Stability improvement methods for soft clays in a railway environment

Méthodes d'amélioration de la stabilité des argiles moles sous remblai de chemin de fer

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ABSTRACT: In October of 2009 a full-scale railway embankment failure experiment was conducted in Finland. The data gathered from the test established a good verification base for the soil models used in this study. In Finland, the most commonly used improvement method for railway embankments with low stability is a counter weight berm, which is designed based on the undrained shear strength of clay. Undrained shear strength is often underestimated and this inaccuracy is constantly leading to overdesigned counter weight berms, which can be tens of meters wide. This paper introduces an evaluation of alternative methods to improve embankment stability with wooden pile structures or with sheet pile walls. The study contains a comparison of different pile elements and an evaluation of piles capability for stability improvements. The evaluation is based on 2D and 3D finite element analysis and to the soil behavior calibrated in the failure test and existing, well investigated Finnish railway embankments with poor stability conditions.

RÉSUMÉ : Une expérimentation grandeur réelle d'une défaillance de remblai de chemin de fer été menée en Finlande en octobre 2009. Les données recueillies à partir de ce test ont fourni une base pour le modèle de géométrie et de simulation du comportement des sols exploité pour cette étude. En Finlande, la méthode d'amélioration la plus fréquente est une berme contre-poids conçue sur la base de la résistance au cisaillement de l'argile. La résistance au cisaillement non drainé est souvent sous-estimée et cette imprécision conduit à des bermes contre-poids surdimensionnées, qui peuvent avoir des dizaines de mètres de largeur. Cet article présente une évaluation de méthodes alternatives pour améliorer la stabilité du remblai à l'aide de pieux en bois ou de murs de palplanches. L'étude présente une comparaison des différents éléments de piliers et une évaluation de la capacité des piliers à améliorer la stabilité. L'analyse s'effectue par éléments finis 2D et 3D, avec un comportement du sol calibré dans le test de défaillance et l'existence bien documentée de remblais finlandais dans des conditions médiocres de stabilité.

KEYWORDS: FEM, 3D analysis, soft clay, embankment, wooden piles, sheet pile wall, stability improvement, railway.

1 INTRODUCTION

Stability of railway embankments on soft clays is commonly calculated with limit equilibrium method (LEM) using undrained strength parameters. In Finland the undrained strength is defined with the Field Vane Test. However, calculations with undrained strength might in some cases underestimate the factor of safety. In some of the soft soil areas the calculated total factor of safety is less than F=1.0 for existing embankments. On the other hand, LEM calculations with effective strength parameters φ' and c' tend to overestimate the safety factor for undrained conditions, when the excess pore pressure is not accurately taken into account.

A major problem in effective stress analyses is the assumptions for stress and pore pressure distribution and the difficulty in accounting for yield induced pore pressure. These can be taken into account with finite element method (FEM), if the analyses are conducted with advanced material models and correctly defined parameters (Mansikkamäki et. al., 2011).

To clarify the real stability conditions of Finnish railway embankments, a full scale failure test was conducted in October of 2009 on a soft marine clay deposit in southern part of Finland. Embankment was loaded to failure in 2 days as shown in figure 1. The goals for the test were to gather data for the purpose of improving stability calculation methods and testing the suitability of different instruments for monitoring embankment stability.

The extensive instrumentation is well documented in the work by Lehtonen (2011). Data considering displacements and excess pore pressure development has given good basis for the evaluation of FE analysis and the material models.

Figure 1 Test site after the failure. Instrumented area is between the containers and the ditch. Loading structure has overturned and the slip surface is protruding from the ditch.

So far FEM stability analyses have been mostly done using plane strain 2D analyses. Recent development of FE programs and increase of the computational capacity have enabled an increasing use of 3D analysis (e.g. Nian et.al., 2012). A stopped freight train on embankment is relatively close to a plane strain stability problem, even though 3D modeling provides possibility to analyze effect of axles or concentrated bogie loads. What comes to stability improvement methods, modeling of three-dimensional structures, for example piles, can be much more precise with a 3D analysis compared to a plane strain approximation.

2 THREE-DIMENSIONAL ANALYSES

The earlier 2D stability analyses with Plaxis 2D 2010 contained evaluation of different material models for soft clays. It was shown that anisotropic S-CLAY1 based (Wheeler et.al. 2003) material models can well express most of the important features of soft clay, such as failure induced pore pressure. The Soft Soil model was also found to be suitable with adjusted soil parameters. It was also found that the counter weight berms can be significantly smaller if design is conducted with the effective strength parameters and a suitable material model compared to the traditional undrained analysis.

