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Design of a reinforced soil capping beam over a soil-bentonite barrier wall

S.J. Goodall
Douglas Partners Pty Ltd, Newcastle, Australia

ABSTRACT: Soil-Bentonite mixtures are frequently used in barrier walls to aid in the control of groundwater flow across contaminated sites. These mixtures typically have low undrained shear strengths with values in the range of 7 kPa to 15 kPa. Issues can arise when infrastructure such as roads are subsequently proposed to cross over these barrier walls. Common practice is to add cement to increase the strength of the mixtures, however, this can be costly and can have detrimental effects on the permeability of the barrier wall. This paper presents an alternative to adding cement by using a reinforced soil capping beam to bridge across the barrier wall. Three design tools are utilised in the current assessment, namely; British Standard 8006-1 design of a reinforced soil embankment over a void; a proprietary program used to model geogrid reinforced pavements; and the numerical software package Plaxis 2D.

1 INTRODUCTION

1.1 *Overview*

As the populations of many industrialised cities continue to grow, the availability of uncontaminated land to develop and to cater for population growth is becoming scarce. As a consequence, contaminated lands are being rehabilitated and used for future infrastructure and development.

One potential form of remediation where groundwater flow is the primary mechanism of contamination migration is a soil-bentonite barrier wall. The walls are used to either cut off the flow of groundwater or redirect groundwater around a contaminated site. The soil-bentonite mixtures within the barrier wall are influenced by permeability requirements, however, strength and compressibility become important when the soil-bentonite cut off walls are installed adjacent to or beneath infrastructure (Baxter et al. 2005).

Typically the shear strength of the soil-bentonite mixture is similar to that of a soft to firm clay, however, it is usually less compressible due to the high content of sand and gravel (Baxter et al. 2005).

To improve the strength of the soil-bentonite mixture in practice, general purpose cement can be add-ed, however, this has implications in terms of cost and material permeability. In general, the strength gains are not usually appreciable and the permeability of the mixture can increase by as much as one order of magnitude (Hale et al. 2015). This in turn diminishes the effectiveness of the barrier wall. As such, an alternative cost effective solution that does not entail adding more cement is the focus of this paper.

Another alternative that does not require adding cement, is to use a reinforced soil capping beam that is designed to bridge over the weaker soil-bentonite wall. The idea is similar to the design of reinforced embankments over weak foundations or voids, where the overall objective is to reduce the shear stress magnitudes and plastic shear deformation on the foundation or to limit surface deformations.

The objective of this paper is to present an overview of some of the analysis methods available that can be used to assist with the design of a reinforced soil capping beam, with a particular focus on providing a trafficable surface over the top of a soil-bentonite barrier wall. It is also the intent of this paper to highlight the importance of how different methods of analysis can be used to complement one another.

A brief overview on some of the general design aspects of a reinforced capping beam such as design criteria, load cases and the geotechnical properties of soil-bentonite mixtures is also provided.

2 DESIGN OF REINFORCED CAPPING BEAMS OVER SOIL-BENTONITE WALLS

2.1 *Design Criteria and Loads*

Surface settlement and distortion are often used as project specific design criteria and are equally relevant for the current problem. In addition to this, creep and permissible strain are also relevant criteria where soil reinforcement (i.e. geogrids) is used, and strength criteria for the bentonite mixture are also essential. An overview on the strength of the soil-bentonite mixtures used in barrier walls is provided in Section Geotechnical Properties of Soil-Bentonite Mixtures.

Design life will usually be dictated by the proposed purpose of the capping beam, however, typically ranges between 40 to 100 years.

Since the focus here will be a capping beam providing a trafficable surface, and also considering the bridging nature of the capping beam, design loads can be derived from Section 6.2 of AS5100.2

(2017). In this instance, the capping beam is designed for the "SM1600" traffic loads. The abbreviation SM1600 represents the design loads W80/ A160, M1600 and S1600.

2.2 *Geotechnical Properties of Soil-Bentonite Mixtures*

Bentonite barrier wall mixtures generally comprise a well graded granular matrix with 20% to 50% plastic fines and a minimum of 1% bentonite (Jones & Taylor 2010, Baxter et al. 2005 and Evans & Ryan 2005), and therefore can typically be described as a sandy clay/ clayey sand.

