AN EXPERIMENTAL INVESTIGATION OF ARCHING IN PILED EMBANKMENTS

Emma Britton[1] & Patrick Naughton[2]

[1] *Institute of Technology, Sligo. (e-mail: britton.emma@itsligo.ie)*
[2] *Institute of Technology, Sligo. (e-mail: naughton.patrick@itsligo.ie)*

Abstract: The design of piled embankments is extremely complex and relies on determining the magnitude of arching in the embankment fill and the tension in a geosynthetic reinforcement layer at the base of the embankment. Several design methods, BS 8006 (1995), Kempfert et al (2004), Russell et al (2003), Jenner et al (1998), Hewlett & Randolph (1988) and Terzaghi (1943), are available for estimating the magnitude of arching and the tension in the geosynthetic reinforcement. However, Naughton & Kempton (2005) showed that significant inconsistencies exist between these design methods in both estimating the magnitude of arching and the tension in the reinforcement.

Naughton (2007) showed that the magnitude of arching and therefore the tension in the geosynthetic reinforcement can be estimated based on the concept of critical height. The critical height is a function of the frictional characteristics of the embankment fill and pile-pile cap spacing at the base of the embankment.

An instrumented laboratory 1:3 scaled model of typical piled embankment geometries was used to investigate the influence of the critical height in the design of piled embankments. A detailed description of the properties of the sand fill and the laboratory model is presented. The model was used to quantify the magnitude of arching and load transfer for the test sand investigated.

The experimental results are used to validate the concept of critical height on the assessment of arching in piled embankments. Recommendations are made on how the concept of critical height can be integrated into routine piled embankment design.

Keywords: Arching, design method, differential settlement, soft soil, piled embankment, shear strength.

INTRODUCTION

In many parts of the world, construction of road and rail networks is challenging due to marginal subsurface soils, such as soil with low bearing capacity or consolidation characteristics which could result in large differential settlements.

Designing structures, such as embankments, on soft foundation soils where the structure will impose a significant load over a large area, raises several concerns. These concerns are related to time constraints, excessive total and differential settlements, large lateral pressures and movement and slope stability. A variety of techniques can be used to address these concerns which include preloading or stage construction, using lightweight fill, over-excavation and replacement, geosynthetic soil reinforcement and piled embankments. The benefits of using piled embankments over other techniques are that superstructures can be built in a single stage without prolonged construction times and significant reduction in the total and differential settlements.

Theoretical studies on geosynthetic reinforced piled embankments have largely focused on the investigation of load transfer mechanisms including soil arching and tension developed along the geosynthetic. However, limited research has been carried out to investigate the true nature of these load transfer mechanisms and the factors which effect them such as dilatancy of the fill material and friction angle.

The more popular design methods used for the design of piled embankments, as outlined below, have a tendency to concentrate on the estimation of the vertical stress at the base of the embankment after arching has occurred. Naughton (2007) looked at an alternative approach based on the concept of the plane of equal settlement which was originally proposed by Marston (Spangler & Handy, 1973). Shear stresses are generated in the embankment as a result of the differential movement in the embankment fill above the rigid pile caps and soft soil. When the height of the embankment is sufficiently large it is assumed that these shear stresses terminate at some horizontal plane in the embankment fill; termed the plane of equal settlement.

DESIGN METHODS

There are various methods available for designing piled embankments. It is generally assumed in all cases that the total vertical load of the embankment is transferred to the piles by either soil arching within the fill material or by basal reinforcement spanning between adjacent piles. Six of the most popular design methods are reviewed in this paper.

The Stress Reduction Ratio, S_{3D}, the ratio of the average vertical stress carried by the reinforcement to the average stress due to the embankment fill, first proposed by Low et al (1994), is used to compare the output from the design methods. Also these methods use the concept of critical height in estimating the magnitude of arching in the fill material. The critical height is defined as the height from the top of the pile caps to the plane of equal settlement in the embankment fill.

BS 8006 (1995)

BS 8006 (1995) has adopted an empirical method initially developed by Jones et al (1990), which is based on Marston's equation for positively projecting conduits. In the BS 8006 method the stress concentration on the piles and consequently the stress remaining to be carried by the geosynthetic, depends on the pile type and the pile support condition.

