NUMERICAL MODELLING OF THE 1962 RED RIVER FLOODWAY TEST PIT EXCAVATION

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ABSTRACT

This paper presents results of a recent analysis completed for the documented failure of a large-scale test excavation completed in 1962 in the high plasticity glacio-lacustrine Lake Agassiz Clays. The paper demonstrates that a fully coupled stress-seepage model is capable of replicating the observed behaviour of constructed deep cuts in real-time under transient stress and/or groundwater conditions. The modelling completed for this paper on the 1962 PFRA test pit excavation has shown very good correlation between the measured porewater pressures and the predicted values from the model. The modelling efforts have shown there are only small differences between using a linear elastic or Modified Cam Clay model to represent the high plastic clays during the excavation. The model has been able to reproduce the failure that occurred along the 1H:1V slope of the test excavation.

RÉSUMÉ

Ce papier présent les résultats récents d'une analyse, qui a été complété pour la documentation de la faillite d'une excavation d'essai à grande échelle, complété en 1962 dans la plasticité élevée glacio-lacustre au Lac Agassiz Clays. Le papier démontre qu'un modèle entièrement couplé de soumettre à une contrainte-infiltration est capable de dupliquer le comportement des coupes profondes construites en temps réel sous un effort passager et/ou des états d'eaux souterraines. Les modèles qui ont été complétés, pour ce papier, au sujet du sondage de l'excavation de mine PFRA en 1962 ont démontré une très bonne corrélation entre les pressions mesurées de porewater et les valeurs prévues du modèle. Les efforts subi ont démontré qu'il existe seulement des petites différences entre l'utilisation d'une élastique linéaire ou l'utilisation d'un modèle Modified Cam Clay, qui représenter l'argile d'haute plastique pendant l'excavation. Ce modèle a été capable de reproduire la faillite qui s'est produit le long de la pente 1H :1V sur l'essai d'excavation.

1.0 INTRODUCTION

This paper presents results of a recent analysis completed for the documented failure of a large-scale test excavation completed in 1962 in the high plasticity glacio-lacustrine Lake Agassiz Clays. The paper demonstrates that a fully coupled stress-seepage model is capable of replicating the observed behaviour of constructed deep cuts in real-time under transient stress and/or groundwater conditions.

In 1962 the Prairie Farm Rehabilitation Administration (PFRA) completed a full-scale test pit excavation to investigate the stability of cut slopes in Lake Agassiz clays as part of the geotechnical design prior to the construction of the Red River Floodway around Winnipeg. Day by day records of the excavation progress (stress unloading), of the porewater pressure response to unloading, and of slope movements were well documented. Previously reported attempts to model the instrumented failure using measured shear strengths and conventional limit equilibrium analysis using the method of slices provided limited success (Mishtak 1964, Freeman and Sutherland 1974).

Because of the inability, of the existing stability modelling tools to replicate the observed failure, the designers of the original floodway channel concluded that channel sideslopes of 6H:1V would most likely remain "stable", although some localized sloughing might occur and would be acceptable (Mishtak 1964). No specific evidence remains or has been found as to how the selection of 6H:1V sideslopes was arrived at, but anecdotal evidence suggests that the work by Baracos (1961) and Sutherland (1966), most likely formed the basis for the decision.

The 1997 Red River "Flood of the Century" imposed hydraulic flows, which exceeded the design capacity of the floodway channel (KGS Group 1999). Following the 1997 flood a series of engineering studies (KGS Group 1999) concluded that the City of Winnipeg narrowly avoided complete flooding and destruction. Subsequently the governments of Canada, Manitoba and Winnipeg have committed to significant expansion of the floodway channel from its current design capacity (approximately a 1 in 100 year return period) to an enlarged channel which will pass a much larger 1 in 700 year return period event.

Recent pre-design studies have been examining the options of deepening the channel, steepening the channel sideslopes, as well as examining how to place additional spoil material or berms next to the top of the excavated channel. Results of the expanded channel design modelling are not described herein, but the recent stability modelling started with a analysis of the results of the 1961 instrumented test excavation failure. That calibration forms the focus of this paper.