Evans & Ryan (2005) conducted vane shear tests on a soil-bentonite wall at various time increments. The soil-bentonite mixture typically comprised 5% bentonite and an average of 23.7% plastic fines. The authors reported that most of the strength gain from the initial "liquid" state during back filling of the wall occurs within 1 month and can be attributed to primary consolidation under self-weight. More importantly, even after 6 months there was no detectable change in the shear strength of the soil-bentonite mixture. Based on their data, Evans & Ryan (2005) suggest a design undrained shear strength in the range of 5 kPa to 10 kPa.

Jones & Taylor (2010) undertook 23 piezocone tests in conjunction with shear vane testing to verify the in situ properties of a soil-bentonite barrier wall. The soil-bentonite mixture is described as clayey sand comprising 75% sand, 23% clay and 2% bentonite. Jones & Taylor (2010) reported corrected cone tip resistance (q_t) in the range of 0.2 to 0.4 MPa and corrected shear vane strengths in the range of 5 to 18 kPa.

In-house testing was undertaken by Douglas Partners (2017) on two mixtures: Blend 1 which contained 25% plastic fines and 5% bentonite; and Blend 2 which contained 30% plastic fines and 5% bentonite. The mixtures were incorporated into a permeability cell for triaxial permeability testing. Unconfined compression strength testing could not be undertaken due to the low strength of the mixtures and therefore pocket penetrometer testing was undertaken on the samples at the completion of the permeability testing to provide an indication of strength. Pocket penetrometers readings were in the range of 30 kPa to 40 kPa, some 60% to 70% lower than the design strength criteria. General purpose cement (1% total) was added to the mixtures to improve the strength, however, the overall strength gains were only marginal (i.e. increase of about 5 kPa shear strength) and the permeability of the mixtures increased, with the results of testing indicating that Blend 2 with cement would no longer satisfy permeability criteria.

2.3 *Methods of analysis*

Three design approaches are considered here: British Standard 8006-1, Design of a Reinforced Soil Embankment over a Void (British Standard 2010); TensarPave, a proprietary program used to model geogrid reinforced pavements; and the numerical software package Plaxis 2D. A discussion on each method is provided in the following sections.

2.3.1 *British Standard 8006-1 2010 Section 8.4* The method is a limit state design approach and is aimed at analysing the reinforcing of a soil embankment over a void based on allowable surface deformations and strains in the reinforcement. The method requires the selection of reinforcement to ensure the serviceability limit state (SLS) is maintained and collapse, i.e. the ultimate limit state (ULS) case, does not occur.

The method is based on two principle assumptions: the volume of soil in the "zone of depression" remains constant; and no arching occurs within the capping beam. These assumptions can lead to conservative designs, particularly if the ratio of the capping beam thickness to the width barrier wall is greater than one due to arching effects in the capping beam.

Further discussion on the method is provided in BS8006-1, however, the general process is summarised below:

- *Determine the maximum acceptable surface deformation which is defined as the ratio of the depression at the surface d_s over the surface deflection zone D_s. This is a function of the road type crossing the void and varies between 1% to 2%;*
- *Determine the tensile properties of the reinforcement needed for the design; and*
- *Based on the anticipated tensile load, estimate the bond length L_b such that the geogrid does not pull out of the embankment material.*

An extract from British Standard 8006-1 2010 Section 4 (BS8006-1) illustrating the problem and nomenclature has been provided in Figure 1. For the purposes of the discussion herein, the embankment height H is the capping beam depth and the void width D is the width of the barrier wall.

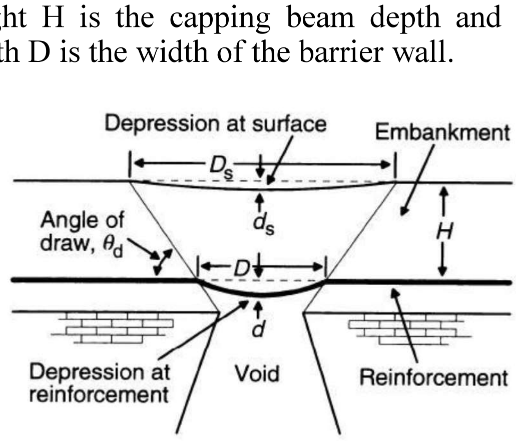

Figure 1. BS8006-1 Problem Definition

Since this method is developed for spanning voids, it is apparent the method could yield conservative results. In reality, the soil-bentonite wall will provide some support to the capping beam so long as the capping beam is constructed following completion of primary consolidation, noting that primary consolidation due to self-weight typically occurs within four to six weeks for soil-bentonite walls (Evans & Ryan 2005).

