Numerical Modeling of the 3D Behaviour and 2D Equivalence of Ground Improvements during Lateral Loading

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ABSTRACT
Providing ground improvements to loose alluvial foundation soils can be used to prevent loss of perimeter containment structures due to liquefaction resulting from earthquake loading. Ground improvements can be provided by treating individual columns of soil or by constructing slurry walls to form soilcrete panels. Common layouts include panels arranged into either square or rectangular sections forming a cellular structure within the ground improved area. When an earthen berm containing waste material is constructed above the improved foundation soils, lateral forces are transferred to the ground improved area from both static and dynamic loading. These lateral loads are assumed to be carried entirely within the soilcrete panels when the foundation soils liquefy. Two-dimensional numerical modelling is commonly used to predict the stresses and displacements within the soilcrete panels during the dynamic loading from an earthquake event. Using weighted averages based on the area replacement ratio, the composite strengths of the soil mass are estimated for complex three-dimensional soilcrete layouts. In this study it is shown that the three-dimensional layout of the ground improvements makes it difficult to determine the appropriate stiffness parameters for a two-dimensional model. However, three-dimensional numerical modelling to analyze the problem is much more computationally intense and can result in extremely long run times, especially when simulating earthquake events. In an attempt to reduce computing times, a method to model the equivalent three-dimensional behaviour of the ground improvements during lateral loading using appropriate composite stiffness parameters in a two-dimensional numerical model was developed.

Introduction

In December 2008, an impoundment failed at Tennessee Valley Authority's Kingston power plant, releasing roughly 5.4 million cubic yards of sluiced, saturated coal ash. About 3.5 million cubic yards of this material was dredged from the adjacent Watts Bar Lake (Emory River) and hauled by rail to an off-site landfill. The remaining ash is being recovered and stacked back inside the site boundaries. For the closed facility, which will be comprised of compacted ash with an engineered cover, seismic containment is a critical design objective. Like similar facilities at many other power plants, the ash storage at Kingston is sited on the alluvial sediments of a major river. The older deposits of sluiced ash in the closed landfill will remain saturated and, together with the deeper alluvial sands, are expected to liquefy in the design earthquake.

A key feature of the closure design is a stabilized perimeter (Figure 1), with a grid of soilcrete walls that extend into bedrock around the two-mile site circumference. The perimeter walls will buttress the ash embankment, acting like a dam to retain the liquefied ash under dynamic loading. Similar designs have been used to stabilize major earth embankment dams with liquefiable foundation deposits. Notable examples in the USA include Jackson Lake Dam (Ryan and Jasperse 1989), Clemson Dam (Wooten et al. 2003; Wooten and Foreman 2005), Sunset North Basin Dam (Barron et al. 2006), and San Pablo Dam (Kirby et al. 2010). Deep soil mixing was used to stabilize the foundation soils in each of those projects, as well as for levees in New Orleans (Cooling et al. 2012; Filz et al. 2012).

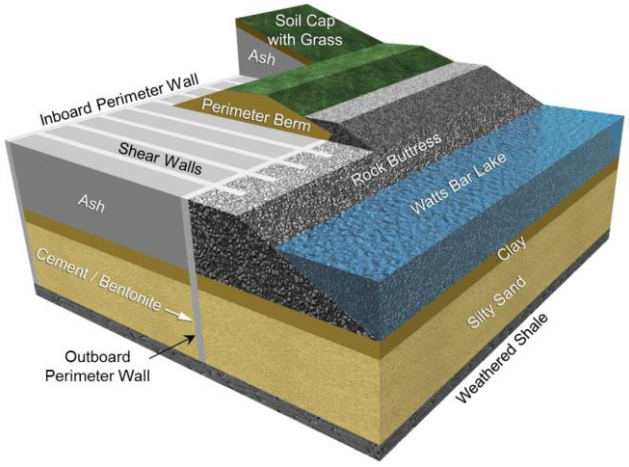

Figure 1: Rendering of the Stabilized Facility Perimeter.

Background

Ground improvement techniques are often used to stabilize poor foundation conditions. Ground improvements can be provided by treating individual

columns of soil or by constructing slurry walls to form soilcrete panels. Common layouts for soilcrete panels include square or rectangular patterns forming a cellular structure within the ground improved area.