The scope of the 3D analyses was to compare them with the 2D analyses and to model stability improvement objects, which would be indefinite to model as plane strain. The 3D FEM analyses were conducted with the Plaxis 3D program (version 2010.2.0.7044). At the first phase the whole test site was modeled to compare 2D and 3D analyses. The geometry model is shown in figure 2.

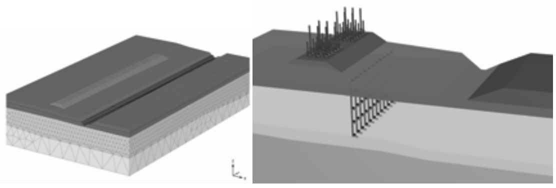

Figure 2. Full 3D geometry model containing 240 000 nodes and the 2 bogie section (12 m) with a pile row.

However, for the needs of modeling reinforcements, geometry was reduced to two different options. The larger model contained a section of two bogies (12 m) and the smaller geometry was only a 1.0 m thick section from the middle of the site. Larger model was used to evaluate different pile row installations and the smaller model to observe an influence of a single pile in more detail. Observations from the latter analyses are shown in this study.

The Plaxis Soft Soil model was used for the soft clay, while the Hardening Soil model was used for the coarse layers. Parameters and soil behavior is calibrated with the displacement and pore pressure data gathered from the conducted failure test. The basic parameters of each soil layer are shown in table 1.

Table 1. Basic material parameters of the soil layers. Corresponding layers are shown in figure 3.

	γ [kN/m^3]	E_{50} [MPa]	λ^* -	φ' [°]	c' [kPa]	POP [kPa]
1 Ballast	20	50		38	0.2	
2 Sandy fill	19	15		35	0.2	
3 Dry crust	16	12		0	30	
4 Clay	15		0.166	25	0.2	13
5 Clayey silt	17		0.08	27	0.2	20

3 MODELING WOODEN PILES

Wooden piles can be a cost-efficient method to improve stability in a railway environment. There is also a lot of research data available about the laterally loaded piles (Cai and Ugai 2000, Thompson et. al. 2005).

There are several options available to model laterally loaded piles in a FEM program. The most convenient way is to use Embedded pile elements, which are special beam (line) elements creating a elastic region around them imitating real structural element with a volume. The elastic region around the pile is equal to the pile diameter. The element does not create new geometry points to the model and therefore the analysis can be conducted with coarser mesh compared to the volume pile. Embedded pile elements cannot take into account a soil-pile interaction. There is no interface between pile and soil and therefore pile always moves with soil without sliding (Plaxis 3D 2010, Dao 2011).

Other options to model piles in a 3D program are a volume pile and a plate element. In practice, the volume pile is a solid soil element, which material model is linear elastic and diameter equal with the pile diameter. To be able to inspect forces affecting the pile, a beam element with very low elastic modulus was inserted to the center of the pile. A plate element is also applicable when the lateral forces are studied. In that case width and stiffness of the plate should be equal to the wooden pile. One should notice that the skin surface area of the plate is not equal to a cylinder shaped pile, which should be accounted in interface strength between soil and pile.

In this case the strength of the soil was fully accounted for the pile skin, even though with a volume pile and a plate element it is possible to use reduced interface strength. The geometry model was a 1 m thick cross section, where one vertical d200 mm wooden pile was inserted 5 m from the center line of the track, equal to 2 m from the embankment toe. The pile was installed through the clayey silt layer to the surface of the sand layer, where the approximated tip resistance of the pile head would be 24 kN.

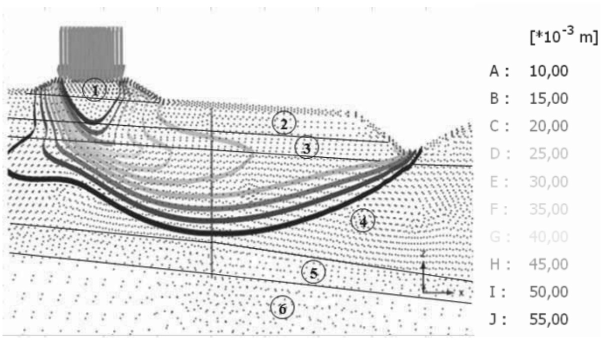

Figure 3. Vertical pile and displacements in 1 m thick cross section. Location of the pile is 5.0 m from the center line. Displacement contours are in 5 mm steps from 10 to 55 mm. Soil layers are sketched and numbered.

