TECHNICAL NOTE

Revisions to BS8006 for reinforced soil – what do these mean for the industry?


Presented at the Ground Engineering Slopes Conference on 16 November 2010.

Over the past 15 years, the design of reinforced soil structures has evolved from the principles set out in the first edition of BS8006, published in 1995. The introduction of Eurocodes made a revision to BS8006:1995, essential as EN1997-1 specifically excludes the design of reinforced soil structures and slopes. The BSI has completed the revision of BS8006 and Part 1, which covers the design of reinforced fills was published at the end of 2010, while Part 2, which covers the design of reinforced insitu soils, is due to be published this year.

Introduction

Since the publication of BS8006 in 1995 after 10 years of drafting in committee, the design of reinforced soil structures has evolved as the results from research have improved the understanding of the behaviour reinforcement in soil.

The publication of the Eurocodes for design has resulted in the need to withdraw or revise all existing design codes in the UK and Europe. The Eurocode EN1997-1 Geotechnical Design does not include any provision for the design of reinforced soil structures and the UK NAD makes a clear statement to this effect.

In particular, the factors to be applied to the strength of the reinforcing elements cannot be determined from the Eurocode.

Structure of the code

The standard will be published in two parts: Part 1: Reinforced Fills – Retaining walls, slopes, basal reinforcement and transfer platforms; and Part 2: Strengthened Insitu Soils – Soil nails.

The two parts recognise that the design of insitu soils reinforced with soil nails is completely different to the reinforcement of soils placed as fills. Part 2 will be based on the CIRIA C637 Soil Nailing – Best Practice Guidance, but with more emphasis on the design of reinforced soil slopes.

The changes

The BSI committee of experts has reviewed the whole document and the following changes have been made. The changes were made public in a draft for public comment, published in July 2009, and the comments received where incorporated in the final document.

Clauses and information which conflicted with BSEN1997-1, BSEN 14475 & BSEN 14490 have been removed, but we have not revised all terminology to agree with BSEN 1997-1.

The correction of some outstanding errors found in text (at least two found in the last published version).

The partial factors were reviewed. The review was to look at the possibility that the partial factors in BS8006 could be the same or very close to the partial factors used in EN1997-1. The current situation is that the partial factors in BS8006 will remain unchanged.

A new section on reinforced modular block walls has been added to cover this new technology.

The design of facing for slopes and walls has been reviewed.

All text previously included by the Highways Agency in BD70, has been moved to BS8006-1 for bridge abutments and bank-seats.

The construction details have been revised, with more information included for information.

Advice on the use of vegetation on steep slopes is included. The advice includes information about the best methods for getting grass and other vegetation to grow and survive.

The problems associated with lateral forces at the edges of embankments with basal reinforcement and short piles or other foundation improvements have been addressed.

In Annex A the derivation of partial factors for polymeric reinforcement has been updated to follow the principles described in PD ISO CR 20432: 2007.

A new Annex has been added with seismic effects on reinforced soil structures, based on experiences form Japan and US, with cross reference to BSEN 1998.

Other Annexes giving test procedures have been reviewed and removed where now described in testing standards.

New features include: The use of recycled materials is encouraged using the Highways Agency document HD33/04 as a basis; cohesive fills can benefit from inclusions of drains within the soil layers to reduce the drainage paths for excess pore water pressure; new reinforcements can be considered, such as electro-kinetic geosynthetics, used to set up an electro osmosis system for rapid drainage; low strain/high strength geogrids.

Sections 6 and 7 reinforced soil walls and steep slopes – concepts and fundamental principles

To satisfy the requirements of a limit state code, partial load factors are applied to the actions and material factors are applied to the material properties.

Once the factors have been applied, the requirement for stability is that the design restoring forces and moments should be greater than the design disturbing forces and moments. In a particular application, the load factors applied to dead loads and live loads can vary depending on the load combination under consideration; in some circumstances, the partial load factors for live loads may be set at zero to produce a worst-case combination for the design load.

The loads, both dead and live, are calculated in their unfactored form as characteristic values, which then have the appropriate partial factor applied to give the design load.

The resisting forces in any application are generated from the available shear strength of the soil and the tensile strength of the reinforcing elements.

The shear strength of the soil, allowing for any pore pressure, should be a characteristic value determined as a cautious estimate of the value recognising the limit state under consideration. For walls and slopes, the peak friction angle is used. In all applications, the consideration of external, internal and compound stability is required. For internal and compound stability, the resistance provided by the soil is supplemented by the strength of the reinforcement.