In this instance, a simple modification to the BS8006-1 method may be considered, where the net vertical pressure at the base of the capping beam is used, rather than the reinforcement sustaining the full magnitude of the applied load, i.e. the difference between the applied load and support provided by the soil-bentonite mixture. For the analysis considered here, both results from the allowable and ultimate bearing capacity of the soil-bentonite mixture are presented. The ultimate bearing capacity is evaluated using the following equation:

$$q_u = N_c \times s_u \quad (1)$$

Where N_c is a bearing capacity factor taken as 5.14 and s_u is the undrained shear strength of the soil-bentonite mixture. The allowable bearing capacity is obtained by dividing Equation 1 by an appropriate factor of safety, taken as 2.5 in this instance.

2.3.2 *TensarPave Version 7.00.12*

TensarPave is a program that models geogrid reinforced pavements according to the AASHTO design method. The procedure uses the number of axle movements in determining a pavement profile similar to conventional pavement thickness design procedures, and therefore differs significantly from BS 8006-1.

Since a backfilled trench cannot be specifically modelled in TensarPave, the subgrade CBR can be set to that of a bentonite clay, for which a conservative California Bearing Ratio (CBR) in the range of 0.5 to 1% can be adopted.

The traffic frequency adopted for this paper is based on two of each load type from AS5100.2 over a design life of 40 years. A total of 1.68×10^6 equivalent standard axles (ESA) has been adopted for the TensarPave analysis presented here.

The software includes a wearing course where values of the structural layer coefficient (based on the Elastic Modulus of the layer material) are assigned to both the granular base layer and sub base. A constant value of 0.14 was assigned to the structural layer coefficient for the granular base layer and sub base since the material compaction must be consistent over the full depth of the capping. The geogrid properties are built into the software and are limited to the application of hexagonally ribbed geogrids used in pavement stabilisation scenarios.

2.3.3 *Plaxis 2D 2018*

The two-dimensional finite element software Plaxis 2D has been used to analyse the capping beam and barrier wall. The package is intended for deformation and stability analysis in geotechnical engineering. Since the soil stratigraphy, construction staging, problem geometry and soil models based on geotechnical data can be incorporated, fewer assumptions and simplifications (compared to the previous methods) are made when analysing the capping beam and barrier wall.

For the purpose of this paper, a typical "pre-construction" geotechnical model for a soil-bentonite wall has been adopted. The Mohr Coulomb (MC) soil model was used to represent the soil materials considered herein. The model comprised 10 m of medium dense sand, overlying very stiff clay. It is assumed that the soil-bentonite wall penetrates 1 m into the very stiff clay, hence creating a barrier wall. The groundwater level was set to coincide with the base of the capping beam.

3 RESULTS AND DISCUSSION

The general approach adopted for the analysis of the capping beam is to use BS8006-1 and TensarPave to converge on a preliminary design, with the main objective to estimate the capping beam thickness and geogrid requirements to satisfy design criteria. Plaxis 2D is used to undertake a more rigorous analysis, where the focus is to develop a better understanding of the mechanics of the problem and undertake more detailed sensitivity analyses.

For the purpose of this paper, the soil-bentonite is assumed to be 0.8 m wide and have a design undrained shear strength of 10 kPa. It has also been assumed that the capping beam itself is a well graded compacted crushed rock (i.e. minimum peak friction angle of 40°). A nominal value of 1% has been adopted for the maximum permissible surface deformation, as required by BS8006-1.

