1. CONCEPTUAL DESIGN

The development of modern soil reinforcing techniques has been rapid, even so the benefits to be gained from their use have been demonstrated not only in the financial savings achieved but also in their ability to produce novel solutions to construction problems.

Due to the extensive lead times involved in civil engineering schemes it is probable that the first consideration of soil reinforcing systems will be as an alternative to a conventional solution. The disadvantages of substitution can be considerable; contractors inexperienced in the technique may tender high, short lead times for material delivery can cause logistical difficulties and the lack of knowledge relating to specific subsoil conditions may create design problems. The fact that reinforced soil can frequently provide financial benefits when used as a late alternative to a conventional design suggests that greater benefits could be obtained if the use of soil strengthening systems were considered at the conceptual design stage of any scheme.

Full benefit of soil strengthening systems can only be obtained if the designer is aware of the advantages and limitations of the technique and has access to the necessary analytical, testing and estimating procedures required for design. An essential requirement is a comprehensive soil survey which must be planned with the understanding that soil reinforcing techniques could form part of the design solution. In particular if finite element techniques are to be used in the analysis the soil survey may need to be supplemented to provide information relating to the initial stresses in the subsoil.

1.1 Retaining Walls

When viewed at the conceptual design stage reinforced soil walls present few problems, although their cost effectiveness may suggest vertical and horizontal alignments which could not be contemplated with conventional structures. Design and analytical techniques for conventional reinforced soil walls have now been established, although the complex mechanisms involved are not properly understood.

1.2 Bridge Abutments

Medium or small span bridges, if constructed using abutments, will have a significant proportion of the total cost invested in the substructure. Split costs of decks and abutments on these bridges have indicated substructure costs rising above fifty per cent of overall cost. Since reinforced soil has been shown to produce economies in abutment costs, significant reductions in total bridge costs are possible. The use of reinforced soil abutments cannot be accomplished without some change to the deck design, the span of which will almost certainly be increased at a cost dependent upon span and skew, Fig 1. In addition the possibility of differential settlement of reinforced soil abutments raises concern in articulation of the deck. However, this problem can be resolved with the use of a low torsion deck.

Fig 1. Deck Costs Relative to Span and Skew
On weak foundations bridge abutments are frequently supported on piles and the same solution can be adopted with reinforced soil abutments. However, an alternative idealisation may be possible based upon a different treatment of the overall stability analysis. In many analytical circumstances the abutment, piles and the adjacent embankment will each be considered separately and the effects on one element superimposed upon the others. If the abutment is subjected to vertical loads from the deck and the piles to lateral loads from the embankment, a frequent conclusion of this analysis is that the abutment will move away from the embankment and the piles should be raked forward. A global assessment of the behaviour of the abutment, piles and embankment, in which the ground movement caused by the placing of the embankment is also considered, may show an alternative behaviour. Abutment rotation may be towards the embankment; introducing an increase in moment in the abutment stress due to increased lateral pressures. Instead of providing compressive reinforcement in the form of conventional piles, a concept of tensile strain arc reinforcement may be used and the abutment reduces to a reinforced earth structure forming one end of the embankment, Fig 2.

![Diagrams](image.png)

**Fig 2.**
Practice indicates that the pile lengths associated with the idealisation of Fig 2(a) may be substantial, >30m on occasions, whereas the reinforcement length needed with the condition of Fig 2(b) may be substantially less. An additional benefit of the reinforced soil solution is that this idealisation effectively eliminates the problem of differential settlement between the abutment and the embankment. The design concept in these circumstances is similar to the procedure adopted in mining areas where piled foundations cannot be used due to the problem of differential subsoil strain caused by a moving subsidence wave. One solution is to provide a substantial bearing pad, up to seven metres thick, of compacted granular material under the abutment and to accept any residual differential settlement. The use of a thinner reinforced soil foundation formed as an integral part of the reinforced approach embankment is a practical alternative which has the advantage of minimising the incident of differential settlement which frequently occurs behind conventional abutments. The same concept can be used under central piers of a two span structure, although a degree of sophistication may be required in the analysis to permit the settlements of the abutments and the pier to be of the same order.

2. **ANALYSIS**

Walls and abutment structures are normally constructed using horizontal reinforcement and take the form illustrated in Fig 3. The vertical spacing of the reinforcement may remain constant throughout the depth, but the density may vary. Analysis covers two stability conditions:

1. External stability.
2. Internal stability.

![Diagrams](image.png)

**Fig 3.** Typical forms of walls and abutments

2.1 **External Analysis**

External analysis covers the basic stability of the reinforced soil structure as a unit, covering sliding, tilt/bearing failure, and slip within the surrounding subsoil or slips passing through the reinforced structure. Failure mechanisms of this nature are represented by
(c), (d) and (e) in Fig 4. In addition, stresses imposed upon the reinforced earth structure due to particular external conditions such as the creep of the subsoil have to be considered. Fig 4(f).

(a) Adhesion failure  (b) Tension failure

(c) Sliding  (d) Tilt/bearing failure

(e) Slip failure  (f) Tear failure

Fig 4. Failure Mechanisms of Reinforced Soil Walls

External Stability: The stability of the structure against forward sliding, cracking, tilting and overall stability of the supporting foundation and the adjacent retained fill may be checked as follows:

Forward sliding:

- Sliding forces \( \frac{1}{2} k_o y H^2 \)
- Resisting force \( \mu_k Y L_i \)
- Factor of Safety \( \frac{2 Y L_i}{k_o H} \)

where \( \mu_k \) is the coefficient of friction at the base of the wall.

Overturning:

Overturning moment with respect to the toe \( M_e = \frac{1}{2} k_o y H^2 \times \frac{H}{3} \)

Resisting moment with respect to the toe \( M_r = \frac{W L_i}{2} \times \frac{H L_i}{2} \)

\( \therefore \) Factor of safety against overturning \( = \frac{3 L_i^2}{k_o H^2} \)

Tilting: For structures situated on good subsoils a trapezoidal pressure distribution beneath the structure may be assumed. An allowable bearing pressure in the subsoil may be taken as half the ultimate bearing capacity, provided any resulting settlements can be tolerated by the wall and any superimposed structure.

From Fig 5(a) \( W = Lo (a+b) \)

\( r_c = \frac{(a-b) l_0}{l_0} \)

from which

\( a = \frac{1}{l_0} \left[ W + 6 \frac{F_a}{l_0} \right] \)

\( b = \frac{1}{l_0} \left[ W - 6 \frac{F_b}{l_0} \right] \)

where \( l_0 = \frac{1}{k_o y H^2} \)

\( j = \frac{H}{3} \)

\( a = \frac{Y}{l_0} \left[ 1 + k_o (H)^2 \right] \)

\( b = \frac{Y}{l_0} \left[ 1 - k_o (H)^2 \right] \)

If the computed value of \( a \) in equation seven exceeds the allowable, the stability of the base with respect to the ultimate bearing capacity may be improved by widening the base width of the structure to \( L_0 + L_0 \), Fig 3(b).

With a poor subsoil widening the base width may not be sufficient to satisfy the ultimate bearing capacity criteria for stability with respect to tilting. In this case support to the base may be provided by external means, Fig 6. Alternatively the overall stability of the structure and the surrounding soil may be considered on a global basis using continuum systems.

(a) Piles  (b) Stone column  (c) Foundation matress

Fig 6.
Rotational Slip: All potential slip surfaces should be investigated, including those passing through the structure, Fig 5(b). Where slip planes already exist, residual soil strength parameters should be adopted. The factor of safety for reinforced soil structures against rotational slip is the same as for conventional retaining structures.

2.2. Internal Stability

The internal stability is concerned with the estimation of the number, size, strength, spacing and length of the reinforcing elements needed to ensure stability of the whole structure, together with the pressures exerted on the facing, Fig 4(a,b).

Numerous analyses to check for internal stability have been developed, most fall into the following categories:

(a) Those in which local stability is considered for the soil near a single element of reinforcement, and

(b) those in which the overall stability of blocks or wedges of soil is considered.

The most widely used analytical procedures are semi-empirical systems which can be considered to represent the serviceability and ultimate limit state.

2.2.1 Coherent Gravity Hypothesis

The coherent gravity hypothesis relates to a reinforced soil structure constructed with a factor of safety in a state of safe equilibrium. The design stresses relate to actual working stresses not to failure conditions. Thus the coherent gravity hypothesis relates to the serviceability limit state.

Fig 7. Coherent Gravity Hypothesis

The coherent gravity hypothesis assumes:

- The reinforced mass has two zones, the active zone and the resisting zone, Fig 7.
- The state of stress in the fill, between the reinforcements is determined from measurements in actual structures constructed using well graded cohesionless fill.
- An apparent coefficient of adherence ($\mu^*$) between the soil and reinforcement is derived from an empirical expression developed from pullout tests.

Fig 8. Variation of apparent friction co-efficient $\mu^*$ with depth. 
(After Schloesser & Segregustin 1979)

For a structure using strip reinforcement, the maximum tension ($T_{max}$) per element at depth $h$:

$$T_{max} = \frac{K \cdot \sigma' \cdot \Delta H}{N}$$

where

- $\sigma'$ = soil strength in zone of action of the reinforcement
- $\Delta H$ = zone of action of the reinforcement
- $N$ = number of reinforcements per area considered
- $K = K_o \left(1 - \frac{h}{h_o}\right) + K_a \frac{h}{h_o}$: $h \leq h_o = 6m$

$$K = K_o \left(1 - \frac{h}{h_o}\right), \quad h > h_o = 6m$$

Similarly, the maximum adhesion force ($T_{ad}$) per element of reinforcement, assuming a well graded, cohesionless fill, $B$=reinforcement breadth, $L_r$=length of reinforcement within resistance zone:

$$T_{ad} = 2B \int_{L_{tr}} \rho^b \cdot C_u \cdot dL$$

where

- $\rho^b = \rho_o \left(1 - \frac{h}{h_o}\right) + \frac{h}{h_o} \ln \left(\frac{h}{h_o}\right)$: $h \leq h_o = 6m$

$$\rho^b = \left(1 - \frac{h}{h_o}\right) \ln \left(\frac{h}{h_o}\right), \quad h > h_o = 6m$$

$\rho_o$ for rough reinforcement is defined empirically as:

$$\rho_o = 1.2 + \log C_u$$

where $C_u$ = coefficient of uniformity

$$\rho^b = 0.4$$ for smooth reinforcement (15)

2.2.2 Tie-Back Hypothesis

The tie-back hypothesis is based upon the following design requirements for vertically faced structures:

- The design criteria is simple and safe.
- The design life of the structure is 120 years.
- The design procedure is consistent with the use of a wide range of potential fill materials, including frictional and cohesive-frictional soil.

