Analysis of a Full-Scale Field Test of Rockfill Columns

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ABSTRACT
In 2008 a natural clay riverbank was reinforced with rockfill columns and then tested under increasing surcharge loading. This paper discusses the use of laboratory measured material properties in the design and analysis of the full-scale field test of rockfill columns. In particular, a large scale direct shear testing apparatus was used to determine the properties of crushed limestone rockfill. This paper presents the methods used for incorporating the stress-strain relationships and other measured material properties into 2-D numerical models.

RÉSUMÉ
En 2008, une rive d'argile naturelle a été renforcée avec des colonnes d'enrochement et ensuite a été examinée sous une charge croissante. Ce document discute de l'utilisation de propriétés des matériaux mesurés en laboratoire dans la conception et l'analyse d'un test à grande échelle de colonnes d'enrochement. En particulier, un test de grande échelle d'appareil de cisaillement direct a été utilisé pour déterminer les propriétés d'un enrochement de calcaire broyé. Ce document présente les méthodes utilisées pour incorporer les relations contraintes-déformations et d'autres propriétés matérielles mesurées dans un modèle numérique de 2-D.

1 INTRODUCTION

Winnipeg is a prairie city that developed around the confluence of the Red and Assiniboine rivers as a trade and transportation hub. These rivers and their banks are still important to the city's character and beauty. With the growth of the city, and the development of the property along the rivers, the riverbank slopes have created some interesting geotechnical challenges.

The slope instabilities discussed in this paper primarily occur in glaciolacustrine clay. They are deep-seated and generally triggered by toe erosion, crest loading or excessive porewater pressures. After significant deformation, they develop slickensided surfaces at residual strength and are typically slow moving.

Rockfill columns were introduced as a stabilization alternative to granular shear keys and other more rigid retaining structures that had been used previously. Rockfill columns have been used to stabilize riverbanks in the Winnipeg area for close to three decades, generally with satisfactory performance. Rockfill columns are considered the technique of choice for many local slope stability projects because of their cost effectiveness. Notwithstanding their success, consultants, contractors and owners have identified some shortcomings in the understanding of rockfill column performance. Slopes stabilized with rockfill columns have undergone relatively large deformations in the construction and post construction stages as identified by Yarechewski and Tallin (2003). This research project seeks to identify the causes and mechanisms of these slope deformations and provide recommendations for design and construction.

The University of Manitoba has completed a multi-phase project investigating the performance of rockfill columns. Research completed to date has included a materials testing program which considered clay, rockfill, rockfill-clay composite and rockfill-cement samples. Much of the strength testing was done with a large scale direct shear machine at the University of Manitoba

1.1 Full-Scale Field Test

The final phase of the research project included a full-scale field test of a natural riverbank, stabilized with rockfill columns along with the numerical modeling. The field test consisted of reinforcing the bank with eleven 2.1m diameter rockfill columns over a 19m reach of riverbank. The slope was then loaded with fill to induce displacements in the clay slope and rockfill columns. Figure 1 shows the site with the final load in place. The site was closely monitored with 11 inclinometers, including two strings of in-place inclinometers. The in-place inclinometers as well as six vibrating wire piezometers were monitored continuously. Thiessen et al. (2007 and 2008) discuss the field test, monitoring and some results. The cross section Figure 2 shows the loading and some of the installed instrumentation.

Numerical models were used in the design process to predict results and tailor the layout and test procedure. The models have now been calibrated with the monitoring results of the field test. The purpose of the numerical modeling effort is to help advance the current practice by providing modeling and analysis recommendations which will help practitioners understand, predict and mitigate the deformations associated with rockfill column stabilization works. Current design methods rely on limit equilibrium stability analysis without consideration for how resisting stresses are mobilized in the rockfill columns or surrounding clay.

Figure 1. Rockfill research site with final surcharge load.

1.2 Regional and Site Geology

Winnipeg rests in the basin of the former glacial Lake Agassiz and has a top down stratigraphic progression of an upper complex zone of organics, clay and silt followed by high plasticity glaciolacustrine clay, and a silt matrix till over top of limestone bedrock. The stratigraphy at the research site consisted of approximately 13m of clay overlying 2m of loose to compact till which became dense with depth.

