Reinforcement techniques in repairing slope failures

R. T. Murray, Transport and Road Research Laboratory

INTRODUCTION

Because of the economic constraints on both materials and land usage, embankment and cutting slopes constructed in cohesive soils are generally designed with relatively low factors of safety. In the longer term, therefore, failures may develop as a result of changes in the pore pressure conditions or in the site drainage characteristics.

Some preliminary results of a survey of 300 km of selected lengths of motorway are presented in Table 1 (Transport and Road Research Laboratory, 1983)

Although the results highlight the more severe examples of the stability survey, they nevertheless demonstrate that significant problems can be expected with overconsolidated clays.

The usual method of repairing such slope failures has involved the replacement of the rammed or compacted granular soil by high frictional strength material. Where a source of suitable granular soil is not locally available, however, considerable haulage costs can be incurred and to minimise costs the Department of Transport has proposed re-using the existing material whenever possible (Department of Transport, 1983). A possible approach involves re-using the rammed or compacted granular soil to ensure the integrity of the reinstatement.

In this paper the technique of repairing slopes employing reinforcement layers is briefly described and some details are provided of the application of the method to

<table>
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<tr>
<th>Geological Stratum</th>
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<th>Total Length of slope surveyed (m)</th>
<th>Percentage of length failed</th>
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<td>4340</td>
<td>15.5</td>
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TABLE 1 More severe examples from TRRL Slope Stability Survey on Selected Motorways


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the repair of a cutting in London clay. A set of design charts are also provided to enable a rapid assessment to be made of the reinforcement requirements for a range of typical slope situations.

Method of Analysis

Details of the method of analyses are provided in earlier publications (Murray et al. 1982; Murray, 1982) but for completeness a brief outline is given below.

The failure surface is represented by a bilinear slip plane as shown in Fig. 1, in preference to the more conventional circular or non-circular slip surfaces; much of the complexity of analysis is then removed and moreover the volume of computation is greatly reduced. It is possible to provide a reasonable representation of most types of failure surface by the use of bilinear slip planes and the accuracy of the results obtained from the analysis agree favourably with those obtained by other methods. This is demonstrated by the comparison provided in Fig. 2, which relates to stability assessments based on the above method together with the results obtained from stability coefficients produced by Bishop and Morgenstern (1961). As can be seen from the figure the results are in very close agreement.

The results shown in Fig. 2, relate to the situation where the slope is unreinforced, and the inclusion of a tensile component into the analysis produces a significant increase in the complexity of the assessment of the stability of the reinforced soil system. It is normal practice to consider two aspects of internal stability, namely adherence and tensile resistance. However, because geotextiles or geogrids are more strain susceptible than metallic reinforcing elements, additional reinforcement layers are generally incorporated to limit these strains. The adherence or pull-out resistance of the tensile elements is determined by the coefficient of friction between the soil and reinforcement. It is frequently the case that the friction coefficient at the soil geotextile interface is a high proportion of the soil friction co-efficient, particularly where geogrids are used. The possibility of an adherence failure occurring in such circumstances thus appears very unlikely. For purposes of the stability assessments therefore, the influence of the reinforcement layers is treated purely as a mobilised or permissible tension component and no consideration has been given to pull-out or adherence type failures.

The result of a series of analyses for 3 different values of pore water pressure ratio (\(\psi\)), have been used to compile the charts shown in Figs. 3 - 5; a unit weight for the soil of 20 kN/m\(^3\), is assumed. These charts can be employed to obtain a rapid assessment of the reinforcement requirements for different slopes. It is assumed in using the charts, that the vertical spacing, \(S_y\), of the reinforcement layers is constant.

In selecting a value of permissible tension to use in a particular situation, due consideration must be given to both the short-term and long-term strain and creep characteristics of the geotextile. On this basis, therefore, the value chosen will generally be much smaller than the known ultimate tensile strength of the material and will frequently correspond to that required to produce no more than about 5 per cent strain in the long term. Some consideration of the procedures for determining the potential deformations of a reinforced slope is given in an earlier publication (Murray, 1982).

The following example has been included to demonstrate how the charts are used:
(a) Select a value of permissible tension in each reinforcement layer, in the appropriate units of kN/m width. The value selected is intended to avoid problems of both short and long term serviceability - say a value of 30 kN/m width has been chosen.

(b) Estimate the height (H) in metres from the lowest to the highest point on the anticipated slip surface. An initial estimate could involve only the height of slope at risk, however it is possible that the slip surface will be more deep-seated and actually dip below the lowest point on the slope face. Assume a value of 12m has been selected in this case.

(c) Select a suitable value of $S_v$ - say 1.0m.

(d) For the known values of pore pressure ratio ($r_u$) and effective angle of internal friction ($\phi'$), select the appropriate chart in Figs. 3-5; say values of 0.25 and 20° for $r_u$ and $\phi'$ respectively apply in this case.

(e) Evaluate the factor $T_{perm}/(H \times S_v)$; for the selected values this is $30/12 \times 1 = 2.5$ kN/m². Obtain a factor of safety for the given slope angle $\theta$ from the chart (Fig 4). For purposes of the example a value of $\theta$ equal to 30° will be assumed. and on this basis the factor of safety from Fig 4 is about 1.4. If this factor of safety is not considered adequate then either $T_{perm}$ must be increased or $S_v$ reduced. A reduction in the vertical spacing, $S_v$, of the reinforcement layers to 0.8m would increase the value of $T_{perm}/H \times S_v$ to 3.1, and the corresponding factor of safety is a little more than 1.6. It is interesting to note that the same chart gives a factor of safety of about 0.4 when the slope is unreinforced.

**Practical Considerations in Slope Repair**

Although a substantial proportion of the total cost of repairing a slip failure by the conventional method of replacing the failed region of soil by granular material arises from the haulage distances involved, it is also necessary to add to these costs those resulting from the impedance to the free flow of traffic that occurs while the construction work is in progress; these are particularly significant where this involves the removal and importation of a large quantity of material along the section of road affected by the failure. The difficulties are likely to be compounded where the repair applies to minor roads or to other roads with limited access.

The repair of a slope by re-use of the founded soil could thus alleviate much of the hindrance to the free flow of traffic by reducing the amount of construction traffic utilising the affected road. Of course, even with the soil reinforcement method of repair, it will be necessary to remove some of the founded soil at the start of the reinstate-
Fig 4 RELATION BETWEEN FACTOR OF SAFETY AND SLOPE ANGLE FOR DIFFERENT VALUES OF $\phi$ AND $T_{perm}$ AT A PORE PRESSURE RATIO OF 0.25.

Fig 5 RELATION BETWEEN FACTOR OF SAFETY AND SLOPE ANGLE FOR DIFFERENT VALUES OF $\phi$ AND $T_{perm}$ AT A PORE PRESSURE RATIO OF 0.5.
REINSTATEMENT OF SLOPES

Application of the method to the repair of a failed cutting

The method was first applied to the repair of a cutting slope in London clay. As this work has been previously reported (Muray et al., 1962) only a brief outline will be given:

The cutting, which extended to a depth of 20m, formed part of the M4 motorway some ten miles west of Reading. Over a period of several years a series of slips had occurred at the particular section and this culminated in a major slope failure, over the top 10m of the cutting, in 1979, with further movement in 1980.

