Advanced Analysis of Reinforced Fills Over Areas Prone to Subsidence

C. R. Lawson & G. T. Kempton
Exxon Chemical Geopolymers Ltd., Gwent, UK

C. J. F. P. Jones & E. K. S. Passaris
University of Newcastle-Upon-Tyne, UK

ABSTRACT: The analysis of the effect of reinforcement on the performance of fills spanning potential voids constitutes a complex interaction problem. Previously, various simplified analysis techniques have been developed - tension membrane, tension membrane with no arching, tension membrane with arching - all of which have limitations concerning accuracy and validity. The paper presents a more sophisticated model using continuum methods. Results obtained from the various models are compared. It is concluded that for this type of problem the continuum method provides the most accurate solutions. The paper also presents a case study where this advanced analysis technique was used to model the effect of reinforcement on embankment performance for a highway constructed over karstic terrain in the North of England. Geosynthetic reinforcement was incorporated into the base of the embankment to prevent a catastrophic collapse should subsidence occur.

1. INTRODUCTION

Subsidence is the surface settlement resulting from removal of support below the ground surface. This loss of support is often due to the formation of a void and subsequent collapse within the ground strata. The formation of these voids can result either from natural processes or from man-made processes.

An extreme form of subsidence result from the formation of a local depression, or hole - a "sinkhole" - at the ground surface. Sinkholes arise from the subterranean erosion of soils in karstic areas (termed "swallow holes"), or the collapse of subterranean rock caverns in karstic areas; from the extraction of brine from subterranean salt deposits; from extensive pumping of groundwater; or from the collapse of underground mines (normally termed "plumpholes" or "crownholes").

The consequences of subsidence occurring beneath structures range from a total loss of structural integrity to a mild serviceability loss. Specific foundation treatments are required to ensure subsidence remains within predefined tolerances. Because many of these treatments are expensive a risk analysis is normally carried out beforehand to determine the most cost effective solution.

A variety of techniques has been used to prevent, or minimise, the effects of subsidence. These range from high cost "active" measures, which may involve grouting up all subterranean voids before construction commences, to "passive" measures, which may involve doing nothing until subsidence occurs and then carrying out maintenance on the structure. Intermediate measures include the use of rigid foundation rafts (made from reinforced or prestressed concrete) or tension membranes (in the form of geosynthetic reinforcement). These are used at the base of structures to restrict the vertical movement of the structure should subsidence occur.

The technique used for embankments, being flexible structures, is either a rigid concrete raft or a tension membrane because these techniques constitute the most cost effective methods of maintaining the differential deformation at the embankment surface within prescribed limits. (In extreme cases foundation grouting is also used.) By controlling the differential deformation at the surface of embankments a pavement can remain in a serviceable condition following a subsidence occurrence. Parry (1983) has recommended that to maintain serviceability the maximum allowable surface differential deformation should be limited to 1% for principal roads and 2% for lower class roads.

When using the tension membrane technique it should be noted that the loading regime is different to conventional reinforced soil applications associated with slopes and walls. In these applications the geosynthetic reinforcement is required to resist tensile loads immediately following installation. With the tension membrane technique the geosynthetic reinforcement remains unloaded until such time as collapse occurs beneath the structure, and only then at the location of the collapse is the reinforcement required to carry tensile loads. Ironically, the ultimate success of the structure is guaranteed if collapse never occurs throughout its design life in which case the reinforcement is never required to carry tensile loads.

When a collapse occurs beneath a geosynthetic reinforced embankment a clear strategy regarding maintenance has to be adopted. Maintenance strategies can range from urgent grouting of the collapsed area to taking no action at all provided the deformation remains within acceptable limits. Generally, for principal roads it is common practice to fill any voids which might occur, while for lower class roads the cost of filling is not normally justified. The maintenance strategy adopted has a direct impact on the properties of the geosynthetic reinforcement used. If the strategy is to fill the void as soon as possible after void formation then the geosynthetic reinforcement has to carry tensile loads for a relatively short period of time. If, however, the maintenance strategy is to do nothing following void formation then the geosynthetic reinforcement has to carry tensile loads for the full remaining life of the structure.

2. EXISTING MODELS

The use of geosynthetic reinforcement to support fills over voids constitutes a complex interaction problem. To analyse the problem a number of models have been developed using various simplifying assumptions, including;

- The assumption that the foundation beneath the tension membrane is rigid.
- Either the acceptance of maximum arching in the supported fill or its neglect.
- The assumed shape of the deformed tension membrane conforms to a simplified geometry.
- The tension membrane deforms only within the void area and the strain within this area is uniform.
- The interaction of the tension membrane with the supported soil during deformation is neglected.