Internal stability considerations include:

- The stability of individual elements, Fig 9.
- Resistance to sliding of upper portions of the structure.

* geogrid may be defined as rough in this analytical model.
The stability of wedges in the reinforced fill, Fig 10. (Note: Centrifuge studies indicate that at failure a wedge failure mechanism can develop. The tie-back hypothesis therefore relates to the ultimate limit state rather than the serviceability limit of the coherent gravity hypothesis.)

The following factors which influence stability are included in the design:

1. The capacity to transfer shear between the reinforcing elements of the fill.
2. The tensile capacity of the reinforcing elements.
3. The capacity of the fill to support compression.

The state of stress within the reinforced fill is assumed to be (K_a). The at rest (K_o) condition measured in some structures, Fig 7, is assumed to be a temporary condition produced by compaction during construction. The active state of stress is assumed to develop during the working life of the structure.

\[ T_{\text{max}} = T_{\text{hi}} + T_{\text{wi}} + T_{\text{ai}} + T_{\text{fi}} + T_{\text{mi}} \]  

where:
- \( T_{\text{hi}} \) = reinforcement tension due to fill above the reinforcement layer,
- \( T_{\text{wi}} \) = reinforcement tension due to uniform surcharge,
- \( T_{\text{ai}} \) = reinforcement tension due to a concentrated load,
- \( T_{\text{fi}} \) = reinforcement tension due to horizontal shear stress applied to the structure,
- \( T_{\text{mi}} \) = reinforcement tension due to bending moment caused by external loading action on the structure.

Local stability: The maximum tensile force \( T_{\text{max}} \) is obtained from the summation of the appropriate forces acting in each reinforcement as follows:

Although both the tie-back - wedge analysis and the coherent gravity analysis may be used with grid reinforcement, simplified analyses based upon the superior adhesion capacity can be employed. Performance of grids have been developed. Analysis is based upon an assumption of a Coulomb failure mechanism, Fig 12.

\[ T_i = K_a \gamma h_i \Delta h \]  

Pullout resistance: The total pullout resistance \( F_T \) is a combination of the frictional resistance \( F_F \) presented by the grid, plus the anchor resistance \( F_R \) of the grid.

\[ F_T = F_F + F_R \]  

The frictional resistance \( F_F \) per unit length of longitudinal wire diameter \( d \):

\[ F_F = \rho \cdot \pi d \sigma_v \]  

The Anchor resistance per transverse member based upon the Terzaghi-Buisman bearing capacity expression is defined as:

\[ F_T = d \cdot N_c \cdot N_f \cdot N_f \]  

where:
- \( N_c \) = number of transverse members outside the Coulomb failure wedge and,
- \( N_f \) = Terzaghi bearing capacity factors

Fig 10. Tie-Back Analysis - Wedge Stability

Fig 11. Angle of Failure Plane

Fig 12. Local tensile stress: For a uniform vertical distribution of horizontal grids the forces exerted (Ti) on geogrid (i) at depth hi:

A Coulomb wedge failure is assumed to be possible within the reinforced soil structure. For design, wedges of reinforced fill are assumed to behave as rigid bodies of any shape and all potential failure planes are investigated (i.e. \( \eta \) and \( \beta \) may both vary, Figs 10 and 11).

2.2.3 Tie-Back Analysis - Geogrids
Since $d$ is small, for a cohesionless fill

$$\frac{F_r}{N_w} = \sigma_d d N_d$$  \hspace{1cm} (22)$$

Total pullout resistance per unit width

$$F_r = (\sigma_d d L) + (\sigma_T d N_d)$$  \hspace{1cm} (23)$$

$$= \sigma_d d (L + N_d)$$

where $M =$ number of longitudinal members in grid per unit width

$N =$ number of transverse elements outside the Coulomb wedge

$\mu =$ coefficient of friction between longitudinal members and the fill.

3. CONSTRUCTION

Construction of reinforced soil structures must be of a form determined by the theory and in keeping with the design and analysis assumed. The theoretical form of the structure might be quite different to an economical prototype, and attention should be paid to the method of construction throughout the design process.

Speed of construction is usually essential to achieve economy, and this may often be achieved by the simplicity of the construction.

3.1 Construction Methods

Constructional techniques compatible with the use of soil as a constructional material are required. The use of soil, deposited in layers to form the structure, results in settlements within the soil mass caused by gravitational forces. These settlements result in the reinforcing elements positioned on discrete planes moving together as the layers of soil separating the planes of reinforcement are compressed. Construction techniques capable of accommodating this internal compaction within the fill are required. Failure to accommodate the differential movements may result in loss of serviceability or worse.

The three constructional techniques which can accommodate differential vertical settlements within the soil masses are shown in Fig 13. Except for some special circumstances, every reinforced soil structure constructed above ground uses one or other of a combination of these forms of construction.

Fig 13. Three Methods used for Constructing Reinforced Soil Structures

Concertina Method: The constructional arrangement of the concertina method developed by Vidal (1963) is shown in Fig 13(a). Differential settlement within the mass is achieved by the front or face of the structure concertining. Some of the largest modern reinforced earth structures have been built using this approach, and it is the form of construction frequently used with fabrics and geogrid reinforcing materials in both embankments and cuttings. Since the facing must be capable of deforming, a flexible hoop shaped unit made from steel or aluminium is normally used with strip reinforcement. Geogrids usually provide their own facing. The method is often used with temporary structures.

Telescopic Method: In the telescopic method of construction the settlement within the soil mass is achieved by the facing panels closing up an equivalent amount to the internal settlement. This is made possible by supporting the facing panels by the reinforcing elements and leaving a discrete horizontal gap between each facing panel (i.e. the facing panels hang from the reinforcing elements). (Vidal 1978). The closure between panels will vary from structure to structure depending upon the geometry, quality of fill material, size of the facing panels and the degree of compaction achieved during construction. Typical movements, reported by Finlay (1978), show vertical closures of 5-15mm for facing panels 1.5m high.

Sliding Method: In the sliding method of construction settlement within the soil mass is accommodated by having the reinforcement embedded within the soil slide down the facing whilst still remaining connected to the facing. The facing may be made up of discrete elements or may be single full height units.

Labour and Plant: Labour and Plant requirements for the construction of reinforced soil structures are minimal, and no specialist equipment or skills are required. Erection of a normal vertically faced structure of 500-1000m exposed area is usually undertaken by a small construction team of 3-4 men deployed to cover the main construction elements, namely, erecting the face, placing and compacting the fill and placing and fixing the reinforcement.

A comparison in labour requirements for different forms of retaining walls has been given by Leece (1979), see Table 1.
Table 1.

<table>
<thead>
<tr>
<th>Type of Wall</th>
<th>Labour Content Manhours/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced earth</td>
<td></td>
</tr>
<tr>
<td>without traffic</td>
<td>4.1</td>
</tr>
<tr>
<td>barrier</td>
<td></td>
</tr>
<tr>
<td>Reinforced earth</td>
<td>4.7</td>
</tr>
<tr>
<td>with traffic barrier</td>
<td></td>
</tr>
<tr>
<td>Mass concrete</td>
<td>11.2</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>11.5</td>
</tr>
<tr>
<td>Crib walling</td>
<td>13.3</td>
</tr>
</tbody>
</table>

The plant requirements during construction normally include aids to the placing and compaction of soil, and some form of small crane or lifting device, although the latter is not required when a non-structural facing is used. Where a method specification is employed, as with DTP Memorandum BE 3/78, the compaction plant used within 2m of the facing normally consists of the following forms:

(a) Vibro tampers,
(b) Vibrating plate compactors with a mass <100kg,
(c) Vibrating rollers with a mass/metre width <1300kg and a mass <1000kg.

Rate of Construction: Construction of reinforced soil structures is normally rapid. Construction rates for vertically faced structures of 40-200m² per day may be expected and usually the speed of erection is determined by the rate of placing and compacting the fill.

However, in some cases the economic production of facing units may determine the construction rate, particularly if an original or unique facing is required.

Construction is normally unaffected by weather except in extreme situations.

Damage and Corrosion: Care must be taken that facing elements and reinforcing members are not damaged during construction. Vehicles and tracked plant must not run on top of reinforcement: a depth of fill of 150mm above the reinforcement is frequently specified before plant can be used. Polymer reinforcement should be stored away from ultra violet light.

Compaction: Compaction of the fill in a reinforced soil structure is desirable as it has a beneficial influence on behaviour and increases the efficiency of the structure. Good compaction also reduces internal differential movements, whilst uniform compaction provides the most stable environmental conditions which are important for durability. Uniform compaction of the fill is achieved by placing the fill in layers varying in depth from 100-300mm, and compacting the soil using suitable plant moving parallel to the facing or edge of the structure.

Any normal compaction plant may be used with reinforced soil structures, selection of the most suitable depending upon the properties of the fill being used.

Distortion: Reinforced soil structures are prone to distortion particularly during construction. Many of the construction details adopted in practice are chosen to minimise distortion or the effects of distortion.

Concertina Construction: Structures built from fabrics and geogrids or constructed as temporary structures using the concertina method of construction are particularly prone to distortion of the face. The degree of distortion cannot be predicted. An accepted method of overcoming the problem is to cover the resulting structure either with soil or with some form of facing. An alternative is to provide a rolling block against which the compaction plant can act.

Telescope Construction: An estimate of the internal movements and distortion of the facing can be made from observations of prototype structures. Horizontal movements of the facing are made up of two components:

(i) horizontal movements at the joints
(ii) tilt of the facing units

(With extensible reinforcement some horizontal movement may be expected, in practice this is minor due to the conservative analytical procedures which are normally used.)

Joint movement during construction is not normally significant and is likely to be between 2-5mm depending upon construction details. Tilt of the facing panels can be significant and may have a marked effect upon the facial appearance of the structure, although other elements of serviceability are unlikely to be effected by tilt of the facing. All facing panels in this form of construction tilt, the pivot point depending upon the geometry of the facing.