The work of reinstatement, which took place in late 1980, was carried out by Berkshire County Council by direct labour contract and involved the excavation and replacement of some 7000 m$^3$ of soil. It was possible to excavate all of this material in a single operation after the owner of the adjacent land had given permission for temporary stockpiling of the excavated material. Clearly greater productivity can be achieved by dealing with larger regions in a single operation rather than as a series of strips but, as discussed previously, the risk of initiating further slips is increased. However, as the work was carried out during a relatively short period and in dry weather, no serious problems were encountered in this instance.

The excavation was taken down to some 30cm below the observed slip plane. Drainage trenches of 1.2m depth and at 5m centres were installed at this level to reduce the possibility of further slips developing below the treated zone. The first layer of reinforcement was pegged down over the base of the excavation and sufficient length was taken beyond the face of the slope to allow it to be folded back with an adequate lap onto the next level of reinforcement. The reinforcement used was a polymer mesh (Netlon CE 131) which was supplied in rolls 30m long and 2m wide.

The fill was placed and compacted to layers of about 25cm thickness. Prior to compaction, about 2 per cent by weight of quicklime was mixed into the upper part of the layer. The reinforcement was spaced at vertical intervals of 0.5m in the lower levels which was increased to 1m in the upper levels. Plate 1 shows the reinforcement spread over the excavated slope. To avoid damage by construction plant, a protective layer of top soil was spread over the lower portion of the slope when this slope had been completed. The work continued in the same fashion until the reinstatement was completed and thereafter the remainder of the slope was top-soiled.

Site observations have indicated that no movement has taken place since completion of the work in 1980.

An estimate of the relative costs of the repair using the soil reinforcement method indicated that these were just over half those estimated


definition operation. The procedure envisaged in this method involves excavating a strip of soil up the failed slope. The width of the excavated zone would be dictated by the requirements to operate the construction plant efficiently in addition to the need to prevent further slips developing as a result of excavating too large a region: it is self-evident that a narrow excavation will assume a greater degree of stability than is produced by a wide excavation. The excavated zone would then be reinstated together with sheets of reinforcement at the required vertical spacing. The soil used for the reinstatement would be obtained from a similar excavation at the other side of the failure region. This procedure would then be repeated until the whole of the foundered soil had been treated as a series of strips.

The depth of the treated zone would need to extend beyond the slip surface in order to permit a sufficient length of reinforcement to be installed to avoid adherence failures occurring. It would also be necessary to check that the stability of the region beyond the treated zone is adequate, otherwise this region also would need to be reinforced to prevent deeper seated failures developing. It is normal practice in placing each reinforcement layer to continue the geotextile up the surface of the slope and for a distance of one or two metres onto the next soil level where a reinforcement layer is to be placed.

Because the foundered soil is generally in a very soft condition, it will usually prove difficult, or even impossible, to operate with conventional construction plant. A technique which has proved very successful in these circumstances is to add quicklime during the placement operations. The quicklime is normally only mixed into the upper part of the compacted layers in order to induce a rapid improvement in the working surface so that the construction plant can operate more effectively. Both the strength and permeability characteristics of the lime treated soil will be increased resulting in a further enhancement of slope stability. However, because the underlying material in each compacted layer remains virtually unchanged, the geotextile reinforcement layers are nonetheless required to ensure the integrity of the reinstated material.

Once having prepared a surface on which a reinforcement layer has to be placed, the geotextile will be positioned over the surface and pegged down, if necessary, to prevent it hampering the construction work. To prevent the geotextile from being damaged during the filling operation, it will normally be necessary to end-tip the fill and work from one edge by spreading in front of the plant.

To prevent the exposed surfaces of geotextile on the slope face from being damaged by ultraviolet radiation, it is advisable to spread a layer of top soil or other suitable material over the surface of the slope soon after installation.
for the conventional approach employing granular soil (£70,000 as against £120,000). Thus very considerable savings were achieved by the former method and in view of the fact that Table 1 shows that significant lengths of motorway cuttings and embankments are at risk the method may well find wide applications for the repair of slopes.

Conclusion

(1) A recent survey into problems of slope stability in motorway cuttings and embankments has indicated that significant lengths of slope constructed in over consolidated clays have failed and, by implication, further lengths are liable to failure in the longer term.

(2) The repair of slope failures is traditionally carried out by replacing the failed soil with a free-draining granular material. However, where a source of such material is not locally available, considerable expenditure can be incurred by the haulage costs involved in the transportation of suitable fills to the site over relatively long distances.

(3) An alternative procedure has been described for the repair of failed slopes which involves re-use of the failed soil in conjunction with reinforcement layers of geotextiles or geogrids. It may also prove necessary to use a relatively small proportion of quicklime in carrying out the reinstatement to enable the construction plant to operate effectively when the soils are very soft. To assist in determining the reinforcement requirements for a particular problem, a number of design charts have been prepared which permit a rapid assessment to be made of slope stability for a range of typical situations.

(4) The application of the technique to the repair of a failed cutting in London clay has been described. The use of a polymer mesh reinforcement (Netlon CE 131) in conjunction with 2 per cent by weight of quicklime enabled the soft, soured, soil to be re-used and since the reinstatement in 1980, site observations have shown no indication of further movement. The cost of the repair by this method amounted to just over half that estimated for the conventional approach; this reflects the fact that haulage distances of some 30 miles were anticipated for importation of the granular fill.

(5) In view of the lengths of motorway, slopes in over-consolidated clays potentially at risk from instability, the application of the method could result in significant savings nationally.

Acknowledgements

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References


La Honda slope repair with geogrid reinforcement

R. A. Forsyth and D. A. Bieber, California Department of Transportation

As a result of a series of storms almost unprecedented in their intensity and duration beginning in January 1982 and continuing through the past winter, the California highway system sustained severe damage. Consequently, the Department faced significant repair and restoration costs. Extensive damage occurred south of San Francisco on Route 84 near La Honda, California caused by the action of a contiguous stream that eroded the highway embankment causing a slippout. The site geometry and right-of-way constraints required restoration of the original embankment to a slope which was somewhat steeper than that which would assure long-term stability for the soils in the area. The site cross-section diagram (Figure 1) illustrates the critical section where the embankment must be reconstructed. After an initial investigation, the Materials and Hydraulics Engineers recommended the use of earth reinforcement to develop embankment stability. After consideration of several systems, Tensar geogrid was selected. Subsequently, a cooperative research agreement between Caltrans and Matlon was negotiated.

SITE PARAMETERS AND BACKGROUND

The slippout site is 70 m in length requiring slopes varying from 1.5:1 to approximately 1:1. The streambed is 14 m below the freeway grade and 11 m laterally from the hinge point on freeway grade at the critical cross-section. Access to the area is limited. The water table fluctuates up to 3 m and corresponds with changes in the stream elevation. Acquisition of additional right-of-way was not possible.

As a result of a series of storms almost unprecedented in their intensity and duration in January 1982, the toe of the highway embankment on Route 84 near La Honda was eroded by the action of a contiguous stream causing a slippout. Site geometry required restoration of the embankment with oversteepened (greater than 1:1) slopes that were strengthened by utilizing Tensar geogrid reinforcement. Embankment design parameters and calculations are presented and design features to mitigate drainage problems and to prevent future erosion at the embankment toe are described. The results of laboratory pullout tests are summarized. Construction, which has been suspended for the winter will be completed the summer of 1984.

