Design of reinforced embankments for Great Yarmouth Bypass

L'étude des remblais renforcés pour la rocade de Great Yarmouth

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SYNOPSIS The paper presents the effective stress design methods developed for reinforced embankments on the Great Yarmouth Bypass. The design methods are discussed in relation to total stress methods and the properties of the very soft underlying soils. The performance of the embankments is outlined with an account of a localised failure, passing through the reinforcement. It is concluded that the use of effective stress methods can produce cost effective reinforced embankment designs.

INTRODUCTION

Embankments reinforced with geogrids and geotextiles have been constructed for the Great Yarmouth Western Bypass in Norfolk, England. The embankments vary in height between 2.0 and 8.0m and consist entirely of imported granular fill. They are underlain by soft and very soft alluvial deposits to a depth of 22m. The history of the site and the construction of the embankments have been presented previously, together with some performance data (Williams 1984).

The concept behind the design was to construct embankments for which stability analyses employing total stress (ϕ=0) criteria, neglecting the reinforcement, result in end of construction safety factors between 0.8 to 1.0. The reinforcing members were not intended to provide means for steepening side slopes, but were considered to be an economic means of increasing foundation safety factors.

The majority of analytical methods used for reinforced embankments reported in recent literature employ limit equilibrium methods applied to circular arc surfaces and total stress criteria. In order to adopt such a method for the Great Yarmouth embankments, reinforcement would be required to increase the embankment safety factor from 0.8 to 1.3. While it appears acceptable to use reinforcement to increase safety factors from say 1.1 to 1.3, increases from below unity appear inexpedient.

In addition to stability, the design for a road embankment over deep alluvial deposits must take account of consolidation of the deposits and effects of residual settlement on the completed road. In order to prevent the occurrence of excessive residual settlement, periods for settlement are often incorporated into the construction programme between filling and road laying stages. Such periods may be reduced by surcharging, whereby the embankment is constructed to a greater height than ultimately required. It then becomes possible to include cessation periods in embankment raising to allow porewater pressure dissipation. Taking account of the increase in strength of the subsoil brought about by porewater dissipation, the embankment design is no longer governed by total stress concepts but is controlled by effective stresses. It appears logical then, where factors of safety are less than unity for a ϕ=0 analysis, to combine effective stress criteria with reinforcing inclusions to provide adequate safety factors throughout the construction period.

At Great Yarmouth the dissipation of excess porewater pressure during and subsequent to embankment construction was promoted by vertical band drains installed in the underlying alluvial deposits.

DESIGN DEVELOPMENT

Milligan and La Rochelle (1984) note, for embankments constructed over soft clay foundations with factors of safety using a ϕ=0 analysis in the range 1.2 to 1.3, lateral strains are several percent and settlement about 10 percent of the embankment height. They further note, where settlement is expected to exceed 10 percent, the design should include a detailed assessment of strain and limit equilibrium methods for reinforced embankments must be used with caution.

In order to assess such strains, determine actual performance characteristics and check the proposed design, trial works were undertaken along the line of the proposed road. These comprised of the construction of a 5m high embankment in two 2.5m lifts over a geotextile reinforced foundation. This embankment was constructed along the line of a much narrower former railway embankment, which is closely followed by the proposed road (Fig 1). Two 2.5m high filling phases were separated by a cessation of 10 weeks duration to permit dissipation of excess porewater pressure. During the first phase construction and the cessation, settlements and lateral strains in excess of 20 percent and

12 percent respectively of the embankment height were recorded without failure. The rate of construction was controlled by stability analyses using values of porewater pressure and effective stress parameters inferred from triaxial tests. Further control of construction rates was provided by observations of the rates of settlement and lateral strain.

In view of the high values for settlement and lateral strain measured during the trial works, the effective stress analysis was extended for the design of the further Great Yarmouth embankments, utilising the results from back analyses of the trial embankment.