Train load was set to 70.0 kN/m^3. With this load the overall safety factor of the embankment is F=1.23 without a pile and maximum displacement of the embankment is 60 mm, as shown in figure 3. Number of nodes was 19700 in the original geometry without a pile. Volume of the elements was 0.02…0.03 m^3 which is very dense mesh for 3D analysis. The embedded pile was modeled using 2 different meshing options. First calculation conducted with the original mesh and then with a refined mesh, where a 200 mm diameter tube was created around the embedded pile. The tube had equal properties with the surrounding soil but it induced a mesh refinement around the embedded pile similar with the mesh, which was automatically created around the volume pile. Otherwise the meshing options were similar for soil layers in the parallel analysis.

In figure 4 a lateral displacement of different pile types from parallel analysis at the end of the loading is shown. From left to right the piles are embedded pile, embedded pile with refined mesh, volume pile and plate element. Maximum displacement was very similar at every case; 29, 31, 32 and 33 mm respectively. Maximum value was slightly smaller for the original embedded pile which could be due to coarser element mesh. On the other hand it also indicates slightly smaller bending moments.

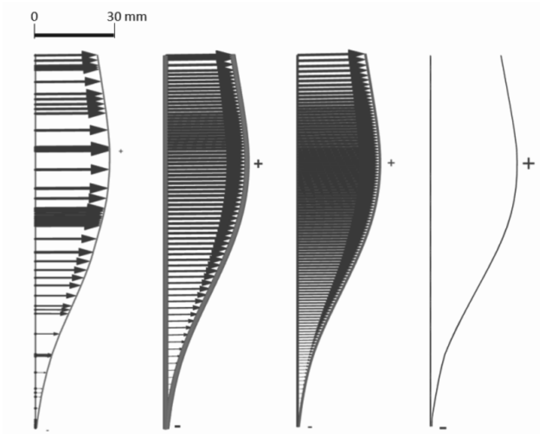

Figure 4. Lateral displacement of a wooden pile. Modeled with embedded piles, volume pile and plate element.

As the displacements and structural stiffnesses of the piles are equal, the bending moments should also be similar. However, notable difference could be found in bending moments as shown in figure 5. The moment distribution of the embedded pile is very irregular, indicating inexact values. The embedded pile with refined mesh whereas produced practically identical bending moments with the volume pile.

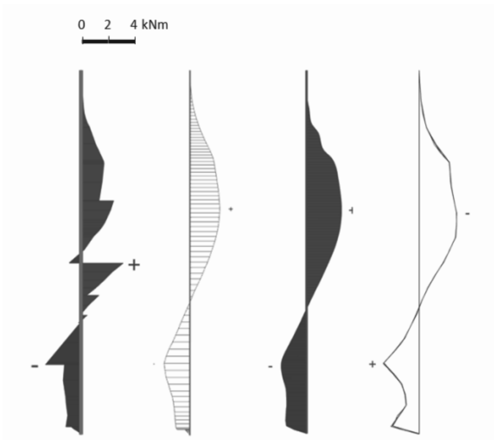

Figure 5. Bending moment of laterally loaded wooden pile. Modeled with embedded pile, embedded pile with refined mesh, volume pile and plate element.

The outcome of the analysis is that the element mesh should be refined around the embedded pile, if accurate structural forces are important to find out. Inaccuracy of the embedded pile element will probably be emphasized in actual design projects, where coarser element mesh is used. Other outcome was that the different pile elements produced very similar displacements and bending moments, if the element mesh around the piles was similar.

In figure 6 the safety analysis conducted for the different pile element types is shown. Initial settlement of 60 mm is caused by 70.0 kN/m^3 train load. One should notice that none of the pile elements have a failure criterion as they are purely elastic. Therefore the safety analysis is not reliable for large displacements as the bending moment of the pile increases beyond the structural capacity of the pile. As the bending moment capacity of a d200mm wooden pile is known to be approximately 15 kNm (Ranta-Maunus 2000), it was further analyzed at which displacement level structural failure may occur. Accordingly the bending moment capacity is reached when the settlement of the embankment is approximately 0.15 m.

It is shown in figure 6 that the safety factor without reinforcements is F=1.23. The volume pile and the embedded pile with refined mesh produces similar safety factors, F=1.29 and F=1.28 respectively for the displacement level of 0.15 m. Safety factor with the plate element is slightly smaller, equal to F=1.26. The embedded pile with the original mesh gives higher safety factor than the other. The factor was found to be F=1.33 indicating that the element can overestimate the stability conditions if the analysis is made without mesh refinement around the pile.