This paper will focus primarily on the rockfill properties but because the columns interact intimately with the surrounding soil, the rockfill behaviour must be considered as part of a composite system of in-situ soils, and arrays of columns.

2 LABORATORY TESTING

The laboratory testing program for the in-situ clay and till, followed standard practice for typical slope stability and embankment construction projects, except with a larger scope enabled by less stringent labour, time and equipment constraints. All testing was done in general accordance with appropriate ASTM standards in the University of Manitoba's geotechnical laboratories.

The rockfill testing focussed on developing the constitutive relationships of the rockfill and identifying the interaction characteristics of composite rockfill-clay samples.

2.1 Clay Properties

The project's geotechnical site investigation was done in August 2006 and consisted of the drilling of 5 test holes for the purpose of sample collection and instrumentation installation. Clay samples were collected from auger cuttings, continuous samples and thin walled shelby tubes.

Testing on clay samples included CIŪ triaxial, direct shear, one-dimensional consolidation (oedometer), and

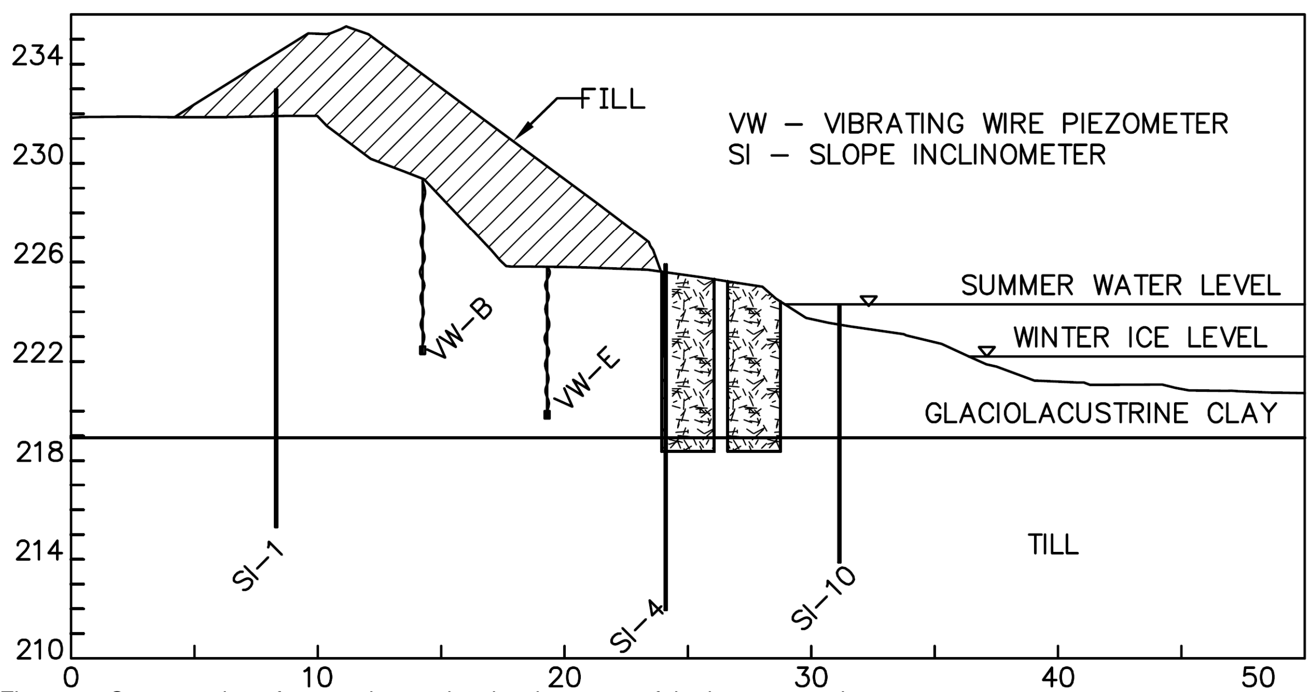

Figure 2. Cross section of research test site showing some of the instrumentation

flexible walled permeameter tests. Other properties measured included plasticity indices, grain size distributions, densities, specific gravities and moisture contents.

