Lateral deformation under the side slopes of piled embankments

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Keywords: piled embankment, lateral loads, deformation, reinforcement tension, Plaxis 2D

ABSTRACT: Piled embankments are used widely for the construction of roads and rails infrastructure over soft and/or compressible soils. The design of piled embankments is a complex soil – structure interaction problem and is not yet fully understood. Traditional design methods, such as BS 8006 (1995), assume that any lateral trust is carried by the geosynthetic reinforcement placed at the base of the embankment directly over the pile caps. At the extremities of the embankment both horizontal equilibrium and strain compatibility between the different components, the embankment fill, the geosynthetic reinforcement, the pile group and the soft soil, must be achieved (Love & Milligan 2003). This paper examines the horizontal and vertical deformation, axial tension developed in the geosynthetic reinforcement and bending moment developed in the pile for a range of complex embankment geometries. The finite element analysis using Plaxis 2D indicates for the geometries examined, that both the horizontal and vertical deformation under the side slope of the embankment can be significant. An increase in the reinforcement tension under the side slope was also observed. The maximum bending moment was found in the outer most rows of piles, while the piles near the center line had a reasonably small moment. The deformations, axial tension and bending moment in the piles were found to be a function of the embankment geometry (height, steepness of side slope and pile spacing) and stiffness of the foundation soil.

1 INTRODUCTION

The design of piled embankments is a complex soil-structure interaction problem involving embankment fill, geosynthetic reinforcement, a pile group and the soft underlying soil. In designing a piled embankment both vertical and horizontal equilibrium must be achieved. The internal horizontal (lateral) force from the embankment fill acting outwards needs to be balanced by a combination of the tension in the horizontal geosynthetic reinforcement, lateral loads on the pile group and the soft soil. In addition to equilibrium considerations, strain compatibility between the displacement of the geosynthetic reinforcement, the pile group and the soft soil must be achieved.

2 OVERVIEW OF LATERAL PILE LOADING

Current design approaches (BS 8006 1995, Russell et al. 2003, Kempfert *et al.* 2004), while identifying the problem of lateral loadings at the outer extremities of the embankment, do not suggest appropriate methods for quantifying either the magnitude of horizontal equilibrium or strain compatibility under the embankment.

BS 8006 (1995) assumes that the lateral thrust is carried by geosynthetic reinforcement placed at the base of the embankment directly over the pile caps. The BS 8006 (1995) limit equilibrium approach requires that strain compatibility be between the deformations of the geosynthetic, movement of the embankment fill and deflection of the pile cap. This study examines the suitability of the BS 8006 design methodology, through a numerical analysis approach. The effects of certain pertinent factors: the pile spacing, stiffness of the geosynthetic reinforcement, height of the embankment and the effect of the soft soil layer under the embankment were investigated. The numerical analysis was performed using the Finite Element code-Plaxis 2D Version 9.0.

3 PLAXIS MODEL

The Plaxis 2D Version 9.0 finite element analysis code was used to examine a series of piled embankment geometries. The developed model was validated against previously performed finite element analysis of piled embankments (Gangakhedkar, 2004, Han & Gabr, 2002 & Fahmi Farag, 2008).

This study focuses on the estimation and investigation of the lateral deformation in reinforced embankment over piles. Therefore, a series of analysis was performed with different embankment geometries. Typical pile spacing, s, range from 1.5m to 4.5m (Han & Gabr, 2002). In this study pile spacing of 1.5, 2.0, 2.5 & 3.0m were investigated. The pile modelled was a 0.3m square concrete end bearing pile with a concrete modulus of 30,000 MPa. The width of the pile cap, a, was varied from 0.75m to 1.0m with a constant thickness of 0.5m, to assess the effect pile cap size had on the load transfer in the embankment fill. The pile depth was kept constant at 10.0m. The piles were modelled as plastic plates with the axial and bending stiffness was modified to take account of the fact that the strength characteristics of the 1.0m plate wall had to equal the structural performance of the 0.3m square pile. The extent of the piles under the embankment was selected based on the recommendation in BS 8006 (1995)

The embankment height, H, chosen for the models was calculated based on H to (s-a) ratios (s = pile spacing, a = pile cap size) of 1, 2 and 4. This enabled a range of embankment heights ranging from 0.5m to 9.0m to be investigated. A constant surcharge loading of 10kN/m was applied along the top of the embankment.

The side slope of the embankment was constructed as 1V:2H and 1V:3H, and are typical of that used in practice. The variance in the side slope allowed an examination of the stability effect on the lateral deformation of the embankment structure itself.

The embankment fill was modelled using typical values for a granular base material which consists of the following; γ_{unsat} = 19.0 kg/m³, E = 20,000 kN/m², c = 0 kN/m², ϕ = 35°, ψ = 0.0° (Gangakhedkar, 2004). The interface friction parameter for the fill and the soft soil model was assumed constant at 0.85. The geosynthetic reinforcement had the following stiffness: 2000 kN/m and 500kN/m. The unreinforced case was also modelled. The geosynthetic was placed 100mm above the top of the pile cap, with the gap between the reinforcement and the pile cap / soft soil layer modelled as embankment fill. The soil models used in the analysis were the Soft Soil Model selected from the Plaxis code due to its Cam Clay like properties especially under primary loading of near normally consolidated clay type soils. The soils models themselves, Table 1, range from a very soft peat (E = 800kN/m²) to a normally consolidated clay (E = 3333kN/m²).