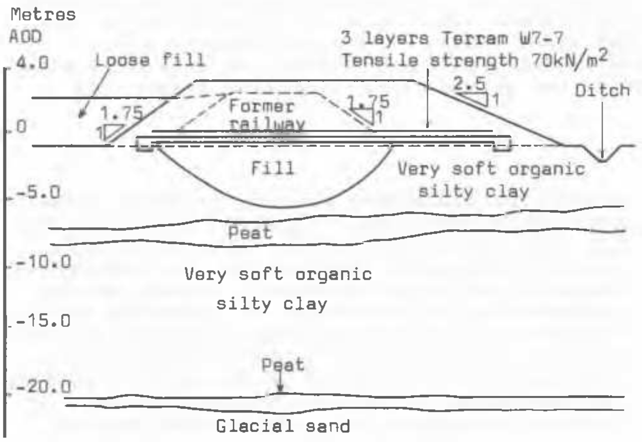

Fig. 1 Simplified Trial Embankment Section

DESIGN METHOD

Three modes of failure were considered (Fig 2)
(1) internal lateral strains causing tensile failure of the reinforcement
(2) circular and non circular failure through the reinforcement and
(3) plastic flow of the foundation

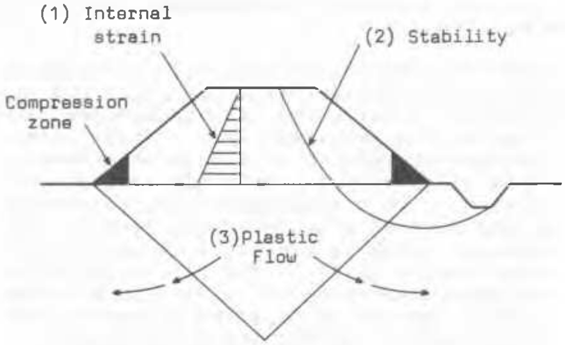

Fig. 2 Failure Modes

Internal lateral strains

In accordance with recent literature it was assumed that the reinforcement must be capable of resisting a horizontal force approximated by the Mohr Coulomb active pressure equation.

$$Pa = 0.5\gamma H^2 \tan^2 (45 - \phi/2) \quad (1)$$

Where γ is the density of the fill and H is the current embankment height.

Furthermore the reinforcement was terminated within the embankment avoiding the zones of zero or compressive strain (McGown et al 1978).

Circular and non circular failure

Using the effective stress design approach developed during the trial works and the rigorous method for stability analysis developed by Sarma (1973), the reinforcement was considered as a cohesive layer after Fowler (1982), or as a resisting moment after Jewell (1982). These approaches were used to equate the porewater pressures around a failure surface, the tensile strength for the reinforcement and the factor of safety. Similar relationships were established for the two methods, however, neither takes account of the disrupting effect a geotextile has on the failure surface. Given adequate bond, a stiff geotextile inclusion reduces the shear stress on the adjacent soil and thereby causes the failure plane to penetrate deeper. This lengthening of the failure plane increases the factor of safety, although in this case no appreciable increase in subsoil shear strength with depth exists.

It was postulated that for a low embankment, reinforcement would steepen the classic shear plane inclination through the embankment to greater than $45-\phi/2$. It was considered that due to the gradual method of filling and compaction of the granular fill, specified in the design, tension cracks would only appear as a result of the development of a failure plane in the foundation.

Plastic flow of the foundation

Stability and bearing capacity failures are known to have taken place during construction of the railway embankments and the site of these failures is traversed by the new road. Site investigation confirmed that the 4m high railway embankment on the poorest soils, in part constructed upon a brushwood mat, caused plastic flow of the subsoil and fill material punched to a depth of 7.0m (Fig 3). In order to prevent a repeat or reactivation of these punching failures, the bearing capacity was analysed using a simplified effective stress approach:

Terzaghi's bearing capacity equation:

$$qf = cNc + \gamma zNq + \tfrac{1}{2} B\gamma N\gamma \quad (2)$$

can be expressed in terms of effective stress to give:

$$qf = c'Nc + p' (Nq - 1) + \tfrac{1}{2} B\gamma' N\gamma + P \quad (3)$$

where $p' = \gamma z - u$ and $\gamma' = \dfrac{\gamma z - u}{z}$

For an embankment above original ground level P=0, and the equation becomes

$$qf = c'Nc + (\gamma z - u) (Nq - 1) + \tfrac{1}{2}B \dfrac{(\gamma z - u)}{z} N\gamma \quad (4)$$

