THREE DIMENSIONAL EFFECTS OF EXCAVATION ON STRESS DISTRIBUTION

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ABSTRACT
During the excavation of a deep (24m) cut near Sarnia, Ontario, unexpected lateral slope movements were observed. This was partially related to the fact that the soil has dissolved gas in its pore fluid. Stress redistribution as a result of soil excavation is another reason. A three dimensional model was required to capture the actual stress distribution under the excavation and provide a justification of the excessive lateral slope movements. This study presents 2D and 3D elasto-plastic effective stress analyses of the excavation.

RÉSUMÉ
Pendant l'excavation d'une fosse profonde (24m) près de Sarnia, Ontario, on a observé des mouvements des versants latéraux inattendus. .Ceci a été partiellement lié à la dissolution de gaz dans le fluide interstitiel des pores du sol. La redistribution d'effort en raison de l'excavation de sol est une autre raison. Le recours à un modèle tridimensionnel a été nécessaire pour reproduire la distribution réelle d'effort sous l'excavation et pour fournir une explication des mouvements latéraux excessifs des versants. Cette étude présente l'analyse élastoplastique drainée 2D et 3D du site en utilisant le logiciel élément finis Abaqus. Les résultats sont comparés aux données recueillies in situ.

1 INTRODUCTION

Ontario's only hazardous waste landfill is located on a 121- hectare site in Moore Township in south western Ontario. The landfill design is based on a progressive trenching and filling technique using conventional excavation and earth moving equipments. During construction of Sub-cell 3 of Cell 18, unexpected slope movements accompanied by evolutions of gas and water from vents in the base of the cell were observed. Construction work was halted in Sub-cell 3 and immediate action was taken to prevent leachate from flowing to the affected area. The venting gas was found to be 95% methane. Extensive studies and site investigations were conducted to explain the observed phenomena (Safety Kleen Ltd, 2000) and the area was repaired by the construction of a massive compacted clay liner over the affected area and no waste was placed in the affected area. There was no escape of contaminants associated with these events.

A similar problem involving unanticipated deep seated deformations had previously been observed at the nearby site of the St Clair river railway tunnel (Dittrich et al, 2002). The study showed that the pore water in the bed- rock and overlying soil has high concentrations of dissolved methane. Ditterich (2000) conducted a 2D effective stress finite element (FE) analysis utilizing an elliptical cap soil model coupled with Biot consolidation theory.

Although both sites share similar soil properties, the St Clair tunnel case could be reasonably approximated by 2D conditions. However it can be hypothesised that 3D effects may have been significant at the hazardous waste landfill. Thus the objective of this paper is to examine the differences between the predicted behaviour based on 2D and 3D FE analyses. A subsequent paper will address the issues of gassy soil behaviour.

2 PROBLEM DESCRIPTION

Cell 18 was divided into sub-cells to facilitate sequential excavation and waste placement. Excavation in Cell 18 was permitted to a maximum depth of 24.4m below original ground surface (i.e., to an elevation of 176m ASL). Construction involved excavation of the whole sub-cell from elevation of about 200 m ASL to 194 m then to elevation of 186m ASL to create an initial plateau. A second plateau was formed at elevation of 182m ASL. Then 6 m deep trenches were excavated with limited length and width to ensure slope and base stability.

2.1 Geologic Conditions

The site is characterised by four main overburden units. The upper most portion of the St Joseph till is weathered, fractured and hydraulically active to a depth of about 6 m. The lower portion of the unit is an unweathered, clay rich till aquitard extending to a depth of about 14 m below ground surface. The St Joseph till is underlined by a 24-26 m of Black shale till which is predominantly a clayey silt of low hydraulic conductivity. However the Black Shale till is characterized by inclusions of pocket's of sand and silty sand in a zone about 24.6 m to 28 m below ground surface. These sand lenses have an average thickness

of about 1.1 m. A thin weak discontinuous layer of sandy clayey silt, known as the Basal till, separates the Black Shale till and the bedrock at some locations. The shale bedrock, known as Kettle point formation, is encountered at a depth below the ground surface ranging between 37 to 45.2 m. The bed rock layer has a relatively high hydraulic conductivity and is an aquifer. The Kettle point formation is known elsewhere as the Antrim Shale (in Michigan) and the Ohio Shale (in Ohio). It is believed to be "both a source and reservoir of methane that is generated dominantly by microbial activity" (Martini et al., 1998). The bed rock elevation reaches its peak under Sub-cell3. The water table is close to ground surface and initial pore pressure distribution was hydrostatic.