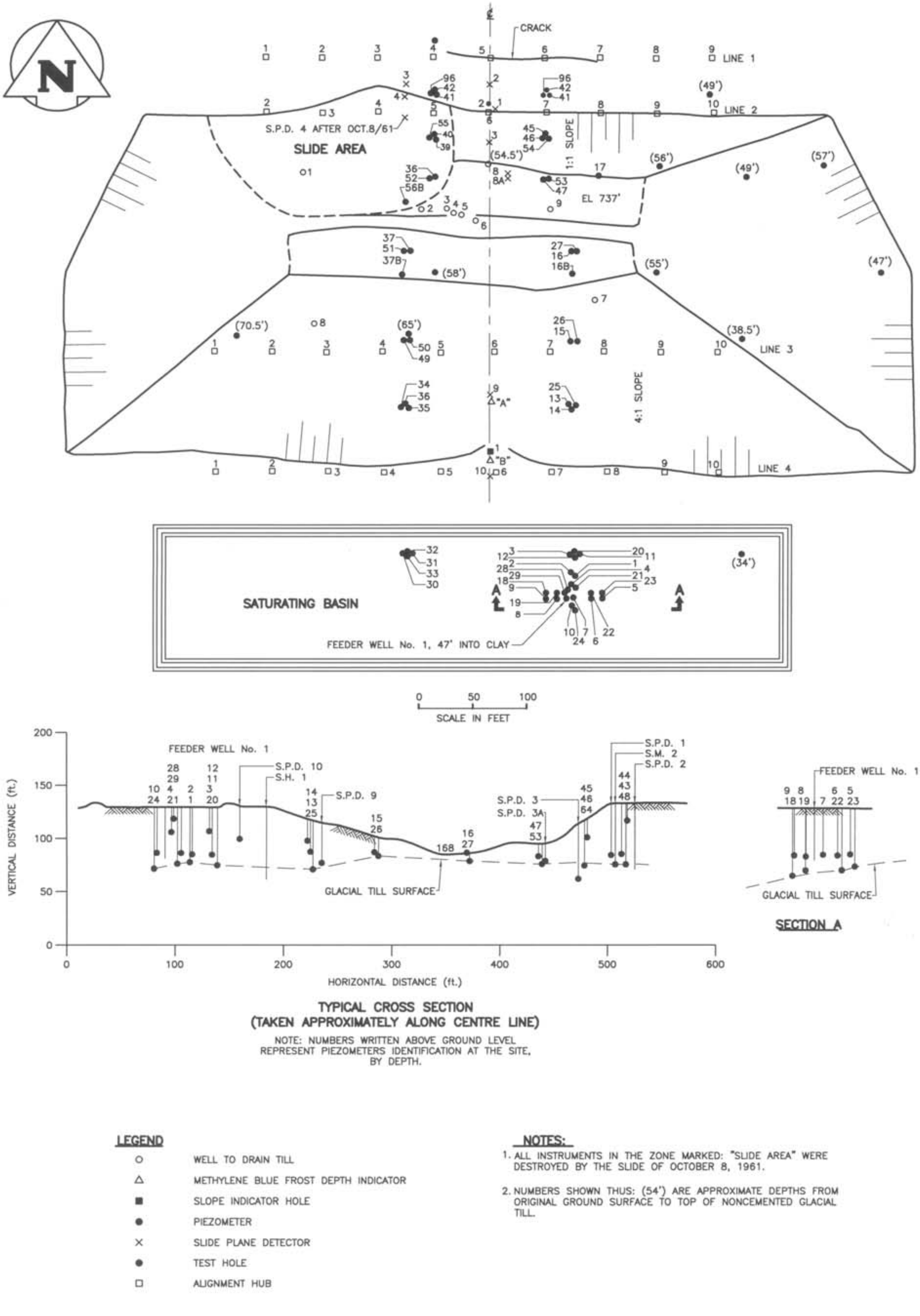

Figure 1. Test pit excavation plan and cross-section showing instrumentation.

2.0 BACKGROUND

Prior to commencing the test excavation in August of 1961, an intensive field and laboratory investigation of the high plastic clay was undertaken along the proposed floodway channel length. These investigations were completed by PFRA, the University of Manitoba, and the Water Control and Conservation Board (WCCB) of the Manitoba Department of Agriculture and Conservation. Meetings were held with Professor Casagrande to review the data, possible methods of excavation, stability of side slopes, and recommendations for further investigations. Based upon these meetings and discussions it was decided to deliberately excavate a large test excavation with side slopes steep enough to generate a slope failure (Figure 1). Full details of the test excavation program are found in the unpublished report by PFRA (1962). Mishtak (1964) and Freeman and Sutherland (1974) have previously presented summary findings of the test excavation as well as limit equilibrium stability modelling using traditional method of slices approaches.