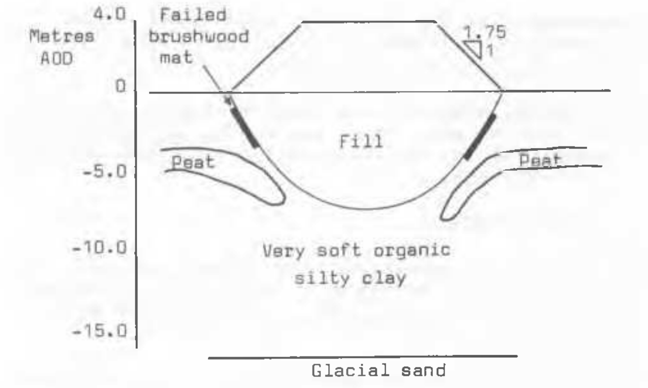

Fig 3. Former Railway Embankment (Bypass chainage 2100)

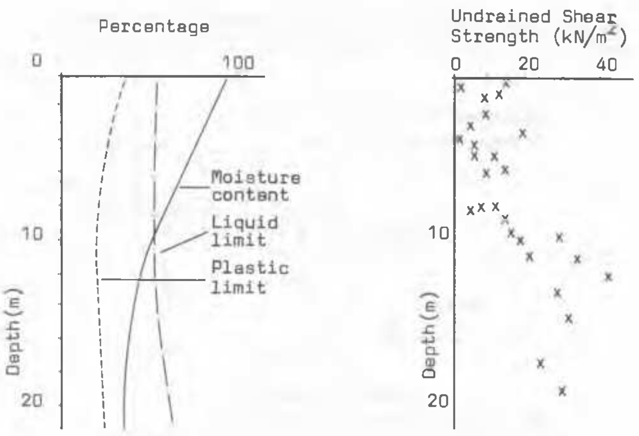

Fig 5. Soil properties - backswamp area

Equation (4) can be solved to give values of porewater pressure (u) for bearing capacity failure.

This approach was used to back analyse the trial and establish values of c' and Ø' for the sub-soil. The method was then used to relate porewater pressure with factors of safety and with controls placed upon lateral movement and settlement, used to control filling rates.

Incorporation of design method

Variations in soil profile, embankment height, the position of the former railway and the requirements for the finished road, resulted in a variety of different levels of reinforcement, construction rates and settlement period durations. Within the scheme various side roads were given lower design and end of construction requirements than the trunk road with the embankment design modified accordingly.

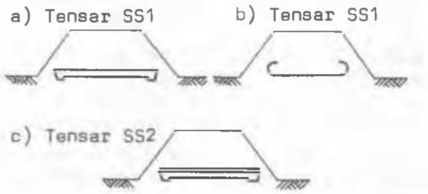

Fig 4. Design variations: a) roundabout chainage 2100 b) Industrial access road c) Embankments over 3m high

PERFORMANCE

The great majority of the 5000m length of reinforced embankment in the scheme performed in accordance with design predictions. However, to report on the total scheme is beyond the scope of the present paper which will concentrate solely on the area of the poorest subsoil. This area thought to be a backswamp to a former river estuary, contains increasingly organic deposits and forms the site of greatest settlement of the former railway embankment.

A roundabout lies in the centre of this area on an embankment 2.5m high which includes 2 geogrid layers (Fig 4a). An industrial access road and cycleway connecting onto the roundabout includes a single geogrid layer (Fig 4b). Identical filling sequences were used for both sections based upon instrument readings under the roundabout. The roundabout was safely constructed to full height in 15 weeks including 2 short cessation periods, however, the cycleway embankment suffered a rotational slip failure along one side approximately 30m long at a height of 1.9m. Soil properties for the backswamp area are shown in Fig 5.