2.2 Problem statement

During excavation of Sub-cell 3 to an approved elevation of 176 m ASL, unanticipated deep seated movements were observed and as a counter measure, the south end of the excavation was filled with excavated clay up to elevation 186 m ASL and trench dimensions were limited. Two months later, as construction and substantial filling procedure progressed, gas and water vents were observed at different points in Sub-cell 3. Excavation and waste placement were halted until further investigations were conducted. Remediation measures were then initiated. No contaminants escaped from the site.

3 FINITE ELEMENT MODEL

The finite element program Abaqus was used to asses the stability of the site. The soil was modelled using an elasto-plastic constitutive model following Mohr-Coulomb failure criterion. The constitutive model followed is an extension of the classical Mohr-Coulomb. Its yield function includes isotropic hardening/softening. The model's flow potential has a hyperbolic shape in the meridional stress plane and has no corners in the deviatoric stress space. A non-associative flow rule was adopted with a dilatancy angle of zero. The elastic behaviour is modelled as linear and isotropic.

For an effective stress analysis, ABAQUS considers the medium as a multiphase material. In case of saturated soil, the porous medium modelling considers the presence of a single fluid "wetting liquid" which is of very low compressibility -or entirely incompressible in the system (ABAQUS, 2003).

3.1 Numerical Model and Parameters

A sensitivity analysis was conducted using a 2D mesh, full 3D mesh, and a 1m thick 3D mesh (to approximate 2D conditions) to define the acceptable mesh refinement. The analysis showed that in the 3D model, 6 m tetrahedron elements will give results with acceptable accuracy. Another analysis was conducted to identify the distance to the lateral boundaries required so as to not to affect the results. The analysis suggested that the distance between the excavation edge and the far field lateral boundaries should be more than 5 times the depth of the excavation.

The excavation was modelled using a symmetrical cross section. The base boundary was taken to be rough and rigid and the lateral boundaries to be smooth and rigid. Hydrostatic pore pressure were specified at the far field lateral boundaries while a zero flux boundary was specified along the base and along the excavation center line. The soil surface and the exposed surfaces of the excavation were prescribed to be free flow, zero pore pressure boundaries. Figure 1 shows the soil profile used in the model.

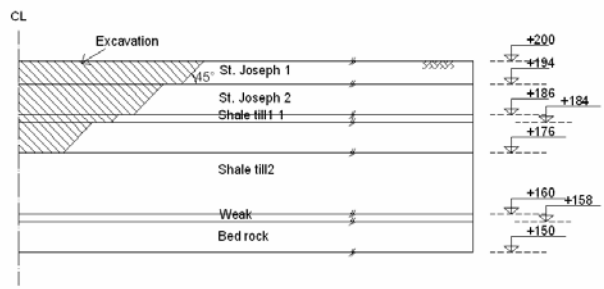

Figure 1 Soil Profile Examined

Soil effective shear strength parameters were collected from field and laboratory data. The properties adopted in the analyses are summarized in Table 1

Table 1 Shear strength parameters

Depth	Layer	δ	c' (kPa)	Φ'
200-194	St Joseph till 1	21.5	24	25
194-186	St Joseph till 2	21	16	27
186-184	Black Shale till 1	20.5	24	24
184-158	Black Shale till 2	19.6	24	24
160-158	Basal till (weak)	18.2	9	18.2
<158	Bedrock	23	40	40
	Sand lenses	19	0	30

Soil stiffness and hydraulic conductivity parameters– shown in Table 2- were assumed based on Ditterich's (2000) assessment of the soil. The model assumed constant hydraulic conductivity with time.