Instrumentation of the test excavation included slope inclinometers, modified slide plane detectors, and alignment hubs to measure ground movements, Casagrande piezometers measured porewater pressures in both the clay and till, and methylene blue frost depth indicators, measured depth of frost at any time. Details of the instrumentation locations are shown on Figure 1 along with a cross section through the test excavation.

With the underlying bedrock under elevated pressures above the excavation level, flow into the excavation was expected to occur as excavation proceeded downwards. These flows were anticipated to either come through existing fissures in the clay or through fractures caused by basal heave or uplift. Render (1970) documented the impact of the bedrock artesian pressures on various aspects of the floodway construction.

3.0 TEST EXCAVATION

The test excavation began on August 9, 1961 and was completed on October 31, 1961. The excavation progressed using rubber-tired scrapers until an average depth of 2.4 to 2.7 m was reached on August 21, 1961. Very little rain had fallen and the temperatures remained in the twenties (Celsius). Below this depth which coincided with the location of the water table prior to excavation, soft spots began to develop causing the rubber-tired scrapers to slip when attempting to climb up the slopes of the excavation. By September 1 the excavation had reached an average depth of 4.6 m. From September 1 to 11, rainfall became more of an issue, as 18 mm fell allowing the equipment to be used only 60% of the time. Regular pumping of the excavation was required. By September 11 the excavation was at an average depth of 6.1 m and clay piezometers recorded approximately a foot drop in piezometer levels for each foot of excavation. Figure 2 shows the porewater response of piezometers that were installed in or near the 1H:1V slope excavation. The first flows from the bedrock up into the base of the excavation were also noticed once a depth of 6.1 m was reached, when flow came from an old borehole. As testholes began to open up and flow, the bedrock pressures dropped but upon sealing the holes the piezometric elevations were recovered as measured by the piezometers in the till.

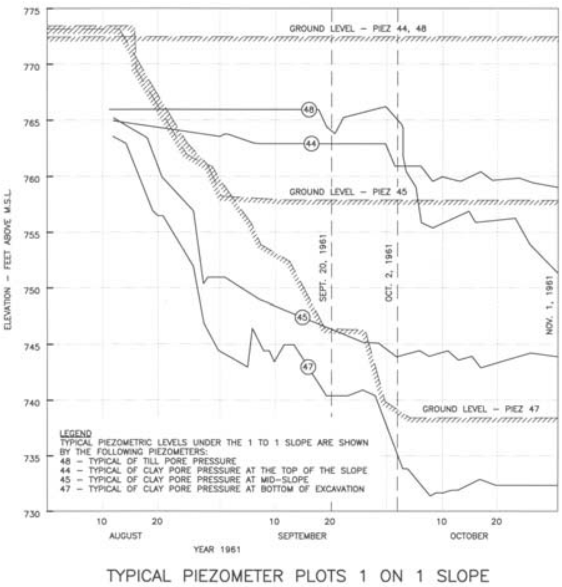

Figure 2. Observed porewater pressure response to excavation.

Work was concentrated at the toe of the 1H:1V slope in order to achieve failure conditions on this slope. A depth of 7.9 m was reached at the 1H:1V slope by September 18. However, 41.9 mm of rain fell between September 18 and 25. All work stopped and pumping of the excavation was undertaken. The first flow from fissures and cracks, due to hydraulic fracturing in the base of the excavation, was observed on September 26 when the base of the excavation was at a depth of 10.1 m. Two large cracks at the toe of the 1H:1V slope opened up and flowed at 5 gpm. After this date flow from the underlying bedrock aquifer was continuous and most of the till piezometers showed decreasing pressure in response to the flows. A depth of 11 m was reached at the toe of the 1H:1V slope by October 2. With increased flows from the till and excessive softening at the toe of the 1H:1V slope, the equipment was moved to the opposite side of the excavation to complete the rest of the excavation.

No evidence of any movements along the 1H:1V slope were recorded until September 15, when the alignment hubs at the top of the slope were noted to have moved 38.1 mm toward the excavation. However, no tension cracks were visible on top of the slope at this time and these movements were concluded to be elastic rebound toward the excavation. First indication that the slope was beginning to shear was given by the slope inclinometer (SH 2) on September 29. The slope inclinometer showed that the shearing was occurring at a depth of 14.6 m (elevation 220.98 m just above the till) when the excavation was at a depth of 10.4 m. By October 7 the slope inclinometer indicator probe would no longer advance past the 14.6 m in the tube, indicating that the casing had sheared off. The first tension crack was noticed 15.2 m beyond the crest of the 1H:1V slope on October 7. On the morning of October 8, 1961 the 1H:1V slope failed along a 41.1 m length of the excavation and the drop was approximately 5.2 m. Figure 3 shows the slope failure of October 8, 1961. The final top dimensions of the excavation were 231 m long by 97 m wide and approximately 11 m deep. Further movements were measured after the failure on October 8, with two other smaller failures occurring elsewhere along the 1H:1V slope on October 31 and December 16, 1961.

