# Introduction to international codes for reinforced soil design

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ABSTRACT: Over the past 15 years, a number of countries have developed and published their own codes for the design of reinforced soil walls, slopes and basal reinforcement of embankments. In this special session, the speakers will present the key aspects of their own codes, illustrated by the results of the design of three examples; a vertical reinforced soil wall, a steep reinforced soil slope and a basal load transfer platform over a piled foundation.

## 1 INTRODUCTION

Since the publication of BS8006 in 1995 after a 10-year period of drafting in committee, other countries have produced their own codes for the design of reinforced steep slopes, walls and basal reinforcements.

The purpose of this session is for the speakers to introduce the main features of their own codes. Part of each presentation will be the results of one or more comparison design models analysed using the principles embodied in the speaker's national design code.

## 2 THE CODES

The session will include presentations covering the following codes:

- UK BS8006-1 2009 Section 6 Walls
- UK BS8006 -1 2009 Section 8 Basal reinforcement and load transfer platforms
- USA Federal Highway Association – FHWA-SA-96-071
- Germany DIN 1054;2005-01
- Japan – JGS TC9 SC3

The standard designs, which each of the presenters will show, based on national or organizational standards are described in this section.

### 2.1 *UK BS8006 -1 2009 Section 8 Basal reinforcement platform over piles*

This section details the design of a geosynthetic reinforced platform over a piled foundation on soft clay soils. Answers being presented include:

- Details of reinforcement,
- Strength,
- Edge restraints,
- Any requirements for the fill in the platform,
- Settlement of foundation soils to be considered

The geometry of the model being analysed is;

- Fill height above Original Ground Level = 6.5m,
- Side slopes to fill 1(vertical) to 2 (horizontal)
- Width of embankment at crest = 35m, with a surcharge = 10kPa over full width,
- Piles on a square grid = 2.5m centre to centre with 0.9mdia round pile caps.

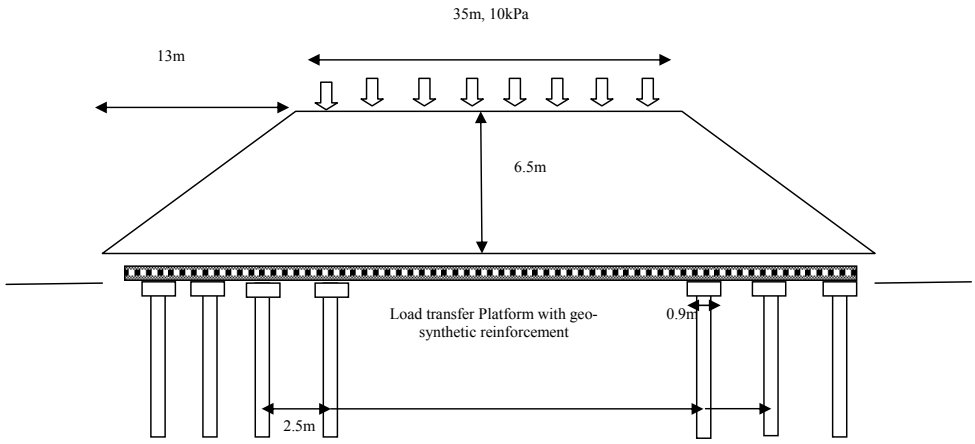


Figure 1. Load Transfer Platform Model

The soil parameters being used in the analysis are:

- Fill over platform – c'=2.5, phi = 39 deg, bulk density = 20.5 kN/m$^3$ taken as dry, ru=0.
- Fill in platform to designers own requirements.
- Foundation soils – firm crust = undrained trength 60 kPa at surface reducing to 15kPa at 1.5m below ground level (BGL), then soft to firm with undrained strength 15kPa to 40kPa at 12m BGL, bulk density 17 kN/m$^3$, highly compressive with $m_v$ = 0.9m$^2$/MN, ground water at 1.25m BGL.

