This paper addresses some of the aspects of design and construction with emphasis on the particular limitations of BS 8006 with respect to serviceability limits, pile/geosynthetic strain compatibility and the implications for pile and geosynthetic design. The paper also describes the installation of instrumentation to monitor reinforcement strains and lateral movement in the piles in an attempt to achieve a better understanding of in service geosynthetic loads and the lateral movement of pile supported embankments.

1 INTRODUCTION

A bypass around the market town of Selby in North Yorkshire, UK, is being constructed to ease traffic congestion caused by the convergence in the town centre of two major roads, the A19 from York to Doncaster and the A63 from Leeds to Hull.

The scheme is being built using a design and construct contract for the Highways Agency following Skanska Construction UK's successful tender to construct the road with High-Point Rendel as designer. Construction started in 2002 and it is anticipated to be open in early 2004.

The route of the bypass runs eastwards from the A63 south west of Selby to the A19 near Barlby. The route crosses the Selby to Hull and Selby to Doncaster Railway lines, Selby canal and the River Ouse, the site of a new swing bridge. The road comprises 10 km of single carriageway highway, generally on shallow embankment although there is a 13m deep cutting through Brayton Barff. However, on the approaches to the bridges near the canal and across the River Ouse flood plain the alignment is constructed on embankment up to 9.5 m in height.

2 EMBANKMENT DESIGN

Design criteria were given for embankments for settlements measured at the end of the five-year maintenance period. These included a maximum allowable settlement of 75 mm and a maximum differential settlement gradient along the carriageway of 1 in 500.

For the shallow embankments, no special foundation treatment was required, although pulverised fuel ash was used in the construction of high embankments adjacent to the canal to minimise ground loading. However, the embankments across the River Ouse flood plain are constructed over thick variable alluvial deposits. It was therefore clear that detailed consideration of embankment foundation design would be necessary to meet the design criteria for settlement.

2.1 *Ground Conditions*

A detailed ground investigation across the flood plain proved typically between 5 m and 8 m of alluvium overlying laminated clay. Sherwood Sandstone bedrock was generally identified at around 15m below ground level.

The upper stratum of the alluvium comprises a desiccated crust of silt and clay between 1m and 2m thick. This is underlain by highly variable deposits of peaty and clayey materials. These comprise between 4m and 6m of soft to very soft highly compressible soils with moisture contents between 50% and 350%. Below the peaty deposits a thin stratum of lacustrine sand was generally identified overlying firm laminated clays. The laminated clay is typically 7m thick overlying the sandstone. Figure 1 shows a simplified long section of the ground conditions across the area of the flood plain.

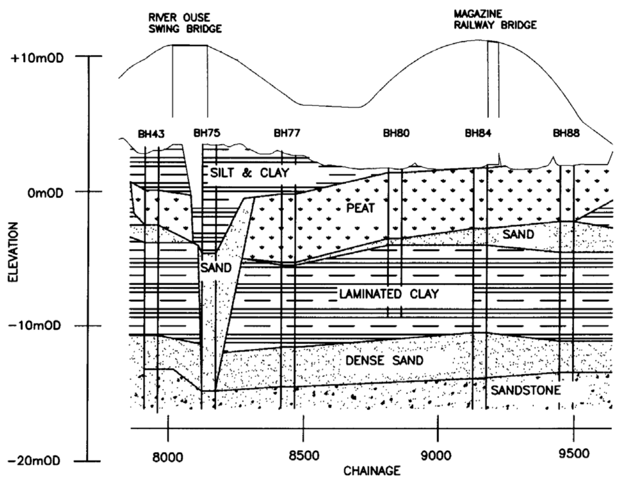

Figure 1 Ground Conditions

2.2 *Form of Embankment Construction*

At an early stage during the tender design period, a detailed comparison of the advantages and disadvantages of supported and unsupported embankments across the flood plain was made. This considered the relative cost, programme and design certainty of the different approaches. Following detailed discussions between the designer and contractor, the decision was taken to adopt a form of supported embankment.

3 TENDER DESIGN

The embankment footprint across the Ouse flood plain was 60,000m^2 and a key criterion for the foundation design was to minimise the number of supporting elements in order to reduce the construction programme for the embankment. As a result, it was decided to adopt piled embankments. During the tender process the outline design for the piled embankment was based on the approach described in BS 8006, BSI (1995) for the design of the piles and determination of both the longitudinal and transverse geosynthetic reinforcement above the piles.