Table 2 Additional soil properties

Layer	E_u(kPa)	ν	e_0	k (m/s)
St Joseph till 1	55000	0.4	0.47	8.3×10^{-8}
St Joseph till 2	55000	0.4	0.49	8.3×10^{-10}
Black Shale till 1	55000	0.4	0.56	2.4×10^{-10}
Black Shale till 2	30000	0.4	0.74	2.4×10^{-10}
Basal till	15000	0.4	1.20	1.0×10^{-8}
Bedrock	200000	0.4	0.25	1.0×10^{-5}
Sand lenses	20000	0.4	0.43	1.0×10^{-7}

The excavation was modelled using Abaqus through series of steps. An initial geostatic step was used to achieve equilibrium under the initial conditions. Through the second and third step, soil layers from 200 m ASL to 186 m ASL were removed to form the first plateau. Step four was a consolidation step where the soil was left to consolidate and stresses to be redistributed for 3 months. During fifth and sixth steps, the soil regions from 186 m ASL to 182 m ASL were removed to form the secondary plateau and left to consolidate for 1 month. Trench excavation was conducted using shorter and smaller steps to monitor the system behaviour. Finally, a consolidation step was applied. This step was used to provide a sense of side slope stability with time if the excavation had remained open for a long period of time. The rates of excavation were selected to be similar to that adopted at the site (Table 3).

Since the Basal till was discontinuous but its distribution is not well defined, analyses were performed for the base case where the Basal till was assumed to be present everywhere. A number of analyses were also performed where it was assumed to be absent everywhere as will be discussed later.

Table 3 Analysis steps

Step	Description	Time days
Geostatic	Geostatic equilibrium step	0
Exc200	Excavate 200-194 m	1
Exc194	Excavate 194-186 m	1
Cons1	Consolidation step 1	90
Exc186	Excavate 186-182 m	1
Cons2	Consolidation step 2	30
Exc182	Excavate 182-179 m	1
Exc179	Excavate 179-178 m	0.5
Exc178	Excavate 178-177 m	0.5
Exc177	Excavate 177-176 m	0.5
Cons3	Consolidation step 3	open

3.2 2D Model

For the 2D model, 8-noded elements with biquadratic displacement, bilinear pore pressure and reduced integration was used. The element has three degrees of freedom (u_x, u_y, and pore pressure at corners). The average element size was 2 m. Figure 2 shows the 2 D mesh.

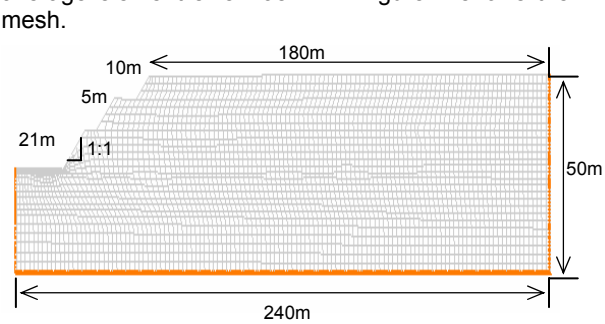

Figure 2 2D mesh showing excavation and plateaus

3.3 3D Model

Ten-noded modified quadratic tetrahedron elements with pore pressures and hourglass control were used. These elements had 4 degrees of freedom (u_x, u_y, u_z and pore pressure at the corner nodes). Element size ranged from about 6 m near the excavation, to 24 m at the boundaries as shown in Figure 3.

As the weak Basal till layer is discontinuous, the same analysis was repeated for two cases – one with and one without the weak Basal till layer. In the latter case the elements that correspond to the basal till were assigned the properties of Black shale till 2.

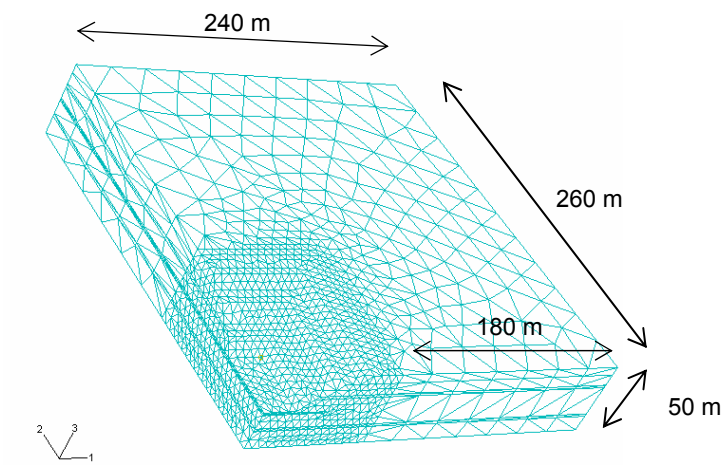

Figure 3 3D mesh

4 RESULTS AND DISCUSSIONS

4.1 Pore pressure response

The bedrock is weathered and has a relatively high hydraulic conductivity. Thus the reduction in overburden stress due to excavation combined with high pore pressures (due to the rapid dissipation of excess pore water pressures at, and near, the bedrock), creates the potential of hydrofracturing of the overlying clayey till and the potential for liquefaction of any granular units.