UK BS8006 -1 2009 Section 8 Basal reinforcement and basal reinforcement over piles (aka.load transfer platforms), updates the previous guidance and attempts to expand/clarify guidance's 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 collaboration between geosynthetics and pile designers
- Time dependant consolidation of soil between piles and consideration of partial support between piles
- Additional guidance on anchorage details etc.

### 2.1.1 *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 $\phi'_p$, the peak effective angle of internal shearing resistance. Larger strains are allowed in other scenarios eg. 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.

The angle of internal shearing resistance for the embankment fill used in the analysis are simply described as having "phi = 39 deg", for the purposes of this comparison then $\phi_{cv}$=39$^o$.

Consideration of foundation soils: The British Standard assumes that all of the embankment loading will be transferred through the piles down to a firm stratum either by direct arching on the piles 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 in subsequently discussed

The original version of BS 8006 published in 1996 was one of the first national codes to adopt limit state principals and these are maintained in the updated code.

Margins of safety, against attaining the limit state of collapse, are provided by the use of partial mate-

rial factors and partial load factors. These partial factors assume prescribed numerical values of unity or greater. Disturbing forces are increased by multiplying by prescribed load factors to produce design loads. Restoring forces are decreased by dividing by prescribed material factors to produce design strengths. Provided the design strength equals or exceeds the design load then there is deemed to be an adequate margin of safety against attaining the ultimate limit state of collapse.

Serviceability limit states are attained if the magnitudes of deformation occurring within the design life exceed prescribed limits or if the serviceability of the structure is otherwise impaired. In assessing deformations or strains to determine compliance with the appropriate limit state, the prescribed numerical values of load factors are assume a value of unity.

Table 1. BS 8006 Section 8 Partial factors

| **Partial Factor** | **Ultimate limit state** | **Serviceability limit state** |
|---|---|---|
| Load factors | | |
| Soil unit mass, e.g. embankment fill | $f_{fs} = 1.3$ | $f_{fs} = 1.0$ |
| External dead loads, e.g. line or point loads | $f_f = 1.2$ | $f_f = 1.0$ |
| External live loads, e.g. traffic loading | $f_q = 1.3$ | $f_q = 1.0$ |
| Soil material factors | | |
| To be applied to $\tan \varphi_{cv}$ | $f_{ms} = 1.0$ | $f_{ms} = 1.0$ |
| To be applied to $c'$ | $f_{ms} = 1.6$ | $f_{ms} = 1.0$ |
| To be applied to $c_u$ | $f_{ms} = 1.0$ | $f_{ms} = 1.0$ |
| Soil/reinforcement interaction factors | | |
| Sliding across surface of reinforcement | $f_s = 1.3$ | $f_s = 1.0$ |
| Pull-out resistance of reinforcement | $f_p = 1.3$ | $f_p = 1.0$ |

2.1.2 *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 (Marstons ratio) or stress redistribution that occurs within the piled embankment. The new guidance recommends reducing the design dimensions of circular pile cap 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 $a_{equ}$,

$$a_{equ} = \sqrt{\frac{\pi D^2}{4}} \quad \text{or} \quad a_{equ} = 0.886D$$

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

The model is based on a circular pile cap with 0.9m dia which is adjusted to an equivalent to 0.7974m

2.1.3 *Determination of load acting across the reinforcement*

Due to the significant differences in deformation characteristics which 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 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 :

$$H \geq 0.7(s - a)$$

where

$a$ is the size of the pile caps (or $a_{equ}$ considering circular pile caps)

$s$ is the spacing between adjacent piles

BS8006 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:

$$H \geq 1.4(s - a)$$

The model embankment fill height is 6.5 with a pile spacing of 2.5 m and pile cap equivalent to $a_{equ} = 0.7974$ m hence the latter relationship is appropriate. The ratio of the vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment ($P'_c / \sigma'_v$) may be determined by use of Marston's formula:

$$\frac{P'_c}{\sigma'_v} = \left[\frac{C_c a}{H}\right]^2$$

where

$P'_c$ is the vertical stress on the pile caps;

$\sigma'_v = (f_{fs}\gamma H + f_q w_s)$ and is the factored average vertical stress at the base of the embankment;

$\gamma$ is unit weight of the embankment fill;

$H$ is the height of the embankment;

$w_s$ is the uniformly distributed surcharge loading;

$a$ is the size (or diameter) of the pile caps;

$C_c$ is the arching coefficient.

