Emergency geotechnical instrumentation for a burning landfill to monitor the impacts of fire-fighting and post-fire reclamation

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Abstract: A state of local emergency was declared at the Delta Shake and Shingle landfill on November 27, 1999. The landfill is situated in Delta, British Columbia within an area referred to as Burns Bog. Surficial soils consist of very soft to soft peats, silts and clays. Two large diameter high pressure gas pipelines as well as a 500KV transmission tower border the landfill to the south while a set of railroad tracks, a 1.0m diameter forced sewer main and a 1.0m diameter high pressure water main border the north side. Previous movements of some of the side slopes of this landfill have shown the sensitivity of the stability of the landfill to the strength and deformation properties of the subsurface soils as well as the landfill materials. One particular event caused the movement of one of the high pressure gas pipelines of the order of 1.5m.

The fire fighting efforts required that large quantities of landfill material be moved to various locations on, and surrounding, the landfill until the fire was completely extinguished and the material could be reclaimed to its original location. This paper describes the emergency geotechnical works carried out which included: preliminary recommendations on material distribution during fire-fighting, prediction of the ground response to fire-fighting activities, and an extensive instrumentation monitoring program to verify that the ground movements were within expectations. The purpose of these geotechnical works was to ensure the safety of the bordering lifeline utilities during the fire fighting and reclamation activities.

Introduction

On Saturday, November 27, 1999, the Corporation of Delta declared a Local State of Emergency regarding the fire at the Delta Shake and Shingle landfill. British Columbia's Provincial Emergency Program (PEP) supplied funding for the firefighting operation. The Local State of Emergency was enacted due to the proximity of the landfill to the high pressure natural gas pipelines and other sensitive lifeline infrastructures (Corporation of Delta, December 9, 2000). An extremely rapid response for geotechnical recommendations was required.

On the evening of Saturday, November 27, 1999, Horizon Engineering Inc was requested by Sperling Hansen Ltd to carry out the geotechnical investigation and assessment necessary to ensure the safety of people, adjacent property and utilities for the fire fighting efforts. The fire fighting approach included significant earthworks in an area bounded by sensitive lifeline infrastructures and underlain by soft subsurface soils. The geotechnical subsurface instrumentation and investigation for the project were initiated on Sunday, November 28, 1999. In the days which followed five drill rigs and two survey crews were mobilized to the site, with full engineering field support, in order to install the instrumentation which would allow monitoring of soil movements in sensitive areas. In addition, the lead author worked closely with the Fire Emergency Command in the development of the fire remedial earthworks program, providing preliminary guidelines which were later confirmed with geotechnical analyses.

On January 14, 2000, the Provincial Fire Commission officially announced that the fire at the Delta Shake and Shingle Landfill was extinguished, thus ending the Local State of Emergency. Subsequently, the Ministry of Environment, Lands and Parks became responsible for the Delta Shake and Shingle Landfill site (Corporation of Delta Regular Council Minutes, January 18, 2000). By May 30, 2000, all of the material that had been removed from the landfill during the firefighting efforts was returned to the landfill.

The landfill was originally constructed in the early 1980's on soft peat and organic silt subgrade. The natural subsurface soils in the vicinity of this site consist of very soft to soft peats, silts and clays. These soft soils are underlain at depth by sands. Due to adverse soil conditions, the fire fighting works required substantial geotechnical effort to ensure the safety of people, adjacent property and major utilities described below. The geotechnical effort required at a landfill site with favourable soil conditions would have been significantly less than that required for the subject site. At the start of the geotechnical work, the owner of the gas pipelines indicated that it had 'zero tolerance' for movement, due to devastating impacts that failure of its pipelines at this location would cause. In designing the investigation and instrumentation program, it was understood that breach of these pipelines in such close proximity to a fire would be immediately catastrophic and have large, financial impacts on 'downstream' users of the gas. There were similar concerns with possible breach of any of the other bordering utility corridors described below.

Site description

The Delta Shake and Shingle Landfill is generally located at the northern edge of Burns Bog and south of River Road in Delta, BC as shown on Figure 1. The landfill is about 18m high and is bounded to the north by a set of railway tracks, a 1076mm high pressure water main, a 1076mm force sewer main, in turn bounded by a property occupied by a trucking company and River Road, respectively. In this area, River Road acts as a dyke for the adjacent south arm of the Fraser River. A drainage ditch is located immediately north of the water and sewer mains. Both water and sewer mains have already undergone large settlements and have ruptured in the past. To the south, the landfill is bounded by two high pressure gas lines (813mm diameter and 914mm, one of which has been locally replaced with a 406mm diameter bypass subsequent to a 1994 displacement) as well as one 500kV transmission tower. A perimeter access road is located on the west, south and east sides of the landfill. At the time that the Local State of Emergency was declared, the fire was located at the northern central portion of the landfill, referred to as the Horseshoe area.