BS 8006 assumes that all the loading of the embankment will be transferred through the piles to a firm stratum. Basal reinforcement spanning across the pile caps transfers the embankment loadings onto the piles, although ignoring the contribution of adjacent soft soils in supporting the embankment. The reinforcement also resists the lateral thrust of the embankment and hence avoids the need for raking piles at the edge of the embankment. The reinforcement allows optimisation of the piles with regard to alignment, spacing, cap size and steel reinforcement within the piles.

At tender stage Skanska's foundation partner, Cementation Foundations, proposed the use of driven-cast-in-situ piles, founded in the dense granular deposits above the Sherwood Sandstone, as the most appropriate piling method. This pile type had the advantages of permitting an integral "mushroom head" pile cap to be constructed simultaneously with the pile and of accommodating local variations in the surface of the Sherwood Sandstone. The geosynthetic design used the approach described in BS 8006 with layers of high strength uniaxial geogrids spanning across the piles longitudinally and transversely. Figure 2 shows a typical section of the embankments.

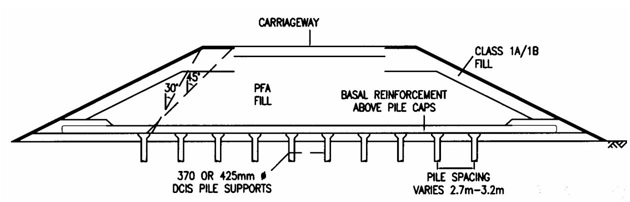

Figure 2 Pile Supported Embankment Section

4 DETAILED DESIGN

Following contract award the design approach for the design of the reinforcement and piles was refined based on the principles set out in BS 8006 using ultimate and serviceability limit state criteria.

4.1 *Geosynthetic Design*

The design considers the two components of stress which influence the required strength of the geosynthetic reinforcement; the weight and vertical surcharge (influencing both longitudinal and transverse reinforcement) and the horizontal thrust associated with this vertical stress (influencing the transverse reinforcement only). The basal reinforcement considered in the detailed design comprised two layers of uniaxial geogrids laid one on top of the other above the pile caps. This consisted of transverse strips of reinforcement spanning over the pile caps and continuous sheets of reinforcement running longitudinally along the embankment.

4.1.1 *Distribution of Vertical Load*

BS8006 notes that the ratio of vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment (p'_c/σ_v) may be estimated by the use of Marston's formula for positive projecting subsurface conduits. It was felt that this method did not adequately model the 3D arching of soil between the grid of the pile caps. In particular, the method described in BS 8006 effectively defines a limiting pressure acting on the geosynthetic equivalent to that of an embankment of height equal to 1.4 times the clear span between the piles.

Alternative methods of determining the stress across the embankment were investigated, including those compared by Russell (1997) & Love (2003). It was considered that the approach outlined by Hewlett & Randolph (1988) was the most rational for determining the distribution of stress on the base of the embankment. This method also allowed the pressure acting on the geosynthetic layers to be determined directly. A general overview of possible design procedures is given by Alexiew (2002).

4.1.2 *Load in Geosynthetics*

The loads in the geosynthetics from the vertical component of embankment load were calculated from the revised approach discussed in Love (2003) for uniaxial reinforcement. The pressure acting on the reinforcement, σ_{si}, is taken to be uniformly distributed between the piles and for pile spacing s and pile cap width a, the average distributed load (W_T) on reinforcement strips spanning between pile caps is:

$$W_T = \sigma_{si} \cdot (s^2 - a^2) / 2(s-a) \qquad (1)$$

Note that equation 1 corrects a typographical error in Wood (2003). However, for layers of uniaxial geogrid, the distribution of load onto the spans between pile caps is not uniform. The equivalent load W_T in the transverse and longitudinal reinforcement was calculated relative to the average distributed load assumed in equation 1. For equal longitudinal and transverse spacing of the pile caps, this results in:

$$W_{Ttrans} = W_T\ (2s) / (s+a) \qquad (2)$$

$$W_{Tlong} = W_T\ (2a) / (s+a) \qquad (3)$$

At locations where the longitudinal and transverse spacing of the piles was different a revised approach was adopted. The distribution of load on to the piles was calculated using the Hewlett and Randolph approach assuming a pile spacing equal to the average of the longitudinal spacing s_L and transverse spacing s_T. This was considered a reasonable simplification as the load transfer tends toward a 2D case. Similar to the situation above, the distributed load in the geosynthetics was then calculated as:

$$W_{Ttrans} = W_T\ (s_T+s_L) / (s_L+a) \qquad (4)$$

$$W_{Tlong} = W_T\ (2a) / (s_T+a) \qquad (5)$$

4.1.3 *Design Strength for Geosynthetics*

The calculation of tension (T_{rp}) in the reinforcement as a result of the vertical load was based on the approach shown in BS 8006 assuming serviceability and ultimate limit strains. The design calculated the factored strength requirement at both service and ultimate limit states based on partial material factors described in BS 8006 and this was checked for strain compatibility against the isochronous stress strain curves for the reinforcement.

In addition, the transverse reinforcement is required to resist the lateral thrust of the embankment fill. The tension in the reinforcement, per metre run, required to resist to outward thrust of the embankment (T_{ds}) was calculated in accordance with BS 8006:

$$T_{ds} = 0.5K_a \cdot (f_{fs} \cdot H \cdot \gamma + 2 \cdot f_q \cdot \omega_s) \cdot H \qquad (6)$$

Where K_a is the active earth pressure coefficient, γ is the unit weight of fill, H is the height of embankment, ω_s is the uniform distributed surcharge load and f_{fs} and f_q are partial load factors (from table 27, BS 8006).

BS 8006 indicates that the transverse reinforcement should be designed for the sum of the two components of tension ($T_d = T_{rp} + T_{ds}$). However, as proposed by Love (2003) the reinforcement was designed only for the greater of the tensile force to transfer loading on to the pile caps (T_{rp}) and the tensile force to resist lateral sliding of the embankment (T_{ds}).

The design force (per metre run) can be multiplied by the pile spacing (s) in the longitudinal direction, to provide the required force to be resisted by geosynthetic reinforcement concentrated on transverse strips across the piles, equation 7. These reinforced strips act as buttresses to resist lateral movements of the overlying embankment.

$$T_{ds} = 0.5K_a \cdot (f_{fs} \cdot H \cdot \gamma + 2 \cdot f_q \cdot \omega_s) \cdot H \cdot s \qquad (7)$$

4.2 *Pile Design*

The piles adopted to support the embankments were driven-cast-in-situ (DCIS) piles. DCIS piles are formed by driving a hollow steel tube, fitted with an expendable steel shoe, to the required depth, or "set". On reaching the required founding criterion a full length reinforcing cage is inserted and concrete is placed in the tube. The tube is then retracted to form the pile in-situ. During the initial driving of the tube a former is used to create a void for the mushroom head enlargement that forms the pile cap. The nominally 900mm diameter head is cast immediately following extraction of the pile driving tube to form an integral element of the pile. The head was reinforced to resist both accidental wheel loads from construction traffic prior to embankment fill placement and the unequal loading imposed by the geosynthetic reinforcement spanning between the pile caps.

The piles were essentially end bearing on the dense granular deposits of sand overlying the sandstone bedrock. The piles were designed to be settlement reducing and consequently an allowable settlement of 25mm under service loading was considered acceptable when taking into account the flexible nature of the embankment and the high dead load component of loading. Pile diameters of 370mm and 425mm were used, and after making allowance for negative skin friction effects, pile spacing was maximised to ensure efficient use of the structural capacity of the pile and the geosynthetic reinforcement. Final pile design lengths were determined by a series of trial drives and preliminary load tests prior to construction of the working piles. Tests on working piles were carried out to confirm compliant performance.

5 SERVICEABILITY LIMIT STATES

BS 8006 suggests that *"Serviceability limit states are attained if the magnitudes of deformation occurring within the design life exceed prescribed limits…"* (Clause 2.2) and that these limits are normally prescribed in terms of acceptable deformations at the face or surface of the reinforced soil structures, slopes or embankments. BS 8006 attempts to ensure that the serviceability of the final embankment is not compromised by limiting the theoretical mobilised strains of geosynthetic reinforcement, both during construction and throughout the service life.