Fig. 1 Cross-section Diagram

The initial storm damage repair report called for rock slope protection 4.5 m high on a 1.5:1 slope placed at the embankment toe. Maintenance forces cleared the streambed of log jams and debris and placed 376,500 kg of rock slope protection as an interim repair. Permanent repair required rebuilding the upper embankment to slope ratios as steep as 0.3:1. Earth reinforcement would be necessary for reconstruction. The Caltrans' Transportation Laboratory initiated design of the reinforced embankment utilizing slope stability analysis and information generated from previous large scale laboratory pullout tests conducted on Tensar ER-2 (renamed Tensar SR-2).
PULLOUT TESTS

Large scale laboratory pullout tests were performed in 1980 on Tensar ER-2(1). The test apparatus consisted of a rigid steel box 46 cm deep, 92 cm wide and 137 cm long in which soil is compacted half way, the geogrid material placed on this layer, and the remainder of the box filled with soil and compacted. A hydraulic ram located above the test box simulates overburden loads up to an equivalent of 15 m of earthfill. A horizontally positioned hydraulic ram attached to the geogrid provides the pullout force. Displacement is adjusted to maintain a controlled strain rate of approximately 2%/min. The pullout tests were performed on Tensar ER-2 in the direction the fabric is drawn from the roll.

Utilizing decomposed granite from an unspecified site as fill (φ equal to 35°) and imposing an overburden load equal to 34.5 kPa, the geogrid was pulled to failure. The Tensar ER-2 failed in tension outside the soil block (Plates 1 and 2) at a load of 44,000 newtons/meter indicating design would be limited by the geogrid material’s maximum tensile strength. Load versus deformation curves were developed from the pullout tests including a comparison between Tensar ER-2 and bar mesh reinforcement (Graph 1). Bar mesh reinforcement of soil has been used by Caltrans to strengthen wall supported embankments(2). The bar mesh (constructed from 0.95 cm diameter reinforcing bar welded to form 10 cm by 20 cm spacings) has sufficient steel to preclude tensile rupture and force a slip failure within the soil block. Thus, the bar mat fails in pullout producing a cone shape failure near the soil face. From the graph, the ultimate strength of the Tensar can be obtained and used in design calculations.

Graph 1: Load Versus Deformation Characteristics of Bar Mesh & Tensar ER-2

Plate 1 Tensile Break, Pull Section

Plate 2 Tensile Break, Face Plate

DESIGN

Computer slope stability analysis was used to determine the safety factor of the reconstructed embankment without reinforcement. From triaxial compression testing of samples of the native soil, properties at the La Honda site were determined to be:

\[
\text{Cohesion} = 2.5 \text{ kPa}, \quad \phi = 32^\circ
\]

These parameters, along with site dimensions, were input into SLOILX, a circular slope stability program utilizing the modified Bishop technique. From the computer analysis, a minimum safety factor of 0.78 was calculated for the unreinforced embankment. Because an overall safety factor of 1.2 or greater was desired, additional resisting moment due to the soil strength increases from the reinforcement had to be quantified and new safety factors generated.
Utilizing the computer generated overturning moment, and friction and cohesion resisting moments, the required reinforcement to increase the resisting moment later was determined. The following equations illustrate the calculations used for estimating safety factor (S.F.) increases as a result of the added reinforcement.

\[
S.F. = \frac{\text{Resisting Moment}}{\text{Driving Moment}}
\]

\[
1.2 = \frac{2.01 \times 10^6 \text{ newton-meters} + \bar{a} \cdot STn}{2.57 \times 10^6 \text{ newton-meters}}
\]

\[
\bar{a} \cdot STn = \frac{1.2 \cdot (2.57 \times 10^6) - 2.01 \times 10^6}{2.01 \times 10^6} = 1.07 \times 10^6 \text{ newton-meters}
\]

where: 2.01 \times 10^6 (nt-m) = Resisting moment

\[
\bar{a} \cdot STn = \text{Total Tensar moment}
\]

The coordinates of the failure arc and the location of the centroid of the reinforcement are used to determine the distance (\(\bar{a}\)) to the centroid of reinforcement. In this case, \(\bar{a} = 14.3\) meters (Figure 2).

\[
\text{Tensar moment} = \bar{a} \cdot STn = 1.07 \times 10^6 \text{ newton meters}
\]

\[
Tn = \frac{1.07 \times 10^6 \text{ newton meters}}{14.3 \text{ m}} = 74,720 \text{ newtons}
\]

The allowable working strength of the Tensar was limited to 6,670 newtons/meters (15% of the ultimate strength). Knowing the working tensile strength contributed per meter, the number of reinforcing layers was determined.

\[
74,720 \text{ newtons} \div 6,670 \text{ newtons/layer} = 11.2 \text{ layers}
\]

Based on these calculations, vertical spacing of the Tensar was standardized at 0.6 meter. Circular failure arcs were plotted to assure that the embankment depth of the reinforcement was sufficient to preclude pullout of the Tensar. Figure 3 illustrates the geometrics of the critical cross-section. The embankment is approximately 14 m high. The lower 4.5 m has a slope ratio of 1.5:1 and is covered with 1 meter of rock slope protection to prevent water scour. The slope ratio in the upper reinforced portion of the embankment is steeper than 1:1. Permeable material lined with filter fabric is placed at the interface of the original ground and the reconstructed embankment. The permeable blanket drains into a horizontal outlet pipe located at the embankment base. The Tensar geogrid extends from the permeable material to the slope face.

Each layer of reinforcement will be fold back a minimum of 1.5 m and anchored in place. The face of the embankment is lined with compacted straw. The total amount of Tensar SR-2 required to complete the slope is 6000 square meters.

**Fig. 3** Final Design Geometrics

**INSTRUMENTATION**

In order to monitor the stability of the reinforced embankment and the performance of Tensar SR-2, instrumentation was incorporated in the fill.

The instrumentation placed in the embankment includes:

1. One Inclinometer installed through the reinforced embankment structure.

2. Extensometers at three levels to monitor lateral movement and internal strains within the reinforced embankment.

3. Survey reference points at the hinge line and toe of the embankment to monitor vertical and horizontal surface deformations.
ALTERNATIVE REINFORCEMENT

Before reaching an agreement with Nelon, tire sidewall reinforcement was considered at the La Honda site. Tire sidewall reinforcement consists of recycled tire sidewalls hooked together with steel rods. The system's material costs are low, but construction is labor intensive. The embedment depth necessary to ensure slope stability was equivalent to that necessary to achieve slope stability with Tensar SR-2. Due to variations in limiting lengths of reinforcement, a total of 4800 m$^2$ of tire sidewall reinforcement was considered sufficient to stabilize the embankment to the desired safety factor.

COST COMPARISON

Comparing the cost of Tensar SR-2 to the cost of the required amount of tire reinforcement revealed a substantial cost savings for the Tensar project. Bar mat cost comparison is included, though actual design was not initiated.