Sliding Method Construction: When a non-structural facing is used distortion of the facing is likely to occur, the degree being dependent upon compaction. The distortion is accommodated by:

(a) using light plant in the 2m zone adjacent to the facing
(b) using bold architectural features to mask the distortion.

When a structural facing is used the horizontal movement of the facing will be limited to the joint movement capacity provided by the reinforcement/facing connection. Typical movements are 2-5mm.

Logistics: The speed of construction must be catered for if the full potential of the use of reinforced soil structures is to be realised. Normally this will cause little or no problems with the reinforcing materials, but the production and delivery rate of the facing units may cause problems, particularly if multiple use of a limited number of shutters is expected for economy.
Transport may cause difficulties and the choice of structural form and construction technique may ultimately depend upon the ease and economy of moving constructional materials. As an example, the light weight of geogrid materials with their ability to be transported in rolls makes them suitable for air freight.

Construction Sequence: Reinforced soil structures encourage the use of non-conventional construction sequences. It is possible to streamline the construction sequence and eliminate some steps as illustrated in Fig 14.

Alternatively it is possible to invert a construction sequence as in Fig 15, where the backfill of a structure is placed before the structure itself. This is achieved by forming the backfill into a temporary reinforced soil structure and using the face of this structure as the back shutter for the permanent structure. This technique has been used successfully in bridge works construction.

REFERENCES


Economics and construction of blast embankments using Tensar geogrids

The paper discusses briefly the various commonly used methods of constructing blast protection and other steep faced embankments.

A typical example is considered from which the economy of the system may be assessed. These savings are generally a combination of total cost and area of land take when compared with traditional systems.

Several structures of this kind have been constructed throughout the world and the various construction techniques are discussed along with surface treatment measures.

Possible future developments are also considered.

J. Paul, Netlon Ltd

Blast Protection embankments and walls are used throughout the world in many different applications.

Obvious examples are ammunition and explosives storage areas, certain types of electrical transformers, some gas and chemicals installations, explosives testing areas and the protection of critical areas from external attack.

Each type of application and each country has its own requirements for blast protection and no attempt will be made to discuss these variations. The example quoted later in this paper is broadly in agreement with the regulations governing the design of embankments around storage buildings containing high explosives ammunition in the United Kingdom. In this case the main aim of the embankment is to provide sufficient mass to contain the horizontal blast and to be sufficiently high to avoid “roll over” of blast and flames into adjoining areas. This latter requirement takes account of the height of the storage stack and the steepness of the face of the containing wall or embankment. With relatively flat sloped embankments the height may need to be increased to avoid the “roll over” problem.

Often, the construction consists of a reinforced concrete wall close to the building with an earth fill behind. The minimum thickness of embankment at the top level of the ammunition stack is 2.4 metres. In normal circumstances the outside face has no retaining wall and is a soil embankment constructed to a face slope suitable for the fill material used.

For lower heights of stack, brickwork or blockwork retaining walls may be constructed rather than reinforced concrete but the general layout is the same.

**Typical Example**

The following example gives an indication of the respective layout and costs for a vertical reinforced concrete faced embankment, a Tensar reinforced embankment and an unreinforced soil embankment using relatively poor quality fill.

In this example the top of stack level is taken as underside of roof level and the stack height is 8 metres.

The reinforced concrete wall is constructed using 250kN/m² concrete and the imported backfill has a compacted density of 19kN/m³ and a friction angle of 30°. The narrow crest width ensures light construction traffic therefore no surcharge loading has been included.

In the Tensar reinforced solution a face angle of 60° is taken and this uses Tensar SR-2 and SS-1 Geogrid reinforcement. Working load in the SR-2 Geogrid is taken as 18.5kN/metre width which ensures adequate overall factor of safety.

Foundation stability has not been considered in this example. In cases of low bearing capacity foundation material there is often a significant benefit when using reinforced soil techniques which can reduce the maximum toe pressure compared with that from a reinforced concrete wall.

**Dimensions**

<table>
<thead>
<tr>
<th>Height to top of stack, Hs</th>
<th>8.0 metres</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building wall thickness</td>
<td>0.3 metres</td>
</tr>
<tr>
<td>Minimum footpath width</td>
<td>1.5 metres</td>
</tr>
<tr>
<td>building</td>
<td></td>
</tr>
<tr>
<td>Minimum height of embankment, H is greater of</td>
<td></td>
</tr>
<tr>
<td>a) Hs + 0.6m.</td>
<td></td>
</tr>
<tr>
<td>or</td>
<td></td>
</tr>
<tr>
<td>b) Hs + 2° rise from stack corner.</td>
<td></td>
</tr>
<tr>
<td>Minimum width of embankment at Hs level</td>
<td>2.4 metres</td>
</tr>
</tbody>
</table>
1 - Reinforced Concrete Wall

Explosives storage

8.0m

2.5m

8.6m

1.5m

Fig 1 - Reinforced Concrete Wall

Design of Reinforced Concrete wall gives the following dimensions,

- Top of stem width = 0.35m
- Bottom of stem width = 0.50m
- Base width = 4.30m
- Base thickness = 0.50m
- Weight of steel reinforcement = 0.10t/m³

Per metre run,

- Volume of concrete in stem = 3.655m³
- Volume of concrete in base = 2.250m³
- Weight of steel reinforcement = 0.59t

Costs:

- Excavation: 2.25 x 5.07 = 11.41
- Provision of concrete: 5.905 x 41.33 = 244.05
- Placing in base: 2.25 x 5.94 = 13.36
- Placing in stem: 3.655 x 19.31 = 70.88
- Rough formwork, base: 2 x 0.5 x 13.84 = 13.84
- Rough formwork, wall: 8.6 x 13.20 = 113.52
- Fair formwork, wall: 8.6 x 15.52 = 133.47
- Steel Reinforcement: 0.59 x 431.13 = 254.37
- Compacted fill behind: 73.36 x 7.20 = 532.51

Wall
- Trim slopes: 18.2 x 0.59 = 10.74
- Grass seed to slopes: 18.2 x 0.32 = 5.82

Total cost per metre run = £1493.67

Land width required out with buildings = 18.9m (allowing 2.5m footpath for construction purposes)

2 - Tensar Reinforced Embankment

Fig 2 - Tensar Reinforced Embankment

A Tie-Back analysis using the given geometry and soil parameters allows the design of a Tensar reinforcement layout to give the required factor of safety.

Taking the working stress in Tensar SR-2 reinforcement (allowing for all safety factors) as 18.5KN/metre width, 9 layers are required. Vertical spacing varies from 0.6 metres at the base to 1.2 metres near the top.

To avoid possible bulging on the face short intermediate layers of Tensar SS-1 reinforcement are placed as shown in Figure 2.

Therefore, Volume of fill = 60.2m³

- Area of SR-2 = 117m²/m
- Area of SS-1 = 21m²/m

Costs:

a. Compacted fill (allowing for working within edge supports) at £7.70/m³ = 463.54
b. Tensar SR-2 at £3.50/m² (laid) = 388.50
c. Tensar SS-1 at £1.20/m² (laid) = 25.20
d. Edge support at £3.00/m² = 60.00
e. Grass seed to face at £0.60/m² = 12.00

Total cost per metre run = 1949.24

Land width required out with building = 13.5m
CONSTRUCTION

While a 60° face slope was chosen in the example for the Tensar Reinforced embankment there is no reason why this could not be increased to 90°. Several vertical, or near vertical Tensar Reinforced embankments have now been constructed in various parts of the world and a variety of construction techniques have been developed.

The first major vertical embankment was constructed at Warton, near Preston, England.

Requirements were for a vertical structure 7 metres high, 16.5 metres wide and a minimum of 3.5 metres deep at the crest. The purpose of this sand embankment is to allow the arming of aircraft standing close to the vertical face, ensuring the interception of any accidentally fired weapons.

Similar structures had previously been constructed as three-sided wooden boxes strengthened with a steel frame and filled with sand. In terms of labour requirements and overall cost the Tensar Reinforced solution was very attractive.

A scaffold frame was erected outside the front and side edges of the embankment with boards fixed vertically to support the faces during construction. As each completed layer of reinforcement and fill is self supporting the framework is relatively light using standard scaffold boards with gaps between, as shown in Figure Number 4.

<table>
<thead>
<tr>
<th>Embankment Type</th>
<th>Land Take</th>
<th>Cost per metre run</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Concrete Wall</td>
<td>18.9m</td>
<td>£1403.67</td>
</tr>
<tr>
<td>Tensar Reinforced Embankment</td>
<td>13.5m</td>
<td>£949.24</td>
</tr>
<tr>
<td>Unreinforced</td>
<td>32.5m</td>
<td>£1022.02</td>
</tr>
</tbody>
</table>

Economy of the Tensar Reinforced system is highlighted by this example.

Even greater savings are likely when blast protection is required between existing buildings, or close to boundaries or other obstructions. In these cases reinforced concrete or other vertical walls are required, sometimes on both sides of the structure.

Figure Number 5 shows a cross-section of the embankment during construction and Figure number 6 shows the completed structure.
Slopes up to 45° have been constructed without the use of edge supports. The face of each layer of fill is dressed by hand and the Geogrid Reinforcement taken up and secured over the top of each lift.

Above 45°, support is required. Blast walls constructed at Catterick Barracks, Yorkshire, England, were reinforced with Tensar SR-2 Geogrid and had a face slope of 60°. Again a scaffold frame and boards were used with a sacrificial vertical tube at approximately 4 metre centres (Figure Number 7).

As can be seen from Figure Number 8, construction of these steep faced embankments is relatively straightforward and requires no specialised techniques or equipment.

A view of this completed project is given in Figure Number 9.

In higher embankments, a horizontal tube is laid within the top layer of fill in each stage of 3 to 4 metres height. This tube projects through the face to support the sloping tubes for the stage above (Figure Number 10).

A sloping face has in several instances been approximated by a series of vertical sections stepped back at each lift. Light plywood shutters, including the standard modular systems have been used with a leapfrogging erection sequence which minimises the number of forms required. Generally, a row of forms may be removed when construction of the stage above is complete therefore only two rows of shutters are required no matter how high the structure (Figure Number 11).