The reinforcement strength is derived following a procedure described in an Annex of the document.

Reinforcement is defined as being extensible if the design strength is sustained at a total axial strain value >1%, and inextensible if the design strength is sustained at a total axial strain <1%. This definition can determine the actual design method adopted for a particular application.

The Code of Practice is applicable to all types of geosynthetic and steel soil reinforcement.

Vertical walls and bridge abutments

The principles described in the previous section apply to both reinforced soil walls and bridge abutments, which are designed as vertical structures.

Other structures with facing...
### Effects

<table>
<thead>
<tr>
<th>Solutions</th>
<th>Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Dead load of the structure</td>
<td>$f_n = 1.5$</td>
</tr>
<tr>
<td>Dead load of the fill on top of the structure</td>
<td>$f_n = 1.5$</td>
</tr>
<tr>
<td>Dead load of bridge and bank seat</td>
<td>$f_1 = 1.2$</td>
</tr>
<tr>
<td>Backfill pressure behind the bank seat</td>
<td>$f_1 = 1.5$</td>
</tr>
<tr>
<td>Backfill pressure behind the structure</td>
<td>$f_n = 1.5$</td>
</tr>
<tr>
<td>Horizontal loads due to creep and shrinkage</td>
<td>$f_2 = 1.2$</td>
</tr>
</tbody>
</table>

### Traffic loading

<table>
<thead>
<tr>
<th>Combinations</th>
<th>Over the entire structure, $f_s = 1.5$</th>
<th>Behind the reinforced zone, $f_s = 1.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge vertical live load</td>
<td>HA</td>
<td>$f_q = 1.5$</td>
</tr>
<tr>
<td></td>
<td>HA and HB</td>
<td>$f_q = 1.3$</td>
</tr>
<tr>
<td>Braking dynamic load</td>
<td>HA</td>
<td>$f_s = 1.25$</td>
</tr>
<tr>
<td></td>
<td>HA and HB</td>
<td>$f_s = 1.1$</td>
</tr>
<tr>
<td>Temperature effects</td>
<td>$f_s = 1.3$</td>
<td>$f_q = 1.3$</td>
</tr>
</tbody>
</table>

### Table 1: Partial load factors for walls

<table>
<thead>
<tr>
<th>Effects</th>
<th>Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil material factors</td>
<td>To be applied $\tan \phi' = f_m = 1.0$</td>
</tr>
<tr>
<td></td>
<td>To be applied $c = f_m = 1.6$</td>
</tr>
<tr>
<td></td>
<td>To be applied $c = f_m = 1.0$</td>
</tr>
<tr>
<td>Sliding across surface of reinforcement</td>
<td>$f_s = 1.3$</td>
</tr>
<tr>
<td>Pull-out resistance of reinforcement</td>
<td>$f_s = 1.0$</td>
</tr>
<tr>
<td>Foundation bearing capacity: to be applied to $q_{eq}$</td>
<td>$f_m = 1.35$</td>
</tr>
<tr>
<td>Sliding along base of structure or any horizontal surface where there is soil-to-soil contact</td>
<td>$f_s = 1.2$</td>
</tr>
</tbody>
</table>

### Reinforced slopes

The design method for reinforced slopes, generally defined as those slopes with face angles greater than $20^\circ$ to the vertical, are all derived from limit equilibrium methods, originally derived for unreinforced slopes. The Code does not limit the methods that can be used for design, provided that they can be adapted to a limit state format. However, it does describe the most common methods in some detail.

The partial factors are defined in Table 4 (overleaf), with only two load cases, ULS and SLS. The friction angle of the fill should be defined as the peak value and, as with the wall design, there are partial factors of safety applied to soil reinforcement interaction and sliding along the base.

The requirement for the inclusion of the Moment Correction Factor $\chi$ has been removed in the revised version of the BS. Any potential failure surface that passes around the reinforced soil zone without cutting a layer of reinforcement is now analysed using the approach defined in Eurocode 7.

For reinforced slopes where the face angle is $45^\circ$ or less the need for a formal facing may not be required. In those situations the reinforcement capacity close to the front face may be limited because of bond, although higher shear strengths may be mobilised because of the low confining stress. A check of superficial stability can be carried out to assess the stability of a free face in resistance to sliding on a plane that is very close, and parallel, to the slope surface.