Tensar Grids

\[
\text{6000 m}^2 \times \$4.50/\text{m}^2 = \$27,000
\]

U.S. Currency 1982

Tire Reinforcement

\[
4800 \text{ m}^2 \times \$13.45/\text{m}^2 = \$65,500
\]

U.S. Currency 1982

Bar Mat Reinforcement

\[
2060 \text{ m}^2 \times \$67.30/\text{m}^2 = \$138,600
\]

U.S. Currency 1982

By using Tensar geogrid reinforcement, Caltrans was able to realize the greatest cost savings.

CONSTRUCTION

Due to inclement weather and restrictions in the bid process, the construction at La Honda has been shut down until the spring of 1984. When construction resumes and the project is complete, Caltrans will publish research results of the slope repair at La Honda using Tensar geogrids.

CONCLUSIONS

As a result of pullout tests and design cost analysis, Caltrans has found that geogrid fabrics can be an economical earth reinforcement system. The long-term benefits realized by using geogrids in corrosive environments are considerable. Though not as strong in tension as other types of reinforcement, the La Honda project should demonstrate that satisfactory results can be obtained with geogrid reinforcement at a cost savings.

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REFERENCES


Stabilization of Canadian Pacific Railway slip at Waterdown, Ontario, using Tensar grid

This paper presents a case history of the use of Tensar geogrid to construct a railway embankment on the side of The Niagara Escarpment, at Waterdown, Ontario. The railway was on a sidehill embankment built on an old landlip in the clayey silt till overlying the dolomitic limestone caprock of the escarpment. The track had a history of movement extending back over many years and it required frequent maintenance. Sudden significant movement of the embankment in December, 1982 caused rail traffic to be suspended. The old failure surface was excavated and the embankment reinstated at a slope of 45 degrees using granular fill reinforced with Tensar SR2 geogrid.

J. R. Busbridge, Golder Associates

INTRODUCTION

The Town of Waterdown is located about 50 km south west of Toronto and 6 km north west of Lake Ontario (Figure 1). South of the town a CP Rail track follows a route along the west side of a deep ravine and at "Mileage 4.0" it is located on a sidehill embankment constructed about 6 m from the top of the ravine. About 12 m below the elevation of the track, rock outcrops on the side of the ravine. From about 6 m below the top of the rock outcrop, a scree slope extends down to a creek over a height of about 35 m at an average gradient of 2 horizontal to 1 vertical (Figure 2).

The track in the area of "Mileage 4.0" had a movement history extending back over many years. The movement was reported to occur generally after periods of heavy precipitation such as during spring melt or after a heavy rainfall. Regular inspection and maintenance were necessary and a reduced speed limit of 8 km per hour was imposed by CP Rail.

In December, 1980, Golder Associates was retained by CP Rail to investigate the reasons for the movement and provide recommendations for stabilizing the track. The investigation carried out included boreholes put down to bedrock in the immediate vicinity of the track and a shallow borehole put down by portable drilling equipment at a location downslope of the track. This information was supplemented with a slope survey and mapping of the bedrock outcrop. In addition, the movements were monitored by means of an inclinometer placed at the shoulder of the embankment.

SITE DESCRIPTION AND INVESTIGATION

Waterdown is located close to the edge of the Niagara Escarpment. The escarpment is a massive topographic feature following the edge of the "Michigan Basin" which was formed during the Silurian Period (Figure 1). Sediments deposited in the basin were eventually compressed into rocks of varying hardness ranging from soft shales and sandstones to the more durable dolomite which generally forms the caprock.

Fig. 1. Site location

Fig. 2. Slope prior to stabilization

Various erosive agents have subsequently removed the softer shales underlying the dolomite. As the underlying material is eroded the dolomite becomes undermined and blocks break off creating the near vertical face of the present day escarpment.

The ravine along which the C P Rail track is aligned has been cut through the escarpment by the erosive action of the creek. The scree slope below the bedrock outcrop is formed from the debris arising from the cycle of erosion and scree formation. Above the bedrock, periodic glacier advances deposited till on top of the bedrock. As the bedrock is being eroded the overlying till is being continuously degraded by a process of gradual instability.

The subsurface investigation indicated that the till was supported on loose fill overlying a very stiff to hard silt till. The till was in turn underlain by a 3 m thick layer of dolomite which forms the caprock of a series of sandstones, mudstones and shales (Figure 2). Monitoring established that the lateral movement beneath the embankment shoulder was confined to the till and the main movement was taking place at the till/fill interface. Over a 3 month period from March to June, 1982, a total downslope movement of 39 mm was recorded.

Piezometers sealed into the till and the underlying bedrock indicated that while the water pressures in the sandstones, mudstones and shales were very low (piezometric level at elevation 190 m, Figure 2), the water pressure in the till was high with the piezometric level close to the top of the stratum. It appeared that the dolomite caprock was preventing underdrainage of the till by the more permeable underlying rock layers.

Periodic monitoring continued over a period of two years. It was concluded that the railway embankment had been placed on an old failure in the till. High groundwater pressure in the till had probably been a major cause of the initial failure. Since then, surficial run-off from snow melt or rain would ingress to the base of the fill and activate movement along the old failure surface, which in turn would reflect in movement of the track.

Effective stress analyses of the slope were carried out using laboratory measured shear strength parameters for the till and fill. Having established the marginal stability of the slope in the computer model, the analysis was extended to investigate the relative merits of alternative remedial schemes. Details of these analyses are given by Bushbridge, Chan and Sward (1984).

With private property immediately above the rail track and the near vertical escarpment rock outcrop located 12 m below, there was no room for regrading the slope. It was evident that the problem of the high groundwater pressures which had precipitated the original failure would have to be solved in any stabilisation scheme. The best opportunity for reducing this pressure seemed to be to utilize the sandstones and silt bedrock as an underdrain by drilling a series of vertical drainage holes down through the till and dolomite caprock into the underlying more permeable layers. These holes would intercept the recharge through the till and lower the phreatic surface. In addition to the drainage scheme, it was proposed to rebuild the railway embankment using earth reinforcement to maintain a side slope of 45 degrees.

On December 25, 1982 an exceptional thaw resulted in a 200 to 250 mm vertical movement of the track and a crack developed along the shoulder of the embankment. At that point, stability of the track could not be assured and rail traffic was suspended. It was decided to implement the track rebuilding immediately, using earth reinforcement.

STABILIZATION WORKS

Excavation of the failed slope section was carried out from December 29 to 30, 1982 using two excavators. Benches, 1.2 to 3 m wide, with steps of about 2 m were made in the till along the natural ground profile below the old failure surface (Figure 3). The excavation was 37 m long. Immediately under the tracks the railway fill was found to rest on undisturbed till. Downslope of the track a discrete shear surface within the till was identified and this was found to extend down to the surface of the bedrock, which was 4.5 m higher than originally inferred from the bedrock outcrop mapping. The extent of the old shear surface and the higher elevation of the bedrock called for a revision to be made to the design. The excavation was taken down to the bedrock and all pre-sheared surfaces were removed (Figures 3 and 4). A 2 m high rockfill toe found on bedrock was built to form the toe of the new embankment. Compacted granular material was placed as a filter between the till and the rockfill. Construction of the rockfill toe and granular filter was completed on December 31, 1982.