Peak and post-peak strengths plus modulus values for the clay were calculated from the consolidated undrained triaxial testing results. A total of 13 triaxial tests were done on samples from select depths over a range of confining pressures. The post-peak strengths from the triaxial testing are shown in Figure 3. A friction angle of 19˚ and cohesion of 5 kPa was used in the modeling. Initial and secant modulus values for clay were also calculated from the undrained triaxial results. Table 1 is a summary of some of the measured material properties used in the numerical modeling.

Oedometer tests provided κ, λ and the overconsolidation ratios required for developing a modified Cam-Clay model.

Flexible walled permeameter tests were done on representative specimens from above and below the water table. The specimens were all saturated prior to testing. It was found that the hydraulic conductivity of the upper, brown clays was about one order of magnitude greater than the conductivity of the deeper grey clays.

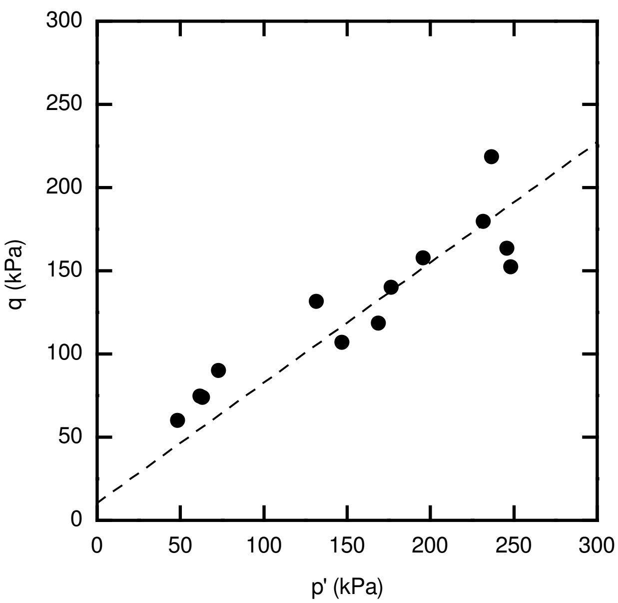

Figure 3. Post-Peak strengths in q-p' space for all triaxial tests.

2.2 Till Properties

The difficulty of sampling silt till limits the types of laboratory tests that can be conducted Standard penetration tests were conducted in the till during the site investigation to estimate the density and for use with empirical correlations. The upper 2-3 m of the till was loose, with blow counts of less than 10. A Shelby tube sample was successfully taken from this upper loose till and a set of three direct shear tests were done on the relatively undisturbed till sample with the results shown in Table 1. The direct shear tests demonstrated that although the till had a high water content and low SPT blow count, its consolidated shear strength was still within the expected range for silt tills. Since the measured strength of the till is much greater than that of the clay, no further strength tests on the till were conducted as shear failure through the till was unlikely.

Table 1. Select material properties of in-situ soil units.

Parameter	Clay	Till
γ (kN/m3)	17	23
k-brown clay (m/s)	8e-11	
k-grey clay (m/s)	8e-10	
E-initial tangent (kPa)	13400	200000 [1]
E-secant to peak (kPa)	7080	
G (kPa)	5070	
$\Phi_{post\ peak}$ -triaxial (˚)	19	
$c_{post\ peak}$ -triaxial (kPa)	5	
Φ_{peak} -direct shear (˚)	15	42
c_{peak} -direct shear (kPa)	28	0
$\Phi_{residual}$ -direct shear (˚)	8	NA

[1] Pressuremeter upper bound from Baracos et al. 1983

2.3 Rockfill Properties

The primary source of rockfill testing data was Abdul Razaq (2007) and Kim (2007). They conducted a series of large scale direct shear tests on limestone rockfill, varying stress levels and densities. The direct shear apparatus handled cylindrical specimens up to 0.6m in diameter and 0.4m in height.