The north, east and west sides of the landfill were sloped at about 45°. Tension cracks visible along the top surface of the landfill adjacent to these slopes were indicative of the marginal stability of these side-slopes. The south slope was more gently sloped at about 26°, but a 1994 movement of the south slope causing displacement of one of the gas pipelines suggests that even the south slope was at least locally over-steepened for the foundation materials present at that location.

Firefighting strategy

The firefighting strategy required that material affected by the fire be excavated from the landfill and trucked to predetermined locations called 'pads', away from the burning area. These pads were located on nearby properties adjacent to, as well as on top of, the landfill as shown on Figure 1. The burning material removed from the landfill was placed on 'hot pads' which were constructed with a leachate containment system incorporating HDPE landfill liners designed by Sperling Hansen Ltd, and extinguished using water. Material which was not burning was removed to 'cold pads'. Once the landfill fire was extinguished, the landfill material that had been removed during the fire remediation earthworks were returned to the landfill. Pads 1, 3, and 4 were designated as 'hot pads', while Pad 2 and the Contingency Pad were designated as cold pads. A 60m long drainage trench was excavated immediately south of the Contingency Pad to allow run-off from this pad to be collected. Pads 1, 3, 4 and the contingency pad were located in close vicinity to the water and sewer mains. Pad 2 was located on top of the eastern portion of the landfill. Soils to be used for lining and capping the replaced landfill material were stockpiled during the fire remediation earthworks in the vicinity of Pad 6.

Therefore, there were geotechnical concerns with regards to: a) construction of Hot and Cold Pads in close vicinity of the water and sewer mains, b) frequent and heavy construction traffic crossing the already over-stressed water and sewer mains, and c) addition of material at Pad 2 in close proximity to over-steepened side-slopes of the existing landfill.

Geotechnical program objectives

The following specific objectives were developed at the start of the geotechnical program, based on existing site conditions and proposed fire-fighting plan, in decreasing order of importance:

- Install monitoring devices on top of and surrounding the landfill, particularly near the gas, water and sewer pipelines to allow monitoring of movements of the soils in the zone between the landfill and the pipelines.
- Provide preliminary geotechnical recommendations to enable the fire fighting crew to proceed with the required earthworks. These recommendations were to be confirmed or augmented as other information became available.
- Collect geotechnical information in order to assess 'safe' fill heights and side slopes at each of the proposed Hot and Cold pads and 'safe' offsets of these pads from nearby existing structures and utilities.
- Collect geotechnical information at the proposed heavy truck crossing(s) of the pipelines and provide recommendations, as required.
- Carry out stability analysis of the pads and the landfill to provide and confirm recommendations which protected site personnel, the pipelines, train tracks, and adjacent developed and undeveloped private properties. The analysis included establishment of 'trigger' levels for the instruments being monitored.

The monitoring devices included inclinometers installed in drill holes extending through the weak soils to the native sand at depth, piezometers installed into the weak soft soils,