The slope was then built up at a gradient of 1.5 horizontal to 1 vertical using the same granular material. A typical grain size distribution curve of the granular material is given in Figure 5. The exposed surfaces of the natural till were free of loose and softened materials prior to filling. Horizontal surfaces of the till were trimmed to a downslope fall of not less than 5 per cent to assist natural drainage into the granular fill. Compaction of the granular material to a density of not less than 95 per cent of the Standard Proctor density was carried out using vibratory smooth drum rollers. The average dry density of the compacted fill as determined by an in situ nuclear density testing device was 2.0 t/ cu. m. Two layers of reinforcing geogrid were placed at 1.2 m height intervals within the lower rebuilt slope. The reinforcement used was Tensar SR2
soil reinforcing grid supplied by Tensar Incorporated. Each strip was 3.7 m long and was placed against the face of the excavation. This portion of the fill slope, which actually forms the base for the steeper earth reinforced slope, was completed on January 2, 1983.

The upper earth reinforced slope is 5.5 m high, and was built at a gradient of 45 degrees. Six layers of reinforcing grid (Tensar SR2) were laid at height intervals of 1.2 m. The length of the reinforcing layers varied from 3.5 to 5.5 m as shown in Figure 3. Each strip of 1 m wide reinforcing grid was cut to the design reinforcing length, plus an additional length for wrapping around the slope surface and overlapping with the next layer. The strips were laid with a transverse overlap of 50 to 100 mm and granular fill was placed with some overbuilding beyond the design gradient to enable adequate compaction near the finished slope surface (Figure 6). When the elevation for the next layer of reinforcing grid was reached, the alignment of the slope was set out and the slope surface was trimmed to the required gradient. The trimmed slope surface was covered by reinforcing grid from the previous (lower) layer, and the grid taken into the embankment for a minimum length of 1 m. The grids were held in place by 0.2 m long steel pins driven into the fill (Figure 7). This process was repeated for each layer of reinforcing grid until the top of the embankment was reached, on January 8, 1983. Train traffic resumed on January 9, 1983, which was 12 days after starting the works. A total of 62 rolls (each 1 m wide and 30 m long) of reinforcing grid was used, while the actual length of Tensar SR2 laid in the slope was about 150 linear metres. Wastage due to cutting the reinforcement lengths from the 30 m long rolls was about 6 per cent.

**Fig. 3.** Beached slope after excavation of old failure surface

**Fig. 4.** Stabilized slope

**DESIGN OF THE EARTH REINFORCED SLOPE**

As in any earth reinforced structure, the first design consideration was to ensure overall stability of the reinforced mass. This is particularly important in stabilization of a natural landslip where critical conditions originally existed in the natural soils. Where groundwater pressures are present, it is necessary to ensure good drainage within and behind the reinforced zone. In addition, the low winter temperatures in Ontario mean that a non-frost susceptible layer of soil should be placed in the outer 1.2 m of fill to avoid stresses in the reinforcement caused by frost heave. In order to meet both these criteria, free draining granular material was specified for the fill.

The design, which had been prepared prior to the emergency which precipitated the construction, had included vertical drains to lower the piezometric level in the fill. Excavation of the till down to bedrock at the bottom of the slope and replacing it with granular fill resulted in greater drawdown of the piezometric level than had been considered in the original design. Analyses of the revised section indicated that the vertical drains could be dispensed with.

**Fig. 5.** Typical grain size distribution curve of fill
Slope stability analyses were carried out using a computer program based on the method developed by Sarma (1973). A back analysis of the original slope profile assuming limiting equilibrium along the observed failure surface indicated an average cohesion along the failure surface of 3 kPa if a friction angle of 31 degrees is assumed (Figure 8). This is only 4 kPa lower than the cohesive intercept measured together with the same friction angle in a laboratory drained triaxial test on an "undisturbed" sample of till.

Since the upper section of the failure surface was between railway fill and undisturbed till, the figure calculated by the back analysis does not accurately reflect the residual shear strength of the till mobilized in the bottom section of the failure surface.

The shear strength of the granular fill used in construction of the earth reinforced slope was established by three direct shear tests. These gave friction angles varying from 32 to 38 degrees with zero cohesive intercept.

For design, a friction angle of 35 degrees and a unit weight of 21 kN/cu.m were assumed for the fill.

Using these parameters for the fill and a friction angle of 31 degrees, together with a cohesion of 7 kPa for the undisturbed till, the factor of safety along the critical failure arc extending behind the earth reinforced structure was calculated to be 1.3 (Figure 9).

An important design consideration was that any remedial measures would not adversely affect the stability of the overall slope including the property above the rail track. Because of the improved drainage of the slope provided by the granular fills, the stability of the area immediately above the track was increased. An analysis of deep seated stability involving the upper slope indicated that the factor of safety for this mode of failure was 1.2 which is the same value calculated for the slope prior to stabilization (Figures 8 and 9).

Internal stability of the earth reinforced slope was established by designing the reinforcement to resist the maximum horizontal earth pressure within the reinforced mass. The general procedure adopted was the "tied-back" method which has been described by Murray (1980) for vertical walls. The design force was obtained by investigating
a series of trial failure surfaces exiting at the toe and at various points on the slope. For critical surfaces, the tensile capacity of the reinforcement intercepting the failure surface was checked together with the pull-out resistance of reinforcement extending behind the critical surface.

The computer analysis adopted has the facility of computing the horizontal force required to maintain stability of any failure surface investigated. Circular arcs and two-part wedge failure surfaces were investigated together with Coulomb planar surfaces. It was found that for the 45 degree slope of the embankment, the planar surface underestimated the restraining force necessary to maintain equilibrium by about 50 per cent. For calculating the maximum forces in the reinforcement, circular failure arcs were adopted.

The following design parameters were adopted:

- Minimum Factor of Safety for overall stability: 1.3
- Allowable Tensile Capacity in Tensar Sn2: 16 kN/m
- Minimum Factor of Safety against pull-out: 2
- Coefficient of friction between soil and geogrid: 0.8 tan φ'

COMMENTS

The Waterdown stabilization, to the best of the author's knowledge, was the first use of geogrid in Canada for slope stabilization. In the months prior to the works described, Golder Associates had been working closely with the Ontario Ministry of Transportation and Communications in designing the earth reinforced slopes for the Highway 410 embankments, which are described in Mr. Devata's paper to this conference (Devata, 1984).

The method of face support adopted at Waterdown, which involved turning the horizontal reinforcement up the face and lapping with the next layer, proved to be labour intensive and time consuming. Using the experience gained at Waterdown, and after consultation with Tensar Incorporated engineers, this detail was revised for the Highway 410 embankment. The detail described by Mr. Devata of providing short horizontal layers of reinforcement at compaction lift intervals between the main reinforcement layers proved to be easier to construct.

Difficulty was experienced at Waterdown in achieving adequate compaction near the edge of the 45 degree slope. Although this is helped to some degree by providing intermediate horizontal strips, in cases where it is necessary to specify clean granular material at the face of slopes steeper than 45 degrees, facing panels or temporary formwork is required to provide the necessary reaction for compaction. For vertical walls, the use of Coulomb's earth pressure theory for determining the maximum force in the reinforcement is adequate for design. However, as the wall/slope becomes flatter, Coulomb's assumption of planar failure surfaces leads to significant underestimation of the forces involved and circular arcs or two-part wedges must be investigated. For the case described in this paper, the method of stability analysis developed by Szczuka (1973), which is readily available in computer programs, proved to be a convenient and reliable design method.