5.1 *Reinforcement Strain due to Vertical Loading*

Strain of the geosynthetic spanning between the piles is necessary to generate a tensile load to resist the pressure acting on it. For the serviceability limit state concerning vertical displacements between the piles, BS 8006 states *"The maximum allowable strain in the reinforcement ε_{max}, should be limited to ensure differential settlements do not occur at the surface of the embankment…This can be a problem with shallow embankments where the soil arch cannot develop fully within the embankment fill."* (Clause 8.3.3.10)

For basally reinforced pile supported embankments BS 8006 imposes an upper limit of 6% initial strain to ensure that all the embankment loads are transferred to the piles. In addition a maximum creep strain of 2% may be allowed over the design life of the reinforcement to ensure that long-term localised deformations do not occur at the surface of the embankment.

These limits appear somewhat arbitrary and/or perhaps influenced by the normal working stress levels and creep strain of many proprietary geosynthetic materials currently available, and less influenced by surface settlement criteria. By means of comparison, in determining the allowable strain for reinforcement spanning over voids, BS 8006 directly relates allowable reinforcement strain to acceptable surface deformations (Clause 8.4.4.2). However, in contrast to the situation described in Clause 8.4, no theoretical basis for determining surface settlements based on strain has been presented for pile supported embankments.

Nevertheless, for the simple situation of geosynthetic reinforcement spanning between two pile caps, the sag can be estimated using the simple expression suggested by Leonard (1988), see equation 8. For example, in reinforcement spanning a distance (s-a) equivalent to 2m, the maximum post-construction deflection y associated with 2% creep strain from an initial strain of 3% is 62mm. This could lead to the settlements at the surface of a shallow embankment exceeding serviceability limits, particularly if the embankment fill does not exhibit dilatant behaviour.

$$y = (s - a)\sqrt{\frac{3\varepsilon}{8}} \qquad (8)$$

Where ε is the average strain. Post-construction settlement prediction also assumes that the reinforcement has reached its theoretical design strain during the construction, i.e. that the rate of settlement of the compressible soil between the pile caps has been rapid enough so that all the embankment loading has been transferred to the piles. Gradual loss of partial support from the compressible soil after construction may increase the apparent long term post construction strain of the reinforcement.

5.2 *Lateral Reinforcement Strain*

With respect to lateral deformations Clause 8.3.2.11 of BS 8006, referring to basally reinforced embankments, states "*...the maximum strain ε_{max} in the basal reinforcement should not exceed 5% for short term applications and 5% to 10% for long term conditions. Observations have shown that reinforced embankments have performed satisfactorily with these limiting values...*" However, in describing the strain in basally reinforced pile supported embankments, Clause 8.3.3.7 of BS 8006 states "*The reinforcement should resist the horizontal force due to lateral sliding...This reinforcement tensile load should be generated at a strain compatible with allowable lateral pile movements...*"

In the absence of the piles, the tension to resist the lateral thrust would be developed as a lateral strain of the reinforcement, as for a basally reinforced embankment. However, the vertical load acting on the reinforcement spanning between the piles develops tension through the strain undergone in the formation of a catenary, i.e. with no lateral movement of the reinforcement or piles. As the reinforcement can only have a single value of tension, this will involve a component of strain as a result of both mechanisms. Therefore, whilst the transverse geosynthetic reinforcement is designed to have sufficient strength to resist the full lateral thrust of the overlying embankment, the lateral strain of the reinforcement will be reduced by the tension in the catenary.

5.3 *Pile Lateral Movement*

Although the transverse reinforcement was not placed directly above the pile caps, it is in close proximity being separated by 100mm of sand fill. No clear guidance is given within BS 8006 on the depth of influence at the geosynthetic/soil interface and it was considered that lateral forces could be transferred from the interface to the piles through the lateral strain of the reinforcement resulting in bending moments in the piles.

The piles needed to be designed to support the vertical loading due to the embankment and to withstand any bending stresses that might be induced due to the bending moments. Precast piles are required to resist lifting and driving stresses and therefore will have reinforcement to resist these installation loads, However, DCIS piles tend to be designed only to cope with the in-service loads and as such may require little or no steel reinforcement under the centre line of the embankment, to cope essentially with vertical concentric loads. Typically the DCIS piles are often only reinforced with a single central bar to provide resistance to possible tension in the piles during construction.