In order to predict stresses and displacements during the dynamic loading from an earthquake event, numerical modelling is commonly used. Utilizing weighted averages calculated from the area replacement ratio for both the strength and stiffness parameters, two-dimensional numerical modelling can be used to predict vertical displacements from loadings such as embankments. However, the three-dimensional nature of the problem needs to be considered when evaluating the response to lateral loading. This is especially important when the existing foundation soils may liquefy during earthquake loading causing the lateral loads to be carried entirely by the soilcrete panels through load transfer between panels.

The disadvantage of three-dimensional numerical modelling is the size and complexity of the problem creating extremely long computational run times, especially when simulating earthquake events. Long simulation run times make it time consuming to conduct parametric studies. The two-dimensional numerical model is more efficient at modelling seismic events but the stiffness parameters for the soils within the stabilized foundation area need to represent the three-dimensional nature of the problem. Therefore, a method to model the equivalent three-dimensional behaviour of the ground improvements during lateral loading using appropriate composite stiffness parameters in a two-dimensional numerical model is required. As can be expected, the composite stiffness of the stabilized foundation zone will vary with the stiffness, the thickness and spacing, as well as the overall layout of the soilcrete panels.

When ground improvements are used to remediate an existing dam, construct an earthen berm on soft soils, or in other applications where lateral loads are induced, some level of numerical modelling is typically required to verify the proposed design. As state of practice for evaluating deep soil mixing moves from limit equilibrium to advanced numerical modelling, it becomes apparent that the complex behavior of three-dimensional layouts must be accounted for. Two-dimensional numerical modelling of deep soil mixing applications has used various approaches to evaluate three-dimensional geometries in two-dimensions. Filz and Navin (2005) indicates that the current state of practice is to use composite soil parameters based on the in situ soils, ground improvements, and the area replacement ratio. The San Pablo Dam deep soil mixing upgrade was evaluated using a two-dimensional model with a composite area with strength parameters obtained by ignoring the contribution of the soils and only using the strength and stiffness of the deep soil mixing elements to calculate composite parameters (Kirby *et al.*, 2008). Seismic remediation of the Clemson Dams was done by using a similar approach (Wooten and Foreman, 2005). These methods have been verified using centrifuge modelling (Filz and Navin, 2005). However, the correlation of area replacement ratio to composite stiffness with respect to lateral movements needs further verification.

Numerical Modelling Details

An earthen berm constructed on soil improvements to retain waste material was chosen as an example problem. The generic layout selected for the deep soil mixing panels was chosen as a simple rectangle spanning a 30 m width for the ground improved area. The thickness of the soilcrete panels was arbitrarily set at 1 m. The upstream and downstream perimeter soilcrete walls were set as continuous along the length of the containment berm and soilcrete shear walls, spaced 6.1 m apart (measured centre to centre), were added to connect the perimeter walls. Figure 2 shows the chosen layout of the ground improvements in plan view. Based on the layout and spacing of the soilcrete panels selected, the area replacement ratio for the soilcrete within the ground improved area is 0.25. The depth of the ground improvements in the foundation soils was assumed to be 12 m and the height of the perimeter containment berm was chosen to be 4 m with 3H:1V side slopes. The waste material being contained within the perimeter berm was assumed to be 8 m in height with a 4H:1V slope starting at the inside crest of the perimeter berm. The water table was assumed to be 1 m below the top of the foundation soils. Figure 3 illustrates the chosen geometry in section.