As the slope flattens from the vertical, the forces in the reinforcing elements reduce significantly and the use of polymer geogrids as earth reinforcement provides an attractive and economical solution to the problem of constructing steep slopes. In the case described, the geogrid reinforcement allowed a railway embankment to be constructed in the limited space and at the same time provided effective stabilization of an old landslide.

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REFERENCES


Repair of landslides in the San Francisco Bay area

R. Bonaparte, The Tensar Corporation, and E. Margason, Edw Margason Geotechniques

INTRODUCTION

Geogrids are high strength, orientated polymer grid structures used to reinforce soils. This paper describes the use of geogrids in a large landslide stabilization and road repair project in San Pablo, California. The repair extended over a road length of about 150 m (500 ft), with excavation depths in the road right-of-way of 15 m (50 ft). Geogrids were used because they were found to be more cost effective than other considered repair alternatives including gabion retaining walls, concrete crib walls, and conventional concrete retaining walls. The paper begins by presenting background on the geologic conditions and history of the slide area, and follows with a site description and results from site soils investigations. The landslide repair is then discussed. An emergency repair to temporarily stabilize endangered homes is described first. A subsequent permanent repair is presented next. The primary components of the permanent repair are a geogrid reinforced soil prism anchored into hard claystone and an extensive subsurface drain system. The design and construction of the reinforced soil prism is discussed in detail. Conclusions are drawn regarding the suitability of geogrids for the stabilization of landslides.

BACKGROUND

The City of San Pablo, California is situated near the east shore of San Francisco Bay, about 15 miles northeast of San Francisco. A hillside area within the city is located along the San Pablo Ridge. This ridge, which lies at the northwest end of the Diablo Range (part of the Pacific Coastal Ranges), is composed of Ordovician claystone with interbedded strata of sandstone and conglomerate, overlain in areas by surficial soils derived from the parent rock. Regional tectonic activity has caused intense shearing and folding of these rock strata, resulting in variable geologic and ground-water flow conditions. Surface faulting reflects the nearby presence of the Hayward Fault.

Hillcrest Road is situated in a north-facing hillside area along the San Pablo Ridge and has long been identified as having zones with actively unstable or potentially unstable slopes. Through the years, numerous geologic and engineering studies have identified many of these zones. In spite of the recognized potential for slope instability, portions of the hillside area have been developed for residential and commercial purposes.

California has experienced two consecutive winters of exceptionally heavy rainfall. In the hills surrounding San Francisco Bay, the rainfall has resulted in numerous landslides. Along transportation routes and in populated areas, these slides have required prompt repair. This paper describes the use of polymer geogrid reinforcement in the repair of a large and potentially disastrous slide along Hillcrest Road in the City of San Pablo. The slide repair had two components: (i) an emergency winter repair consisting of driven steel piling to temporarily stabilize endangered homes at the headscarp of the slide; and, (ii) a permanent summer repair. The permanent repair consisted of excavation of a deep prism of soil below the road right-of-way. The road excavation was refilled with compacted on-site soils reinforced with horizontal layers of geogrid. The steep face of the reinforced soil prism was formed using the "geogrid wrap-around" technique. The soil-geogrid composite, keyed into unweathered claystone beneath the deepest active slide plane, is designed to provide passive resistance against further movement in the headscarp area of the slide.

SITE DESCRIPTION AND HISTORY

Fig. 1 shows a site plan of Hillcrest Road and its surroundings. In the area of interest, the road runs in an east-west direction approximately 400 m (1300 ft) south of San Pablo Dam Road. From the "Dam" Road, the ground slopes upward, at first gently, then more steeply, to a series of homes that line the south (upvalley) side of Hillcrest Road. With the exception of a church and an apartment complex adjacent to the Dam Road, the downslope area is undeveloped.

Hillcrest Road had been disrupted by landslides in the past. In the 1950's, a slide damaged two homes and destroyed the road at the location shown in Fig. 1. A second slide disrupted the road in 1969. Smaller slides, slumps, tension cracks, and creep movements have been observed periodically, and geologic hazard studies of the hillside neighborhood (Woodward-Clyde Consultants, 1978 and 1983a) ranked this area as having a high potential for slope instability. In mid-February, 1983, a major slide scarp appeared along the south shoulder of Hillcrest Road just west of the 1950's slide. This large headscarp, associated with the heavy winter rains of 1981/1982 and 1982/1983, endangered the houses along the south side of the road. In order to save the homes, the City of San Pablo commissioned an immediate emergency headscarp stabilization program, followed by a permanent repair of the road and headscarp area the following summer.

Fig. 1 Hillcrest Road and surrounding area.

SITE AND SOIL INVESTIGATION

Five exploratory soil borings were made during February and March 1983 (Fig. 3). The borings identified a road fill and an upper soil zone of stiff to very stiff, light brown sandy and/or silty clay, underlain by a thinner zone of weathered, sheared, siltstone and claystone. Bedrock consisting of hard gray Orinda Claystone lay below the weathered siltstone and claystone. Slope indicators were installed in each finished boring.

Analysis of the slope indicator data suggested several active slide planes beneath Hillcrest Road (Fig. 2). An upper and most active slide plane was found about 3 m (10 ft) below road grade. A second active slide plane was found at a depth of about 6 m (20 ft) to 3 m (10 ft) below road grade. Movements along both of these slide planes were of the order of 0.03 m (0.1 ft) per day. Deeper seated movements were observed in the weathered zone, but these were much slower than in the upper more active zones. As Fig. 2 indicates, the slide planes dipped to the north. The shallow active slide plane was situated approximately at the base of the old road fill, while the lower active slide plane was controlled by the boundary of the light brown clay and the weathered, sheared siltstone and claystone. The deep, slower movements appeared to be taking place on top of unweathered Orinda Claystone.

The site investigation revealed that the Hillcrest Road slide was actually the head scarp of a much larger slide covering a hillside area of about 50,000 m² (12 acres). The toe of the slide was evidenced in the parking lots of the church and apartment complex shown in Fig. 1. At both locations, ground heave resulted in structure distress and/or disruption of parking facilities.

REPAIR OF HILLCREST ROAD SLIDE

The Hillcrest Road slide repair had two components: (i) an emergency winter repair to temporarily stabilize the endangered homes; and, (ii) a subsequent permanent road repair. The emergency stabilization plan called for the driving of a series of steel H-piles along the upslope edge of Hillcrest Road. The permanent repair plan consisted of excavation of the slide mass in the road right-of-way north of the pile line, followed by construction of a drained prism of reinforced or retained engineered fill anchored into the hard claystone. While the conceptual design of the permanent repair was integrated with the emergency stabilization plan from the start, final design of this permanent repair was carried out after the emergency program had been completed.