Estimating the bending moments in the piles requires determination of the lateral movement of the piles, which will be governed by the lateral strain of the reinforcement. Furthermore, the lateral movement of the piles will be resisted by the soil reaction around the piles. Calculation of the net lateral strain is highly complex, involving the coupling of the two components of load. Initially the tensile load developed in the reinforcement is dependent upon the strain of the catenary. However, in order to develop a tensile force to resist lateral thrust there may be additional strain leading to lateral movement of the piles. This may increase the span of the reinforcement, which in turn will increase the load and strain in the reinforcement acting in catenary limiting the lateral strain.

Love (2003) suggests a method, which was initially considered here, whereby the lateral resistance of the piles and passive restraint from the granular piling mattress, ΣP_h, analogous to a stiffened ground beam connecting the piles, could be considered in the formulation for T_{ds}:

$$T_{ds} = 0.5K_a \cdot (f_{fs} \cdot H \cdot \gamma + 2 \cdot f_q \cdot \omega_s) \cdot H \cdot s - \Sigma P_h \qquad (9)$$

Subsequently, in order to estimate the movement of the pile heads and hence the bending moments in the piles, numerical models of laterally loaded pile groups were developed. Initially, simple structural 2D frame models of vertical piles with geosynthetic between the pile caps were considered, neglecting the soil resistance on the piles. The imposed loads modelled distributed shear stresses on the base of the reinforced embankment, e.g. after Hird (1989) and differential tensions in the catenaries under the shoulders of the embankment due to the lower height of fill above the piles. Later, pile group models were developed which included the resistance of the subsoil.

The analyses were used to estimate bending moments for the design of the piles and indicated that the outer rows of piles under the shoulders of the embankments required heavier steel reinforcement than piles under the main body of the embankment. Hence the pile reinforcement for the outer piles that were subject to the highest lateral loads was increased from a single central bar to a conventional 6 bar reinforcement cage over their upper 10 m.

6 GEOSYNTHETIC DESIGN

6.1 *Stress/Strain Behaviour*

Determination of the appropriate design strength for the geosynthetic material required a detailed knowledge of the stress/strain behaviour of the proposed material. The calculation of tension in the reinforcement in catenary includes the strain of the reinforcement, as given in BS 8006 based on Leonard (1988):

$$T_{rp} = \frac{W_T \cdot (s-a)}{2a}\sqrt{1+\frac{1}{6\varepsilon}} \qquad (9)$$

Where ε is the average strain in the reinforcement. The calculated design strength increases with lower strain limits for a given imposed loading W_T. Therefore, it was necessary to check that the design strength calculated from the consideration of the tension in the membrane could be mobilised at the strain limit assumed. This was obtained from the stress/strain curves as shown in figure 3.

6.2 *Creep Strain*

Creep strain is a well-documented characteristic of polymeric materials (Greenwood, 1990), therefore the quoted ultimate tensile strengths of geosynthetic needed to be considered with respect to the rate of strain and ambient temperature.

When estimating deformations in the embankment consideration needs to be given not only the initial tensile stiffness of the geosynthetic (see figure 3), but also the stiffness relevant to the embankments design life, and additional settlement of the embankment fill between the piles due to creep strain. This is of particular relevance for piled embankments where consideration of strain levels within geosynthetics is critical with respect to ultimate and serviceability limit states.

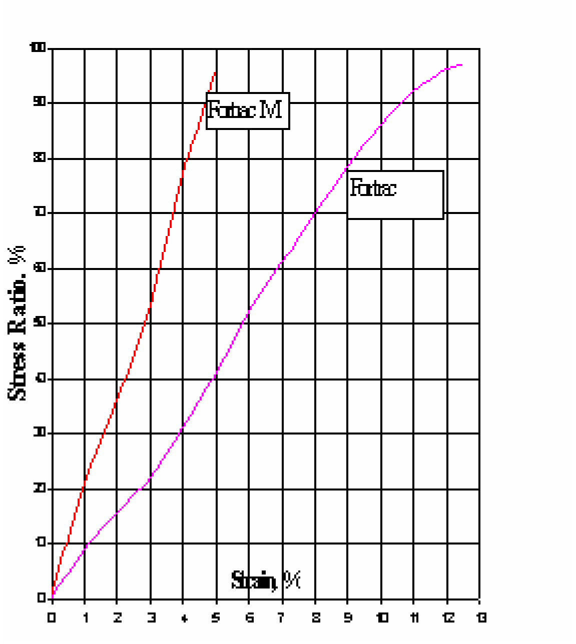

Figure 3 Initial stress/strain relationship of Fortrac & Fortrac M

6.3 *Geosynthetic Details*

The preliminary design was based on the use of high tenacity polyester (PET) reinforcement. The partial material factor approach advocated in BS 8006 was used to determine the long-term design strength of the geosynthetic.