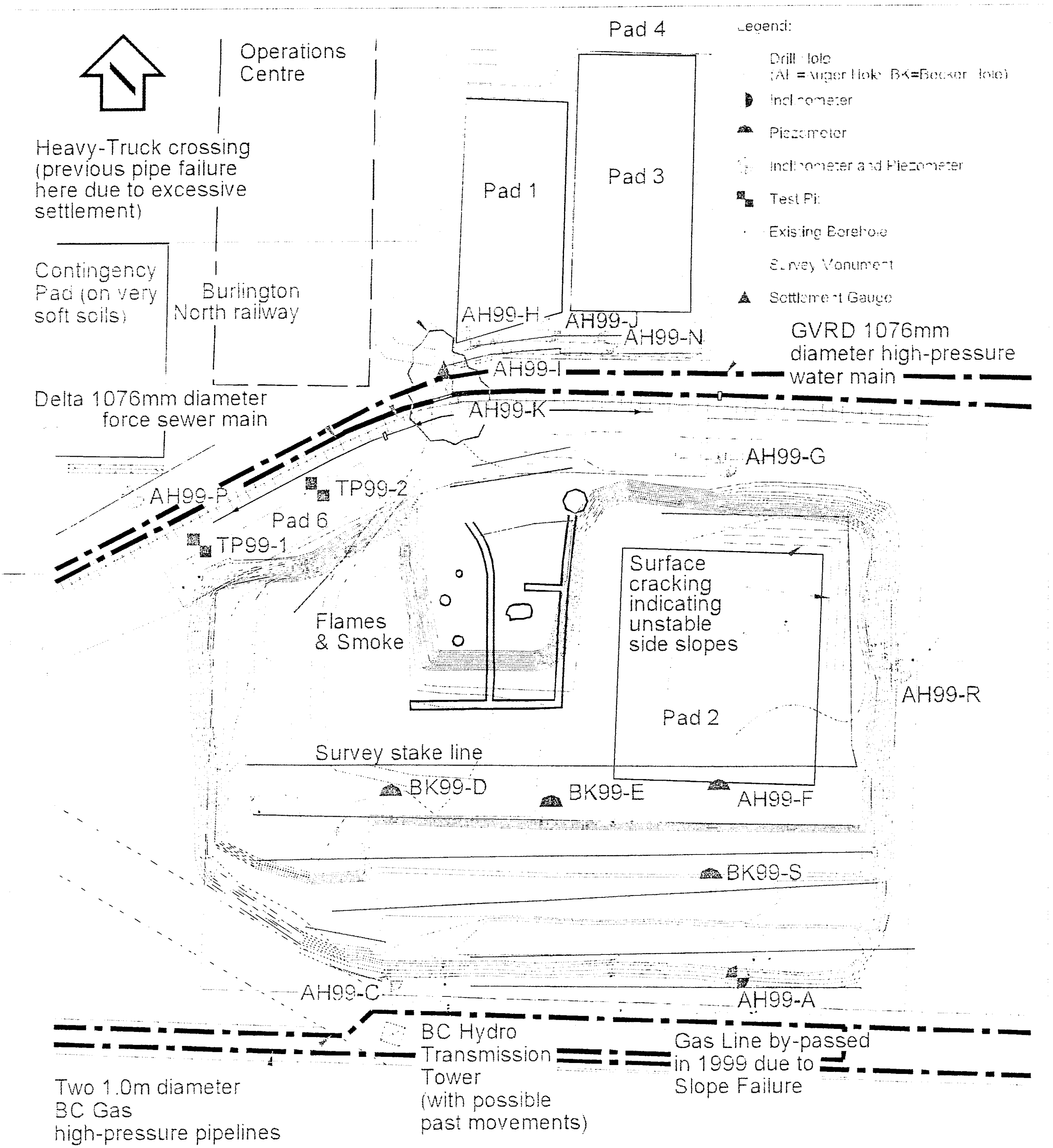

FIGURE: 1 SITE PLAN

Not to Scale

and surficial survey reference points. As well, a settlement monitoring gauge was installed at the heavy truck crossing an connected to a datalogger.

The geotechnical program objectives were prioritized on site in consultation with the Fire Emergency Command. Resources were deployed to meet these objectives in a manner which held safety of the public and the emergency response crew paramount.

Emergency geotechnical recommendations

It was not possible to delay the fire fighting earthworks until geotechnical assessment and instrumentation were completed. A number of preliminary geotechnical recommendations had to be provided to enable the fire fighting crew to proceed with the required earthworks. The following is a summary of these recommendations:

- Fill heights at pads 1, 2, 3 and the Contingency Pad should be less than 3 m.
- Side slopes of the pads should be maintained at no steeper than 1.0 Horizontal to 1.0 Vertical.
- Setback distances from the water and sewer mains should be no less than 25 m.
- Pad 2 should be constructed a minimum of 150m from the gas pipelines.

Cold fill material was stockpiled on top of the existing landfill at Pad 2. The following preliminary set-back values from the new cold fill to the crest of the slope of the existing landfill and to the gas pipeline to the south of the landfill, were provided:

- 25m from the top of the slope of the existing landfill on the north, east and south sides of the landfill, and
- 150m from the gas pipeline on the south side of the landfill

These recommendations were confirmed or augmented as appropriate information and analysis results became available.

Literature review

Prior to the landfill fire, a number of geotechnical consultants had carried out various investigations at and near the site. The reports from these investigations, although not necessarily related to the purpose of this project, provided a basis for the initial geotechnical recommendations and a way to compare results. In particular, it is very difficult to estimate the geotechnical properties of a relatively random landfill for modelling purposes. The information within some of these reports provided a starting point for the analyses.

Geotechnical investigation & instrumentation program

A geotechnical investigation and instrumentation program was carried out on the site from November 28, 1999 to December 22, 1999 which included the completion of:

- four Becker Hammer holes,
- thirteen auger holes,
- six Dynamic Cone Penetration Tests, and
- two test pits.

Geotechnical instrumentation installed in the drill holes included:

- 11 inclinometers,
- 3 standpipe piezometers, and
- 15 vibrating wire piezometers.

Survey monuments were installed in stake lines between December 6, 1999 and December 15, 1999 along the north side of the railway tracks, and the north, east and south sides of the landfill. The purpose of these was to monitor surface movements. Areas that were of particular interest included the gas, water and sewer pipelines.