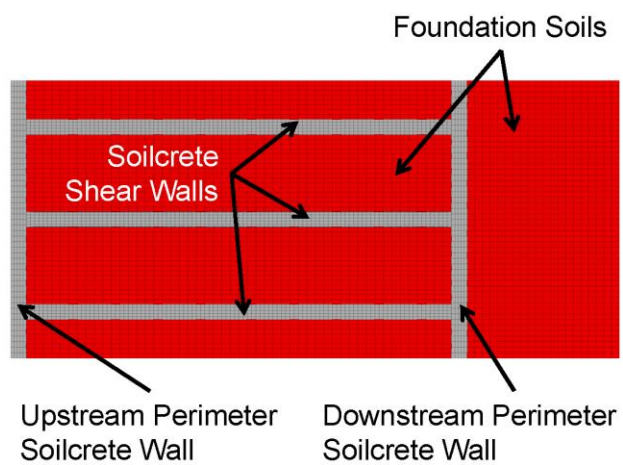

Figure 2: Plan view of details for ground improvements at foundation grade

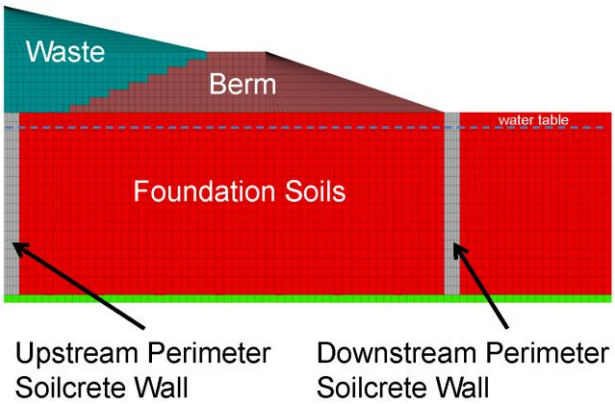

Figure 3: Cross section through foundation, berm, and waste material

Three-dimensional numerical modelling using the finite difference program FLAC3D (Itasca 2009) was used to evaluate the response within the ground improved area to lateral loading from an earthquake. Two three-dimensional models were constructed to determine the equivalent two-dimensional composite stiffness parameters to be used within the area of the ground improvements. The first model (baseline) represented the actual three-dimensional layout within the ground improved area. The second model (composite) contained only the equivalent improved foundation soils and the upstream and downstream perimeter soilcrete (Figure 4).

The soilcrete shear walls were replaced with composite strength and stiffness parameters. Walls were maintained in the three-dimensional composite model since they are continuous and appear in any two-dimensional section. Therefore, the three-dimensional equivalency analysis was based only on the composite behavior of the soilcrete shear walls and in-situ foundation soils.

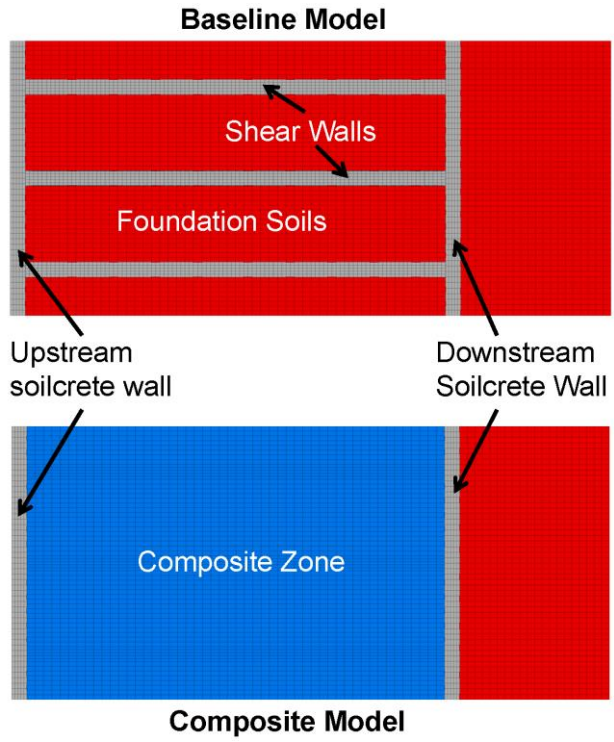

Figure 4: Plan view of the layout of the ground improvements versus the layout of the composite layout.

The total number of zones per model was kept below 100,000 to maintain reasonable simulation times without compromising the accuracy of the overall model behavior. In spite of these measures, run times per simulation were approximately 4 hours in duration (using a Core 2 Duo T7700 / 2.4GHz Centrino Pro processor).

Horizontal displacements at the upstream perimeter soilcrete wall were tracked for both the baseline model and the corresponding composite model. Displacements were measured at both the midpoint of the soilcrete shear wall and where the in situ soils were present. The average horizontal displacement with height was calculated to represent the overall movement

Soil Parameters

The soil parameters used for the numerical modelling analysis are showing in Table 1.