The Plaxis analysis used a plastic updated mesh analysis with staged construction and drained conditions as a simplifying assumption. Gangakhedkar (2004) suggested that it is unclear when exactly to use an ordinary plastic analysis or an updated mesh analysis. It is suggested to inspect the deformed mesh after conventional plastic analysis. Should excessively deformations occur, updated mesh analysis is more appropriate (Gangakhedkar, 2004).

Table 1. Soil model parameters used in this study

Model	SSM 1	SSM 2	SSM 3	SSM4
Description	Peat	-	-	Ncc Clay
γ_{sat} (kg/m³)	12	14	17	19.5
γ_{unsat} (kg/m³)	12	14	17	19.5
λ	0.12	0.0975	0.052	0.03
k	0.04	0.03	0.02	0.01
c (kN/m²)	5	4	2	1
φ ° (deg)	15	17.5	22.5	25
Ψ ° (deg)	0	0	0	0

4 DISCUSSION OF RESULTS

The results of the analysis are discussed in relation to the vertical and horizontal deformations, the tension recorded in the geosynthetic reinforcement and the bending moment generated in the piles.

4.1 *Deformation and tension in the geosynthetic reinforcement*

The horizontal deformation increased from a relatively small magnitude at the centreline to a maximum value near the crest of the embankment (Fig. 1 and 2). The magnitude of the horizontal deformation increased with increased embankment height (Fig. 1), and increased steepness of the side slope (Fig. 2). The peak horizontal deformation occurred at approximately the same location for a given height, irrespective of the steepness of the embankment side slope (Fig. 2).

The vertical deformation of the reinforcement followed a continual catenary shape between adjacent pile caps, from the centre line of the embankment to the outer row of piles for all geometries examined. The vertical deformation increased significantly from the outer row of piles to the toe of the embankment. This would indicate that the piles should be continued further outwards towards the toe of the embankment than currently recommended in BS 8006 (1995). The magnitude of vertical deformation also increased with increased embankment height (Fig. 3).

Between the centreline and near the crest of the embankment the axial tension in the reinforcement followed the same general pattern; a maximum values was observed at the edge of the pile cap, while near zero tension was recorded at mid span. However, from the crest to the toe the axial tension increased, approximately linearly, before dropping sharply to zero at the toe of the embankment, (Fig. 4, 6 and 7). The increase in axial tension was attributed to the reinforcement attracting load as the embankment deforms horizontally. The magnitude of the tension in the reinforcement under the side slope increased for increased embankment height and decreased for increased subsoil stiffness. For low height embankment the axial tension was not influenced by the steepness of the side slope (Fig. 5), however for high embankment the 1H:3V gave slightly low tension than the steeper 1H:2V side slope.

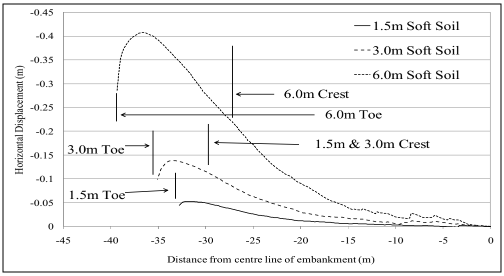

Figure 1. Horizontal displacement in reinforcement for a range of embankment heights

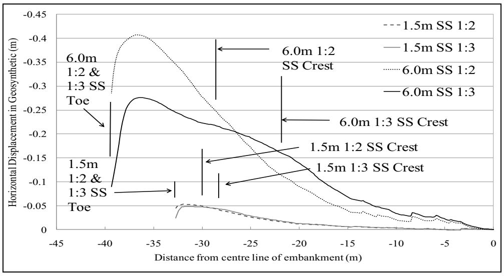

Figure 2. Horizontal displacement in reinforcement for a range of embankment heights and steepness of side slopes

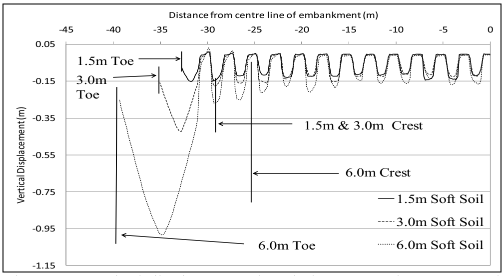

Figure 3. Vertical displacement in reinforcement for a range of embankment heights on soft soil

4.2 *Bending moment in pile*

The bending moment in the outer rows of piles, which was the maximum in the pile group (Fig. 8 and 9), increased as the strength of the reinforcement increased. The bending moment in the outer row of piles increased for a decrease in the stiffness of the soft soil, as the lateral resistance provided to the pile by the soil decreased. The ability of a pile to resist a lateral force is a combination of the capacity of the pile itself and the support provided by the soil surrounding it. For a 76% decrease in stiffness of the soft soil (going from soil model SSM 4→SSM 1, Table 1) the bending moment in the pile increased by 281% (Fig. 10).