Figure 1 shows the locations of the above and pre-existing borehole locations (with the inclinometers and piezometers noted), and survey stake lines.

Two test pits were completed to depths of about 6m by a tracked excavator. The maximum depths of the Becker Hammer and auger holes were 39.6m and 21.3m respectively. Drill holes were located at areas for which both geotechnical instrumentation data and soils information were required. In situ Nilcon Vane shear and pocket penetrometer tests were carried out on select samples. In addition, approximately 200 grab samples were collected for laboratory analyses.

A permeable sand layer was penetrated at depth beneath the landfills and underlying peat and soft silt during the drilling program. It was recognized that this is an environmentally sensitive area and there would be a potential for contamination of this sand layer by leachate from the landfill through our drill holes. To prevent contamination, the drill holes on and near the landfill were grouted or were backfilled with a low permeability mixture of sand and bentonite. For a similar reason, all instruments that were decommissioned were similarly grouted.

The DCPTs were carried out using track-mounted auger drill rigs. The DCPTs were generally carried out north of the landfill adjacent to the auger holes. In situ Nilcon Vane shear tests were performed at two of the Becker and two of the Auger test holes. These tests were performed to determine the current, in situ, undrained shear strength of the peat and silt beneath varying heights of the landfill. Table 1 indicates the drill hole, test depth, soil stratum being tested and strength results (peak and residual) for each test. Pocket Penetrometer tests were also perfomed on selected samples and were found to correlate well with the Nilcon Vane. These results were used as a basis for selecting key parameters for the geotechnical analyses.

Table 1: Vane Shear Results

Hole	Fill Depth	Depth (m)	Material Type	Su (kPa) Peak	Res.	Rem.
E	24 m	25.6	Organic Clay	140	-	42
E	24	26.5	Organic Clay	92	-	38
E	24	27.9	Organic Clay	96	-	25
E	24	31.1	Sandy Silt	155	57	33
G	6	6.4	Fibrous Peat	38	21	2
G	6	6.7	Fibrous Peat	49	31	15
G	6	7.9	Fibrous Peat	49	18	3
G	6	8.2	Fibrous Peat	58	19	14
G	6	9.5	Organic Silt	24	9	7
G	6	9.8	Organic Silt	44	16	9
G	6	11	Organic Silt	51	15	11
G	6	11.3	Organic Silt	40	12	4
G	6	12.5	Organic Silt	56	11	7
G	6	12.8	Organic Silt	59	23	14
G	6	15.5	Organic Silt	65	19	11
G	6	15.9	Organic Silt	63	25	17
R	5	5.3	Amorphous Peat	120	61	36
R	5	5.6	Amorphous Peat	93	40	20
R	5	6.7	Organic Silt	44	17	5
R	5	7.6	Organic Silt	45	16	11
R	5	7.9	Organic Silt	46	21	9
R	5	9.1	Sandy Silt	59	12	2
R	5	9.5	Sandy Silt	69	26	10
R	5	10.7	Sandy Silt	56	19	10
R	5	11	Sandy Silt	62	21	9
R	5	12.2	Sandy Silt	67	18	11
R	5	12.5	Sandy Silt	79	42	27
S	14.5	16.2	Peat	63	9	0
S	14.5	16.5	Peat/Org. Silt	192	120	56
S	14.5	17.4	Organic Silt	83	36	27
S	14.5	17.7	Organic Silt	103	44	16
S	14.5	18.3	Organic Silt	98	23	2

Laboratory analyses including moisture contents, gradation analyses (including sieve and hydrometer testing), Atterberg Limits and specific gravity analyses were conducted on selected samples in order to allow classification of the soils and define soil index properties for the geotechnical analyses. The results of the gradation analyses are summarized in Table 2.

Table 2: Sieve and Hydrometer Gradations

Sieve and Hydrometer Gradations				
Borehole	Depth (m)	% Sand	% Silt	% Clay
99-A	7.8	3.2	88.9	7.9
99-I	7.3	18.6	77.3	4.1
99-E	26.2	9.2	82.8	8
99-C	7.3	3.2	88.6	8.2
99-G	9.8	9.8	82.1	8.1
99-P	6.1	10.8	78.9	10.3
99-K	8.8	7.2	80.6	12.2
99-D	30.0	1.8	86.2	12
99-L	5.5	1.8	82.2	16
99-R	7.0	1.4	84.2	14.4
99-N	10.4	2	88.1	9.9
99-G	2.7	16.4	No Hydrometer	
99-E	34.1	0.1	No Hydrometer	

The results of the Atterberg Limits and Specific Gravity testing are summarized in Table 3.