The Tensar Geocell Mattress may also be used to give a vertical or stepped-vertical face.
Due to the unique, elongated apertures of Tensar uniaxial reinforcement a cellular mattress system is easily erected on site using pins to secure the joints (Figures 12 and 13). The 1 metre high cellular structure thus formed is filled with a granular material and they may be stacked one above the other to form an extremely rigid reinforced embankment. They may be used as a facing to the embankment (similar to gabions) but do not require to be filled with large stone which can cause an additional hazard in blast situations. When filled with sand or other fine material a geotextile liner is used to retain the fill. In narrow, steep sided structures the mattress extends from face to face and as may be seen from the diagrammatic view (Figure 14), damage to any area of the face is contained by the Tensar diaphragms bounding that cell. This is of special importance in the case of an embankment protecting buildings or equipment from external attack.

**FACE TREATMENT**

Treatment of the face of the embankment depends on the requirements of each individual project.

In many cases no surface treatment is specified and through time, weeds and grasses tend to establish themselves on the face, the seeds being trapped in the Tensar apertures.

To encourage this vegetation cover grass seed may be sown either dry or in hydraulic mulch, the seeds again being held in the Tensar apertures.

Some projects have used a different quality fill (including topsoil) close to the face to ensure rapid vegetation growth in aesthetically sensitive areas. Turf, placed against the face during construction gives even more rapid cover to the exposed geogrid.
FUTURE DEVELOPMENTS USING TENSAR GEGRIDS

Surface treatment of the steep embankment type of blast protection has, to date, consisted of natural vegetation cover. In general this is the most desirable form of treatment giving the most natural appearance. There may be some situations however where a harder finish is required and there are several possible alternatives.

A large aperture Tensar Geogrid may be fixed close to the face and sprayed concrete applied over the whole surface. This could be a useful solution in desert areas where no vegetation cover is possible and some protection is required for the Tensar against extremely high levels of ultra-violet radiation.

Work has been carried out on assessing the performance of Tensar reinforced cement composites under dynamic loading and positive results are reported in Session 7 of this Symposium.

Precast, Tensar reinforced concrete slabs may also be used to cover the face. These may be constructed with exposed-aggregate or other featured surface to give a pleasant visual appearance and a cast-in tail of Tensar would allow them to be securely fixed to the reinforced slope (Figure 15).

Heavier concrete panels have been used as facings to vertical, Tensar reinforced soil walls as have brickwork and blockwork. These are almost certainly lower in cost than the traditional hard surface of a reinforced concrete wall.

Some low intensity blast situations are contained by walls of the building housing the equipment or by relatively thin walls outside. The development of Tensar reinforced cement composites will undoubtedly lead to applications in this area.

CONCLUSIONS

Tensar-reinforced blast protection walls and embankments are in many cases a feasible alternative to traditional methods of construction. Major cost savings are possible and in some cases there are even greater benefits.

Land take may be of primary importance when working between existing buildings or close to other obstructions. A steep faced, vegetated embankment can often be the most acceptable solution.

In some areas of the world suitable fill for embankment construction may be difficult to obtain and concrete plus reinforcement almost
impossible. Tensar reinforcement is light, easily handled and installed and can reduce fill volume to a minimum. The Geocell mattress has proved to be an efficient system to use when a steep wall is required in areas where fine sand is the only fill material available.

The variety of Tensar reinforcement systems suggests that few applications would not warrant investigation of a Tensar alternative.
Construction of a steep sided geogrid retaining wall for an Oregon coastal highway

J. R. Bell, Oregon State University, and T. Szymonik and C. R. Thommen, Oregon State Highway Division

INTRODUCTION

In December 1981 a slide occurred on an Oregon coastal highway closing the main entrance to the popular Devil's Punch Bowl State Park 25 km north of Newport, Oregon, U.S.A.

The soil profile consisted of 3.5 m of medium to yellow brown sand over a layer of soft gray silty clay varying in thickness from 0 to 3.5 m underlain by gray shale. The failure plane was at the clay-shale interface. Figure 1 shows a typical cross section of the slide and the failure plane.

Several alternatives were considered for stabilizing the slide. A nonwoven geotextile retaining wall and a geogrid reinforced wall had the lowest estimated costs. The geogrid wall was chosen for two reasons: 1) the geogrid retaining wall had the lowest estimated cost, and 2) the open face of the geogrid wall allowed establishment of vegetation on the wall to provide a natural appearance compatible with the state park.

The geogrid wall had the lowest estimated cost because it did not require a facing or protection from ultraviolet (UV) light as did the conventional geotextile wall. Other than the facing, the geogrid wall and the conventional wall had nearly identical estimated costs.

The wall is 21 m long at the bottom and 52 m long at the top. The top is stepped to fit the vertical curve of the roadway. The minimum height of the wall is 9 m. Common backfill is placed over the lower face of the wall to reestablish the natural ground surface. Above the natural ground line sod is placed between the gravel backfill and the geogrid facing. The sod was believed to be the most economical way to establish vegetation on the wall. To accommodate growth, a dirty backfill (Class B) was placed in the first 0.6 m behind the sod.

The Oregon State Highway Division constructed a vertical Tensar® SR-2 geogrid reinforced wall to stabilize a landslide on the Oregon (USA) Coast. The wall was approximately 10 m high and 50 m long at the top. The site was adjacent to a park and special considerations were given to providing a natural appearance. Soil was placed behind the grid to establish vegetation on the face of the wall.

Initial construction was with forms which were removed and moved up as each segment of the wall was completed. Problems were experienced with the initial form design and a modified system was developed during construction. The grid wall was an economical solution to the special problems of the site.

GEOGGRID WALL DESIGN

Tensar® SR-2 was selected for the wall reinforcement grid. The backfill materials were graded crushed basalt with 50 mm maximum size; the general backfill material had a maximum of 10 percent fines, and the Class B material had approximately 20 percent fines. Specifications required at least 95 percent of standard optimum dry unit weight (MASHTO T99). The bulk density and angle of internal friction for the backfill were assumed to be 22 kN/m³ and 40°, respectively.

To limit possible creep, the working stress for the geogrids was taken as 40 percent of the ultimate strength. Because of the open structure of the grids, the full soil friction was assumed at the soil-geogrid interfaces.

The wall was designed assuming the grids had to resist the active Rankine lateral earth pressures by the portion of the reinforcement extending beyond the theoretical Rankine failure surface. The method of analysis was described by Lee, et. al. (1) and Haussmann (2) for Reinforced Earth® walls and modified for geotextile walls by Bell and his coworkers at Oregon State University (3,4). This method has been used by the U.S. Forest Service (5,6), New York Department of Transportation (7), Colorado Department of Highways (8), and others to construct geotextile walls in the United States.

Geogrid lengths and vertical spacings were calculated to provide minimum safety factors of 2.0 for dead load only, and 1.15 for dead load plus live load. The reduced factor with live loads was allowed because: 1) after construction, truck traffic would be limited to recreational vehicles and an occasional service vehicle; and, 2) the allowable working load included a safety factor of 1.5 against a short-term grid failure.

For appearance and construction considerations, the wall was detailed with 0.9 m steps. Each step was set back 150 mm from the one below to give the wall an average batter of 1:16. The lower three layers were given reinforcement spacings of 0.3 m, the mid-height layers spacings of 0.45 m, and the top two layers reinforcement spacings of 0.9 m. To give a uniform appearance the geogrids were folded back into the backfill at mid-layer height for the top two layers. This fold was anchored a distance of 1.5 m into the backfill. This anchored distance was the same as the 1.5 m overlap embedment used for each layer.

The geogrid reinforcement lengths were 4.9 m. This length was required at the top for pullout resistance and at the bottom for resistance to horizontal sliding.

**CONSTRUCTION**

To keep the costs of the geogrid wall competitive, it was necessary to select a simple effective method of supporting the face during construction. Because of the steep site, wall geometry, and the need to operate equipment in front of the wall, scaffolds were not considered practical for this wall. Instead, movable self-supporting forms were used on geotextile walls (5,7,8) were proposed.

This wall required forming 0.9 m steps. Experience on a geotextile wall in Glenwood Canyon, Colorado, indicated that the simple movable forms previously used were not suitable for layers greater than about 0.4 m (9). Therefore, a forming system was suggested that incorporated the concepts of the previously used geotextile forms, but had special features to allow for thicker layers. This forming system is illustrated in Figures 3 and 4.

The suggested form consisted of a 0.9 m by 2.45 m sheet of 19 mm plywood held in place by a form support. The form support was anchored in the backfill. There was concern that if the form support base extended into the backfill far enough to provide stability, friction would make it very difficult to pull the base out at the completion of the layer. Therefore, a sacrificial reaction pipe was anchored in the backfill, and the rod on the form support was inserted into the pipe. Since there was little friction on the form support base, an anchor rod was used to provide lateral resistance.

As shown by the typical installation in Figure 4, it was anticipated that the forms for a completed step would be left in place while the next higher step was constructed. The lower form would add stability to the upper form and help maintain vertical and horizontal alignments. When the upper step was completed, the lower forms would be removed and moved up to form the next step. It was believed this system and procedure would be expedient and stable for the 0.9 m steps.

The general procedure followed by the contractor in the early stages of the construction was:

1. Prefabricate 3 m by 4.9 m geogrid panels.
2. Set the proposed forms at gradeline.
3. Lay out prefabricated grid panels, drape the grid over the forms, and secure the panels with anchor pins.
4. Secure panels to one another at the face.
5. Place backfill in 0.15 m lifts to desired layer thickness.
6. Place sod in position behind the geogrid and place Class B backfill.
7. Pull back and secure grid overlap.
8. Repeat Steps 3-7 until the top of the form is reached, then remove forms and move up for the next layer.

Figures 5 through 10 illustrate this procedure. Figure 5 shows a worker securing the strips of geogrid into panels and splicing the ends of the grid with No. 3 rebar. A 300 mm circular saw was used to cut the panels to length. Figure 6 illustrates the initial forming system with the grid draped over the form. Figure 7 is an overview of the wall construction which shows the restricted space and the placement of the backfill. Figure 8 shows a worker hanging sod strips on the forms, and shows the space left for the dirty Class B backfill. In Figure 9 workmen are pinning the overlap. The completed wall is shown in Figure 10.