It is noted in the code that compound stability considerations where a slip surface passes partially through the reinforced fill and partially through the unreinforced fill may well be the critical situation.

### Section 8 – basal reinforcement and load transfer platforms (Horgan)

UK BS8006-1 2009 Section 8 – basal reinforcement and basal reinforce over piles (aka load transfer platforms), updates the previous guidance and attempts to expand/clearly guide considered ambiguous in the previous draft and to reflect changes in best practice since BS8006 was first published in 1996.

The main additional issues addressed in the updates relate to:
- Consideration of pile cap geometry;
- Consideration of alternative arching mechanisms;
- More emphasis on assessing/controlling the settlements between piles;
- Greater emphasis on collabo-
Selection of soil parameters and partial factors

The British standard differentiates between soils/fills used in slopes and walls and those used in embankments. Since the strains induced in multiple layer reinforcement such as walls and slopes are deemed small, the frictional strength is represented by \( \tan \psi' \), the peak effective angle of internal shearing resistance. Larger strains are allowed in other scenarios, for example an embankment subject to differential settlement. In these cases, the frictional strength is represented by large strain values. For cohesionless soils this is the value when the soil shears at constant volume.

Consideration of foundation soils: BS8006-1 assumes that all of the embankment loading will be transferred through the piles down to a firm stratum either by direct arching on the pile caps or by the load carried to the piles by the geosynthetic reinforcement.

Consequently, the characteristics of the soft foundation soil is considered only with regard to the type of piles used and their installation. However, the time dependant consolidation of soil between piles is subsequently discussed.

BS 8006:1996 was one of the first national codes to adopt limit state principles and these are maintained in the updated code.

Consideration of pile cap geometry

The original guidance did not differentiate between circular or square pile caps of equal dimension yet the area ratio of the square pile cap (\( a_2 \)) is different to that of a circular cap (\( \pi \, D^2/4 \)). Yet this dimension is used subsequently used to determine the amount of arching (Marston’s ratio) or stress redistribution that occurs within the piled embankment. The new guidance recommends reducing the design dimensions of cir-

Table 3: Partial factors for reinforced slopes

<table>
<thead>
<tr>
<th>Partial Factors</th>
<th>Ultimate limit state</th>
<th>Serviceability limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil unit mass eg slope fill</td>
<td>( f_s = 1.5 )</td>
<td>( f_s = 1.0 )</td>
</tr>
<tr>
<td>External deal loads eg line or point loads</td>
<td>( f_d = 1.2 )</td>
<td>( f_d = 1.0 )</td>
</tr>
<tr>
<td>External live loads eg traffic loading</td>
<td>( f_q = 1.3 )</td>
<td>( f_q = 1.0 )</td>
</tr>
<tr>
<td>Soil material factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>To be applied ( \tan \phi_p )</td>
<td>( f_{ms} = 1.0 )</td>
<td>( f_{ms} = 1.0 )</td>
</tr>
<tr>
<td>To be applied ( c' )</td>
<td>( f_{ms} = 1.6 )</td>
<td>( f_{ms} = 1.6 )</td>
</tr>
<tr>
<td>Reinforcement material factor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>To be applied to the reinforcement base strength</td>
<td>( f_s ) should be consistent with the type of reinforcement to be used and the design life over which the reinforcement is required (see 5.3 and Annex A)</td>
<td></td>
</tr>
<tr>
<td>Soil reinforcement interaction factors</td>
<td>( f_s = 1.3 )</td>
<td>( f_s = 1.0 )</td>
</tr>
<tr>
<td>Pull out resistance of reinforcement</td>
<td>( f_p = 1.3 )</td>
<td>( f_p = 1.0 )</td>
</tr>
<tr>
<td>Partial factors of safety</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sliding across surface or reinforcement</td>
<td>( f_s = 1.2 )</td>
<td>NA</td>
</tr>
</tbody>
</table>
circular pile caps to consider a square pile of an equivalent area. Where circular pile caps are to be used, the diameter of the pile should be reduced to produce an effective pile cap width, 
\[ d_{cap} = (\pi D^2/4)^{1/3} \text{ hence } a_{cap} = 0.886D \]
where:
- \( a \) is the size of the pile caps (assuming full support can be generated at the edges of the caps);
- \( D \) is the diameter of the pile cap.