Table 1. Soil Parameters for Numerical Modelling

Soil Type	Density (kg/m^3)	Bulk Modulus (Pa)	Shear Modulus (Pa)	Friction Angle (degrees)	Cohesion (kPa)
Perimeter Berm	1900	3.44E7	1.54E7	35	25
Waste Material	1600	3.44E7	1.54E7	28	0
Sand	1450	3.44E6	1.54E6	8	0
Soilcrete	1450	3.44E8	1.54E8	0	1000

The cohesion of the soilcrete was set to 50% of the assumed unconfined compressive strength of 2 MPa. The tensile strength of soilcrete was assumed to be zero for this analysis and the strength for the sand was assumed to be low, representing post liquefaction conditions after an earthquake.

The composite three-dimensional model used composite strength parameters within the ground improved area, which were determined by using the weighted averages of the soil and soilcrete parameters as outlined in the following equations:

$$c_{composite} = (1 - \alpha)\, c_{soil} + 0.5\, \alpha\, q_{u\text{-}sc}$$

$$\Phi_{composite} = \tan^{-1}[(1 - \alpha) \tan\Phi_{soil}]$$

Where c is cohesion and Φ is the friction angle for the in situ soil and composite zones. The area replacement ratio is represented by α and $q_{u\text{-}sc}$ is the unconfined compressive strength of the soilcrete.

Earthquake Loadings and Pressure Boundary

Seismic loads were simulated in the three-dimensional model by applying the horizontal pressure recorded from the two-dimensional dynamic model. The horizontal pressure boundary was determined using a two-dimensional model created in FLAC (Itasca 2008) based on the three-dimensional model being evaluated. Estimates were made for the composite stiffness parameters in the two-dimensional model for the initial trial simulation used to obtain the horizontal pressure used in the three-dimensional model. Based on the results, the 2D equivalency analysis was completed using the three-dimensional models and if the final composite stiffness parameters determined differed significantly from the values initially assumed, then the entire process was repeated to determine an updated pressure boundary.

The earthquake event used for this analysis was a magnitude 6.0 with a peak ground acceleration of 0.11 g and duration of 12 seconds. Based on the two-dimensional seismic analysis, a maximum pressure boundary was determined by monitoring the horizontal pressures on the upstream perimeter soilcrete wall during the earthquake.

Three horizontal pressure boundaries were evaluated to determine the sensitivity of the results to different applied horizontal pressures. The initial horizontal pressure distribution evaluated was based on the results from the two-dimensional seismic FLAC analysis. The horizontal pressure boundary was converted to a linear non-uniform distribution with 325 kPa applied at the base of the ground improvements and 275 kPa applied at the top of the ground improved area (Figure 5). The other two pressure boundaries were simply set 10% lower and 10% higher.

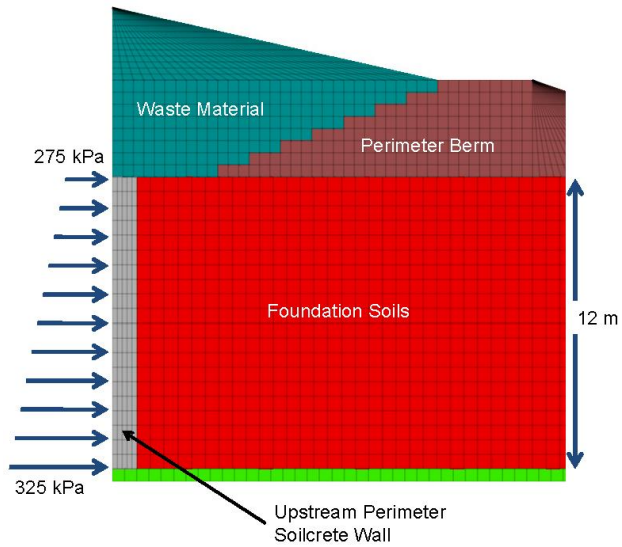

Figure 5: Horizontal pressure boundary applied to the upstream perimeter soilcrete wall (+/- 10%).