3.1 *BS8006-1*

The BS8006-1 method can be used as an iterative procedure, however, it is useful to generate a series of curves that are a function of the bond length L_b required to sustain the applied loads (noting that the bond length is a function of the load on the geogrid) that have been normalised by the depth of the capping beam H, plotted against the depth of the capping beam H normalised by the width of the barrier wall D. Three cases were considered based on the ULS case according to BS8006-1: Case 1 assuming a void 0.8 m wide is being spanned; Case 2 where the method has been modified and support is limited to the allowable bearing capacity of the soil-bentonite; and Case 3 where support is provided based on the ultimate bearing capacity of the soil-bentonite.

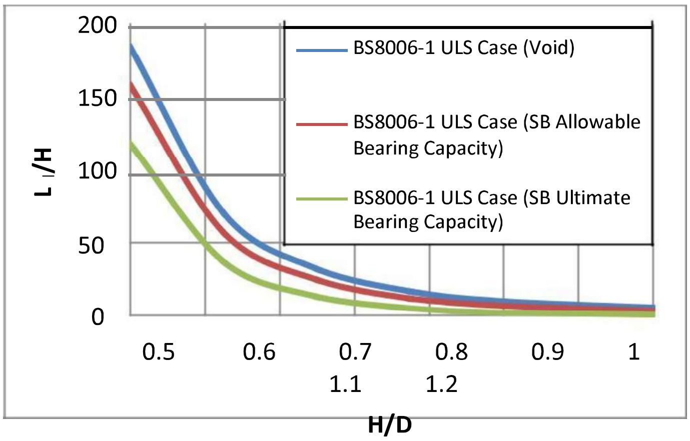

Figure 2. Plot of normalised bond length based BS8006-1

Figure 2 indicates that by considering the support provided by the soil-bentonite, reductions of the required bond length L_b are reduced by about 25% on average for the case where the allowable bearing capacity is adopted, and up to about 50% on average for the ultimate bearing capacity for H/D up to 0.7.

As could be expected, the required bond length decreases as the depth of the capping beam increases. However, beyond a ratio of about H/D ≈ 0.8, the support provided by the soil-bentonite and the reduction in bond length becomes far less appreciable with respect to the thickness of the capping beam. This can be attributed to the effective load on the reinforcement reducing as the depth of the capping beam increases, and also, as the thickness of the capping beam converges to the same width as the barrier wall, the surface deflection zone D_s reaches a limiting value, hence the incremental increases in strain in the reinforcement reduce as the capping beam thickness is increased.

According to the BS8006-1 method, a capping beam of 0.7 m thickness would satisfy the maximum permissible surface deformation of 1% (i.e. about 25 mm based on a surface deflection zone of 2.5 m), provided the reinforcement is capable of sustaining an ultimate tensile force (T_{ult}) of 31 kN/m and serviceability strain of approximately 2%. Geogrids capable of sustaining this tensile loading, for example, include Tensar SS30 and Macgrid EG 30S. It must be noted that the above ultimate tensile force has been obtained by assuming that the ultimate bearing capacity of the soil-bentonite is mobilised. This is to be consistent with the Plaxis 2D analysis which uses un-factored parameters of soil strength (i.e. working stress analysis).

The BS8006-1 method has been derived assuming a single layer of reinforcement, where the method indicates a required bond length of approximately 3 m either side of the barrier wall to sustain the estimated tensile force. In practice this may not be achievable, particularly since reinforcement rolls are typically 3.8 to 4 m wide. An alternative is to incorporate two layers of reinforcement to sum up to the total reinforcing bond length required. It must be recognised that the reinforcement effect provided by geogrids (or other soil reinforcement types) depends on a complex interaction between the reinforcement and soil that it is embedded within, and that it is unlikely that the load would be shared equally over the reinforcement layers. Further guidance on multiple layers of reinforcement is provided in British Standard 8006-1 2010.

Based on the discussion above, two layers of geogrid embedded 1.5 m either side of the barrier wall may be adopted.

3.2 *TensarPave*

Unlike BS8006-1, the Tensar software uses an empirical approach, hence, the capping beam thickness and reinforcement requirements are obtained directly. It is worthy to note that the method relies heavily on the subgrade CBR, hence, sensitivity analyses are prudent. For the purposes of this paper, a sensitivity analyses was undertaken for CBR values ranging from 0.5% to 2%, where five cases were analysed: no reinforcement; a case with a single layer of Tensar TX 160 geogrid; and three cases with two layers of reinforcement using Tensar TX150, TX160 or TX170 geogrids, as illustrated in Figure 3.