The existing models fall into two general categories - tension membrane theory with no soil arching, and tension membrane theory with soil arching. A model from each category is described in further detail below.

2.1 BS 8006 model

The BS 8006 (1994) model is based on the use of tension membranes to restrict the amount of surface deformation should a void form beneath an embankment. This model is particularly relevant to highway embankments and pavements traversing areas prone to subsidence, Kempton (1992). The basic model geometry and variables used are shown in Figure 1(a). The model is used to define two conditions;

(a) design to resist complete collapse into the void while accepting a loss in serviceability or,

(b) design to limit deformations so as to maintain serviceability of the structure over the void.

The model assumes that there is no arching in the fill above the tension membrane. This is considered to be a conservative approach but is acceptable as soil arching cannot always be relied upon in situations where the edges of the void undergo considerable movement and are naturally unstable. In addition, vibrations due to traffic may break down arching in shallow embankments.

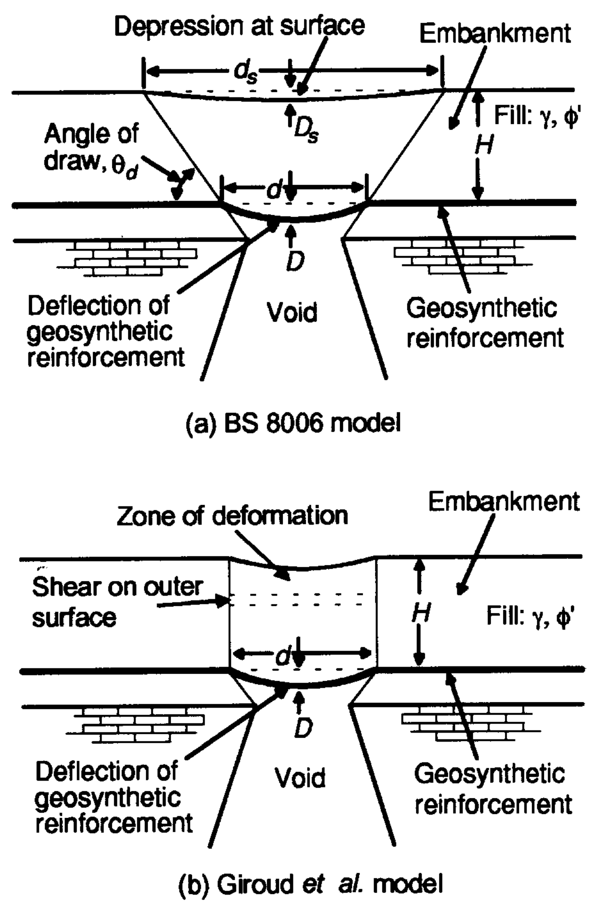

Figure 1 Two models describing tension membrane theory.

To simplify the analysis it is assumed that the fill loading is distributed along the horizontal span of the reinforcement. In this case the shape of the deflected geosynthetic reinforcement is assumed to be parabolic although it is known to form a catenary in the ideal case. The equations governing the extension of the unsupported geosynthetic reinforcement are:

For plane strain conditions (i.e. long voids):

$$\epsilon_{max} = \frac{8\left[\frac{D_s}{d_s}\right]^2\left[d + \frac{2H}{\tan\theta_d}\right]^4}{3\,d^4} \qquad (1)$$

For axisymmetric conditions (i.e. circular voids):

$$\epsilon_{max} = \frac{8\left[\frac{D_s}{d_s}\right]^2\left[d + \frac{2H}{\tan\theta_d}\right]^6}{3\,d^6} \qquad (2)$$

where ϵ_{max} is the maximum allowable strain at the base of the fill spanning the void, and D_s, d_s, d, H and θ_d are as shown in Figure 1(a).

The tension in the geosynthetic reinforcement spanning the void is determined using the following equation:

$$(T_{rs})_{BS} = 0.5\,\lambda\,(\gamma H + q_s)\,d\sqrt{1 + \frac{1}{6\,\epsilon}} \qquad (3)$$

where $(T_{rs})_{BS}$ is the tension in the reinforcement according to BS 8006, q_s is the surcharge on top of the embankment, ϵ is the maximum allowable strain in the reinforcement, γ, H and d are as shown in Figure 1(a), and λ is a load shedding factor. For one-way reinforcement $\lambda = 1.0$; for two-way reinforcement $\lambda = 0.67$.