BS 8006 identifies a critical height concept whereby the depth of fill is sufficient for the full arch to be deemed to have developed and any additional overburden or surcharge loads do not influence the tensioned membrane, but distribute to the boundary supports, i.e. the pile caps. It assumed that the plane of equal settlement occurred at a height of 1.4 times the clear spacing between adjacent pile caps in a square grid.

Kempfert et al (2004)

The Kempfert et al (2004) method which was derived from 1:3 laboratory models of piled embankment problems. The magnitude of load on the soft soil, without reinforcement, is first estimated before the tension in the reinforcement is determined. It was observed from the laboratory study that in the reinforcement between adjacent piles a higher tension was generated. The tension in the reinforcement is estimated based on the theory of elastically embedded membranes.

Following finite element and experimental investigations, the height of the plane of equal settlement was deemed to be located at a distance of half the pile spacing above the pile caps.

Russell et al (2003)

Russell et al (2003) found from numerical analysis of the piled embankment problem that reinforcement tension was concentrated in the area directly between the pile caps. Because of this the geosynthetic reinforcement was divided into two types; primary, which spans between the pile caps and secondary, which covers the entire piled area. Also this design method allows for support from the subsoil, the magnitude of which can be determined from compatibility checks on the deformations of the reinforcement and subsoil.

This design method proposed that for ultimate limit state (ULS) the critical height equal to the embankment height should be used, and for serviceability limit state (SLS) the critical height is located at 0.8 times the embankment height.

Jenner et al (1998)

The Jenner at al (1998) design method was developed from the results of plate loading tests on samples of reinforced granular material in a confined rigid box reported by Guido et al (1987). The method assumes that the arching mechanism in the fill above the pile caps is increased with the inclusion of geogrid and therefore the tensile forces within the geogrid is lower than that assumed by the other design methods. The difficulty with the Jenner et al method is that gravity acts in the opposite direction to that used in the laboratory trials on which the method is based (Love & Milligan, 2003). Jenner et al (1998) does not refer to a plane of equal settlement.

Hewlett & Randolph (1988)

The Hewlett & Randolph method is based on data observed from experimental tests carried out on free draining granular soil. It was observed that the region of sand between the pile caps comprised of a series of hemispherical domes having radii approximately equal to half the diagonal pile spacing therefore resulting in a critical height of 1.4 times the spacing between pile cap edges. As the stress reduction ratio is calculated assuming limited plastic stress in the arch, it was also found that there are two critical locations within these domes, which were shown to be at either the crown of the arch or at the pile cap. The higher stress reduction ratio is to be used in design.

Terzaghi (1943)

Terzaghi (1943) examined arching in sand directly above a yielding trap door. When the trapdoor was lowered the load in the sand was redistributed to the non yielding surrounds, i.e. the load on the trapdoor reduced while the load on the non yielding supports increased. It was found that at a height of more than 2.5 times the clear spacing of the yielding trap door there was no effect on the state of stress in the sand. Therefore it was assumed that the shearing resistance of the sand was only active up to this height. The Terzaghi equations for the rectangular trapdoor problem were extended by Russell & Pierpoint (1997) to take account of the typical cruciform shape of a piled embankment.

Horgan and Sarsby (2002)

Horgan & Sarsby (2002) carried out plane strain model tests using a sand box with a Perspex front and incorporating a trap door. The tests were performed using two fill types, a course sand and a 10 mm stone. The results obtained showed that disproportionate additional stress redistribution occurred when the depth/span ratios increased from 1.545 to 1.92. This illustrated that the critical height for the materials used was located between 1.545 and 1.92 times the clear spacing between the supports.

Naughton (2007)

Naughton (2007) presented a method for estimating the magnitude of arching based on the plane of equal settlement. The critical height was calculated using a log spiral shaped shear plane in the embankment fill, above the pile caps. By applying boundary conditions to the general equation for a log spiral an expression for the critical height was determined and found to vary between 1.24 (s-a) and 2.40 (s-a) as the angle of friction increased from 30° to 45°.

The critical height recommendations for the design methods considered in this paper are summarised in Table 1.