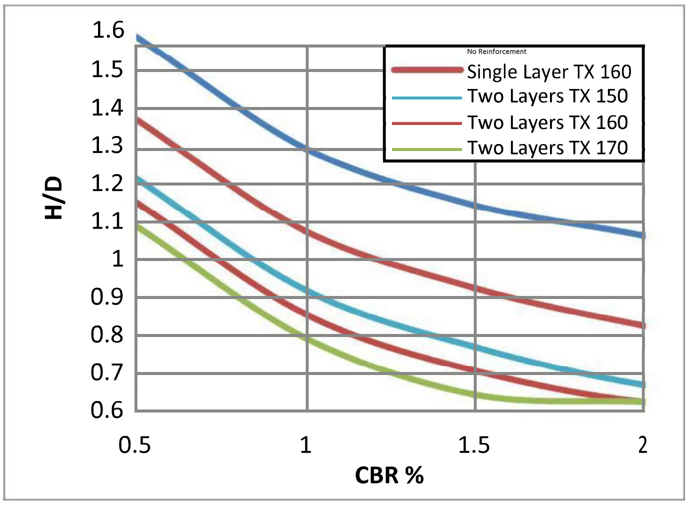

Figure 3. Plot of CBR vs. normalised capping beam depth

Figure 3 presents the five cases for a range of subgrade CBR plotted against the resulting capping beam depth H normalised to the width of the barrier wall D. As could be expected, appreciable reductions in the depth of the capping beam are observed when reinforcement is incorporated. A CBR of 0.5% leads to increases in the capping beam thickness of about 30% to 40%. Beyond a CBR of about 1.5%, the change in capping beam thickness becomes less appreciable and is generally less than about 10% for the scenarios with two layers of geogrid. It must be noted that for the scenarios where two layers of reinforcement are incorporated, the minimum total thickness permitted in TensarPave for the base and sub base is 500 mm.

Based on the assumed traffic loading, a design CBR of 1%, and that two layers of geogrid are provided, the capping beam needs to be between 635 mm to 685 mm, depending on the reinforcement specified.

3.3 *Plaxis 2D*

The estimated thickness of the capping beam from the BS8006-1 and TensarPave methods agree well, however the BS8006-1 method governs the minimum thickness of the capping. Considering the outcomes from the former two analyses, Plaxis 2D is used to analyse a capping beam of 0.7 m depth with two layers of geogrid reinforcement embedded within the capping beam (spaced 100 mm apart). A bond length of 1.5 m either side of the wall, as evaluated by BS8006-1, is adopted. An axial stiffness (EA) of 444 kN/m is adopted for the geogrid, commensurate with the types of geogrids discussed earlier. The surface deflection profiles from each load case based on AS5100.2 are presented in Figure 4.

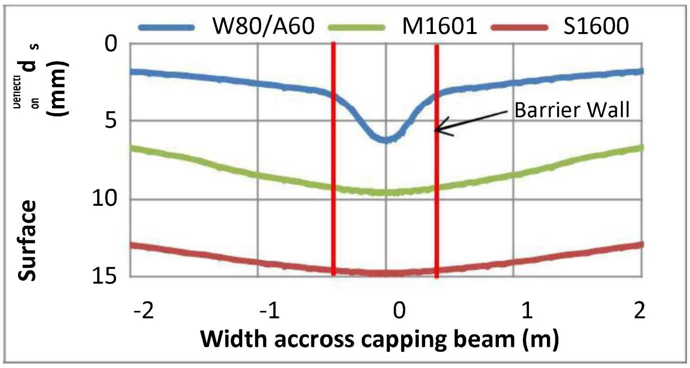

Figure 4. Plaxis 2D surface deflection for AS5100.2 load cases

The Plaxis 2D analysis indicated a maximum surface deflection of 15 mm resulting from the S1600 load case. It is noted that the S1600 is a stationary load case and is a conservative upper bound since it is highly unlikely that the loads over the capping beam will be stationary. Furthermore there is some inherent conservativism associated with converting the AS5100.2 load cases to plane strain equivalent loads. Despite the load being conservative, it is noted that the corresponding allowable surface deformation is less than the criteria of 1% used in the BS8006-1 analysis (i.e. based on a surface deflection zone of 2.5 m, the maximum surface settlement permitted is 25 mm).