Emergency Stabilization of Hillcrest Road Slide

Emergency stabilization consisted of driving 63 steel H-piles along the south curbline of Hillcrest Road, adjacent to the residential properties. The piles were driven through the active slide planes into hard claystone. The layout of the 12WF53 steel H-piles is shown in Fig. 3, and the slide scarp, pile-driving rig, and the endangered homes are shown in Fig. 4. The steel piles had two functions: (i) to act as temporary cantilever supports to prevent further uphill headscarp movements (and therefore movements of the homes) until the permanent repairs; and, (ii) to provide a shoring system so that the slide material in the road right-of-way could be safely excavated during the subsequent permanent repair. The piles were spaced at 1.2 m (4.0 ft) centers and, as a general rule, were driven to depths of about 10 m (33 ft) to 14 m (46 ft), or at least twice the depth of the lower active slide plane beneath the pile line. The piles stabilized the upslope headscarp area. By winter's end, continued movement downslope of the pile line had caused a 1.0 m (3 ft) high scarp along the downslope edge of the pile line.

Permanent Repair of Hillcrest Road

In the assessment of permanent repair alternatives, two requirements were: (i) the repair structure had to be built within the road right-of-way; and, (ii) the repair had to be completed quickly, before the 1983/1984 rainy season. The road was in the headscarp portion of the slide—so repair plans based on active lateral earth pressures were considered. It was concluded that these pressures could be resisted by a reinforced or retained soil prism (soil batter), keyed into hard claystone, and extending up to original road grade. The prism would be designed to stand unsupported, should future slope movements occur downslope of the road. Four alternative soil prism designs were considered: (i) geogrid reinforcement; (ii) gabion retaining wall; (iii) concrete crib retaining wall, and (iv) conventional concrete retaining wall. The decision to use the geogrid reinforcement was based on its lower cost.

Fig. 3 Hillcrest Road showing the slide scarp and the line of emergency stabilization steel H-piles.

Fig. 4 Hillcrest Road (looking west), showing a portion of the slide scarp.

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DESIGN OF PERMANENT REPAIR

A typical design cross-section of the slide repair is shown in Fig. 5. Plans called for construction to proceed with excavation of the area encompassing the reinforced soil prism as well as an area north of the reinforced soil prism extending at a 45° angle from the base of the prism, as shown in Fig. 5. The area between the anchored prism and the 45° slope was to be filled using random on-site fill compacted to 93% maximum dry density determined by ASTM Test Method D-1557. It was recognized that if the hillside area downslope of Hillcrest Road continued to move, the unanchored zone of random fill would gradually slope and the face of the reinforced soil prism would be unsupported and exposed. These factors were considered in the design.

An important component of the permanent slide repair was the subsurface drain system. The functions of the drain system were (i) to keep the prism fill drained; and, (ii) to keep water pressures along potential slide planes to a minimum. As shown in Fig. 5, the drain system beneath the reinforced soil prism consisted of a minimum 0.3m (1.0 ft) thickness of crushed gravel. Along the face of the fill, the drain system comprised a 6mm (0.25 in) thick plastic net (Netlon DN1) wrapped in a needlepunched, nonwoven geotextile (Trevira Spunbonded). The function of the geotextile is to act as a filter, allowing the flow of water into the plastic net drainage layer while preventing the movement of soil particles. Additional drain system details are given subsequently.

Design of Geogrid-Reinforced Soil Prism

The internal stability of the reinforced soil prism was analyzed using active Rankin lateral earth pressures. Geogrid spacings were based on the geogrid Tensar SR2 in the lower portion of the soil prism and the lower-tensile strength geogrid Tensar SS2 in the upper portion. The Tensar SS2 was used in the upper portion of the soil prism for two reasons: (i) the larger roll size would allow quicker placement of the geogrid; and, (ii) the quantity of reinforcement needed in the upper portion of the prism was small. The design was based on an allowable working stress in the geogrid equal to 40% of the strength measured in constant rate-of-strain tensile tests.

The reinforced soil prism was constructed using the on-site silty and sandy clay soils compacted to 92% of maximum dry density (ASTM Test D-1557). From experience on previous projects in the area, an effective cohesion of 24 kN/m² (500 psi/ft²) and an effective friction angle of 20° was used for the compacted fill. The factor of safety against sliding and overturning of the reinforced soil prism was evaluated using the sliding wedge of soil shown in Fig. 5. Because of eventual failure of the geogrid, support provided by the steel piles was neglected in the sliding and overturning analyses. A large strain effective friction angle of the preloaded soil along the slide plane of 10° was assumed, based on large-strain direct shear tests.

The design called for a key trench to be excavated through the lower active slide plane into hard claystone. To speed construction, sections of the key trench deeper than 12m (40 ft) were to be backfilled with imported graded aggregate. Above 12m (40 ft), the reinforced soil prism was to be constructed with compacted on-site fill and horizontal layers of geogrid. The vertical spacings of the geogrid layers were as follows: from a depth of 8.5m (28 ft) to 12m (40 ft), Tensar SR2 at 0.4m (1.3 ft) lifts; from a depth of 4.9m (16 ft) to 8.5m (28 ft), Tensar SR2 at 0.6 (2.0 ft) lifts; between the ground surface and a depth of 4.9m (16 ft), Tensar SS2 at 0.6m (2.0 ft) lifts.

Due to limited right-of-way, the geogrid-reinforced soil prism was designed with an average batter of 4:1, vertical to horizontal. To simplify construction, the battered face was stepped, 0.3m (1 ft) set-back at 1.5m (5 ft) vertical steps. Alignment of the key trench was controlled by the north curbing of Hillcrest Road and the 4:1 prism batter.

Geogrid Wrap-Around Facing Method

The geogrid wrap-around construction technique was used to form the face of the reinforced soil prism. This method had been used on previous geogrid projects, with scaffolding or formwork required to temporarily support the wrap-around face. On this project, the random fill provided the temporary support and no special scaffolding or formwork was needed.

The following procedure was developed to construct the wrap-around face for each 1.2m (4 ft) prism step: (i) compact the random fill one full step above the prism fill; (ii) using a small dozer, trim a vertical face in the random fill along the north boundary of the reinforced prism; (iii) place the geogrid reinforcement across the width of the prism with an extra length of geogrid draped up and over the vertical face of the random fill; (iv) place and compact the required thickness of prism fill; (v) pull the extra length of geogrid back over the prism fill to form the wrap-around and fill of the reinforcement layer; (vi) secure the geogrid tail to the prism fill using 0.15m (0.5 ft) steel pins; (vii) repeat steps i through vi for the next one or two layers of geogrid to complete the prism steps; (viii) repeat through vii to form subsequent prism steps.

CONSTRUCTION OF PERMANENT REPAIR

Excavation of Slide Material

Repair of the Hillcrest Road slide commenced in September 1983. Excavation of the roadway was carried out to a depth of 3m (10 ft), with the excavated soil being stockpiled in an open downslope area. A line of drilled-in anchored tiebacks was then installed along the active slip line to provide construction-phase pile support. Each tieback was anchored (grouted) in unweathered claystone, connected to an adjacent pile and preloaded. After tieback installation, excavation continued according to the plan outlined in Fig. 5. Below 6m (20 ft), a series of soil benches were left standing against the pile wall to provide resistance.
against kicking out of the toe. The maximum excavation depth varied from approximately 6m (20 ft) at the west end of the 150m (500 ft) long excavation, to 15m (50 ft) near the east end. These depths were required to get the key trench anchored into hard claystone.

The trench was constructed in two sections corresponding to the east and west sides of the site. Fig. 6 shows the excavation (looking west) at a time just after the east key trench had been excavated and filled, and prior to excavation of the west key trench. The random fill, raised one step above the geogrid-reinforced prism fill can be seen in Fig. 6, as can the geogrid wrap-around tail. The small dozer in the right foreground was used to cut the vertical face for the random fill step.