Lateral strains

Lateral strains measured across the width of the embankement throughout the area under consideration were in the order of 10 percent of the height or between 1 and 2 percent of the width. At the slip these figures were not exceeded due to the rotational nature of the movement. Investigation of the slip revealed a clean break in the geogrid with the shear plane passing through at 90°. The geogrid either side of the break showed no apparent distress and a stitched joint some 100mm distant, displayed no signs of having been subjected to excessive tension. The break appeared to be a form of local shear failure of the geogrid with no destructive tensile component. It is postulated that tensile forces induced by the shear force are extremely localised and transferred by bonding to the fill material, due to the high normal stress on the geogrid and surrounding fill.

Stability analysis

Porewater pressures in the vicinity of the slip were found to be in excess of those under the roundabout and exceeded those used in design. Fig 6 shows the mode of failure and the soil properties measured along the failure plane.

Back analysis of the slip using the effective stress approach gives a factor of safety of 0.9. Similar analyses for the roundabout give a factor of safety of 1.1 whilst total stress analyses show factors of safety of 0.8 and 0.9 respectively.

Plastic flow

The large railway embankment failures have not been reactivated or repeated and settlements are in accordance with predictions based upon elastic analysis. Fig 7 shows relationships between the porewater and settlement rates. It can be seen that settlement rates increase rapidly when porewater pressure limits calculated using equation (4) were approached and exceeded.

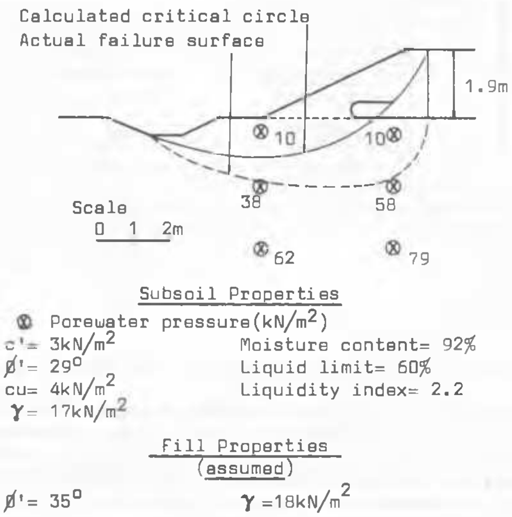

Fig 6. Cross Section of Failure and Measured Soil Properties

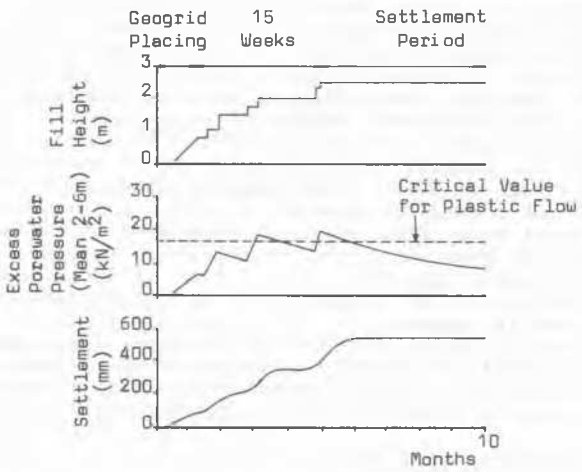

Fig 7. Backswamp Area (Ch2100) (a) Filling Rate (b) Excess porewater pressure (c) Settlement

CONCLUSIONS

By combining the benefits induced by disrupting a potential failure plane using layers of reinforcement, with those of effective stress analyses, cost effective reinforced embankments have been constructed.

In addition to reinforcement and effective stress analyses controlling the stability of the embankments, similar approaches have also been used to reduce settlement by controlling plastic movement of the foundation, using a modified bearing capacity analysis.

Tensile forces induced in a geogrid reinforced embankment at failure have been shown to be extremely localised with rupture taking place in the form of a shear failure.

The design approach and control exercised during construction have been successful in preventing a repeat of the failures which occurred previously on this site.

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The paper is given by permission of Mr.D.I.Evans BSc Tech MSE C Eng Director (TP) Eastern Region, Department of Transport.

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