In addition to the assumptions highlighted above the BS 8006 model also assumes that all of the geosynthetic deformation occurs within the unsupported area and the extension of the reinforcement is uniform throughout the unsupported length. Neither of these assumptions are valid.

2.2 Giroud *et al.* model

The Giroud *et al.* (1990) model is used primarily to determine the tension and extension of the geosynthetic reinforcement only. Other aspects, such as the serviceability of the associated structure are not considered. The basic model geometry and variables used are shown in Figure 1(b). The model defines one condition only - design to resist complete collapse into the void.

The model assumes a rigid base with a well defined collapse geometry, and consequently, full arching is taken into account when assessing the vertical stresses acting on the tension membrane. Arching is assumed to occur according to classical theories (Terzaghi, 1943).

The shape of the deflected membrane across the unsupported span is assumed to be circular to simplify the analysis. The equation governing the extension of the unsupported geosynthetic reinforcement is:

For plane strain conditions (i.e. long voids):

$$\epsilon = 2\,\Omega\,\sin^{-1}\left(\frac{1}{2\,\Omega}\right) - 1 \text{ for } D/d \leq 0.5 \qquad (4)$$

and

$$\Omega = 0.25\left(\frac{2D}{d} + \frac{d}{2D}\right) \qquad (5)$$

where ϵ is the strain in the geosynthetic reinforcement spanning the void, Ω is in radians, and D and d are as shown in Figure 1(b).

The vertical stress acting on the unsupported geosynthetic reinforcement is reduced by arching in the fill and is:

$$p'_c = \frac{\gamma\,d}{2K\tan\phi'}[1 - e^{-2K(H/d)\tan\phi'}] + q_s\,e^{-2K(H/d)\tan\phi'} \qquad (6)$$

where p'_c is the vertical stress acting on the unsupported reinforcement, K is the coefficient of horizontal earth pressure, q_s is the surcharge on top of the fill, and γ, ϕ', d and H are as shown in Figure 1(b).

Consequently, the tension developed in the unsupported geosynthetic reinforcement becomes:

$$(T_{rs})_G = p'_c\,d\,\Omega \qquad (7)$$

where $(T_{rs})_G$ is the tension developed in the reinforcement according to Giroud *et al.* (1990).

The Giroud *et al.* model also assumes that all of the geosynthetic deformation occurs within the unsupported area and the extension of the reinforcement is uniform throughout the unsupported length. As stated above, neither of these assumptions are valid.

One weakness with this model is that it has to assume strain compatibility between the soil and the reinforcement, and yet there is no evidence that this is occurring.

3. MODELLING USING CONTINUUM METHODS

A logical approach to resolving the problem of strain compatibility when analysing reinforced fills over voids in which arching can occur is the use of continuum methods. The advantage of these methods is that they can accurately predict soil behaviour and model complex geometries without the need to predetermine the failure mode. There are two approaches to the use of continuum methods - the finite element method and the finite difference method. Both have particular benefits depending upon the nature of the problem to be solved. Most reinforced soil is modelled using the finite element method, in which the reinforcement is described as being either a tie or a truss element. In the case of classical reinforced soil this is an acceptable assumption as significant deflections along the length of the reinforcement are not expected. In the case of reinforcement supporting soil over a void where significant deflections can be expected the use of a tie or truss element to describe the reinforcement is not acceptable as the resultant model would not represent accurately the real life performance. In this study the reinforcement is modelled as an elastic beam.

The development of a void beneath the reinforced fill results in physical instability and large strains. This is best modelled by the finite difference method, and in the numerical procedure developed this problem was modelled using a modified version of FLAC, Cundall (1980).

Successful modelling of a reinforced fill over an area prone to subsidence can only be achieved if the parameters used in the analysis are described accurately. The critical parameters include the size of the void, the surface geology including knowledge of whether the reinforced embankment is supported on a rock base or a soil foundation, the embankment height and material properties, and the reinforcement characteristics. The geology can be identified and the embankment fill can be modelled in accordance with a range of acceptable soil models. An accurate description of the reinforcement used in this case was achieved by undertaking a full scale experiment in which the reinforcement was suspended over a void of known dimensions and a uniformly distributed load applied.

In addition to modelling the development of soil arching for different geometries and void dimensions, the numerical procedure developed as part of the study also took into account the development of bond length in the reinforcement and the presence of weakened support provided by the foundation at the edge of the void.