As the wall gained in height, problems began to occur with sagging and bulging of the wall face. This was due to the excessive flexibility of the forms and the loss of Class B backfill through the grid where sod was not used between the geogrid and the backfill. Between the times when the forms were removed and when the face was covered by common backfill significant amounts of the fine Class B backfill fell out from behind the grid. Where sod was placed against the geogrid reinforcement, the fines were retained and the wall face was near vertical. The bulging problems were most important in the lower layers, since the layer would be covered. However, the problem resulted in the contractor modifying the forming method before constructing the higher layers.

The combination of the thin plywood and short form supports on wide centers resulted in deflection of the forms. An even more serious problem resulted from the loss of support from under the forms. It was expected that the forms for a completed 0.9 m step would be left in place until the forms above were set and at least the first lift of that step was in place. The contractor elected, however, not to follow this procedure and moved the forms as each 0.9 m step was completed. Also, the contractor used plastic rather than steel reaction pipes. This resulted in the forms being dependent on the support of the previous layer directly under the metal plate of the form support, see Figure 4. Without the lower form in place, the slight inevitable bulging of the face results in tipping of the form support. Loss of the backfill through the grid compounded the problem and with the form support stiffened only.
by the plastic reaction pipe, the form tipped even further. Loss of backfill material also reduced the effectiveness of the form support anchor.

The contractor's solution to the forming problem is shown in Figures 11 and 12. The forms were stiffened and braced against a 50 mm by 100 mm member anchored 1.2 m into the backfill. The protruding end of the horizontal anchor was supported by a vertical member. The bottom of the 19 mm plywood form was held in place by a 50 mm by 100 mm block nailed to the anchor support. At least 3 braces were used on each 2.45 m forming unit. The new forming system required considerably more time to construct, but did provide a stable face to build against.

COSTS

The engineers estimated the project cost to be $165,802 (U.S. dollars), the low bid was $166,328 and the actual cost was $183,395. For the wall items of grid, backfill, and sod facing, the corresponding values were $81,710, $97,146, and $85,496 respectively. This cost translates to $255 per m² of wall face. The inplace costs for the geogrid including the grid material, forming, and handling were $6.60 per m² of grid and $120 per m² of wall face.

EVALUATION AND RECOMMENDATIONS

The geogrid wall appears to have stabilized the site. The sod facing was growing and the appearance was very satisfactory at the end of construction; however, lack of irrigation killed much of the sod in a few weeks. Where vegetation is to be established on a wall face, adequate maintenance must be provided.

Geogrid walls are economical and have great potential; however, improvements in construction techniques are necessary to fully utilize their potential. At suitable sites, scaffolding may be the solution to the forming problems. In other situations, a modification of the movable forms suggested for this project are recommended. The following modifications are proposed:

1. Stiffen the plywood along the top edge and secure adjacent plywood sheets to each other with battens.
2. Lengthen the upright on the form supports and use at least three form supports on each 2.45 m form section.
3. Eliminate the reaction pipe and all anchor pins and extend the base plate of the form support 1 m into the backfill.
4. Weld rings on the short end of the form support base plate so mechanical aids can be used, if necessary, for extraction.
5. Use backfill significantly coarser than the grid openings, or use a fine mesh grid or geotextile behind the face of the wall.

With these changes, the forms may be removed and moved up with each layer.

ACKNOWLEDGEMENTS

The project was constructed as an FHWA Experimental Features Project. The wall was built by Dan D. Allsup, Contractor, Eugene, Oregon. Special acknowledgments are due Chuck Elroy, the Project Manager and Claudius Groves, the construction inspector. The authors also wish to acknowledge James Paul of Netlon, Limited for providing advice during the early stages of the construction. Finally, special thanks to Laurie Campbell for her expert and expeditious preparation of the manuscript.

REFERENCES

The design and construction of a reinforced soil retaining wall at Low Southwick, Sunderland

D. R. Pigg and W. R. McCafferty, Tyne and Wear County Council

THE SITE

The subject of this paper was instigated by a design brief which called for a retaining wall to support a new roadway. This would provide access to a future industrial development to the west of the centre of Sunderland on the north bank of the River Wear.

Fig 1

A large proportion of the site was occupied by the remains of a concrete products factory which had occupied two separate levels. These levels were separated by an existing reinforced concrete retaining wall, with an access road at the higher level which was to form part of the new roadway. The retaining structure was required to support the new access road, with the existing retaining wall remaining in position. The length of wall required was a little short of 100 metres and the maximum retained height was 3.5 metres. Approximately 2 metres away from the front face of the proposed retaining structure, a new D.I.Y. warehouse was to be constructed before wall construction could commence. This factor would greatly affect site access to the highway scheme and complicate construction of the new wall. In addition to the retaining structure, provision had to be made for the installation of a type P2 vehicle/pedestrian parapet at the rear of the footway adjacent to the wall, see Fig. 3.

GEOTECHNICAL CONSIDERATIONS

At the initial design stage, our own soils information was not available but a soils report for the warehouse project was in our possession. It was decided to undertake an initial reinforced concrete cantilever wall design based upon the available information pending the results of our own site investigation. The lower level of the site was overlain with concrete to a depth of 300mm beneath which variable fill material extended to different depths. In some areas, this fill was in the form of a clay, brick, rubble and concrete, whereas in other areas the fill comprised black ash and gravel. This variable fill extended over the full area of the warehouse site extending to depths between 3.7 metres and 6 metres. Below the fill layer, a fairly thick band of soft to firm clay covered the site which, in some areas overlayed a stiffer clay, and in other areas, a rocky sand layer, above rockhead at some 8 metres below ground level.

Geotechnical section based upon Tyne and Wear County Council Commissioned Survey.

Our own soils investigation revealed that maximum formation level for the R.C. wall should be not less than 2 metres below existing ground level, thus founding in the upper portion of the soft to firm clay, see Fig. 2. An allowable bearing capacity of 70 kN/m² was calculated for this layer, and because of the variation in fill material, it was felt that differential settlement of any retaining structure would be inevitable. We were informed that the piles for the warehouse project had been driven to a depth of some 20 metres before reaching bedrock, a factor confirmed by our own soils investigation. Initial predictions of differential settlement based upon our own survey were in the order of 200mm. As a result of this new information, the reinforced concrete wall design was abandoned at an early stage and alternative methods investigated.

**SELECTION OF WALL TYPE**

The problems associated with both the low bearing capacity of the soil and the expected differential settlement could have been overcome by the design of a piled reinforced concrete wall. However, due to the excessive lengths of the piles used for the warehouse construction, this solution was abandoned for reasons of expense. In addition, access for construction of the wall and the vibration and noise effects of piling on the new D.I.Y. warehouse would have caused severe problems.

The natural choice to deal with the prevailing ground conditions, was a reinforced earth retaining wall, none of which had been constructed on a highway scheme in the County of Tyne and Wear. The main reasons for this choice included low associated bearing capacity of subgrade; a 'flexible construction'; and of prime importance, the finished structure would have very little drawback effect on the piled foundations of the D.I.Y. warehouse.

The design parameters peculiar to our retaining structure were: a maximum free height of retaining wall of 5.5 metres, comprising difference in level of 3.5m and a depth to subgrade of 2m, supporting a total width of single carriageway and footways of 16 metres. In addition to the retained height of soil, the structure was also required to support full H.A. loading as a working load case. It is general practice to consider this H.A. loading surcharge as an additional 600mm of backfill for the purpose of analysis. A vehicle/pedestrian barrier was required to contain vehicles at the higher level to the rear of the footway and also to protect the adjacent D.I.Y. warehouse. As previously mentioned, the safe bearing capacity of the ground below the reinforced earth wall is 70 kN/m² and the new wall was to be constructed between an existing retaining wall and the warehouse property, see Fig. 3. The condition of this existing retaining wall was unknown and it was to be shored during construction.

Before the design of the reinforced earth wall could commence, a problem concerning the length of the reinforcing elements had to be resolved. B.E. 3/78 which governs the design, states that the minimum length of reinforcing elements should be the greater of 80% of the retained height or 5 metres. As the available space between the existing and new walls was a minimum of 3.1 metres, we could not fulfill this requirement over the full length of the wall. However, after reading an article describing the use of a polymer reinforcement fabric to construct a wall at Newmarket Silkstone Colliery, Ref 1, it was felt that this problem could be overcome. Unlike the normal strip system of reinforcing elements, the Tensar grid system promotes good aggregate interlock between the plastic "fingers" and the possibility of corrosion is completely removed. As BE 3/78 was written before the advent of grid reinforcement and is based upon the "strip" reinforcement principle, it was felt that this minimum length criterion could be relaxed. The design was commenced assuming the use of Tensar SR-2 reinforcing elements and frictional fill material. The top of the existing retaining wall was to be removed where the Rankine active wedge was at its widest in order to accommodate the calculated grip length.

**THE DESIGN PROCESS**

The general concept of reinforced earth is now well understood. The soil is reinforced using layers of material placed horizontally. These layers are capable of transmitting tensile forces which effectively bind the soil structure to form an integral mass.

The design of a reinforced earth structure consists of the evaluation of lateral stresses within the soil over the full height of the structure and the provision of horizontal reinforcing elements to dissipate these stresses, by friction, to the soil beyond the Rankine active zone. The required length of embedment outside the Rankine active zone, or grip length, is generally shorter at the top of the soil structure than at the bottom. Thus for equal length reinforcing elements there is a balance between a wide Rankine active wedge zone and short grip length at the top and a
A frictional fill material was chosen for its good internal friction and drainage characteristics. The former enables short grip lengths to be used while good drainage reduces the lateral pressures which have to be accommodated. For the purposes of analysis the following properties were assumed: cohesion = 0 kN/m²; bulk density = 2038 kg/m³; effective angle of internal friction = 35 degrees.

A vertical spacing of 500mm for the reinforcing elements was chosen as it optimized the grip length properties of the Tensar SR-2 within a narrow site and suited the size of facing units selected. The force in each reinforcing element was calculated and, after summation, the factor of safety against overall tensile failure was computed using the permissible working load supplied by Netlon. The grip length required to resist the tensile force in each reinforcing element was then calculated with a factor of safety of 2.5 against local bond and tensile failure being applied. The factor of safety against overall bond failure was then calculated.