Construction of Geogrid-Reinforced Soil Key Trench

The east key trench was constructed first, in one long section. On the east side, the maximum excavation depth was about 18m (52 ft.). Since the east trench was greater than 15m (40 ft) deep along its entire length, it was filled with graded aggregate. No geogrid reinforcement was used in the east trench.

As the long west key trench was excavated, several failures developed in the side slopes of the trench in front of the pile line. Blocks of soil began to move from the adjacent soil benches into the trench. These failures were associated with the loss of support of the slide material on the two sides of the trench. On the south side of the trench the failures were taking place on the pre-existing slide planes. Movement occurred both north and south of the trench, but the south movement was of greater concern because it reduced the support at the toe of the steel piles. The scarp that developed along the south side of the key trench can be seen in Fig. 7. As soon as the instability was identified, the west key trench was refilled.

The construction procedure for the west key trench was modified to reduce the risk of trench collapse and pile toe failure. The modified procedure included the following steps, designed to keep short sections of trench open for short periods of time: (i) excavate a 3m (10 ft) section of key trench using a backhoe (Fig. 8); (ii) place preassembled segments of 3-roll wide geogrid along the bottom and face of the trench; (iii) quickly dump 2m (6 ft) of gravel on top of the geogrid; and (iv) wrap the geogrid back over the gravel. This modified construction procedure, while slowing progress, allowed the key trench to be completed without further stability problems.

Construction of Geogrid-Reinforced Soil Prism

Construction of the main portion of the reinforced soil prism was carried out as soon as the west key trench was completed. Figs. 9 and 10 shows the methods and equipment used to construct the prism. Each step of the reinforced soil prism was started by cutting a vertical face into the random fill and placing the geogrid (Tensar SR2 in Fig. 9 and Tensar SS2 in Fig. 10). The geogrid extended from the gravel drain (on the left side of both Figs. 9 and 10) end up and over the vertical face of the random fill. The prism fill was placed using a 6m$^3$ (8 yd$^3$) scraper (Fig. 9). To speed construction, the scraper was allowed to drive directly on the geogrids. The acceptability of direct contact between geogrid and scraper was established through site trials. The prism fill was placed in 0.2m (0.67 ft) lifts and compacted using a sheepfoot roller (Fig. 10). At the soil prism face, compaction was carried out using a hand-operated vibratory compactor. After compaction, the geogrid was pulled back over the prism fill to form the wrap-around and tail (Fig. 6), as described previously. The minimum geogrid tail length was 1.2m (4 ft). Where required, the geogrids were cut with a hand-held power saw. To maintain geogrid continuity during construction, adjacent rolls of Tensar SR2 were connected with metal rings, and adjacent rolls of Tensar SS2 were overlapped a minimum of 0.15m (0.5 ft).
CONCLUSIONS

The Hillcrest Road project represents the first use of geogrids to repair a landslide in California. Significant aspects of geogrid use on this project included: (i) the reinforcing and retaining systems evaluated, the cost of the geogrid-reinforced system was lower than the estimated costs of the other systems; (ii) construction with the geogrids was rapid, allowing the project earthwork to be completed prior to the winter rainy season; (iii) geogrids allowed the use of the on-site clay fill materials; (iv) as shown by the slope problems in the west key trench, geogrid design and construction was adaptable to the specific site conditions; and, (v) geogrids provide a means of constructing cost-effective, steep-faced reinforced fills.

Based on the successful completion of the Hillcrest Road project and a second smaller project as well, geogrids are now being considered as a reinforced soil repair alternative for a number of other slides in California.

ACKNOWLEDGEMENTS

Both authors began work on the Hillcrest Road slide while members of the Geotechnical Engineering Group of Woodward-Clyde Consultants. Preliminary design of the geogrid-reinforced soil prism was carried out with the assistance of J. Dixon and J. Paul of Netton, Ltd., England. The authors are indebted to J. Dixon for valuable comments, and S. Hartmater and N. Stuart for assistance in the preparation of this paper. The authors thank Gary Leah and Gil Zermino from the City of San Pablo for providing information used in the paper.

REFERENCES


Reinstatement of slopes: report on discussion

R. T. Murray, Transport and Road Research Laboratory

With reference to paper 2.1, it was noted that in his presentation Dr Murray referred to the use of quicklime when reinstating the slope with Nelson reinforcement mesh. As quicklime or hydrated lime is very corrosive and yet commonly used in building sites and road sites, Dr Murray was asked whether any research has been carried out into this topic and whether any adverse effects were noted where the mesh and quicklime were in contact.

It was confirmed that no research is being carried out at TRRL into the corrosivity of quicklime. As described in the paper, the quicklime was employed to improve the properties of the London Clay such that the construction plant could operate effectively, particularly after wet weather. No adverse effects were noted to the exposed mesh in direct contact with the quicklime and as the inclinometer measurements through the reinstated slope showed no movements subsequent to completion, it is reasonable to suppose that the mesh at deeper levels was unaffected.

Mr A Burt, Project Engineer for the scheme, stated that there was no evidence of the quicklime having had any ill effects on the mesh. In any case the high moisture content of the clay produced a rapid and complete hydration of the lime such that this would have been unlikely.

Dr Murray confirmed that in his paper bilinear slip surfaces are sliding surfaces formed by two planes, corresponding to a two-part wedge. A figure is included in the paper showing the arrangement.

Dr Bonaparte was asked to comment on the reference in paper 2.4 to the scraper being permitted to operate directly on the Geogrid. In other papers the authors have placed considerable emphasis on trying to prevent this occurring.

In reply Dr Bonaparte agreed that he was aware of these recommendations and at the outset had specified that plant should not be permitted to work directly on the Geogrid. However, time was very short to complete the project before the rainy season, with the result that construction had to be expedited as much as possible. Field trials were therefore carried out to ascertain whether the requirement could be relaxed.

The trials demonstrated that the relatively small (7 cubic yard) scraper with large rubber tyres which was to be employed did not damage the Geogrid and therefore work proceeded accordingly. He did make the recommendation that without the field trials such an approach is not acceptable.

It was not obvious to another contributor where the excavation from the east trench was stockpiled. As there appeared to be quite a large volume involved, this could have proved to be a destabilising factor if located in the wrong place.

Dr Bonaparte stated that there were a number of off-site areas to stockpile. Permission was obtained to stockpile in these areas, although it was not permitted to erect the permanent structure on the owner's property. Some of the stockpile areas were on the 12 acres of land that was actively sliding and the contractor stockpiled some of the fill here initially. Some tension cracks developed at the top of the fill and action was taken quickly to move these stockpiles.

Dr Bonaparte, commenting on the relationship between estimated and actual costs, confirmed that the estimated cost of the Geogrid repair alternative was 60 per cent of the next closest alternative which involved the use of plastic coated steel gabions. It transpired that the final costs were fairly close to the estimated figures. However, the size of the job, and therefore the amount of soil and Geogrid involved, had changed dramatically during the course of the reinstatement works.

Participants: Dr Murray
Dr Bonaparte
Mr Burt
Mr Strudley
Mr Rankilor
Mr Paine