Determining Equivalent 2D Stiffness

To determine the equivalent two-dimensional stiffness parameters within the composite model that would match the horizontal displacements of the inboard perimeter soilcrete wall within the baseline three-dimensional model, the bulk and shear modulus values were adjusted, maintaining a constant value for Poisson's ratio. A single uniform composite stiffness for the full thickness of the foundation soils was used. Since the upstream and downstream perimeter soilcrete walls are continuous and exist for any two-dimensional section, they were modeled with the properties of soilcrete.

Using the horizontal pressure boundary determined from the preliminary two-dimensional seismic FLAC analysis, several trials were run using different composite stiffness ratios. The results were then compared to the horizontal displacement at the upstream perimeter soilcrete wall within the baseline model. Composite stiffness ratios equal to 0.1 and 0.15 times that of soilcrete were found to bracket the results from the baseline model. Subsequent trials found that a composite stiffness ratio equal to 0.14 times that of soilcrete yielded the best fit to the results observed with the baseline model. Figure 6 presents the results from these first trials (the solid line corresponds to the baseline model results).

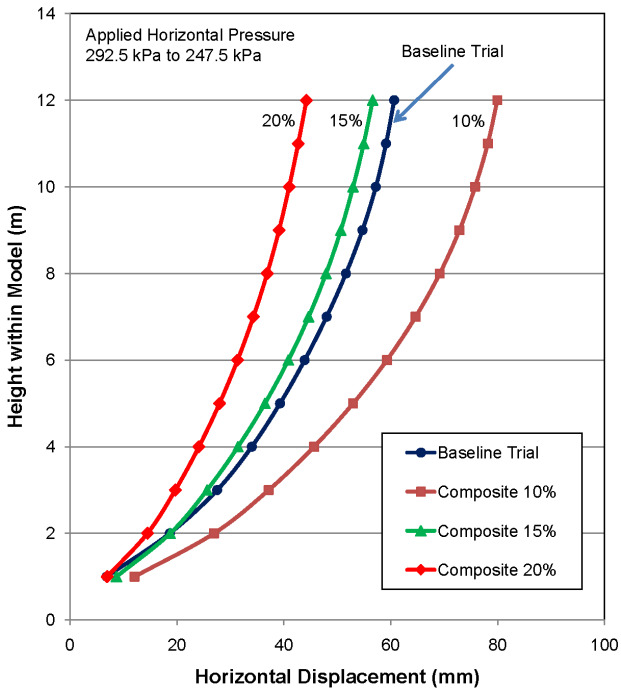

Figure 6: Horizontal displacements at upstream perimeter soilcrete wall with different composite stiffness values.

The FLAC3D simulations were then repeated for the other two horizontal pressure boundaries at +/- 10% of the pressure in Figure 5. Figure 7 shows the best fit results for all three cases. The composite stiffness ratios found to give the best fit with the baseline model displacements ranged between 0.12 and 0.14 times that of soilcrete for the pressure boundaries modeled. Based on the small range in values, an average composite ratio value of 0.13 times that of soilcrete could be used for the final two-dimensional seismic FLAC analysis if the horizontal pressures fall within the limits of the three-dimensional comparison. For comparison, if the composite stiffness parameters would have been simply calculated based on the weighted average of the in situ soils and the soilcrete shear walls, then the composite ratio values would have been 0.16 times that of soilcrete.

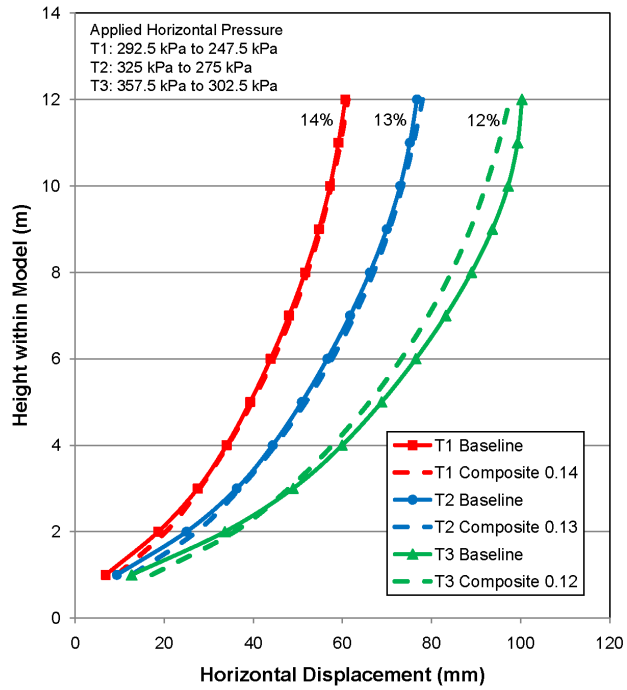

Figure 7: Composite stiffness values to match horizontal displacement at upstream perimeter soilcrete wall for the different applied pressure boundaries.