The reinforced "block" of frictional fill and reinforcing elements was then considered as a mass gravity wall supporting not only its own dead and imposed loadings but also the fill material placed behind it. Factors of safety against sliding, bearing capacity failure, overturning and slip circle failure were then calculated. In addition, sliding on any horizontal plane within the frictional fill mass and wedge stability above any horizontal failure point were also considered. None of the aforementioned calculations resulted in a factor of safety less than 2.0.

**PRACTICAL CONSIDERATIONS**

It was decided to use cruciform shaped precast facing units reinforced on each face with a structural mesh fabric and with an exposed aggregate finish on the front face. At this early stage, a decision was taken to cast Tensar "starters" into the rear of the facing units, these "starters" to be lapped to the main reinforcement, in order to obtain a simple connection between facing units and the main Tensar grids. The panel reinforcement was designed as a continuous beam carrying the tensile forces from the reinforcing elements. A plain concrete strip footing was detailed for the bottom facing units to bear upon and a small upstand was cast on the top surface of this concrete footing to aid the location of units in line and level. The front of the facing unit was placed against this upstand and the unit was inclined towards the backfill at a slope of 1:20. This detail was adopted to allow the facing unit to rotate about its lower edge and move outwards during filling and compaction adjacent to its rear face. The joints between facing units were detailed as being a half tongue, half groove arrangement.

An area of poor ground at depth existed over a length of some 40 metres. It was felt that taking the reinforced earth wall facing units to this depth was uneconomic but that the frictional fill should still be reinforced. The solution adopted was to provide Tensar SR-2 running parallel to the wall to form a hammock arrangement which was continued up into the reinforced earth wall by interleaving the layers of Tensar over a depth of 1.5 metres.

Regarding the vehicle/pedestrian parapet, it has been decided at an early stage in the design that the parapet foundation should be totally independent of the wall facing units to avoid any vehicle impact load being transferred to the wall. The parapet base layout was designed to fulfill this criterion and an additional use for the Tensar was adopted to enhance the base stability. It was imperative that the parapet base should be accommodated within the proposed footway and be of reasonable dimensions. A width was thus selected such that the overturning moment due to vehicle impact was resisted by the dead weight of the base providing a factor of safety of unity under parapet impact loading (the wheel loads associated with impact would increase this value). To increase this factor of safety to a more reasonable figure in the order of 1.4, at ultimate, a grid of Tensar was cast into the bottom of the slab and lapped to the main reinforcement. The free end of this Tensar grid was then passed through the frictional backfill and into the carriageway.
sub-base to prevent pullout. In addition to increasing the factor of safety against overturning, the Tensar was also required to enhance the resistance to horizontal movement during vehicle impact. The parapet base slab was formed such that it acted as a capping beam to the reinforced earth wall and its profile was tailored to overhang the top of the uppermost facing units, thus giving weather protection to both the facing units and the fill material. As the main reason for the selection of a reinforced earth wall had been the expected differential settlement, it was desirable that the aesthetic appearance of the structure should be protected during the service life of the wall. To achieve this it was decided to cast the uppermost Tensar "starter" in the top facing unit into the parapet base slab. The free length of the Tensar and the overhang of the parapet base was calculated such that should the top facing unit be displaced either downwards or outwards, the front face of the unit would not protrude beyond the front face of the parapet base. Therefore, rather than the front face of the units being the focal point to an observer, the front face of the parapet base, which would always remain fixed, would perform this role. It was felt, however, that the construction of the parapet base should be one of the final operations in order that initial wall settlement would have taken place. To allow some differential movement the parapet base/capping units were cast in 3m lengths and separated by transverse dowelled joints while the steel parapet was of bolted construction.

THE CONTRACT

The contract drawings and bills of quantities were prepared in accordance with the Specification for Road and Bridgeworks, the Standard Method of Measurement for Road and Bridgeworks and BE 3/78. The Select List of Tenderers for the scheme comprised five civil engineering contractors in addition to the Borough of Sunderland Public Works Department. The lowest tender price for the full scheme of £120,000 submitted by the Borough of Sunderland, included a cost of £57,000 for the construction of the reinforced earth wall and the vehicle/pedestrian parapet. The contract was awarded to the Borough of Sunderland in January 1981 with construction commencing in March 1981.

CONSTRUCTION

The contractor chose to use a burnt colliery shale material which complied with the specification for frictional fill in BE3/78. In order to verify the design assumptions, the University of Newcastle upon Tyne was commissioned to carry out 300mm direct shear box tests on the material and on its interaction with Tensar SR2. The results of these tests are contained in Table 1.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Dry Density (kg/m³)</th>
<th>Moisture Content (%)</th>
<th>Normal Stress (kN/m²)</th>
<th>Peak Shear Stress (kN/m²)</th>
<th>Angle of Friction (Deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burnt Colliery Shale</td>
<td>1909</td>
<td>11</td>
<td>12.4</td>
<td>266</td>
<td>65</td>
</tr>
<tr>
<td>Burnt Colliery Shale + Tensar</td>
<td>1909</td>
<td>11</td>
<td>12.4</td>
<td>177</td>
<td>55</td>
</tr>
<tr>
<td>Design Assumption</td>
<td>1836</td>
<td>11</td>
<td>—</td>
<td>—</td>
<td>35</td>
</tr>
</tbody>
</table>

These results were better than those assumed for the design and, accordingly, the material was approved. The high sulphate content of the material necessitated the use of Class 3 sulphate resisting concrete throughout and in addition the rear of the facing units were painted with a bitumen emulsion after erection.

As previously mentioned, access available for the construction of the wall was extremely limited and the existing retaining wall had to remain intact wherever possible. Excavation for the strip foundation and wall mass was carried out using a crawler back actor machine. The excavation commenced at the west end of the site and the only means of access at this lower level was from the east, along the proposed line of the new wall. After removing a bucketful of material from the foundation trench, the excavator had to slew through 180° with the bucket held at full vertical reach, before discharging into a tipper directly behind. The existing retaining wall was supported as the trench and wall excavations progressed eastwards, a factor which added to the access difficulties.

The concrete was placed to the strip foundations, the first "course" of facing units erected, then the placing of the reinforcement and backfilling and compaction of the frictional fill was carried out in 10 metre increments along the full length of the wall. Only when the wall was complete at any one level, was the next level of facing units erected and backfill placed. It was decided to carry out construction in this way for several reasons: to reduce differential settlement between adjacent lengths of wall; to allow vehicle access at low level; (rather than from the top of the existing wall for the purpose of depositing frictional fill while the existing wall was undermined by the excavation) and to ensure, as much as possible, that the facing units interlocked as desired. The complete operation, i.e. the construction of the wall to parapet base formation level took some eight weeks, a time which could have been reduced considerably had possession of the site been obtained before construction of the warehouse project.
At the present time, some 2½ years after construction, vertical and horizontal movement of the wall has almost ceased. The total movement recorded to date has not exceeded 50mm vertically or 30mm horizontally.

In conclusion, this wall has proved to be a practical and economic solution for the site conditions. It was constructed by labour with no previous experience of this type of structure in a reasonably efficient manner considering the physical restrictions of the site.

REFERENCES


ACKNOWLEDGMENTS

1. P. Morris B.Sc., C.Eng., M.I.C.E., Executive Director of Engineering, Tyne and Wear County Council who was the Engineer for the works.

2. Dr. C.J.F.P. Jones C.Eng., F.I.C.E., Assistant Director of Structural Engineering Unit, West Yorkshire Metropolitan County Council for his assistance during the planning and design stage.
Constructional details of retaining walls built with Tensar geogrids

Although each reinforced soil retaining wall is unique, elements forming each structure are often similar.

For reasons of economy and serviceability consistent reliable constructional details are required. This paper provides details of some structural systems which have been shown to be efficient when used with retaining walls built with Tensar Geogrids.

C. J. F. P. Jones, West Yorkshire Metropolitan County Council

INTRODUCTION

Although the form of any soil structure may vary, the structural elements required to produce different structures are often similar. In keeping with other forms of construction, poor detailing will produce an inadequate structure. whilst good detailing will ensure success. The difference between good and poor detailing can be subtle and weaknesses and deficiencies may only become apparent during the construction phase or later during the life of the structure. In either event remedial measures may be costly.

Many situations produce common structural problems. The constructional details shown below, although not necessarily the best possible, have been shown to be efficient and effective in general conditions. In some cases the details represent a compromise between constructional efficiency, structural requirements and aesthetics.

FOUNDATIONS AND DRAINAGE

Normally the need for foundations is minimal with reinforced soil, a mass concrete strip footing being sufficient. If the bearing capacity of the subsoil beneath the footing is insufficient additional support may be provided by conventional means such as by the use of piles; alternatively a geogrid foundation matress may prove suitable, Fig 1. The latter construction has the added advantage of providing subbase drainage to the reinforced structure. An alternative drainage detail used successfully beneath reinforced soil structures is shown in Fig 2. Also shown are details of the sand drainage layer behind the facing units which has been shown to be successful when the bulk fill contains a high percentage of fine material as in the case of pulverised ash. When a well graded fill is used within a reinforced soil structure vertical drainage behind the facing is not usually required although care must be taken to avoid erosion of fines through joints between facing units.

Fig 1. Combined Geogrid Foundation and Drainage Blanket

Fig 2. Drainage Beneath Reinforced Structure

CONNECTIONS - FACINGS AND GEOGRIDS

The connection between the facing and the reinforcing material can influence the distortion developed in the structure during construction, to reduce distortion to a minimum there should be no slackness within the connection. In addition a full strength connection is required so as to make the most effective use of the reinforcement. A common detail between the facing and the reinforcement is a bolted connection, the bolt passing through a hole formed in the reinforcement. Although effective, this method reduces the total carrying capacity of the reinforcing element by up to fifty per cent.

An attribute of Tensar Geogrid is that full strength connections may be obtained easily and economically. With the telescope method of construction, short lengths of geogrid can be cast into the facing element to which the main geogrid sheet may be attached using a simple galvanised pin, Fig 5.

The connection used with the sliding method of construction differs in that the reinforcement is attached to the facing in such a manner that movement in the downward vertical plane is possible, Fig 6.