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

Table 2: Arching coefficient $C_c$

| Pile arrangement | Arching coefficient |
|---|---|
| End-bearing piles (unyielding) | $C_c$ = 1.95H/ $a$ -0.18 |
| Friction & other piles | $C_c$ = 1.5H/ $a$ -0.07 |

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 (Alexiew 2002, Eekelen 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

$$H \approx 1.4(s - a)$$

Embankment heights just under this limit would result in considerable more design load across the reinforcement then embankments 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 dependant solely on pile type and independent of the type of fill used within the embankment. Finally a criticism that vertical equilibrium is not satisfied (Eekelen, 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 − $E$).

For consistency with the original guidance Marston's formula will be used and end-bearing piles assumed to determine the distributed load acting across the reinforcement

$$W_T = \frac{1.4 s f_{fs}\, \gamma (s-a)}{s^2 - a^2}\left[s^2 - a^2 \left(p'/\sigma'_v\right)\right]$$

where

$W_T$ is the distributed vertical load acting on the reinforcement between adjacent pile caps

$f_{fs}$ is the partial load factor for soil unit weight (see Table 1);

$f_q$ is the partial load factor for external applied loads (see Table 1).

$W_T$ = 84.57 kN/m (SLS) & 109.94 kN/m (ULS)

To satisfy vertical equilibrium then the remaining load must be carried directly by the pile caps, $W_P$

$$W_P = \left(f_{fs}\gamma H + f_q w_s\right)s^2 - W_T\left(s^2 - a^2\right)$$

(although this formulation is not provided in BS8006 -1 2009) Once the distributed load $W_T$ acting across the reinforcement is determined, then for an extensible reinforcement the tensile load $T_{rp}$ per metre run, generated in the reinforcement resulting from the distributed load $W_T$ is determined by:

$$T_{rp} = \frac{W_T(s-a)}{2a}\sqrt{1 + \frac{1}{6\varepsilon}}$$

where

$T_{rp}$ is the tensile load in the reinforcement,

$\varepsilon$ is the strain in the reinforcement.

The above equation has two unknowns $T_{rp}$ and $\varepsilon$, and may be solved for $T_{rp}$ by assuming a maximum allowable strain in the reinforcement and by an understanding of the load/strain characteristics of the reinforcement at different load levels. Initial tensile strain in the reinforcement is needed to generate a tensile load; BS8006 stipulates a practical upper limit of 6% strain should be imposed to ensure all embankment loads are transferred to the piles.

Assuming a polyester based reinforcement then a typical working strain of 5% can be assumed for the SLS load and strain of 11% for the ULS load. Hence the tensile load required in the reinforcement can be determined.

$T_{rp}$= 187.9 kN/m (SLS) & $T_{rp}$= 186.1 kN/m (ULS)

In addition to the load on the reinforcement generated by the redistributed vertical embankment load, the reinforcement needs to resist the outward thrust from the embankment. The reinforcement tensile load $T_{ds}$ needed to resist the outward thrust of the embankment may be taken as:

$T_{ds} = 0.5K_a(f_{fs}\gamma H + 2f_q w_s)H$

where

$T_{ds}$ is the tensile load in the reinforcement per metre run needed to resist the lateral thrust of the embankment fill;

$K_a$ is the active earth pressure coefficient [ = $\tan^2(45° - \phi_{cv}/2)$];

$H$ is the height of the embankment;

$\gamma$ is the unit weight of the embankment fill;

$w_s$ is the surcharge intensity on top of the embankment;

$f_{fs}$ is the partial load factor for soil unit weight (see Table 1);

$f_q$ is the partial load factor for external applied loads (see Table 1).