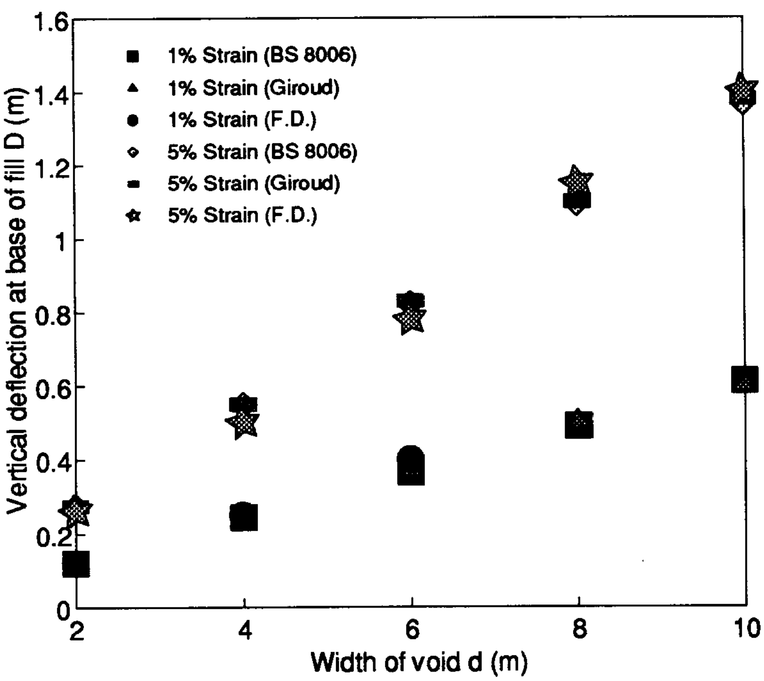

Figure 2 Comparison of the three models in determining the maximum vertical deflection at the base of the fill spanning the void.

Comparison of some of the results obtained using the three models are presented below. Figure 2 compares the vertical deflections at the base of the fill using the three models and assuming a 1% and 5% strain level in the geosynthetic reinforcement. All three models give identical results. Thus, while different deflected geosynthetic geometries may be assumed in order for the analysis to be carried out, at the strain levels typical of geosynthetic reinforcement these differences are negligible.

Figure 3 compares the reinforcement tension determined using the three models and assuming a 5% strain level in the geosynthetic reinforcement and a void span of 4m. The BS 8006 model, which assumes no soil arching, shows a constant rate of increase in reinforcement tension with embankment fill height. The Giroud *et al.* model, which assumes full soil arching, shows a decline in the rate of increase in reinforcement tension with embankment fill height. For all embankment heights greater than 2m the Giroud *et al.* model predicts a lower reinforcement tension than the BS 8006 model.

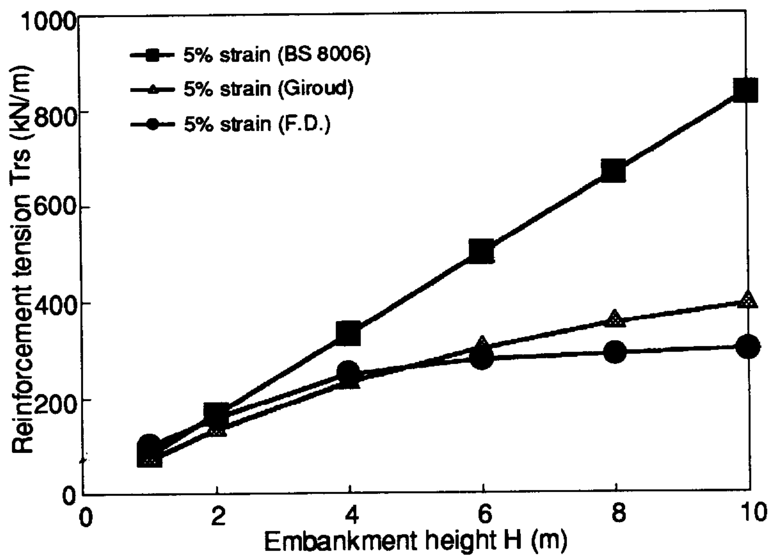

Figure 3 Comparison of the three models in determining the reinforcement tension for a void span of d = 4m (γ = 20kN/m^3, ϕ' = 35°).