In the low permeability Black Shale till using conventional soil mechanics as used in the design of the excavation plan, the reduction in total stress is accommodated by a decrease in pore pressure. With time, excess pore pressure will dissipate leading to reduction of soil shear strength and this eventually may lead to failure. Thus the rate of pore pressure dissipation may be expected to control the maximum time that trench can be left open before being filled with waste. Figure 4 shows total pore water pressure response directly below the midpoint of the excavation for points at depth 38 m (in the bedrock layer) and a depth 36 m (in Black Shale till) during excavation. At the point in the bedrock, the pore pressure rapidly returned to its hydrostatic value after each excavation step

and this caused a rapid reduction in effective stress and strength in adjacent granular units. However for the point in the Black Shale till, the pore pressure dropped significantly with the reduction of total stress, followed by gradual dissipation of excess pore pressure during the consolidation step.

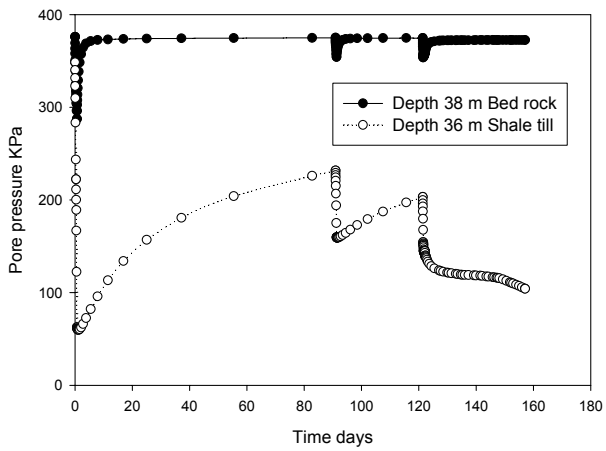

Figure 4 Pore Pressure response in bedrock and Black Shale till: 2D model

4.2 Shear zone

The 2D model resulted in deep seated failure when soil was being excavated from 177 to 176 m. The plastic zone extended to as far as 100 m from the excavation edge. The plastic zone at the time step before development of contiguous plasticity and failure is shown in Figure 5. Figure 5 shows two types of plastic zones, a zone under the excavation, and an overall slope failure.

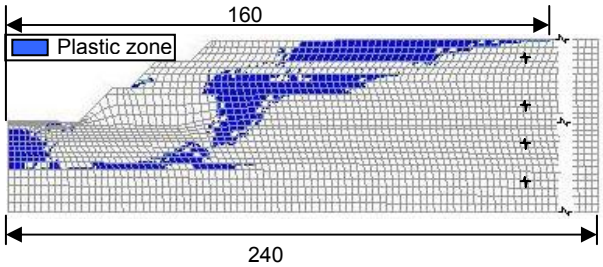

Figure 5 Plastic zone in 2D model one time step before failure

The 3D model was stable during excavation to elevation 176 m ASL. However, the spreading of plastic shear zones suggested that the model is on the verge of shear failure. Figure 6 shows the plastic zones in the 3D model.

Trow Ltd prepared a geotechnical report calculating the expected time for the slope to reach its drained (long-term) conditions in Cell 16 (Tricil, 1988, 1989 and 1990). Trow Ltd installed piezometers around the excavation and monitored pore pressure changes during and after sub-cell excavation. They concluded that undrained (short-term) conditions would prevail within the slope for about four months after excavation. It was reported that the key pore pressure dissipation occurred between four and eight months after which drained conditions prevail and the potential for slope failure is the greatest. On this basis, it was recommended that the trenches to be filled in a period less than eight months after excavation (Jagger Hims Ltd, 1996).