Table 1. Summary of critical heights for various design methods

Design Method	**Critical Height, H_C**
BS 8006 (1995)	1.4 (s-a)
Kempfert et al (2003)	s/2
Russell et al (2003)	H (for ULS)
Hewlett & Randolph (1988)	1.4 (s-a)
Terzaghi (1936)	2.5 (s-a)
Horgan & Sarsby (2000)	1.545 (s-a) to 1.92 (s-a)
Naughton (2007)	1.25 (s-a) to 2.40 (s-a)

EXPERIMENTAL MODEL

A 1:3 laboratory model of the piled embankment problem was developed as part of this study, Figure 1. The model consisted of a 1 m^3 box with a movable base. Four pile caps in a unit cell of a piled embankment are represented in the model by blocks of plywood. Load cells are located beneath the pile caps to measure the change in load due to arching as the cruciform shaped base between the pile caps is lowered.

Figure 1. Experimental model

Sand samples, with homogeneous densities, were formed in the apparatus using a raining deposition technique similar to that described by Schnaid (1991). The target sample densities were achieved using a combination of different shutter plates and diffuser sieves. Dense samples were obtained by passing the sand through perforated plates having 6 mm holes on a 80 mm triangular grid and raining through 2 No. 6 mm sieves located 150 mm and 250 mm respectively from the base if the hopper. Loose samples were obtained by passing the sand through perforated plates having 20 mm holes on the same triangular grid, and omitting the diffuser sieves. The model was filled in four lifts and densities were measured at each lift to check the homogeneity of the sample density.

MATERIAL PROPERTIES

The sand investigated in this study, Figure 2, was uniformly graded, rounded, medium sand which was recovered from excavations (close to the ocean) at Ballyshannon, Co. Donegal, Ireland. The sand properties, Table 2, were determined in accordance with BS 1377 (1990).

Table 2. Properties of sand investigated in this study

Specific Gravity, Gs	2.66
Coefficient of uniformity, Cu	1.49
Coefficient of Curvature, Cc	1.10
Maximum Void Ratio, emax	0.84 ± 0.01
Minimum void ratio, emin	0.39 ± 0.006
Maximum particle size Dmax	2 mm

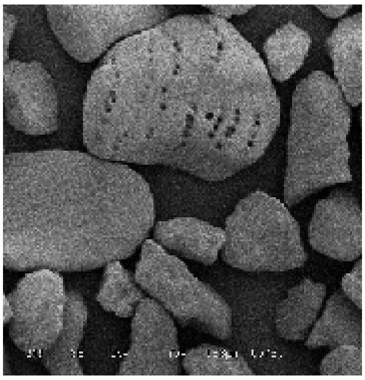

Figure 2. Electron microscope view of Sand at a magnitude (x75)

The shear strength and dilatancy characteristics of the sand were obtained by direct shear tests. The samples were tested under normal stresses ranging from 123 kPa to 368 kPa. The angle of internal friction was found to be 36°for $I_D = 0.798$ and 43° for $I_D = 0.949$, with the angle of dilation of 10° ± 1.5%. The limiting densities and density indexes are presented in Table 3.

Table 3. Limiting Dry Densities and Density Index's

Dense			Loose		
Max density, ($\rho_{D,max,}$) kg/m^3	Sample density, (ρ_D) kg/m^3	Density Index, (I_D)	Min density, ($\rho_{D,min}$) kg/m^3	Sample density, (ρ_D) kg/m^3	Density Index, (I_D)
1.550	1.537	0.949	1.33	1.500	0.798

EXPERIMENTAL TESTING

The trapdoor at the base of the model was cruciform in shape; two pile cap sizes were used. The first occupied 51 % of the 1 m^2 base, with each pile cap having a plane area of 0.1225 m^2. The second occupied 64 % of the 1 m^2 base, with each pile having a plane area of 0.16 m^2.The height of the sand above the pile caps was 1 m in both cases. The reason for the variation in pile cap size was to represent two height/span ratios.

The sand was placed evenly on the pile caps and the trapdoor in 250 mm lifts using the sand raining technique. Densities were checked at each deposition and were found to range between 1394 kg/m^3 and 1413 kg/m^3 with an average density of 1400 kg/m^3.

After placing the sand the trapdoor was lowered and the sand allowed yield. The output from the load cells was monitored every 3 seconds.

ANALYSIS OF RESULTS

Initial readings of the load on the pile caps were taken when the model was full. When the trapdoor was released the load on the pile caps increased dramatically in all cases and then remained relatively constant. The increase in load on each pile cap ranged from 115 kg to 125 kg, Figure 3. The load transfer is due entirely to arching within the fill material as geosynthetic reinforcement was not incorporated into the experimental model.