The shortcomings of direct shear testing have been well documented by Saada (2000) and others. The main criticisms are the severe stress concentrations, the thickness of the shear zone is unknown, no parameter is measured directly, the sample is not realistically confined and most soils do not deform and fail along a plane (with the noted exception of overconsolidated clays). For the current project, large scale direct shear testing was considered suitable because the loading on the columns by a failed slope will somewhat resemble the direct shearing.

Due to limitations of the testing apparatus, the samples were not sheared to peak or critical states. The shear stress-shear strain curve was adequately represented by a hyperbolic curve and these curves were used to extrapolate the peak strength for each individual test.

Abdul Razaq (2007) determined the friction angle at maximum contraction which was identified as the transition friction angle Φ_t. The maximum contraction friction angle has been shown to reasonably approximate the constant volume critical state friction angle. With the measured shear strengths, Abdul Razaq created a two-part failure envelope as shown in Figure 4, consisting of a curved portion caused by dilation of the dense sample and a linear Mohr-Coulomb envelope for the higher stresses at which the rockfill will behave compressively. (Negussey et al. 1988)

As illustrated in Figure 4 effective compaction of the rockfill greatly improves its peak strength. Loose is used here in the relative sense. The loose samples were placed in the direct shear box without any compactive effort (loosely) but during shearing they did show expansive behaviour. Given the large maximum grain size and relatively small sample thickness (400mm), it may not possible to achieve purely compressive behaviour with this test apparatus. The loose and dense conditions correspond to relative densities of approximately 15% and 90% respectively. (Abdul Razaq 2007)

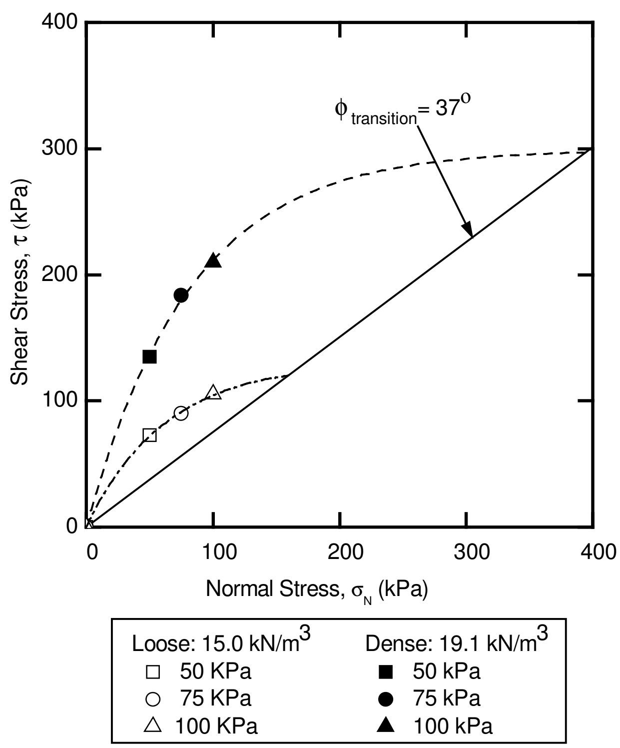

Figure 4. Shear stress vs. normal stress for dense rockfill samples. (Modified from Abdul Razaq, 2007)

Dilation effects were measured with a series of three LVDT's spaced across the top of the specimen aligned in the direction of travel. These LVDT's identified a progression of shear stress mobilization across the sample starting at the leading edge.

The rockfill testing data includes direct shear testing results completed by UMA Engineering Ltd. (1992) for the Mager Drive stabilization project. The limestone rockfill used in the Mager Drive tests were from a different source, but the same geological formation. The test apparatus used for these tests was less sophisticated, but the results fit well with those measured by Abdul Razaq (2007).

3 MODEL CONSTRUCTION

The following discussion is not intended to comprehensively review the modeling process or results but rather highlight some of the considerations regarding the application of material properties to in-situ, stress-deformation and slope stability models. Geo-Studio from Geo-Slope Int.[1] was used for the finite element and limit equilibrium analysis.