Hence the tensile load in the reinforcement per metre run needed to resist the lateral thrust of the embankment $T_{ds}$

$T_{ds}$= 113. 31 kN/m (SLS) & $T_{ds}$= 147.3 kN/m (SLS)

### 2.1.4 *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 $T_{rp}$

$T_{rp}$= 187.9 kN/m (SLS) & $T_{rp}$= 186.1 kN/m (ULS)

The transverse reinforcement is designed to resist both redistributed vertical load $T_{rp}$ and to resist the lateral thrust of the embankment $T_{ds}$

$T_{rp} + T_{ds}$ =113. 3+ 187.9 = 301.2 kN/m (SLS)

$T_{rp} + T_{ds}$ =147.3+ 186.1 = 333.4 kN/m (ULS)

In order to mobilize these loads the reinforcement should achieve an adequate bond with the adjacent soil at the extremities of the piled area; this is 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 ($L_b$) beyond the outer row of piles, see figure 2.

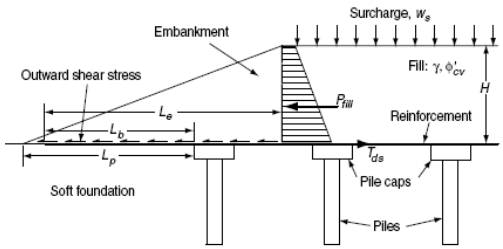


Figure 2. Anchorage length $L_b$ at the edge of the piled area

Depending on the geometry of the embankment, it may be difficult to achieve an adequate bond length at the extremity of the piles by maintaining the reinforcement in a horizontal alignment as depicted in Figure 2. One solution that may be considered is to use a row of gabions, see Figure 3, as a thrust block along the top of the outer row of piles.

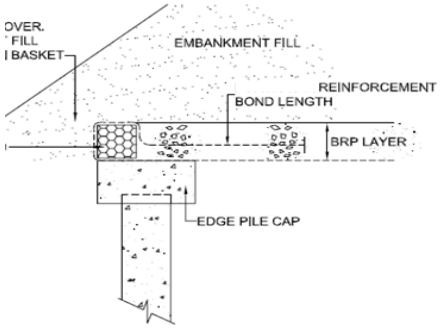


Figure 3. Gabion Anchor at the edge of the piled

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

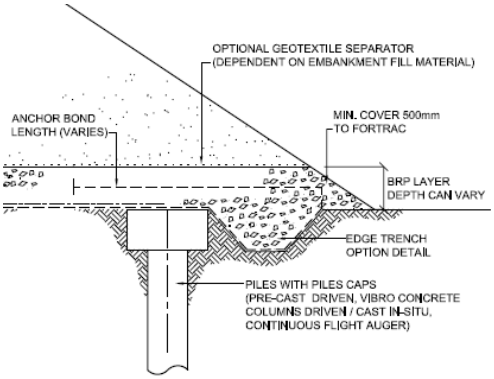


Figure 4. Periphery trench anchorage detail

Another detail that may be considered, see Figure 4, is the inclusion of a small periphery trench just beyond the edge piles, running parallel to the centre-line 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.

### 2.1.5 *Time dependant consolidation of soil between piles*

The British Standard assumes that all of the embankment loading will be transferred through the piles down to a firm stratum either by direct arching on the piles 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 seasonal fluctuations in the ground water level increasing the effective stress on the in-situ soil. Hence the time dependant 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 initially 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).

$$y = (s - a)\sqrt{\frac{3\varepsilon}{8}}$$

where

$a$ is the size of the pile caps

$s$ is the spacing between adjacent piles;

$\varepsilon$ 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

## 3 CONCLUSIONS

BS8006 was first published in 1996. Section 8 of the code has been used to successful design a vast number of piled supported embankments worldwide. UK 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 current BS 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 piles 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 dependant consolidation of soil largely academic.

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