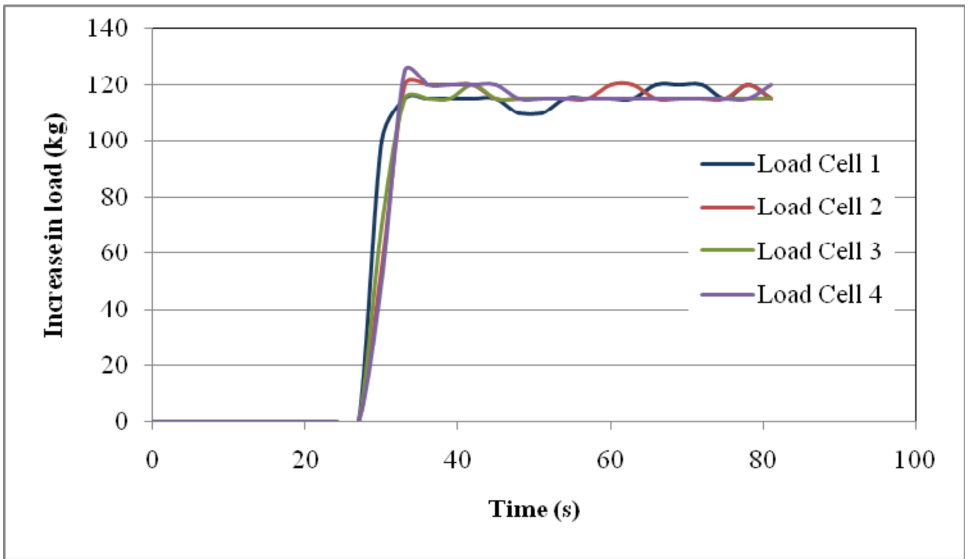

Figure 3. Load increase on pile caps after trapdoor is released (ρ= 1400 kg/m^3)

The weight of sand on the trapdoor after its release was calculated by subtracting the sum of the load recorded by the 4 load cells from the total weight of the sand within the model. As this weight was acting over a known area and the sample was at a constant known density, the height to the top of the arch could be estimated assuming a uniform critical height over the cruciform shaped base. The values for the critical height obtained from the experimental tests are compared with those given in the design methods, shown graphically in Figure 4.

The experimental models value for the critical height was found to be in close agreement with the value suggested by Naughton (2007) and also within the range given by Horgan & Sarsby (2000). The values obtained from the Russell et al (2003), Terzaghi (1936) & Kempfert et al (2003) design methods seem to over predict the critical height thus making them conservative while the values obtained from BS 8006 (1995) and Hewlett & Randolph (1988) are under conservative.

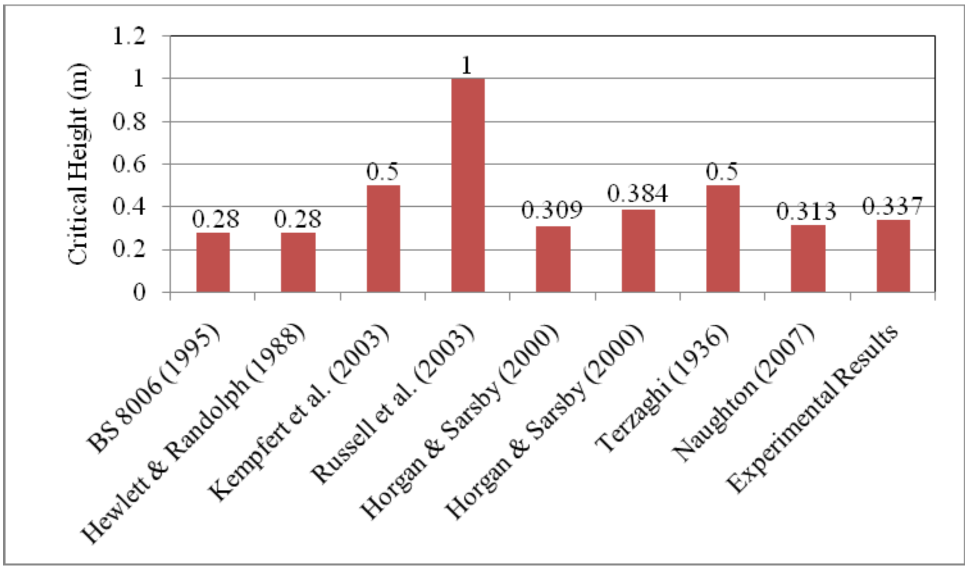

Figure 4. Comparison of the critical heights from design methods & experimental results