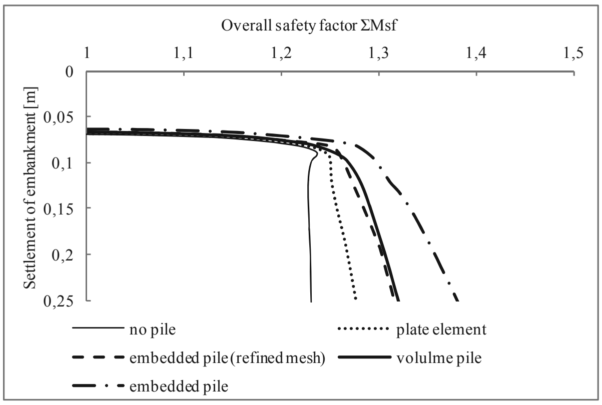

Figure 6. Safety analysis of different piles as a function of embankment settlement.

In general it can be said that the different structural elements produced similar results under operational loading conditions. Embedded pile was influenced by the coarser mesh even thought the magnitudes of forces were correct as an average. In all cases the mobilized forces are clearly smaller than the structural capacity of the piles. The value of maximum mobilized bending moment was 3.78 kNm, when corresponding lateral displacement was 33.4 mm.

A reason for this kind of behavior is a failure mechanism, where the piles are tilting with the soil mass. The foot of the pile has a hinged joint with soil, which causes smaller forces compared to a rigid connection that would be plausible if piles are driven deeper into the dense soil layers.

The installation effects or the effect of interface elements were not taken into account in this study. Obviously these effects should be considered if the piles are used near the railway track. One should also notice that even if the soil behavior is well known due to failure test, the study considering piles is theoretical as no piles were installed for the conducted failure test.

4 SHEET PILE WALLS

Permanent sheet pile walls are used occasionally for the stability improvements. The reason for using this method is usually the lack of space around the embankment and therefore a counter weight berm is not possible.

In the following, a case study from western Finland near Seinäjoki town is presented. A double track was supported with sheet pile walls anchored through the embankment as shown in figure 7. Sheet piles are installed through the soft clay layer (+27…+38) to the hard soil layer. There are no triaxial test results available from this site and therefore the FEM analysis are conducted using typical effective strength parameters of soft Finnish coastal clays. Thus the real stability conditions of this specific site can differ from the factors presented here.

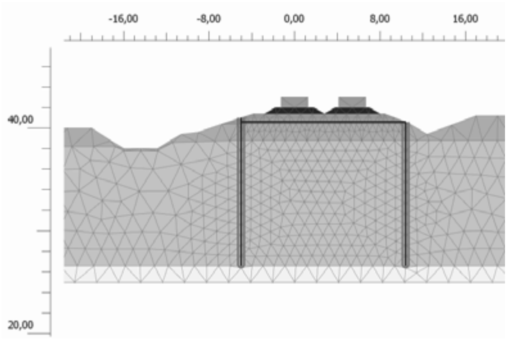

Figure 7. Embankment supported with the sheet pile walls.

Overall safety factor of the cross section is traditionally calculated in LEM so that the slip surface goes under the foot of the sheet pile wall. Often the adequate safety level is not reached until the wall is extended deep to the hard soil layers.

In figure 8 the results from the FEM stability analysis with the strength reduction method is shown. The initial overall safety factor is F=1.15. With the sheet pile wall, stability is improved so that the safety factor is F=1.76. However, the failure surface is not passing under the wall but through the wall. In this case the wall is modeled as an elasto-plastic plate element which bending moment capacity is 426 kNm, which corresponds a section modulus of w=1200cm^3. In this case the failure mechanism includes a structural failure of the sheet pile wall. It was further observed that also the tensile stress of the anchors was very close to failure at this safety level.

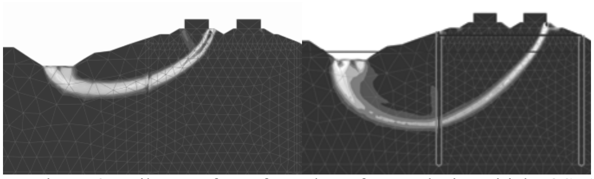

Figure 8. Failure surfaces from the safety analysis. Initial FOS=1.15 without the reinforcement and FOS=1.76 with the sheet pile wall.