The maximum tensile force (T_{max}) in the geogrid resulting from the AS5100.2 load cases ranges from 0.25 kN/m to 0.61 kN/m, and is considerably lower than the estimated ultimate tensile force (T_{ult}) of 31 kN/m using BS8006-1. The corresponding strains are also much lower than the estimated 2% allowable from the BS8006-1 method. The relatively low tensile force that develops in the geogrids can be attributed to the arching of loads in the capping beam, as illustrated in Figure 5.

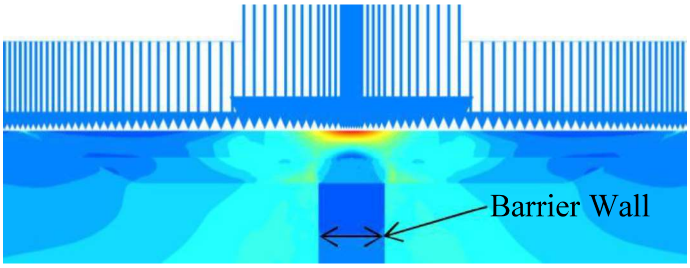

Figure 5. Plaxis 2D Mobilized Shear Stress S1600 Load case

Figure 5 also illustrates that the overall shear stress magnitudes on the barrier wall are minimised.

A range of sensitivity analyses were undertaken to evaluate the effects of some of the input parameters on the estimated surface deflection. The parameters selected for the analysis are as follows:

- *Undrained shear strength S_u of the soil-bentonite;*
- *Elastic modulus E'_{sb} of the soil-bentonite;*
- *Friction angle φ' of the in situ sand;*
- *Elastic modulus E' of the in situ sand; and*
- *Axial stiffness EA of the geogrid:*

Two additional cases were also examined where the reinforcement was reduced to a single layer and removed entirely. A summary of the analysis cases and parameters has been provided in Table 1.

The resulting surface deflection from the sensitivity analysis for each case has been presented in Figure 6 and the maximum tensile force (T_{max}) and resulting strain in the base geogrid from selected cases in Table 2. The outputs have been limited to the M1600 load case.

Table 1. Plaxis 2D sensitivity analysis cases

Case	S_u (kPa)	E'_{sb} (MPa)	φ' (deg)	E' (MPa)	EA (kN/m)	#Layers geogrid	$\frac{E'}{E_{sb}}$
0*	10	5	35	35	444	2	7
1	5	5	35	35	444	2	7
2	15	5	35	35	444	2	7
3	10	2.5	35	35	444	2	14
4	10	7.5	35	35	444	2	4.7
5	10	5	30	35	444	2	7
6	10	5	40	35	444	2	7
7	10	5	35	15	444	2	3
8	10	5	35	80	444	2	16
9	10	5	35	35	200	2	7
10	10	5	35	35	600	2	7
11	10	5	35	35	444	1	7
12	10	5	35	35	-	0	7

*Case 0 is the reference case

The sensitivity analyses on the selected input parameters indicated that the surface deflection is not overly sensitive to the properties of the soil-bentonite or the stiffness of the geogrid. However, the elastic modulus E' of the sand material (Case 7 and 8) of which the barrier wall is constructed in has a significant effect on the resulting surface deflections.

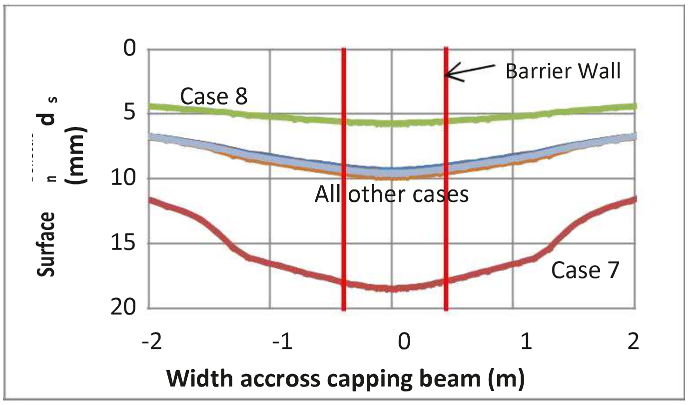

Figure 6. Plaxis 2D sensitivity analysis surface deflections

Table 2. Plaxis 2D sensitivity analysis geogrid outputs

Case	T_{max} (kN/m)	ε (%)
0	0.43	0.10
7	0.85	0.19
8	0.16	0.04
9	0.23	0.11
10	0.55	0.09
11	0.47	0.10