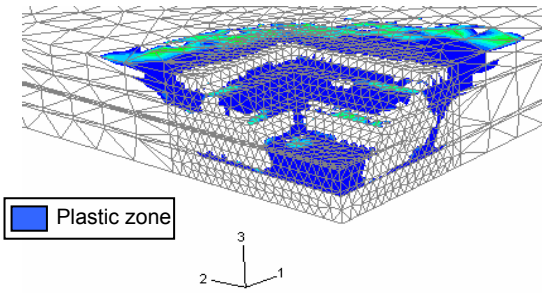

Figure 6 Plastic zone in 3D model

In the 3D model, plastic failure occurred in the consolidation step 10 months after excavation. This is reasonably consistent with the predictions made by Trow based on their field monitoring

4.3 Basal till

The Basal till (weak layer) separating the Black Shale till from the bed rock at some locations is discontinues. The presence of this layer could affect the model in two ways. First, its low shear strength will increase the potential of shear failure. Second, its intermediate hydraulic conductivity value between the extremes of very low permeability Black Shale till and the permeable bedrock may affect pore pressures. To asses the effect of this layer, 2D and 3D analyses were performed where this layer was given the properties of the Black Shale till rather than the Basal till.

The analyses without the weak Basal till layer were more stable than those with the weak layer. In the 2D analysis, the soil was stable during excavation but failed after one month of pore pressure dissipation.

Figure 7 shows the plastic zones one time step before failure from the 2D analysis. The plastic zone is directed towards the excavation perimeter and the development of a potential deep failure mechanism is evident.

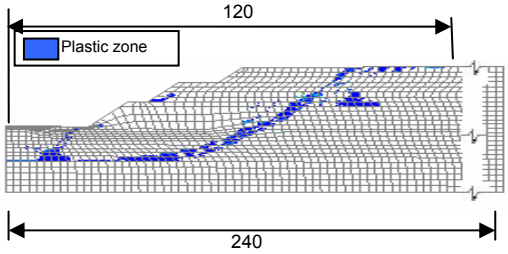

Figure 7 Plastic zone of 2D model without basal till (one time step before failure)

Figure 8 shows the plastic zones calculated from the 3D model 4 months after the excavation. The shear zone can be seen to be progressing towards the excavation perimeter but the excavation is stable

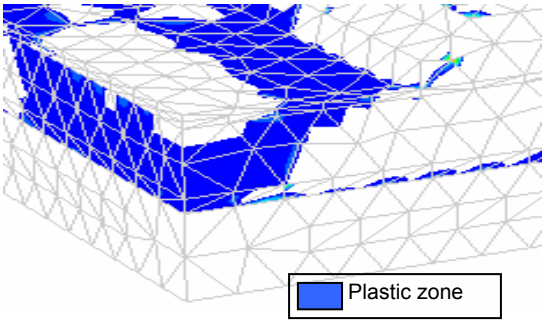

Figure 8 Plastic zone of 3D model without basal till

4.3.1 Heave

The maximum observed heave in Sub-cell 3 was 100 mm at monitor HG9923 located at mid span of trench 307 when the excavation reached 176mASL level (Safety Kleen Ltd, 2000). This was of similar magnitude to that measured during excavation of Sub-cell 2.

Figures 9 and 10 show the calculated heave progression at mid span of excavation base in the FE model when excavation reached 176 m ASL. The 2D and 3D models with the weak Basal till layer predicted a heave exceeding 200 mm before failure (the 2D model failed at 177m ASL). The 2D and 3D analyses without the weak layer predicted heaves of 146mm and 116mm respectively at the end of excavation and are much closer to the observed value. This suggests that the presence of the weak layer has major effect on results and potentially on the stability of the excavation.

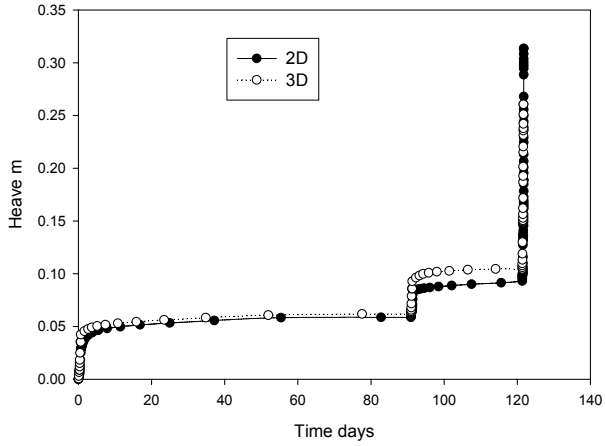

Figure 9 Predicted heave at the base of the excavation (elevation 176mASL) based on 2D and 3D analyse with the presence of weak Basal till layer.