The results from the finite difference model shown in Figure 3 is the case where a well defined rock stratum lies beneath the geosynthetic reinforcement. In this instance arching in the fill above the reinforcement will be a maximum. Consequently, the tension developed in the reinforcement using this model closely relates to the Giroud *et al.* model. For $H/d > 1$ there is little increase in the reinforcement tension. When soil layers are introduced below the reinforcement less arching occurs in the fill and consequently the reinforcement tensions determined using the finite difference model are greater than those shown in Figure 3.

It is considered that the BS 8006 model and the Giroud *et al.* model provide the upper and lower extremes (respectively) when calculating reinforcement tensions and that these can differ by several orders of magnitude. The finite difference model has the degree of sophistication to differentiate between different foundation conditions and provide a more realistic assessment of the actual reinforcement tensions.

4. RIPON BYPASS REINFORCED EMBANKMENTS

Ripon, an historic Cathedral City in the North of England, is situated on karstic terrain. The area has long been affected by surface instability due to sinkhole development emanating from the underlying gypsum strata. A general geological profile of the area is shown in Figure 4.

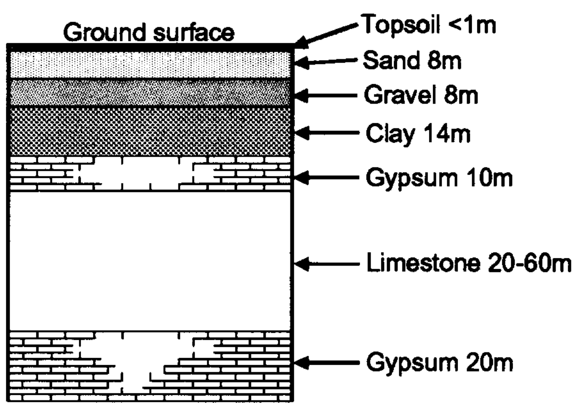

Figure 4 Geological profile of the Ripon area.

Much of the area is thought to be undermined by vast caves eroded in the two layers of gypsum running beneath the town. These interconnecting caves are up to 200m long and 15m high and have been created by circulating groundwater dissolving the calcium sulphate within the soft gypsum rock. Often the entire thickness of the gypsum band has been dissolved. As weaknesses in faults and jointing migrate upwards, large sinkholes can suddenly appear at the surface. The largest of hundreds of surface sinkholes around Ripon is 80m diameter and 30m deep.

A detailed study (Cooper, 1992) of the route of the proposed bypass around the town listed ten cases of subsidence recorded within 500m of the proposed bypass. Out of the ten occurrences, three had measured diameters in excess of 6-8m, and seven of the subsidence events have occurred since 1970.

To ensure stability of the completed bypass embankments a variety of foundation stabilisation techniques were investigated. These included piling and grouting, both proving to be very expensive. An alternative approach was adopted using geosynthetic reinforcement as a tension membrane at the base of the bypass embankments to prevent the occurrence of excessive differential deformations at the pavement surface should a catastrophic collapse occur in the foundation.

In addition to the soil material parameters to be used in the design analysis it was necessary to determine a design void diameter as well as a strategy covering remedial activity should a potential collapse occur. To minimise risk a design void diameter of 10m was chosen. While this void diameter was greater than that which had already been observed in the vicinity of the bypass route it was less than the maximum potential void diameter. A remedial strategy was formulated based on the requirement for the geosynthetic reinforcement to support the embankment for a period of 24 hours following a design collapse. This period of time was considered to be adequate from the viewpoint of preventing an immediate catastrophic collapse and enabling a traffic diversion programme to be put in place.

For design purposes, the embankment sections were divided into two categories based on height - less than 1m height and 1-6m height. The void parameters along with the soil material parameters for the embankment and foundation were used in the finite difference model to determine the required geosynthetic reinforcement characteristics for the 1-6m high embankment sections. The geosynthetic reinforcement characteristics required for the less than 1m high embankment sections were determined using the BS 8006 method (using the plane strain model). The results obtained are listed in Table 1.

Table 1 Design strength/extension characteristics for unidirectional geosynthetic reinforcement.

Embankment height	1m	1 - 6m
Design tensile strength	350kN/m	750kN/m
Design tensile extension	5.0%	5.0%

To ensure a consistent coverage across the base of the embankments it was decided to use a double layer of geosynthetic reinforcement - each layer having half the design strength listed in Table 1.