Summary and Conclusions

The three-dimensional layout of ground improvements makes it difficult to determine the appropriate stiffness parameters for an equivalent two-dimensional numerical model. The example problem presented herein shows that the equivalent composite stiffness parameters cannot be simply estimated using the weighted average based on the area replacement ratio, but must be based on matching the deflection to loading conditions.

The following method has been proposed to determine the equivalent two-dimensional stiffness parameters from a three-dimensional layout:

- Assume initial composite stiffness parameters within the area of the ground improvements;
- Calculate the horizontal pressure boundary on the upstream limits of the ground improved area using seismic two-dimensional modelling;
- Using the horizontal pressure from the seismic two-dimensional model, determine the composite stiffness parameters that match the response of the baseline model in three-dimensions;
- If the composite stiffness parameters do not match the assumptions used within the seismic two-dimensional model, then repeat the procedure until convergence is achieved; and
- Use the composite stiffness ratio determined from the static three-dimensional model in the seismic two-dimensional modelling.

The results presented in this paper cannot be generalized. However, the methodology presented here can be repeated for different problems. Different layouts, parameters, and loading conditions will result in different composite stiffness parameters and therefore a similar study must be completed for each project.

References

Barron, R. F., Kramer, C., Herlache, W. A., Wright, J., Fung, H., and Liu, C. (2006). "Cement Deep Soil Mixing Remediation of Sunset North Basin Dam." Proc. ASDSO Dam Safety 2006, Boston, Massachusetts, September.

Cooling, T., Boeckmann, A., Filz, G., Cali, P., Evans, J., and Leoni, F. (2012). "Deep Mixing Design for Raising Levee Section, LPV 111 New Orleans, LA." Proc., 4th International Conf. on Grouting and Deep Mixing, New Orleans, Louisiana, February.

Filz, G. M., Navin, M. P., 2005. "Final Contract Report: Stability of Column Supported Embankments". Department of Civil and Environmental Engineering Virginia Polytechnic Institute & State University.

Filz, G., Adams, T., Navin, M., and Templeton, A. E. (2012). "Design of Deep Mixing for Support of Levees and Floodwalls." Proc., 4th International Conf. on Grouting and Deep Mixing, New Orleans, Louisiana, February.

Itasca, 2008. Fast Lagrangian Analysis of Continua, FLAC version 6.0. Geotechnical modelling software developed by Itasca Consulting Group Inc. Minneapolis, Minnesota.

Itasca, 2009. Fast Lagrangian Analysis of Continua in 3 Dimensions, FLAC3D version 4.0. Geotechnical modelling software developed by Itasca Consulting Group Inc. Minneapolis, Minnesota.

Kirby, R. K., Roussel, G. L., Barneich, J. A., Yiadom, A. B., Todaro, S. M., 2010. "Design and Construction of Seismic Upgrades at San Pablo Dam Using CDSM". United States Society on Dams 2010 Proceedings.

Ryan, C. R., and Jasperse, B. H. (1989). "Deep Soil Mixing at the Jackson Lake Dam." Proc. ASCE Foundation Engineering Conference, Evanston, Illinois, June, pp. 354-367.

Wooten, L., Castro, G., Gregory, G., and Foreman, B. (2003). "Evaluation and Design of Seismic Remediation for the Clemson Upper and Lower Diversion Dams." Proc., 23rd Annual Conf. USSD, Charleston, South Carolina, April.

Wooten, R. L., Foreman, B. 2005. "Deep Soil Mixing for Seismic Remediation of the Clemson Upper and Lower Diversion Dams". Presented at the 25th USSD Annual Meeting and Conference.