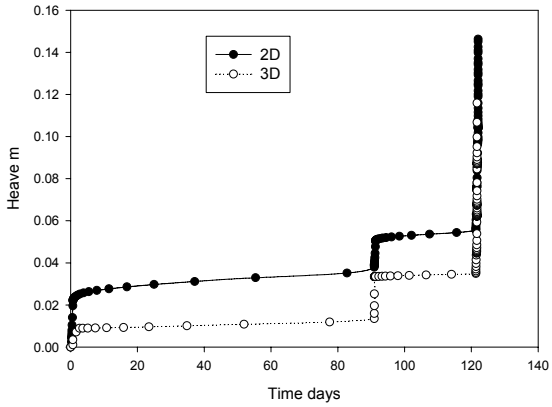

Figure 10 Predicted heave at the base of the excavation (elevation 176mASL) based on 2D and 3D analyse with the absence of weak Basal till layer

4.3.2 Lateral movement

A relatively large lateral movement (about 270 mm) was monitored at inclinometer SI 9825 at the crest of excavation (Safety Kleen Ltd, 2000)). The inclinometer recorded consistently moderate movements. However, unusual large lateral movements were monitored between elevations of 171m ASL and 179 m ASL. The maximum lateral movement occurred at a depth of 22 m. It was concluded that soil below elevation 180 m ASL had a lower undrained shear strength than the other overburden soil layers.

Figure 11 shows the calculated lateral displacements along a vertical line starting from excavation crest; down to 50 m depth (same location as SI9925). The maximum calculated lateral displacement ranged from 150-200 mm at 18 m depth. Lateral movement is in the range of what was monitored in site. However, the model didn't capture the excessive lateral movements that were recorded at 22m and 28m depth at SI9925.

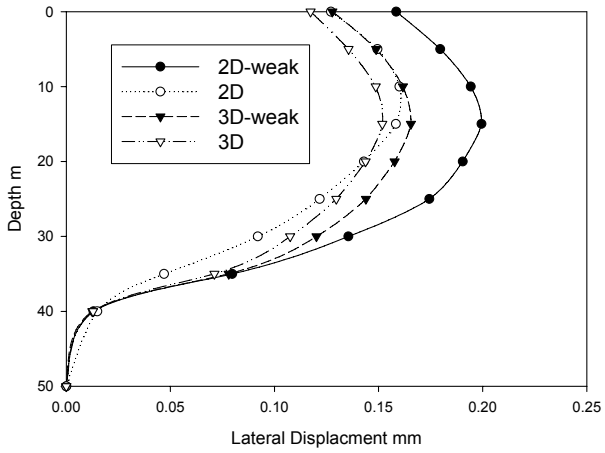

Figure 11 Lateral displacements in FE models

The heave and lateral displacement results (Figures 9, 10 and 11) highlight the potential significance of the presence or absence of the thin Basal till layer. The predictions closest to the field observations are those of the 3D model with no weak Basal till layer. There are no deep bore holes in the critical area so it is not known whether the Basal till is present or not. Other deep bore holes clearly showed the discontinuity of the Basal Till layer. However, it can be concluded that the presence of the basal till reduces the system stiffness significantly, and greatly affects the failure mechanism.

5 SUMMARY AND CONCLUSIONS

Unanticipated slope movements and the evolution of gas and water vents were observed at a deep excavation near Sarnia. Both 2D and 3D FE analyses of the excavation were performed using Abaqus. Coupled effective stress analysis simulated both excavation and post excavation pore pressure dissipation. The analyses adopted a conventional soil model (without modifications to account for gassy soil). It was concluded that 2D analyses over-predicted the deformations and under-predicted the stability of the excavation both compared to physical observations and the results of the 3D analysis. The presence or absence of the Basal till layer was shown to have a significant effect on the predicted response and it was inferred that this unit is likely absent as a significant continuous layer beneath Sub-Cell 3. This study's findings are consistent with Trow Ltd recommendation of filling trenches with wastes within the span of 8 months. This analysis used conventional soil mechanics similar to what was assumed in the design. This approach appears to predict gross behaviour in terms of stability and deformations. However it does not capture the mechanism that led to gas venting and the upward flow of water from the aquifer through about 14m of Black shale till and into the base of Sub-cell 3. Additional ongoing studies are needed to address this issue.

ACKNOWLEDGEMENTS

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