3.1 In-situ stresses

Developing a reasonable numerical representation of the in-situ stresses is important when doing finite element based stability modeling or considering deformations in soils using advanced constitutive models. Achieving accuracy can be very difficult for a number of reasons. Firstly, it is very difficult to measure in-situ stresses, and this has not been attempted for this project. Secondly, the stress history will result in complex relationships between horizontal and vertical in-situ stresses. In soils with high overconsolidation ratios, relationships such as those proposed by Mayne and Kulhawy (1982) predict horizontal stresses greater than the vertical stress. Considering a cross-section with sloped ground caused by erosion and relict landslide features complicates the true in-situ stress regime further.

Sigma/W uses the Poisson's ratio to determine the relationship between the major and minor principal stresses. The downward acting self weight provides the initial vertical force in each element with the model calculating the horizontal and shear stresses to achieve a solution.

Since Poisson's ratio is limited to values less than 0.5, horizontal earth pressures must be lower than the vertical pressure. This method does produce in in-situ stress regime similar the solution of Silvestri and Tabib (1983).

3.2 Porewater pressure conditions

The modeled in-situ porewater pressure distribution was calibrated to match the measured pressures at the piezometer locations. These in-situ conditions then provided the baseline for the pressure response due to the placement of the fill during the column field test.

In the laboratory, the hydraulic conductivity of the clay was measured in the vertical direction. Lake Agassiz clays are known to have a horizontal conductivity twice the vertical conductivity (Render 1970). The test results indicated that the natural local soil variability can result in hydraulic conductivity variations up to an order of magnitude and thus measuring or modifying the horizontal conductivity would be trivial. Additionally, the horizontal drainage path was much greater than the vertical drainage path because of the relatively large test loading area.

A hydraulic conductivity function for the clay was estimated using Van Genuchten's method. A soil water characteristic curve was developed based on the grain

[1] GeoStudio 2007 (Version 7.13) Geo-Slope International Ltd.

size distribution of the soil. The unsaturated hydraulic conductivity and soil water characteristic curves developed for this project were comparable to the measured values reported by Garinger (2002) for Lake Agassiz clays. These functions are important because the groundwater table was approximately 8m below the surface. The numerical porewater pressure response to loading and unloading matched quite closely both in magnitude and spatially with the response measured in the field.

A hydraulic conductivity of the till layer was not measured at the test site, but a regional average of $3x10^{-8}$ m/s has been reported (Render 1970). Piezometer observations of the till pressure heads during the test loading indicated that there little to no porewater pressure response, and therefore, the till has been modeled as a drained layer, essentially maintaining the base boundary condition throughout the layer. The two order-of-magnitude difference between the conductivities of the till and clay layers affirms this assumption.

3.3 Stress Deformation and Constitutive Models

The rockfill material was modelled with three different constitutive models: linear-elastic, elastic-perfectly plastic (EPP) and hyperbolic elastic. The linear-elastic model is useful for doing sensitivity analysis as it is numerically the most stable. The EPP and hyperbolic models present a more accurate picture of the soil response to additional loading because they are dependent on in situ stress conditions.

The hyperbolic model, based on the formulation of Duncan and Chang (1970) is the preferred constitutive model for rockfill as it closely “fits” the stress strain curves from the lab tests as shown in Figure 5. Fitting the model to the test data required replacing the elastic modulus E with the shear modulus G according to the Equation 1:

$$E = 2G(1+\nu) \quad [1]$$

The hyperbolic model was fitted to the loose, medium and dense test results by appropriately varying the strength definitions.

The compacted rockfill was considered drained based on its grain size distribution and the fact that the columns were surrounded by high plastic clay with a much lower hydraulic conductivity. This assumption was confirmed by monitoring of porewater pressures in the clay in-front of, in-between and in the rockfill columns. The piezometers installed in the rockfill, till and the remaining clay between the columns did not measure a porewater pressure response to the loading whereas the piezometers installed upslope of the rockfill columns did report a porewater pressure response (Thiessen et al. 2008).

The clay layers were also measured with three constitutive models: linear elastic, elastic perfectly plastic and Modified Cam-Clay. A Modified Cam-Clay constitutive model has been developed and used in the numerical modeling with good results. The Cam-Clay model more accurately predicts the pore-water pressure response and handles yielding more realistically than elastic or elastic-perfectly plastic formulations.