Typically, material with an in service strain below the upper initial strain limit of 6%, such as high tenacity polyester (PET), will have with an associated creep strain (at this constant stress level) of <2%. The design was based on transverse strips of reinforcement of strength between 1200 and 1600kN/m (Fortrac 1200 & Fortrac 1600) and continuous longitudinal sheets of strength 400kN/m (Fortrac 400).

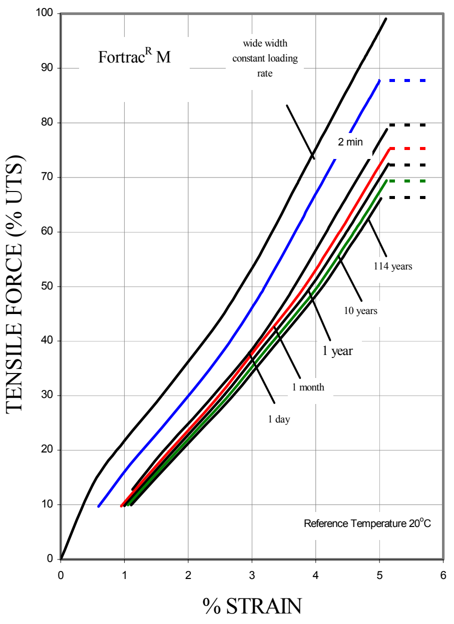

Figure 4 Isochronous curves for Fortrac M

With regard to pile lateral movement considerations (see above) it is desirable to minimise lateral strains in the transverse reinforcement and for construction, higher modulus Fortrac M reinforcement made from Polyvinyl alcohol (PVA) was used for the transverse reinforcement.

PVA offers advantages over PET in that it has a higher tensile modulus, and hence creep strains at in service strains of, say, 4% are negligible. However, the material has a lower rupture strain than PET reinforcements and hence lower ultimate and serviceability limit strains had to be adopted. The original design was verified for the alternative materials, including consideration of the lower strain limits.

Real time creep data and accelerated creep testing was used to generate a family of isochronous curves of stress against strain for a given time, as shown for Fortrac® M in Figure 4. These were used to determine the partial material reduction factor for creep for the geogrids.

6.4 *Instrumentation*

In order to gain knowledge for use in future projects with respect to the uncertainties regarding the interaction mechanism between lateral spreading of the embankment and development of catenary strain in the geosynthetic, it was considered desirable to instrument a section of the embankments. A section at chainage 8967m was chosen, being sufficiently far from any structures that might affect the monitoring but representative of higher sections of the embankment (circa 7.5m).

6.5 *Monitoring*

The instrumentation comprised:

- A horizontal magnet extensometer tube spanning transversely across the embankment, with magnets located at each pile head location, but buried in the sand fill levelling layer immediately above the pile heads. This necessitated the assumption that friction was sufficient to prevent slippage between the transverse geosynthetic reinforcement and the piles.
- Two parallel hydrostatic profile gauges running transversely through the embankment above the reinforcement layers, one located over a pile row and one located midway between pile rows.
- Linear variable displacement transducers (LVDT) fixed to the geogrid at the midpoint of the spans in both directions of the embankment.

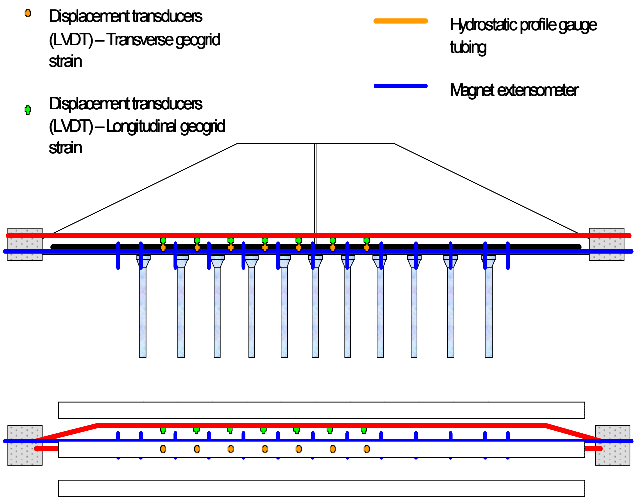

Figure 5 Schematic plan and cross-section of the instrumented embankment

The magnet extensometers and hydrostatic profile gauges were anchored into a stable datum block to give a fixed reference and were manually monitored. The LVDT's were remotely interrogated using a solar powered data logger with a telephone modem connection. Figure 5 shows a schematic of the instrumentation installed.