Figure 3. Photo of October 8, 1961 failure.

4.0 STRATIGRAPHY AND MATERIAL PROPERTIES

Typically, there was a 3.0 to 4.6 m thick brown clay layer, complex in structure, with alternating thin silt and clay layers overlying a relatively uniform high plasticity grey lacustrine clay layer, which was 9.1 to 15.2 m thick. Underlying the clay was a medium plasticity clay or silt till. According to PFRA index test results, the upper complex clay had a plastic limit (w_p) of 17% and a liquid limit (w_L) of 53% and an average water content (w.c.) of 30%. The lacustrine clay had a plastic limit (w_p) ranging from 17% to 31% and a liquid limit (w_L) ranging from 68% to 116%, with an average w.c. of 55%.

Figure 4 plots the results of the 1962 oedometer testing that was completed at test excavation as well as the results from the recent 2003/2004 investigation for the floodway expansion. Good agreement between the data of the two testing programs is apparent. It is seen from this plot that clays were heavily over-consolidated (OCR ratio greater than 4) down to elevation 227 m, and medium over-consolidated (OCR from 2 to 4) from 227 m down to elevation 223 m. The clays were lightly over-consolidated (OCR from 1 to 2) immediately above the till.

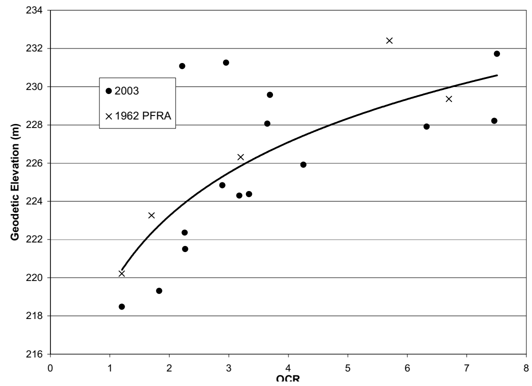

Figure 4. Over-consolidation ratio versus elevation.

A significant number of triaxial tests were completed both prior to the 1962 test excavation and then again during the 2003/2004 investigations for the floodway expansion. The triaxial testing included consolidated drained and consolidated undrained compression tests with porewater pressure measurement. There were 79 and 24 triaxial tests completed for the 1962 test excavation and 2003/2004 testing programs respectively. The 2003/2004 testing was completed on 100 mm diameter thin walled Shelby tube samples trimmed to minimal dimensions of 73 mm in diameter by 150 mm long. Figure 5a and 5b summarize and compare both sets of test data in p'-q space where the failure criteria has been selected using peak deviator stress.

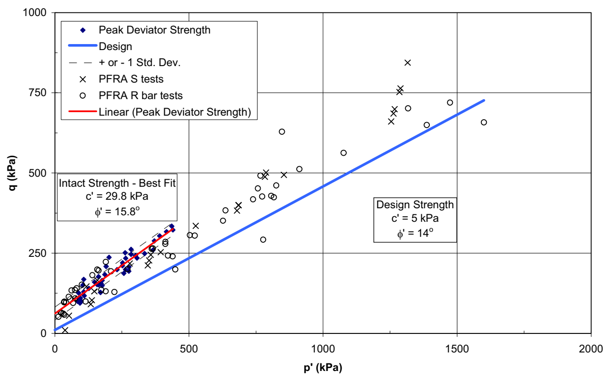

Figure 5A. Triaxial test results for grey clay.

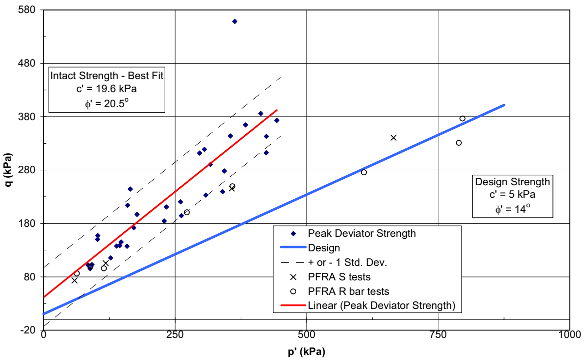

Figure 5B. Triaxial test results for brown clay.