**Determination of load acting across the reinforcement**

Due to the significant differences in deformation characteristics that exist between the piles and the surrounding soft foundation soil, the vertical stress distribution across the base of the embankment is assumed to be non-uniform. Soil arching between adjacent pile caps induces greater vertical stresses on the pile caps than on the surrounding adjacent soil. BS8006-1 identifies a minimum height whereby partial arching begins to develop but additional loads placed at the surface of the embankment still influence the load carried by the reinforcement. This minimum height is:
\[ H \approx 0.7(s - a) \]
where:
- \( a \) is the size of the pile caps (or \( a_{cap} \) considering circular pile caps)
- \( s \) is the spacing between adjacent piles.

BS8006-1 also identifies a critical height concept above which any additional embankment weight or surcharge loading placed at the surface of the embankment is deemed to pass directly to the pile caps and not influence the reinforcement.

The ratio of the vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment may be determined by using Marston’s formula:
\[ \frac{P}{\gamma H} = \left[ \frac{C_a}{H} \right]^3 \]
where:
- \( P \) is the vertical stress on the pile caps;
- \( \gamma \) is the unit weight of the embankment;
- \( H \) is the height of the embankment;
- \( s \) is the uniformly distributed surcharge loading;
- \( a \) is the size (or diameter) of the pile caps;
- \( C_a \) is the arching coefficient.

Where the value of the arching coefficient will vary depending on the type of piles.

Several authors have studied and compared the phenomena of soil arching using a variety of physical, analytical and numerical models to try to gain a better understanding and quantify the load acting across the reinforcement and distributing directly to the pile caps (Alexisw 2002, Eckelen 2008, Kempton et al 1998, Love and Milligan 2003, Rogbeck 1998, Stewart and Filz, 2005).

One criticism of the original code is that the distributed load acting across the reinforcement was very sensitive to embankment heights around the critical height.

\[ H = 1.4(s - a) \]

Embankment heights just under this limit would result in considerably more design load across the reinforcement than embankment heights just above this level. Additional criticism related to the fact the ratio of the vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment was dependent solely on pile type and independent of the type of fill used within the embankment.

Finally a criticism that vertical equilibrium is not satisfied (Eckelen, 2008). However, neither the load acting across the reinforcement nor the increased load acting on the pile caps are calculated directly, but rather indirectly from the ratio of increase in stress concentration on the pile caps to the average vertical stress at the base of the embankment.

BS8006-1 2009 Section 8 allows for an alternative theoretical solution which may be used to determine the vertical load acting across the reinforcement based on work presented by Hewlett and Randolph, 1998, which was based on the observed failure mechanism from model tests and considers a series of hemispherical domes. The theory determines the efficacy \( E \) as the proportion of the embankment weight carried by the piles, hence the proportion of the embankment weight carried by the geosynthetic reinforcement may be determined:
\[ 1 - E \]

The original guidance utilises Marston’s formula to determine the distributed load acting across the reinforcement.

![Figure 4: anchorage length \( L_b \) at the edge of the piled area](image)

![Figure 5: Reinforcement anchorage at edge of fill](image)

**Table 4: BS8006 Section 8 Partial factors**

<table>
<thead>
<tr>
<th>Partial factor</th>
<th>Ultimate limit state</th>
<th>Serviceability limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil unit mass, e.g. embankment fill</td>
<td>( f_{us} = 1.3 )</td>
<td>( f_{us} = 1.0 )</td>
</tr>
<tr>
<td>External dead loads, e.g. line or point loads</td>
<td>( f_e = 1.2 )</td>
<td>( f_e = 1.0 )</td>
</tr>
<tr>
<td>External live loads, e.g. traffic loading</td>
<td>( f_l = 1.3 )</td>
<td>( f_l = 1.0 )</td>
</tr>
<tr>
<td>Soil material factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>To be applied to ( \tan \phi' )</td>
<td>( f_{ma} = 1.0 )</td>
<td>( f_{ma} = 1.0 )</td>
</tr>
<tr>
<td>To be applied to ( c' )</td>
<td>( f_{ma} = 1.6 )</td>
<td>( f_{ma} = 1.0 )</td>
</tr>
<tr>
<td>Soil/reinforcement interaction factors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sliding across surface of reinforcement</td>
<td>( f_s = 1.3 )</td>
<td>( f_s = 1.0 )</td>
</tr>
<tr>
<td>Pull-out resistance of reinforcement</td>
<td>( f_p = 1.3 )</td>
<td>( f_p = 1.0 )</td>
</tr>
</tbody>
</table>