In the present design codes the design values of maximum bending moment and anchor force is defined by applying partial safety factors for the permanent and variable loads. Factor is lower for permanent, and higher for variable load. In this case the characteristic train load was 40.4 kPa and design load 50.9 kPa. This design load was used to calculate the bending moment M_k and anchor force F_k. The design values for bending moment and anchor force are calculated as follows; M_d=1.15M_k=114.3 kNm and F_d=1.15F_k =96.3 kN/m.

Next, a parallel analysis was conducted, as it can be argued that the loads are quite well known compared to the strength parameters of the soil. The strength parameters of the soil layers were reduced using a partial factor of γ_φ=1.20. Calculation was conducted with the characteristic train load 40.4 kPa. In this analysis maximum bending moment was M=157.5 kNm and the anchor load T=105.0 kN/m.

Hence, a relatively small decrease in soil strength caused higher bending moment and anchor force with the characteristic loads than the design values are. The overall safety margin for the bending moment by the means of soil strength was F<1.20. When the stability of the embankment is poor, a small change in soil strength parameters builds up a significant amount of excess pore pressure, which significantly increases the stress in the supporting structure.

Sensitivity analysis with FE shows that the structural forces are in this case sensitive for soil strength variation. This kind of sensitivity analysis would also be beneficial in practical design cases to ensure a sufficient safety margin.

5 CONCLUSION

3D FE analysis can provide useful and valuable information in geotechnical projects even though robustness and mesh independency are not yet at the same level than in the 2D programs.

The embedded pile element seems to give imprecise results when it is used with standard element mesh. Performance is clearly improved when the mesh is refined around the pile element. In that case the results are similar with the volume pile. This feature slightly reduces calculation performance and handiness of the element.

Wooden piles can be used to improve embankment stability if the demanded supporting forces are reasonable. Still, several piles per track meter should be used, as the mobilized lateral forces are quite small.

If sheet pile walls are used to improve embankment stability, FEA can provide valuable additional information on how sensitive the structural forces are for the soil strength variation and what is the real nature of the failure. FEA was found to be a useful tool for these evaluations as the structural behavior is also accounted for the analysis. It was shown that the bending moment and the anchor force can be so sensitive for soil strength variation that the safety margin can be lower than expected.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Cai F. & Ugai K. 2000. Shear Strength Reduction FEM Evaluating Stability of Slopes with Piles or Anchors. Proceedings of an International Conference on Geotechnical & Geological Engineering Melbourne

Dao T.P.T. (2011).Validation of PLAXIS Embedded Piles For Lateral Loading. Delft University of Technology, Plaxis bv.

Dawson, E.M., Roth, W.H. and Drescher, A. 1999. Slope stability analysis by strength reduction, Geotechnique, vol. 49, no. 6, pp. 835-840.

Farias M.M., Naylor D.J. 1998. Safety analysis using finite elements. Computers and Geotechnics, Vol.22, No. 2, pp.165-181.

Lehtonen, V (2011). Instrumentation and analysis of a railway embankment failure experiment. 29/2011 Research report of the Finnish Transport Agency.

Mansikkamäki J., Länsivaara T. 2012. 3D Stability Analysis of a Full-scale Embankment Failure Experiment. Conference proceedings, NGM2012–Nordic Geotechnical Meeting. Copenhagen, Denmark.

Mansikkamäki, J.; Lehtonen, V; Länsivaara, T. (2011). Advanced stability analysis of a failure test on an old railway embankment. Conference proceedings, GeoRail 2011, Paris.

Nian T.-K., Huang R.-Q., Wan S.-S. and Chen G.-Q. May 2012. Three-dimensional strength-reduction finite element analysis of slopes: geometric effects. Canadian Geotechnical Journal. Volume 49, Number 5. NRC Research Press Journal.

Plaxis 3D. (2010). Reference and Material Models Manual.

Ranta-Maunus A. 2000.Bending and compression properties of small diameter round timber. Proceedings of World Conference on Timber Engineering. Whistler Resort, British Columbia, Canada July 31 - August 3.

Thompson M., White D. J. and Schaefer V. R. Dec. 2005. Innovative Solutions for Slope Stability Reinforcement and Characterization: Vol. III. Final Report. Iowa State University, Center for Transportation Research and Education.

Wheeler S.j., Näätänen A., Karstunen M., Lojander M. An anisotropic elastoplastic model for soft clays. Canadian Geotechnical Journal 40: 403–418. 2003.