From PFRA (1962), the shear strength testing of the brown clay produced a ϕ' = 15°, the over-consolidated grey clay had a ϕ' = 14° and the lower normally consolidated grey clay had a ϕ' = 15°. A value of c' = 5 kPa and ϕ' = 14° was interpreted for design to represent the shear strength parameters for both clays from the 2003/2004 test data. It is important to note that the selected strength values form a lower bound of all the measured peak deviator strengths. As such, this design value represents a conservative estimate of shear strength for these materials.

5.0 NUMERICAL MODEL

The numerical modelling used the commercially available software package Seep/W, Sigma/W and Slope/W developed by GeoSlope International Ltd. of Calgary, Alberta. The GeoSlope applications were selected for the numerical modelling task due to the ability to fully couple the stress-deformation and groundwater flow (both transient and steady-state conditions) aspects of the test excavation. The coupled finite element model calculates the actual effective stress conditions in the soil as a function of geometry, material properties, and the groundwater flow regime. The resulting stress state is the input to the slope stability model (Slope/W) for slope stability modelling. The model allows for estimation of the stress deformation response of the soil due to excavation to be coupled to the impact on porewater pressures. This is an important feature of the model as there was documented porewater response to excavation.

Table 1 summarizes the material properties that were assigned in the modelling. Properties such as λ, κ and Γ, which are essential for the modelling effort along with the hydraulic characteristics, were interpreted from the testing data that was reported by PFRA (1962). First attempts to model the stress deformation response using a Modified Cam Clay (MCC) model were undertaken to assess the importance of including the plastic component of material behaviour. Subsequent modelling used a linear elastic model to define the material behaviour. Comparison of the use of the two different models is discussed in the following sections.

Table 1. Material properties

Property	Upper Brown Clay	Upper Grey Clay	Lower Grey Clay	Glacial Till
$c'_{(measured)}$ (kPa)	5	5	5	5
$\phi'_{(measured)}$ (deg.)	15	14	15	30
$c'_{(design)}$ (kPa)	5	5	5	5
$\phi'_{(design)}$ (deg.)	14	14	14	30
γ (kN/m^3)	18.0	16.5	16.5	22.0
κ	0.033	0.125	0.124	-
λ	0.102	0.371	0.44	-
E (kPa)	20000	20000	20000	200000
ν	0.4	0.4	0.4	0.4
OCR	7	3	1.2	-
Γ	1.67	4.3	5.19	-
M	0.49	0.53	0.57	-
θ_{sat}	0.49	0.59	0.59	0.27
k_{sat} (m/sec)	$1x10^{-9}$	$1x10^{-10}$	$1x10^{-10}$	$1x10^{-4}$
k-ratio	1	1	1	1

The initial groundwater conditions were created using a constant head boundary on the left and right boundaries of the model representing a phreatic condition in the clay (2.4 m below prairie elevation). A constant head boundary was defined at the base of the model to represent the piezometric condition measured in the underlying till. The initial steady-state groundwater regime was then calculated in the FEM groundwater model for the initial groundwater conditions. Then by coupling together seepage and stress-deformation modelling it was possible to calculate time-dependent changes in stress due to the excavation activity (change in geometry) and corresponding changes in the porewater pressure regime. The hydraulic conductivity and volumetric water content functions were taken from standard curves provided in the software package for similar materials (based upon physical description).

The model time steps were set up to replicate the excavation as it was documented to occur in the PFRA 1962 report. With the geometry and depths of the test pit excavation known at different times during excavation, these geometries and depths were used to govern the excavation as it progressed in the model. With the stresses and porewater pressures calculated, these results were imported into the slope stability model and an analysis of the measured failure in the field was conducted using the finite element method in Slope/W.

6.0 RESULTS

Figures 6 and 7 show typical porewater pressure values measured in 1961 versus pressures that were calculated during the modelling of the excavation. The data shown on Figures 7 and 8 correspond to piezometers 47 and 48 installed in the clay and till layers in the test excavation. Figures 7 and 8 show the model was able to capture the general trend of each of the piezometers as observed in the field and shows close agreement. Using the Modified Cam Clay model to represent the clay behaviour produced slightly closer agreement for estimating the porewater pressures versus using the linear elastic model as shown on Figures 6 and 7, but the results are slightly different.