6.6 *Preliminary results*

Measurements of level changes using the hydrostatic profile gauges and lateral spreading using the magnet extensometer were only successful for a limited period up to an embankment fill height of about 3m. This was primarily due to deformation of the monitoring tubes between the transition from the piled to unpiled toe zones of embankment preventing the passage of the hydrostatic profile and magnet extensometer probes. Despite this, the early level data however clearly illustrated the support provided to the fill by the pile/reinforcement system in contrast to fill placed directly on the subgrade. The hydrostatic profile gauges suggested that lateral spreading of the embankment was occurring, with a relative movement of approximately 80mm across the piled width of the embankment of the order 32m. This suggested an outward movement of around 40mm at the edge piles relative to the pile centreline, equivalent to a strain of 0.25%, Figure 6.

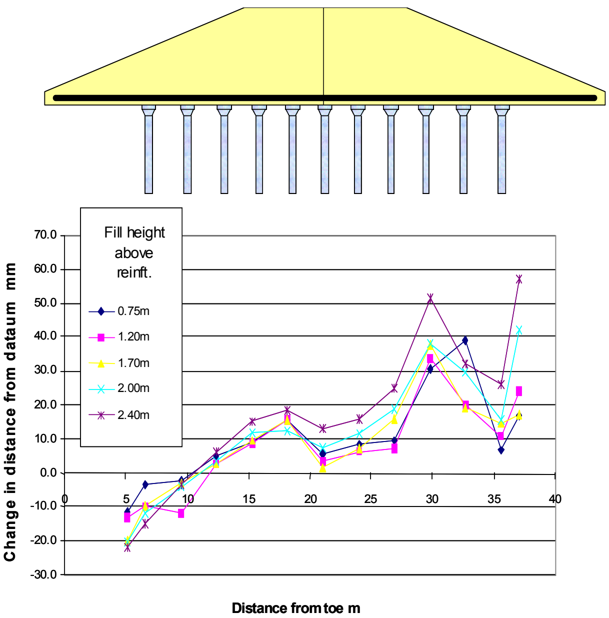

Figure 6 – Measurement of lateral spreading of the embankment

The recorded strain measured in the LVDT's appears to be that taking place during the initial placement of the layer of protective sand fill as any "slack" was taken up. Subsequent monitoring as the embankment has been filled to its full height, over a period of 10 months, has shown little or no increase in strain in the geogrid in both transverse and longitudinal directions.

7 CONCLUSIONS

The design of 1.6km of piled embankment adopted the principles of BS 8006, considering both serviceability and ultimate limit states. However, the distribution of stress across the base of the embankment and the calculation of load on the geosynthetic reinforcement were derived using methods that have been developed specifically for piled embankments.

Detailed consideration to the strain of the reinforcement at serviceability limit state in respect of the lateral movement of the embankment and displacement of the pile heads and the difficulties of estimating the net lateral strain of the reinforcement and potential lateral movement at the level of the pile heads have been highlighted.

The importance of the initial stiffness and subsequent creep behaviour of geosynthetic materials in determining the serviceability of piled embankments has been identified as an essential element of the design that is currently not well described in BS 8006.

Although BS 8006 provides a thorough overview of the issues relating to the design of piled embankments, experience in the design of basally reinforced piled embankments has developed significantly since the publication of the Standard. This has been matched by ongoing research in to the detailed behaviour of piled embankments including the instrumentation and monitoring of full-scale structures. These confirm that the behaviour of these structures is highly complex, however, they suggest that alternative design approaches that have been utilised in the design of a number of piled embankments produce appropriate designs. Hence there are a number of areas where further guidance and development of the Standard would be useful.

The instrumentation installed provided a useful insight into the behavior of the embankment. However, the large relative settlement between the piled and non-piled toe of the embankment meant that data could only be recorded for a limited period. This factor needs to be taken into account if future attempts are made to investigate pile supported embankments

8 ACKNOWLEGEMENTS

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