Table 3: Atterberg Limits and Specific Gravities

	Atterberg Limits					
Borehole	Depth (m)	Plastic Limit	Liquid Limit	Plasticity Index	Type	Specific Gravity
AH99-G	9.8	60.1	67.1	7	MH	2.64
BK99-E	26.2	33.1	39.3	8.2	ML	2.68
AH99-C	7.3	35.8	52.1	16.3	MH	2.5
AH99-P	6.1	24.5	26.3	1.5	ML	2.63
AH99-K	8.8	81.9	93.5	11.6	MH	2.57

Geotechnical analyses

Geotechnical analyses were performed to verify that the redistribution of material into both Hot and Cold Pads, necessary to fight the fire would not result in unacceptable deformation of the landfill or the pads. Both limit equilibrium and finite difference modelling methods were utilized. In all cases, short-term stability was assessed, meaning that undrained strength parameters were used in materials with low hydraulic conductivity and drained parameters were used for the remaining materials.

The computer program SLOPE/W (GeoSlope International, 2000) was used to perform the limit equilibrium analyses, and the computer program FLAC (Itasca Consulting Group, 2000) was used for the finite difference analyses. FLAC is a versatile program that is capable of modelling post-failure plastic flow and includes large strain logic for computing deformations. The Mohr-Coulomb constitutive model was used in all analyses. In general, for the determination of the factor of safety using FLAC, small strain logic was used as this method more closely represents the approach taken in a limit equilibrium analysis. Large strain logic was used when a prediction of the magnitude of deformation was desired.

Strength and stiffness parameters were derived for the prediction of the response of both the landfill and the Hot Pads to the fire-fighting efforts. These parameters were developed by combining information from previous studies on this landfill with new site investigation results and back-analyses. For the Hot Pads, the plastic landfill liners were not modeled in the analyses, as the underlying foundation materials were viewed as the weak link in the system and the reinforcing effect of these liners was conservatively neglected.

The site investigations included in situ Nilcon Vane shear tests, summarized on Table 1. A conservative lower bound strength of 45 kPa was used in the analyses for the undrained strength of the organic silt/clay and the peat confined under more than 5 metres of sandy fill or under the landfill.

Table 4 presents the material parameters used in the predictive analyses, and also indicates the source of each parameter, whether from previous work and experience, in situ or laboratory testing or back-analysis.

Back analyses

Two back analyses were performed for existing landfill slopes of marginal stability to establish representative strength parameters for the various material.

The first back-analysis was of a steep, high portion of the existing landfill slope, where tension cracks and other observations indicated that the stability of a shallow slip surface in the landfill material was marginal. With the estimated landfill density, limit equilibrium analysis indicated that an effective cohesion (c') of 5 kPa and a friction angle (f') of 23° resulted in a factor of safety of about 1.0. This back analysis was repeated using FLAC with small strain logic and confirmed the effective stress parameters.

Table 4. Material Parameters

Material	r_{bulk} t/m³	Cohesion c', kPa [1]	Friction f', deg	Strength Source [2]	Shear mod. G, kPa	Shear mod. Source [2]	Poisson's Ratio	Bulk mod. B, kPa
new cold fill	1.22	5	23	BA	385	PW/E	0.3	835
existing landfill	1.01	5	23	BA	770	PW/E	0.3	1670
sand and gravel fill	1.79	0	30	PW/E	25000	PW/E	0.3	54200
unconfined peat, near south toe	1.1	11 / 6	0	BA	105	PW/E	0.2	140
consolidated peat, confined under fill	1.1	45 / 20	0	IST	105	PW/E	0.2	140
soft organic silt, near south toe	1.5	11 / 6	0	BA	1720	LT & PW/E	0.45	16700
stiff organic silt, confined under fill	1.51	45 / 20	0	IST	3450	LT & PW/E	0.45	33300
sand	1.8	0	35	PW/E	25000	PW/E	0.3	54200

1. peak / residual strength in the in-situ shear vane tests
2. Sources: BA=Back Analysis, IST=In Situ Test, LT=Laboratory Test, PW/E=Previous Work and Experience

These strength values were used for the landfill material in all subsequent predictive analyses.

To estimate the strength of the soft organic silt/clay and the peat along the south side of the landfill, a second back analysis was performed of a previous deformation that displaced one of the gas pipelines. At this location near the east end of the south slope of the landfill, the organic silt/clay and peat were present under the toe of the landfill slope and the relatively strong sandy fill was quite thin. In both limit equilibrium and FLAC analyses, it was found that a peak undrained shear strength of 11 kPa in these weak materials resulted in a factor of safety of about 1.0.