The stress reduction ratio of the experimental data was calculated based on the initial weight of sand over the pile cap and the increase in load recorded once the trap door was dropped. A comparison of the design methods and the test data based on stress reduction ratio is presented in Figure 5. The experimental models stress reduction ratio was similar to those predicted by both the Naughton, (2007) method for critical height and the Terzaghi (1943) arching theory. The values obtained from the BS 8006 (1995), the Hewlett & Randolph (1988) and the Kempfert et al (2004) arching theories were slightly lower and are therefore less conservative, underestimating the load at the base of the embankments between pile caps.

A possible explanation for the discrepancy in the stress reduction ratio may have been due to the cruciform shape of the trapdoor. The majority of previous experimental studies on piled embankments used a square or rectangular trapdoor as opposed to the cruciform shape which occurs in an actual piled embankment, (Russell et al 2003).

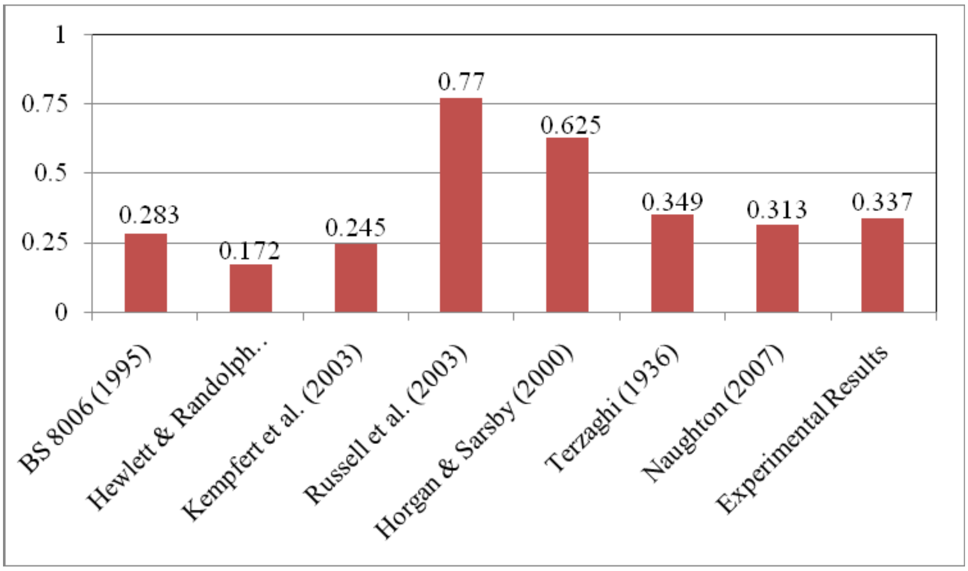

Figure 5. Comparison of the Stress Reduction Ratios, (S_{3D}), from design methods & experimental results

CONCLUSIONS & RECOMMENDATIONS

Piled embankments are becoming increasingly popular in construction of road and rail networks on marginal subsurface soils as they are often the only practical and economic method available. The piled embankment application is truly a three dimensional problem and therefore should be modeled as such.

A series of model tests have been carried out to investigate both arching and the concept of critical height in piled embankments. The results from the experimental model were compared to current design methods using both the critical height and the stress reduction ratio. The value for the critical height obtained from the model was found to be in close agreement with the value suggested by Naughton (2007) and also within the range given by Horgan & Sarsby (2000). The model results for the stress reduction ratio also compared well with Terzaghi (1936) arching theory and the Naughton (2007) method for critical height.

The experimental work discussed in this paper is currently ongoing and samples of various densities over different pile cap sizes are to be tested using the model, so as to determine whether the strength and dilatancy of the fill material has an effect on the critical height and arching behaviour.

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Corresponding author: Miss Emma Britton, Institute of Technology, Sligo, Ballinode, Sligo, Ireland. Tel: 00353719155481. Email: britton.emma@itsligo.ie.

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