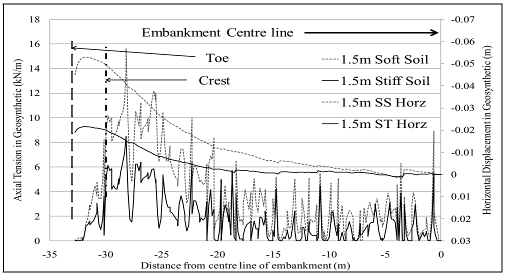

Figure 4. Tension and horizontal displacement developed in reinforcement for a range of soil stiffness

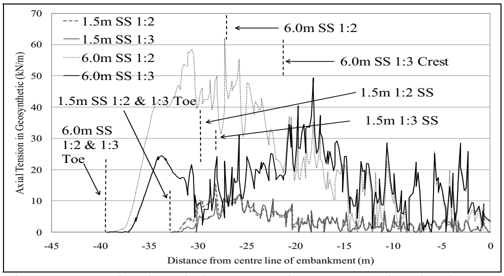

Figure 5. Tension in reinforcement for a range of embankment heights & side slopes

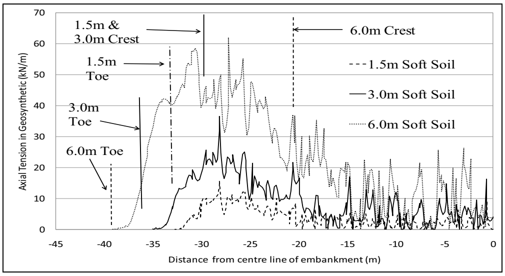

Figure 6. Axial tension in reinforcement for a range of embankment heights

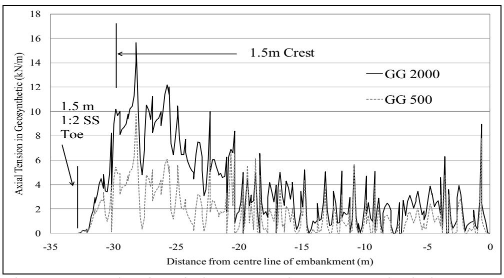

Figure 7. Tension in reinforcement for a range of reinforcement strengths

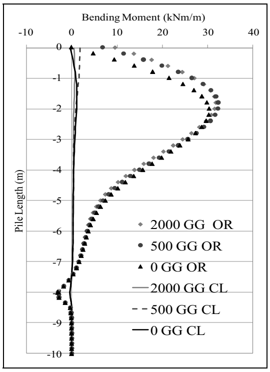

Figure 8. Bending moment of pile in outer row (OR) and centre line pile (CL) for a range of reinforcement strength on stiff soil

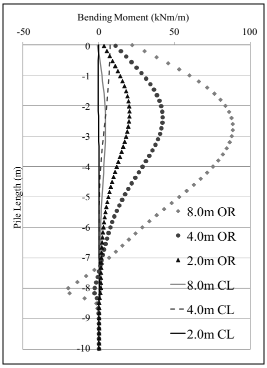

Figure 9. Bending moment of piles in outer row pile (OR) and centre line (CL) for a range of embankment heights

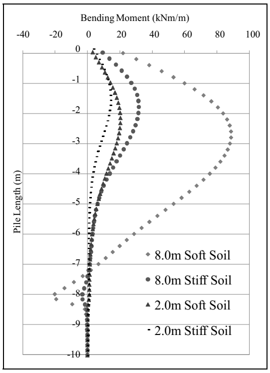

Figure 10. Bending moments in outer row pile for a range of embankment heights

The bending moment in the pile increased with an increase in embankment height (Fig. 9). As the embankment height increased from 2.0m to 4.0m the bending moment increased, almost proportionally, going from 19.47kNm/m to 41.43kNm/m (a 112 % increase).

The bending moment in the outer row of pile was also found to increase as the steepness of the embankment side slope increased and the spacing of the piles increased.

5 CONCLUSIONS

The analysis indicates that significant horizontal and vertical deformations occur in the reinforcement under the side slopes of piled embankments.

The axial tension in the reinforcement was seen to increase from near the crest of the embankment, approximately linearly, until dropping shapely to zero at the toe of the embankment.

Significant bending moments were observed in the outer row of piles.

The magnitude of horizontal and vertical deformations, axial tension in the reinforcement and bending moment in the outer row of piles was a function of the embankment height, pile spacing, steepness of the side slope and stiffness of the soft soil.

The analysis indicates that the pile group should extend for a greater distance towards the toe of the embankment than that currently recommended in BS 8006 (1995) to control vertical deformations. The tension in the reinforcement due to lateral thrust was concentrated under the side slope of the embankment.

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