To determine the initial ultimate mechanical properties of the appropriate geosynthetic reinforcement the principles proposed by Jewell and Greenwood (1988) and Lawson (1992) were adopted. However, where the geosynthetic reinforcement is used as a form of insurance against a potential future event, as is the case with spanning voids, the procedures presented by the above authors have to be modified to take into account the different loading regime. Based on this modified approach relevant partial material factors covering installation damage, creep rupture and durability were determined, as well as an economic ramifications of failure factor.

A cross section of the embankments up to 6m in height is shown in Figure 5. The reinforcement used in these locations, ParaLink 700S, satisfied fully the reinforcement design requirements listed in Table 1 following application of the various partial material factors.

Prior to the placement of the reinforcement a 350mm thick layer of Type 6C granular fill was placed. This class of material is crushed rock with a maximum size of 125mm diameter, and can be highly aggressive to most geosynthetics. Thus, the partial material factor dealing with installation damage resistance was considered an important component in the buildup of the overall partial material factor for the geosynthetic reinforcement.

The geosynthetic reinforcement materials were laid generally parallel to the road centreline. Two layers were installed with a vertical spacing between the two layers of 150mm. The use of two layers in this way minimised the difficulties associated with joints between the geosynthetic reinforcement layers along their edges and at the end of each sheet. The upper layer was staggered with respect to the lower layer both sideways and lengthways such that any potential collapse would have at least one layer of continuous reinforcement above it running beyond the edge of the collapse a sufficient distance to prevent bond failure.

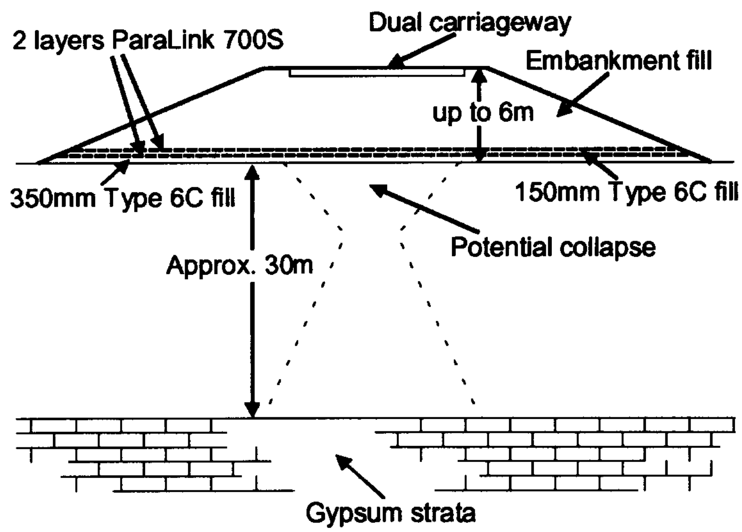

Figure 5 Design cross section through reinforced embankments.

The geosynthetic reinforcement lengths were placed beyond the edge of the carriageway a sufficient distance to support the edge of the road adequately. Further layers were placed at the edge of the carriageway at positions where the reinforcement intersected the edge at an angle. This problem occurred particularly at roundabouts. At such locations the geosynthetic reinforcement was used primarily parallel to one of the intersecting roads with secondary layers to support the ends of the primary layers. The direction chosen for the primary layer was normally parallel to the contours of the existing ground for ease of laying.

The geosynthetic reinforcement was delivered to site in 100m rolls. These weighed 1.4 tonnes and required careful handling during transportation and laying. The length of the rolls was dictated largely by the need to avoid joints in the layers. A specially adapted fork lift truck was used to lift and position the rolls. Once in the correct position they were easily unpacked and rolled out. Minor adjustments to position were possible after laying by rearranging the layers in small lengths.

Following an initial learning period the contractor was able to install the geosynthetic reinforcement in a systematic manner commensurate with construction operations.

5. CONCLUSIONS

The use of geosynthetic reinforcement as a means of preventing a catastrophic collapse of embankments should subsidence occur would appear to be highly cost effective. However, while relatively simple analysis models exist, the use of sophisticated analysis modelling would appear warranted to understand fully the mechanisms involved.

While each of the three models reviewed in this paper give identical results regarding the deflection of the geosynthetic reinforcement, there are significant differences regarding the calculated reinforcement tensions. These differences are due to the degree of soil arching assumed to exist in the reinforced embankment. Of the three models, it is considered that the finite difference model provides the best approximation for reinforcement tension, with the other two models providing the upper and lower extremes.

The use of the geosynthetic reinforcement at the base of the highway embankments at Ripon would appear to provide a highly beneficial form of insurance by preventing a catastrophic collapse should subsidence occur.

6. REFERENCES

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