**Table 5: Arching coefficient \( C_a \)**

<table>
<thead>
<tr>
<th>Pile arrangement</th>
<th>Arching coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>End-bearing piles</td>
<td>( C_a = 1.95H/a - 0.18 )</td>
</tr>
<tr>
<td>Friction &amp; other piles</td>
<td>( C_a = 1.5H/a - 0.07 )</td>
</tr>
</tbody>
</table>
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\[ W_s = \frac{1.4 \left(1 - \frac{(e - \varepsilon)^2}{s^2} \right) \left(1 - \frac{(e' - \varepsilon')^2}{a^2} \right)}{s^2 - a^2} \]

where:
\( W_s \) is the distributed vertical load acting on the reinforcement between adjacent pile caps;
\( \varepsilon \) is the partial load factor for soil unit weight (see Table 1);
\( \varepsilon' \) is the partial load factor for external applied loads (see Table 1).

Reinforcement details

The design approach essentially determines the requirement for two orthogonal layers of geosynthetic reinforcement at the base of the piled embankment, one longitudinal layer parallel along the centre line of the embankment and one transverse layer perpendicular across the line of the embankment.

The longitudinal reinforcement is designed to resist the redistributed vertical load \( T_{p,v} \).

The transverse reinforcement is designed to resist both redistributed vertical load \( T_{p,v} \) and to resist the lateral thrust of the embankment \( T_{p,t} \).

To mobilise these loads the reinforcement should achieve an adequate bond with the adjacent soil at the extremities of the piled area to ensure that the maximum limit state tensile loads can be generated between the outer two rows of piles.

Across the width of the embankment the reinforcement should extend a minimum distance \( L_n \) beyond the outer row of piles, (see Figure 4).

The reinforcement may be extended around the row of gabions and returned into the embankment fill to develop the necessary bond length (see Figure 5).

Depending on the geometry of the embankment, it may be difficult to achieve an adequate bond length at the extremities of the piles by maintaining the reinforcement in a horizontal alignment as depicted in Figure 4. One solution that may be considered is to use a row of gabbions, as a thrust block along the top of the outer row of piles.

Another detail that may be considered is the inclusion of a small periphery trench just beyond the edge piles, running parallel to the centreline of the embankment; the trench is typically only as deep as the piling mat or pile cap depth. The reinforcement can be extended into the trench and when backfilled, will return into the embankment fill to develop the necessary bond.

Time dependent consolidation of soil between piles

BS8006-1 assumes that all of the embankment loading will be transferred through the piles down to a firm stratum either by direct arching on the pile caps or by the load carried to the piles by the geosynthetic reinforcement. In reality, the soil between the piles will need to undergo some initial degree of consolidation to enable tensile forces in the geosynthetic to be mobilised.

This settlement can be related to an increase in stress from the installation of the temporary working platform or as a result of initial fill placement or by external factors such as seasonal fluctuations in the ground water level increasing the effective stress on the insitu soil. Hence the time dependent consolidation of soil between piles is important in reaching the assumed end design condition of no partial support between piles.

Consideration can also be given to the introduction of a compressible layer between the pile caps to ensure deflection of the geosynthetic during the initial placement of the overlying fill. The maximum mid-span deflection \( y \) of extensible reinforcement, spanning between pile caps, may be determined from the formulation below (after Giroud):

\[ y = (a - s) \frac{3e}{8} \]

where:
\( a \) is the size of the pile cap;
\( s \) is the spacing between adjacent piles;
\( e \) is the strain in the reinforcement.

The thickness of the compressible layer can be such that it can accommodate the deflection of the geosynthetic during the early stages of embankment construction, ensuring that the assumed design deflections are achieved.

Conclusions

BS8006 was first published in 1995. Section 8 of the code has been used to successfully design a vast number of piled supported embankments worldwide.

BS8006-1 2009 updates the previous guidance to reflect better industry understanding of the behaviour of such complex earthworks and to reflect changes in best practice.

Currently there is a variety of different design approaches, which differ mainly in the amount of arching considered and the degree of support offered by the existing sub soil.

The BS8006-1 assumption that all of the embankment loading will be transferred through the piles or by the geosynthetic reinforcement onto the piles can be considered conservative.

However, the potential use of compressible layers beneath the geosynthetic reinforcement between pile caps can ensure that the end design condition of no partial support between piles can be achieved during construction.

This would make consideration of the time dependent consolidation of soil largely academic.

References