In the case of the hybrid structure shown in Fig 4, two alternative connection systems are possible, Fig 7(a,b). In figure 7(a) the geogrid is connected to the facing by a galvanised steel bar passing in front of the steel columns, the grid passes through the facing and vertical settlement within the structure is accommodated on the telescope principle, Jones (1984).

The alternative method of connection is shown in figure 7(b) where the grid is attached via
a reinforcing bar held in U lugs welded to the back of the steel columns. Vertical settlement of the connecting bar and geogrid is possible and the connection fulfills the requirement of the sliding method of construction. With this detail the bonded cork gaskets placed between the infill planks shown in figure 7(a) are not used.

CONCLUSION

Geogrid reinforcing material can be adopted for use with any of the current construction systems and may easily be incorporated into new details. Care must be taken not to damage the material during construction, either with construction plant or with sharp fill. The light weight of the material must also be considered and it is particularly important to eliminate slackness from the material and connections during fill placing.

REFERENCES


The design and construction of polymer grid boulder barriers to protect a large public housing site for the Hong Kong Housing Authority

L. Threadgold, Leonard Threadgold
Geotechnics Ltd, and D. P. McNicholl,
Hong Kong Housing Authority

1. INTRODUCTION

Hong Kong Housing Authority administers the largest public housing programme in the world. Since 1953 the Authority has housed over 2,000,000 people and the current construction programme provides for some 35,000 flats per year. Within Hong Kong there is a shortage of suitable housing sites and hence development has to take place in areas which frequently pose significant civil engineering problems. This paper describes one such area where extensive site formation works are required to create platforms in sloping terrain. These platforms are being used for the construction of residential blocks with associated roads and services for a projected 50,000 inhabitants.

The natural slopes which extend above the site are a potential source of boulder fall. In view of the exposure of the housing site and its occupants to any future boulder movements, Hong Kong Housing Authority commissioned a study to investigate the extent of possible boulder instability and movements and to advise on suitable protective and preventive measures.

Throughout the study there was close liaison between the Consultants, the Housing Authority and other Government Departments. This ensured that the work proceeded in a positive manner and took account of technical, practical and administrative requirements.

2. SITE DESCRIPTION

The study area is located on the southern slopes of Lion Rock ridge which dominates the skyline to the North of Kowloon peninsula in Hong Kong, see Figure 1. The area is bounded to the south by the new housing estate. The ground slopes at angles of 30° to 60° adjacent to the ridge line and reduces in a southerly direction to the range 0° to 30° at the foot of the slopes. Exposed rock masses on or close to the ridge

The shortage of sites for housing in Hong Kong has required the construction of estates on hillside where boulders and rock exposures are a prominent feature. Protection of sites against potential boulder or rock falls requires measures to prevent boulder movement or barriers to intercept falling boulders. This paper concerns the design of protective barriers on a site in which the topography allowed their use, together with preventive measures, to achieve an economic solution.

Barriers constructed using rock-filled polymer grid mattresses which have both mass and flexibility were selected and preventive works comprising boulder removal, buttressing or erosion protection used.
line exhibit overhang and vertical surfaces.

The slopes are divided into a series of North-South trending valleys and ridges resulting from the erosion of the soil matrix or derived from rock falls are evident on the slope surfaces. They range in size from less than a metre up to 30m in width although this latter size is exceptional.

At the time of the study, parts of the lower slopes were covered by squatter huts which obscured some boulders.

Grass, trees and shrubs obscured many of the ground features although periodically the vegetation has been removed by man or slope fires, thus allowing the unprotected ground surface to be eroded by wind and water.

3. Boulder Survey

The size of the area from which boulders could fall was such as to render the task of identifying and stabilising all boulders practically unrealistic in terms of time and cost. Furthermore, rock exposures close to the Lion Rock ridge which have been the source of boulders in the past would probably continue as such and the task of stabilising these would also be time consuming, expensive and for this site, probably aesthetically unacceptable. The surveying and preventive option would therefore be more practical and less costly.

For the foregoing reasons consideration turned towards a combination of protection and prevention with protection being of primary importance.

4. Literature Review

Currently available literature on the problem of boulder falls is relatively limited. Work has tended to concentrate on the stability of rock faces, remedial action to maintain stability or measures to protect structures such as roads or buildings which would be threatened by any falls from these rock faces.

Work by Ritchie (1) in the United States of America provides a basis for rational design of protective works, with particular reference to highways threatened by rock face instability, but considers slopes no flatter than 35°. Solutions to the problems are presented in terms of the design of rock ditches and the location and height of associated fences. It is largely empirical in character and does not attempt to quantify the loads involved in stopping or retaining the boulders.

In India, problems with roads passing through the Himalayas have been studied by Bhandari (2, 3) of the Central Building Research Institute in Roorkee, and protective measures such as fences, netting, trenches etc. designed. Poole & Sweeney (4) have studied remedial and protective works in relation to rock faces whilst DeFreitas and Watters (5) have studied toppling failure, predominantly in relation to rock exposures. A paper by Merer (6) deals with measures adopted to deal with rock falls in Trenadog, North Wales.

The papers most appropriate to this study are from a meeting of the I.S.M.E.S. in Bergamo, Italy on 20-21 May 1976, entitled "Rockfall Dynamics and Protective Works Effectiveness" (7). Papers describe experimental studies, model tests, computer models, mathematical work and field observations relating to rock or boulder movements. The papers consider the form of movement, factors affecting the distance of movement and the form of protective works using a site near to Lecco in Italy as the study area.

Campolongo describes model tests for the slopes of St. Martino. He demonstrated the effectiveness of barriers a few metres high, particularly where the natural ground formed a series of gorges or "forced passages". He also observed that, in the absence of barriers, the finer material tends to form accretes, whilst larger and heavier blocks tend to break away and roll further down hill.

Lied of the Norwegian Geotechnical Institute indicated that the maximum reach of a boulder fall could be defined using an angle of reach between the source of boulders on the talus slope and the valley floor. In his paper (8) he indicates an angle of 28° to 30°, but in a private communication to the Author a tentative angle of 23° to 25° was postulated for Norwegian conditions.

Other papers deal with mathematical models for boulder falls and employ coefficients of restitution to deal with the bouncing mode. The rolling mode is less easily dealt with.

It is clear that many factors influence boulder behaviour and the variables of any site would require an extremely large modelling and testing programme. Even then a large number of unknowns would remain.

The overall conclusion from the above papers is that the main factors of practical significance to any design are the path of any falling boulder and its velocity pattern along this path.

5. Location of Protective Works

The ridge and valley terrain divides the area into catchments appropriate not only to water, but more importantly, in this context, to boulders. Study of the contoured plans of the area allowed the construction of "flow lines" for boulders perpendicular to the contours. These flow lines enabled protective barriers
to be located in the most efficient position towards the bottom of each catchment to intercept the vast majority of boulder trajectories. There remained areas near to the site, however, where the flow lines would not be intercepted by the barriers. For the boulders in these areas simple remedial works such as erosion protection, buttressing or in some cases removal were required.

This approach gave a very practical method of efficiently locating barriers and minimising boulder survey and treatment work. Effort was therefore directed to the location and design of the barriers.

6. Boulder Velocity

A major problem is the determination of the forces involved in stopping a boulder. This in turn is related to the size of the boulder, its velocity and the rate of deceleration which the obstruction imposes. Whilst an estimate of the size and shape of boulder to be dealt with could be made from the ground or aerial survey, determination of its velocity is particularly difficult in analytical terms. After considering several alternatives, a model consisting of a sphere rolling down a slope and hitting a series of steps of varying height and spacing was used. Figure 2(a)

This model appeared to predict reasonable relationships between slope angles, roughness and velocity. At a later stage, the concept of a cubic or hexagonal boulder was incorporated following a similar analogy, to predict velocities. Figure 2(b). The terminal velocity was used as the value for design purposes and typical relationships between step heights and spacings and boulder shapes are presented in Figure 3 (a) (b) (c) (d)

In practice the velocity would be influenced by other factors such as the dynamic modulus of the ground over which the boulder travels, the presence of vegetation, variations in slope and unforeseen obstructions etc. It was felt that this work provided an upper bound velocity and a practical basis for design.

There remains an element of doubt in respect of this upper bound velocity, and hence it was felt that any solution adopted should reflect this uncertainty and be able to accommodate variations without failure.

![Diagram](image-url)
7. PROTECTIVE WORKS

In the past the design of remedial or protective works has proceeded in a largely intuitive or empirical manner. A wide range of solutions have been adopted on an ad-hoc basis to deal with the problems of most concern to the feature to be protected. Alternative solutions have included:

- Trees or Vegetation
- Netting or Wire mesh
- Energy absorption areas
- Fences
- Trenches
- Structural walls
- Mass barriers

Initially, a toe-of-slope rockfall trap was considered with a geometry designed to bring boulders to rest by a controlled vertical impact. Preliminary estimates indicated that at a cost of some HK$9-12 millions this would be too expensive. Furthermore for this site, the use of traps or trenches had adverse implications for slope stability and drainage for the site below. Hence this solution was not used.

Structural concrete or steel barriers can only tolerate small deflections before failure. The
THREADGOLD AND MCNICHOLL

Forces required to stop a boulder are inversely proportional to deflection and are proportional to the square of the boulder velocity. Small deflections therefore imply high loads and these loads are sensitive to small changes in velocity. If structural barriers of this type fail the boulder would continue on its path.

Sacrificial fences and netting have been used to achieve some control but this has not been quantified. It is recognised that for sites which are restricted these are frequently the only practical solutions but for this site, space was not a major constraint.

The concept of a barrier having mass as its primary characteristic and deformation without failure as another was therefore favoured for this site. Within fairly broad limits, boulders of a given size travelling at a velocity greater than that estimated would cause greater movement but would not cause "failure".

8. BARRIER DESIGN PHILOSOPHY

The concept embodied in the barrier design was that following impact by a boulder, it would deform, in part within itself and in part by displacement of the base. The momentum would be transferred into the barrier and the whole retarded by the braking action of the friction between the barrier and the base.