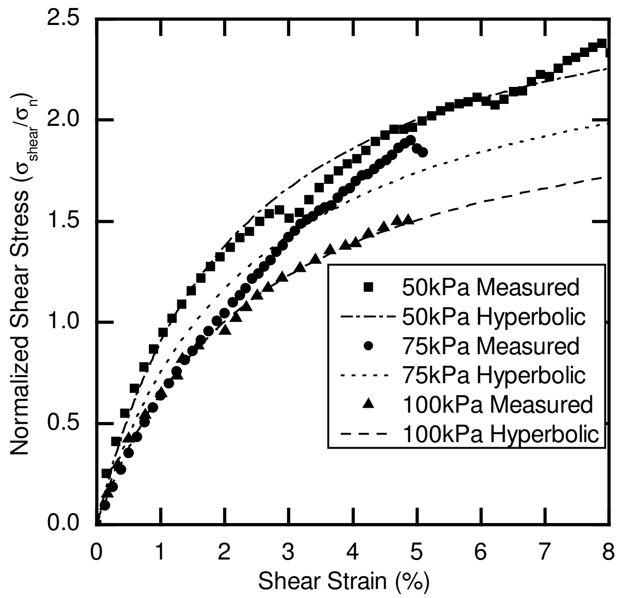

Figure 5. Normalized shear stress vs. shear strain curve from direct shear test for dense rockfill

3.4 Slope Stability Analysis

Slope stability analysis was not the primary objective of the modeling, but it is useful for identifying how constitutive models and the resulting stress distributions affect the assessed stability of a slope.

When doing stability analysis of overconsolidated clay slopes, the question arises whether peak, post-peak/critical state or residual shear strengths should be used. When a slope has failed, the strength can be back calculated. Historical photos and observations confirmed that the slope at the research site “failed” around 2003, but the site was not instrumented at that time, and the depth of the failure was unknown. Inclinometer surveys of the mid and upper slope from 2006 to 2008 showed very limited movement (<2mm) at depth. Over the same period, visual observations did identify movements in the lower slope, below the normal regulated summer river level. These movements coincided with the fall drawdown of the river.

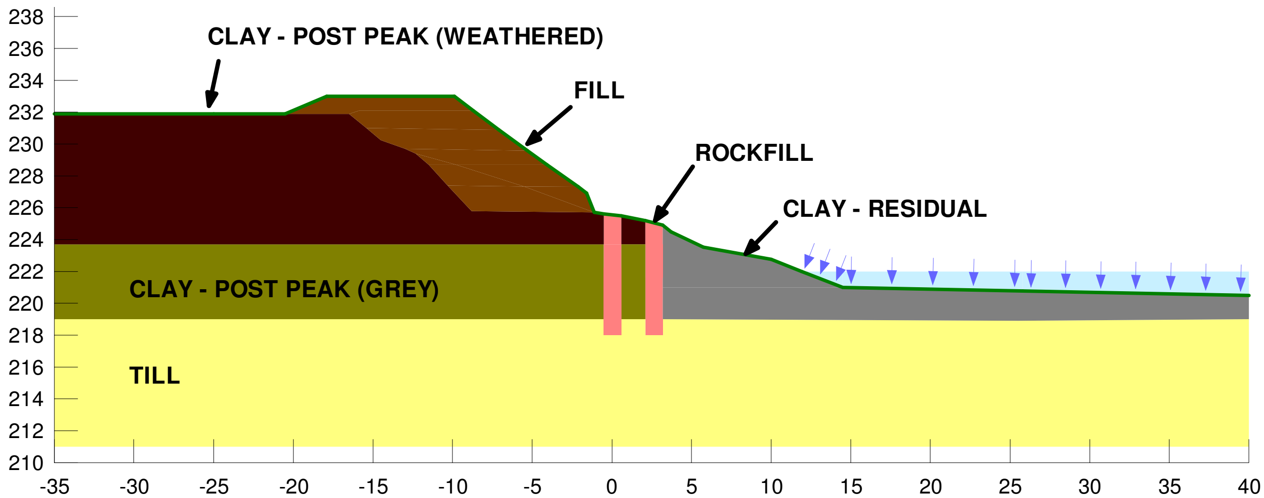

Figure 6. Slope stability modeling cross section

Common practice, based on recommendations by Baracos and Graham (1981) is to model moving slopes with residual strengths and stable slopes with fully softened strengths. This methodology has been applied to the stability analysis of the current project by applying residual strengths to soil down-slope of the summer river level while attributing post-peaks strengths to the soils up-slope of the normal summer river level. The residual friction angle was measured with multiple reversal direct shear tests. Figure 6 shows the material types and regions they were applied to.