Back analyses with FLAC using small screen logic confirmed that a peak undrained strength of approximately 11 kPa results in a factor of safety of about 1.0 for the geometry existing at the time of the movement. In order to estimate the residual strength of the soft peat and organic silt/clay existing after the movement took place, FLAC analyses were completed wherein the strength of the materials was progressively decreased until the pattern and magnitude of deformation reasonably matched those of the actual slope movement. In this case, large strain logic was used and a reasonable match occurred when the strength of the soft materials was decreased to approximately 6 kPa. Results for this analysis are shown on Figure 2 and Table 5 summarises the results of this case.

Table 5. Comparison of model and measured displacements

Comparison Item	**Measured Value**	**Model Prediction**
Vertical drop of the crest of the slope (m)	0.6	1.5
Vertical heave at the toe of the slope (m)	1.5	1.2
Horizontal displacement of the BC Gas pipeline (m)	3	2

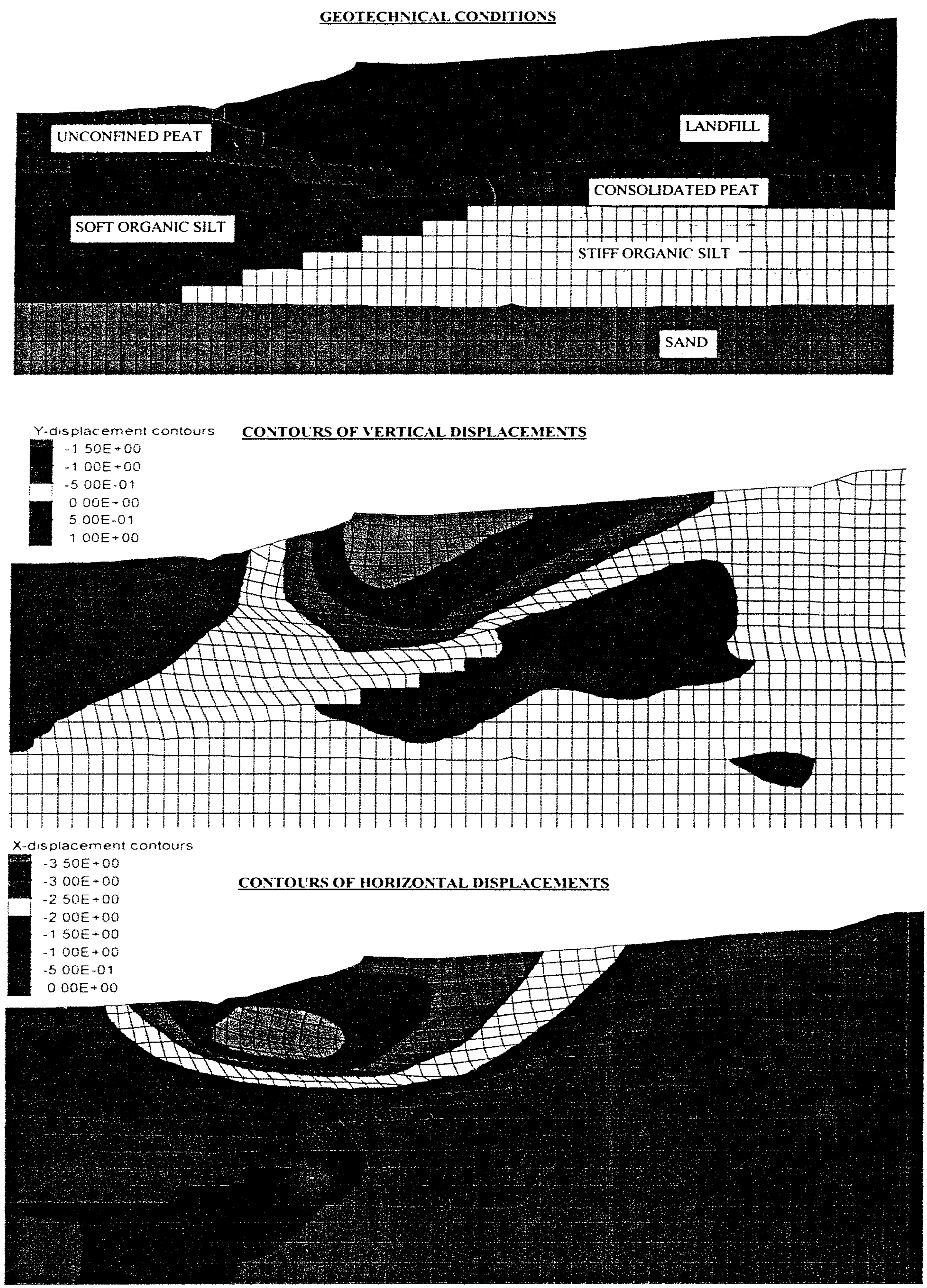

Figure 2. Back analysis of previous slope deformation

It is recognized that a better match between the model and measurements could be obtained through more complex material parameters and modelling. However, the primary goal was to determine appropriate strength parameters for predictive analyses, and the level of knowledge of the details of the slope movement was limited.