The elastic modulus of the sand is not a parameter that the earlier methods of analysis consider. For Case 7 where the modulus is reduced to that of loose sand, the peak deflection is nearly doubled. Whereas Case 8, based on dense sand, the deflection is halved and is more uniform across the capping beam. These results are not unexpected since the capping beam arches over the barrier wall, hence, transferring the applied load to the sand material either side of the barrier wall. This highlights the importance of numerical analysis, particularly for cases where E' tends to E'_{sb}.

The analysis also confirms that the variations in tensile forces in the geogrid are minor, a likely result of the capping beam arching. In the instance where a single geogrid is used, the peak tensile force is comparable to the reference case. In fact, the peak tensile force in the second layer of geogrid used in the reference case is also 0.4 kN/m. This highlights that load sharing across geogrids is not necessarily straightforward and that caution should be exercised if the required bond length is shared over more than a single layer of reinforcement to sum up to the total bond length required, as was the case in the BS8006-1 analysis.

4 CONCLUSIONS

The analysis methods presented in this paper can be used to develop an understanding of the mechanics of a capping beam intended to provide a trafficable surface over a barrier wall. Whilst it is not recommended that each method is used in isolation, collectively they can be used to demonstrate the performance of a reinforced capping beam bridging a barrier wall.

The BS8006-1 method provides a good first pass assessment of the capping beam to bridge the barrier wall, however, is likely to yield conservative results due to the assumption that no arching occurs. On the contrary, the method provides valuable insight to the scenario where arching in the capping beam potentially breaks down. Where the method suggests substantial bond lengths, caution must be used if more than one layer of reinforcement is required to achieve the bond length due to the complexity of load sharing across any additional layers of reinforcement.

The TensarPave software can be used to analyse the performance of the capping beam as a pavement. It must be appreciated that the reinforcement available in the software is limited to applications where stabilisation of the pavement is the primary focus.

Due to the simplifications and assumptions made in the BS8006-1 and TensarPave software, more rigorous analysis using a numerical software package, such as Plaxis 2D, is recommended. The Plaxis 2D analysis considered here demonstrated that arching of the load occurs and as a result the overall magnitude of shear stress on the barrier wall is minimized. A sensitivity analysis using Plaxis 2D highlighted important parameters, such as the stiffness of surrounding soil, which are not considered in the former two methods of analysis.

5 ACKNOWLEDGEMENTS

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REFERENCES

Australian Standard. 2017. *Australian Standard Bridge Design Part 2: Design Loads.* Standards Australia.

Baxter, D.Y., Filz, G.M. & Heslin, G.M. 2005. Strength and compressibility of soil-bentonite mixtures for cutoff walls. *Waste Containment and Remediation*. American Society of Civil Engineers.

British Standard. 2010. *Reinforced embankments over areas prone to subsidence*. BSI Standard Publication.

Evans. J.C. & Ryan.C. 2005. *Time-dependant behaviour of soil-bentonite slurry wall backfill.*

Hale.J., Hubaut.A. & Vincent.P. 2015. Utilisation of deep groundwater barrier walls using soil-bentonite and biopolymer slurries in geotechnical and environmental applications. *Australian Geomechanics Society Sydney Chapter Symposium.*

Jones, S.R., Taylor, G. 2010. *Verification of a deep soil-bentonite barrier wall using CPTu; Proceedings, 2nd International Symposium on Cone Penetration Testing., Huntington Beach, May 2010*. California.

Plaxis 2D [Computer Software]. 2018. Plaxis bv. Delft, Netherlands.

TensarPave Version 7.00.12[Computer Software]. Tensar International Limited.