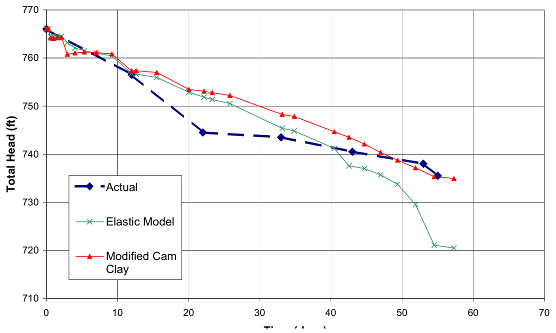

Figure 6. Measured vs. modeled porewater response to excavation in clay.

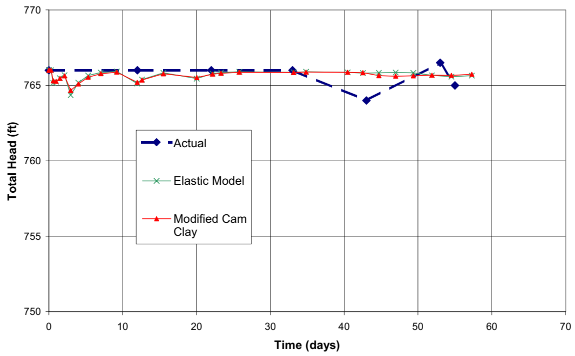

Figure 7. Measured vs. modeled powerwater response to excavation in till.

The porewater pressures and stresses as calculated from the coupled consolidation analysis were imported into the stability model and a stability analysis was conducted based on the surveyed geometry at different times during the excavation. The estimated factor of safety (FS) at each stage using the two models are summarized in Table 2 and a comparison of the critical failure surface and the approximate failure geometry are shown on Figure 8.

With movement first being measured on September 29 behind the 1H to 1V slope, the estimated FS should be relatively close to unity. As seen in Table 2, the estimated FS using the elastic model is equal to 1.00 and the Modified Cam Clay model estimated FS = 0.92.

Table 2. Estimated factor of safety during excavation

	Factor of Safety	
	29-Sep	8-Oct
Elastic Model	1.00	0.77
MCC Model	0.92	0.57

7.0 DISCUSSION

The results have shown that there was a slight difference between using either a linear elastic or Modified Cam Clay model to represent the clay soils during excavation. The difference in the results can be associated with the fact that the Modified Cam Clay model has allowed for yielding to occur during excavation while the linear elastic model has not. Since a majority of the soil is over-consolidated to some degree and the primary stress changes during excavation are unloading (stress decrease), the yielding that is associated with the unloading due to excavation is minimal resulting in only a small difference in the estimated porewater pressure response between the two models.

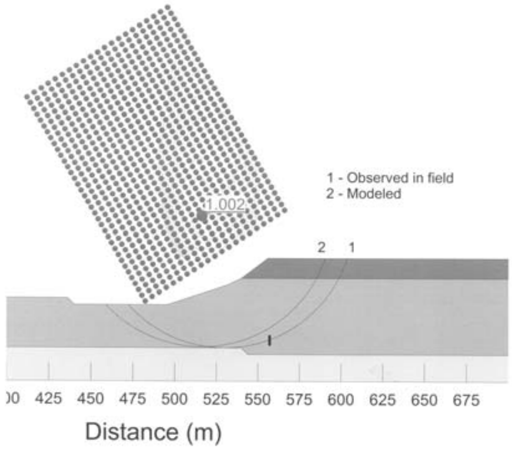

Figure 8. Modeled vs. observed failure geometry

The modelling efforts reported herein have been able to analyze the failure of the 1H:1V slope of the test excavation. The model has been able to capture the transient nature of the excavation process. A key feature that has been left out of previous analyses is the upward gradient that occurs as the excavation progresses. Blowouts were documented to occur in the field and were continuous after the date of first measured movement on September 29, 1961. The upward gradients served to lower the effective stresses at the toe of slope, which ended up contributing to the failure that occurred on October 8, 1961.

8.0 CONCLUSIONS

The modelling completed for this paper on the 1962 PFRA test pit excavation has shown good correlation between the measured porewater pressures and the predicted values from the model. The modelling efforts have shown there are only small differences between using a linear elastic or Modified Cam Clay model to represent the high plastic clays during the excavation. The model has been able to reproduce the failure that occurred along the 1H:1V slope of the test excavation.

9.0 REFERENCES

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