The strength parameters estimated from the back analyses were subsequently use to determine the likely effect of the addition of 3 metres of cold fill on top of the existing landfill.

Predictive analyses

Using the material parameters from the back analyses and the strength and stiffness parameters in Table 4, stability analyses and geotechnical modelling were completed. Limit equilibrium analyses were completed first to determine factors of safety against failure for short-term loading conditions for the addition of cold fill material to the landfill and for the new Hot Pads. Finite difference analyses were completed only for the main landfill, and provided more accurate factors of safety as well as predictions of movements due to placement of the cold fill on top of the main landfill. The predictions of movement were used to establish trigger levels for the inclinometers and survey monuments surrounding the main landfill. If actual movements exceeded the predicted values, then remedial action would have been immediately carried out.

Hot pads

A preliminary set-back value of 25 meters was provided for the distance from a typical 3 meter high Hot Pad to existing structures or roadways. There was concern that the rapid placement of material on the new pads could cause failure of the slope of the Hot Pad through the weak peat and silt/clay foundation materials. Limit equilibrium analyses were performed to verify that the proposed set-back and height of the Hot Pads would be acceptable. Two cases were analyzed to encompass the typical soil profiles identified by the site investigation work: Hot Pad 1 and the Contingency Pad. In both of these cases, a surficial sandy fill layer approximately 3 meters thick was modeled under the Hot Pad fill. The soft peat and silt/clay layers were conservatively assigned an undrained strength of 6 kPa, while the stiffer silt/clay layers were assigned a strength of 10 kPa. Factors of safety for short-term loading were above 1.2 and considered acceptable for the new Hot Pads. Further, potential failure surfaces did not extend out to the 25-meter limit, so the critical services were unlikely to be affected.

Main landfill

Preliminary set-back values were provided for the distance from the new Pad 2 to the crest of the slope of the existing landfill and the gas pipelines to the south of the landfill. These set backs were: 25 meters from the top of the slope of the existing landfill on the north, east and south sides of the landfill, and 150 meters from the closest gas pipeline on the south side of the landfill. Analyses were performed for the main landfill to ensure these preliminary set-backs were adequate for the addition of 3 meters of cold fill on top of the landfill.

Predictive analyses were performed to estimate factors of safety using limit equilibrium and finite difference methods for two main cross-sections. The first was the cross-section representing the south slope of the landfill where the previous movement had occurred and displaced the gas pipeline. There was concern over re-mobilization of this area, even though the slope of the landfill had been flattened after the slope movement occurred. The second section studied was representative of the north and east slopes, where the landfill slope was steepest and high. In all cases, the geometry used was that existing immediately prior to the fire, plus the proposed 3 metres of cold fill to be added using the set-backs described above.

For the north and south slopes, the factors of safety in limit equilibrium analyses exceeded 1.4 using peak strengths, indicating that the proposed placement of 3 meters of cold fill material on the landfill would not cause failure of the landfill. Using residual strengths the factors of safety exceeded 1.1, indicating that progressive failure was not predicted nor was re-mobilization of the previous movement at the south toe of the landfill. Hence, the preliminary set backs were deemed adequate and placement of the cold fill commenced.

As time permitted, finite difference analyses were completed for the main landfill to provide more accurate factors of safety as well as predictions of movements due to placement of the cold fill on top of the main landfill. Because of the considerable stiffness contrasts between the various materials in the landfill, finite difference modelling was considered necessary to accurately predict the response of the landfill to the addition of the new Pad No. 2. Using finite difference analyses with peak strength parameters, the factor of safety against a new failure of the north, south or east slopes exceeded 1.3. The failure planes did not extend back to the new cold fill, indicating that failure surfaces involving this new material have a higher factor of safety than 1.3. Figure 3 shows the results of the case for the north slope.

A further check was made for re-mobilization of the previous movements at the south toe of the landfill. In this case, large strain logic was used with the residual strength in the soft peat and organic silt/clay (6 kPa). The strengths were reduced in all materials until significant deformation was initiated. Figure 4 shows that these deformations did not extend back to the new cold fill, indicating that the failure block would not be affected by the addition of the cold fill.