Various momentum equations were derived which allowed the relationship between boulder velocity, boulder size relative to the barrier and the distance moved to be determined. From this work, the following relationships were derived.

\[
S = \left[ \frac{V_B}{R} \right]^2 \cdot \frac{1}{2g \tan \phi}
\]

Where \( S \) = distance moved by the barrier
\( V_B \) = Velocity of Boulder
\( R \) = Ratio of Mass of Barrier to Mass of Boulder
\( g \) = acceleration due to gravity
\( \phi \) = angle of shearing resistance along the base of the barrier.

also \( F = \left( \frac{M_B \cdot V_B}{M_B + M_P} \right)^2 \times \frac{1}{2g} \)

Where \( F \) = Force applied
\( M_B \) = Mass of boulder
\( M_P \) = Mass of barrier

Examples of these relationships have been plotted in graphical form. Figures 4 (a) to 4 (c).
particular importance for the terrain in which the barriers are formed. The fabrication does not require sophisticated equipment or very experienced personnel, provided that guidance is given initially and the work supervised throughout.

The use of other materials having characteristics equivalent to those indicated was permitted under the contract, but in the event, the contractor used Tensar S82, the material on which the design had been based.

Whilst no loading tests of the polymer grid have been performed at the high rate of strain that would be experienced in the event of a boulder strike, tests so far indicate that the strength of the polymer grid increases with increased rate of strain.

Concern was expressed regarding the long term durability of the materials under the heat, humidity and ultra violet radiation to which the material would be subjected by Hong Kong's sub-tropical climate. Available test data shows that any deterioration in strength is unlikely to be significant. Also it should be noted that the structural elements of the material would be mainly within the body of the barrier. External faces would not be subjected to high structural load.

10. DESIGN CONDITIONS
The following potential failure modes were considered:

1) Punching Shear
The boulder was assumed to cause the displacement of a horizontally orientated truncated "pyramid" of barrier material. This form of movement would be resisted by the polymer grid reinforcement and friction between the base of the pyramid and the barrier platform or ground. It is in this mode that the ability to form the polymer grid in a diagonal pattern was felt to be most advantageous.

2) Overturning
The possibility of overturning about a series of points on the downslope side of the barrier was investigated under a series of loading conditions and boulder impact points. The structure was shown to be stable in this mode.

3) Horizontal Shear
The possibility of horizontal shear displacement within the barrier itself was considered and adequate resistance shown to be present.

4) Water Pressure
The consequence of blockage of drainage provided under the barrier structure and a build up of water pressure for the full height of the barrier was considered and
the structure shown to be stable.

5) **Excessive Strain**

Strain within the grids was predicted from a consideration of the scale and geometry of the anticipated displacements and shown to be well within acceptable limits.

11. DESIGN DETAILS

The final design adopted is shown in Figure 5.

Some particular features are worthy of note.

1) Since the boulder barriers were constructed in the base of valleys it was necessary to provide culverts beneath the barriers to take the predicted flow. In the event, larger pipes than necessary were used to reduce the chances of blockage and facilitate cleaning out. Energy dissipation blocks were included on the down-stream side.

2) Erosion protection both uphill and downhill of the barrier was provided.

3) The base layer of fabric was placed between two 150mm thick layers of well graded granular material to ensure good frictional interlock between the gabions and the supporting base structure.

4) To prevent the fine material from washing out of the base of the barrier the sides were retained by no-fines concrete.

5) No horizontal sheeting was placed between the "mattresses" in order to ensure good interlock between them. Horizontal layers were only used at the base and at the top surface of the barrier.

6) Lateral reinforcement is provided by the vertical diagonal sheeting which also provided support to the vertical faces.

12. BARRIER CONSTRUCTION

After an initial familiarisation process by the Contractor, fabrication of the gabion structures proved to be extremely rapid. It is to the credit of the main contractors for the work, Kumagi Gumi Hong Kong Ltd., that these unique structures were built quickly and neatly in difficult terrain.

Table I gives an indication of the times taken to fabricate the barriers above culvert structure level, with materials and fabrication costs.

13. FUTURE WORK

It has not been possible to conduct controlled impact testing on the barriers at this site. Such testing would not only aid the prediction of boulder velocity and mode of movement but also lead to refinement of the design in relation to barrier deformation and stress redistribution.

Tests of the materials at the likely ambient temperature and at the rapid rate of strain which is appropriate to boulder impact would be extremely helpful to future design.

14. ACKNOWLEDGEMENTS

This paper is published with the permission of the Hong Kong Housing Authority and their Site...
Formation Consultant, Peter Y.S. Pun and Associates. We would like to thank them and in particular acknowledge the contribution of Mr. M.C. Gregson of Hong Kong Housing Authority, Mr. T.A. Rogers of P.Y.S. Pun and Associates and Dr. Y.C. Suen of Leonard Threadgold Geotechnics Limited, Sub-Consultant for the work.

REFERENCES


Retaining walls: report on discussion

C. J. F. P. Jones, West Yorkshire Metropolitan County Council

The Chairman discussed the anchoring effect of different types of reinforcement and described the results of research work at Sheffield University into the performance of 2m long reinforcing elements buried in a tank with a surcharge capacity of 200 kN per square metre.

In the case of steel strip reinforcement, the ultimate pulling resistance was found to be directly proportional to the overburden pressure. With smooth steel strip failure could be induced and the steel strip pulled out after very few cycles. The use of ribbed steel strip increased the number of cycles to failure by a factor of 10. A very marked difference was noted in the performance of Tensar SK2. The Tensar SK2 was subjected to 100,000 load cycles with the pulling load varying from between the limits of 70 per cent of ultimate and zero. The results show that the system became stiffer and stiffer as the number of load cycles increased. Polymer grid reinforcement was therefore believed to provide excellent performance if subjected to severe repeat loadings, as would be the case in bridge abutments.

Referring to paper 5.1 the optimum angle for placing of reinforcement was discussed by a number of contributors. Dr. S. Murray described some model tests at the TRL where horizontally placed reinforcement had been compared with inclined reinforcement. It has been found that the inclined reinforcement provided twice as much effective reinforcement as the horizontally placed layers.

References were made to the fact that ground anchors and structures often utilise inclined anchors, and it was felt practical to install reinforcement at an angle by first placing a wedge of soil down at a slope of 1 in 4.

It was reported that research work at the University of Strathclyde had illustrated the effect of the method of construction on the principal strain directions. Having discovered the principal strain direction of the soil, geotextile reinforcement was placed at angles calculated to make the reinforcement most effective, only to discover that changing the mode of construction had in itself changed the principal strain direction.

The conclusions which can be drawn from this work are that the principal tensile strain directions were the directions in which the reinforcement could be placed but that it is important to take into account the method of construction in order to determine where these will be. Once the principal tensile strain directions had been identified, it was possible to analyse, measure and design structures properly.

In reply Dr. Jones recommended that for practical reasons the reinforcement would usually be laid horizontally. In many cases it was not the cost of the reinforcement which was dominant, but the cost of the fill. An empirical approach had to be taken. If the installation of the reinforcement at an incline proved more economical then the contractor would suggest that this mode of construction be undertaken.

Dr. Bassett illustrated that in the case of reinforced soil retaining walls there was a quadrant in which the reinforcement could be inserted with positive effect. With a frictional material this quadrant would have an angle of $(45 \text{ degrees} + \phi/2)$. The effect of surcharge as described by the Chairman, provided an increase in efficiency.

Dr. Bassett commented that Dr. Jones was making an important point with regard to the construction when he recommended pulling the reinforcement tight. The result was to provide stress right the way through the structure and the effect was to produce anchored walls with the anchorage zone extending beyond the failure plane.

Dr. Bassett disagreed with Dr. Jones' comparison of a reinforced soil system with piles, particularly with regard to foundations where the piles pass through the foundations.

He commented that the foundation could be reinforced and used as the classic fan for a 45 degree stress field and the principal stress directions in compression and tension to illustrate the problem.

The same origin of planes and therefore the same rotation of principal stresses was noted, but the piles would have had to be bent to come out horizontally through the fill material if they were to be compared with this. Theoretically the tensile reinforcement should be horizontal under the foundation.

Dr. Bassett concluded that piles could be used as reinforcement if provided, for example, at 14" centres to make a wall of vertical timber props with soil back between them.

Mr. Pigg was questioned on the foundation conditions for the reinforced soil retaining wall at Low Southwick, reported in paper 6.4, in view of the very narrow length of reinforcement to height of wall ratio used.

It was pointed out that the Department of Transport Technical Memorandum called for a reinforcement of length 0.8 times the height or a minimum of 5 metres long strips.

The design concepts in the Technical Memorandum recognised the possibility of constructing a reinforced soil wall on poor foundations, whereas with a conventional wall pile foundations would be required. If, however, a narrow wall was build the situation reverted to the condition where the structure creates very high stresses in front or below the toe of the wall.

In reply Mr. Pigg explained that when excavation took...
place for the wall foundation, some fairly poor soil was found, and it was decided to extend the frictional fill to a greater depth. In addition, the foundation was reinforced using a hammock construction formed from Tensar running along the length of the wall and tied into the good clay on the inside of the dip.

The longitudinal reinforcement was interleaved with the reinforcement extending back in a direction normal to the wall. The interleaving was carried on up to a height of 2 metres. In view of the fact that the reinforced soil wall was being constructed, the subgrade was made as strong as possible.

In reply to a question on machines operating on top of the reinforcement it was confirmed that no tracked vehicle was ever allowed to run directly on the reinforcement, regardless of the type of reinforcement selected. Good practice demanded that soil was pushed on to the reinforcement.

The use of colliery waste was discussed with particular reference to the problems of pre-nationalisation dumps being mixtures of burnt and unburnt shales and the potential problems of unburnt shale being subject to internal combustion, or other forms of degradation.

It was pointed out that the slides shown in the presentation of paper 6.5 had shown that unburnt colliery shale had been used in reinforced soil structures.

In reply Dr Jones stated that colliery waste had been used with both Tensar and fibreglass reinforcement. The mining authorities used unburnt colliery shale, and it appeared to perform extremely well. Structures in which unburnt shale had been used had been designed to Design Memorandum BE 3/78 - as far as it was possible, although from a specification point of view, BE 3/78 did not permit the use of unburnt shale.

The potential dangers of internal combustion were recognised but experience had shown that if the material is properly compacted, in accordance with BE 3/78, that this problem could be eliminated.

Participants: Professor Hanna
Dr Murray
Dr McGown
Dr Jones
Dr Bassett
Dr Merrifield
Mr Pigg
Mr Stebbings