3.5 Preliminary Results

Figure 7 and Figure 8 show some of the results of the numerical modelling. The modeled porewater pressure response illustrated in Figure 7 is slightly greater at this node (vibrating wire piezometer B) than the measured response. The averaged response at all piezometer locations matched very closely with the averaged modeled response at those locations.

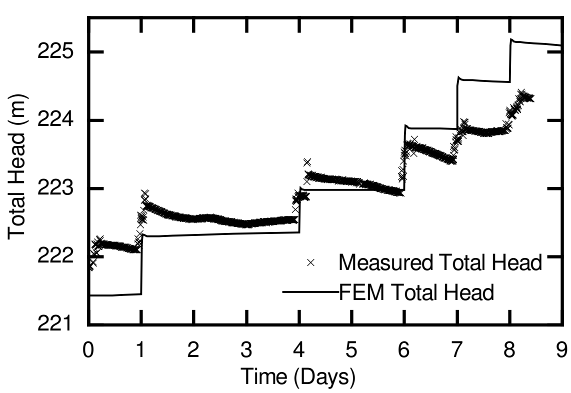

Figure 7. Measured and modeled total heads at vibrating wire piezometer B.

Figure 8 shows a good agreement between modeled and measured deformations. The deformations shown were modeled using a linear elastic constitutive model for the clay and the hyperbolic model for the rockfill. It was found that using a modified Cam-Clay model overestimated the deformations.

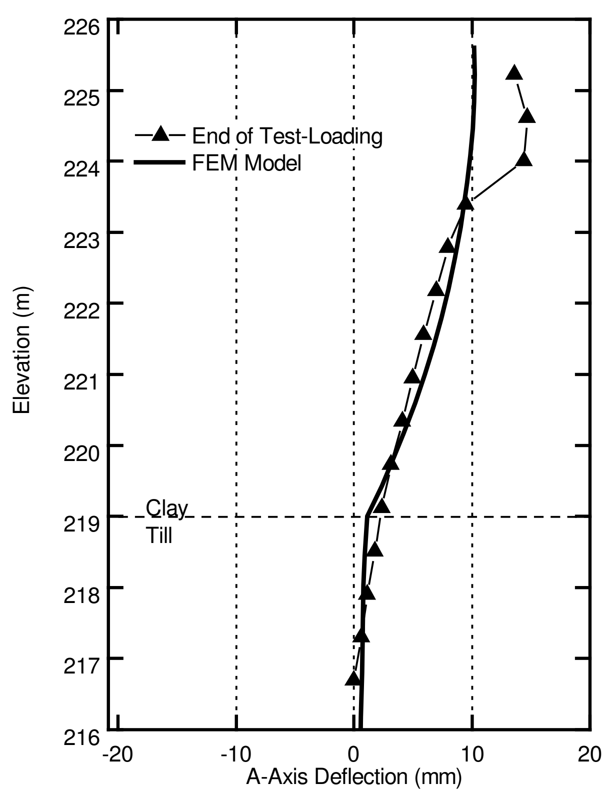

Figure 8. Measured deformations from SI-7 installed in a rockfill column and modeled deformations.

4 CONCLUSIONS

Using laboratory test results in developing numerical models requires some compromise with respect to complexity, given the limitations of the input data. Laboratory results require some subjective interpretation for application because of the spatial variation of soil properties and testing limitations. Although there have been major advances in constitutive models, they can not accurately predict soil behaviour in all stress conditions. Additionally, as more advanced constitutive models are applied, numerical instability becomes a greater problem. Despite the shortcomings, this project has shown that with care, models can provide reasonable results which can further our understanding of geotechnical system behaviour.

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