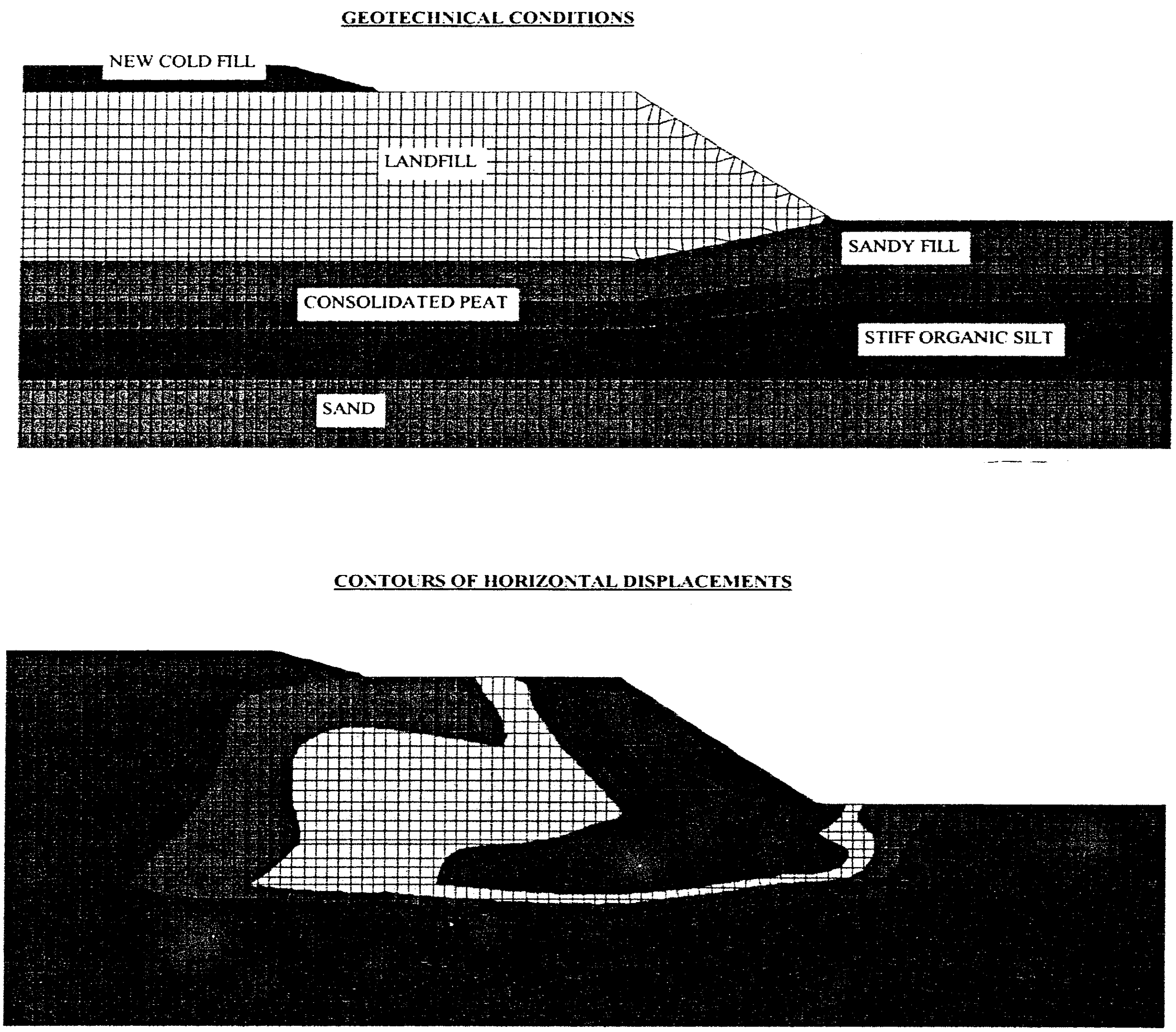

Figure 3. Finite difference analysis of north/east slopes

Based on consideration of the results of all the predictive analyses for the main landfill, it was concluded that the addition of the new cold fill to a height of 3 metres and at the proposed set-backs was acceptable. The reconstruction of the horseshoe area would be no higher than its original level before the fire, and to a lower level than the top of the new Pad 2. Hence, reclamation and refilling of the horseshoe area was also not predicted to result in failure.

Deformation and trigger levels

Having determined that the main landfill was not expected to fail from the addition of the new cold fill on top, it was necessary to estimate deformations of the landfill due to this addition. These estimates were completed with FLAC and provided trigger levels for the instruments installed around the landfill. Trigger levels for the hot and cold pads outside the main landfill were based on experience and extrapolation of the deformation analyses for the main landfill.

For the trigger levels analysis, the entire north-south cross section of the landfill was analyzed using large strain logic. The predicted deformations should be considered short-term movements, as undrained shear strengths and high bulk moduli were used in the low permeability materials such as the organic silt/clay. The results of the analysis are shown on Figure 5. Based on the analysis, trigger levels for the main landfill were set at measured movements of less than 25 to 50 mm. Movements greater than this amount would have been cause for concern. Table 6 presents estimated horizontal deformation values for the landfill and the expected direction of movement. Also shown on the table are the measured deformations and the locations of these points can be found on Figure 1.

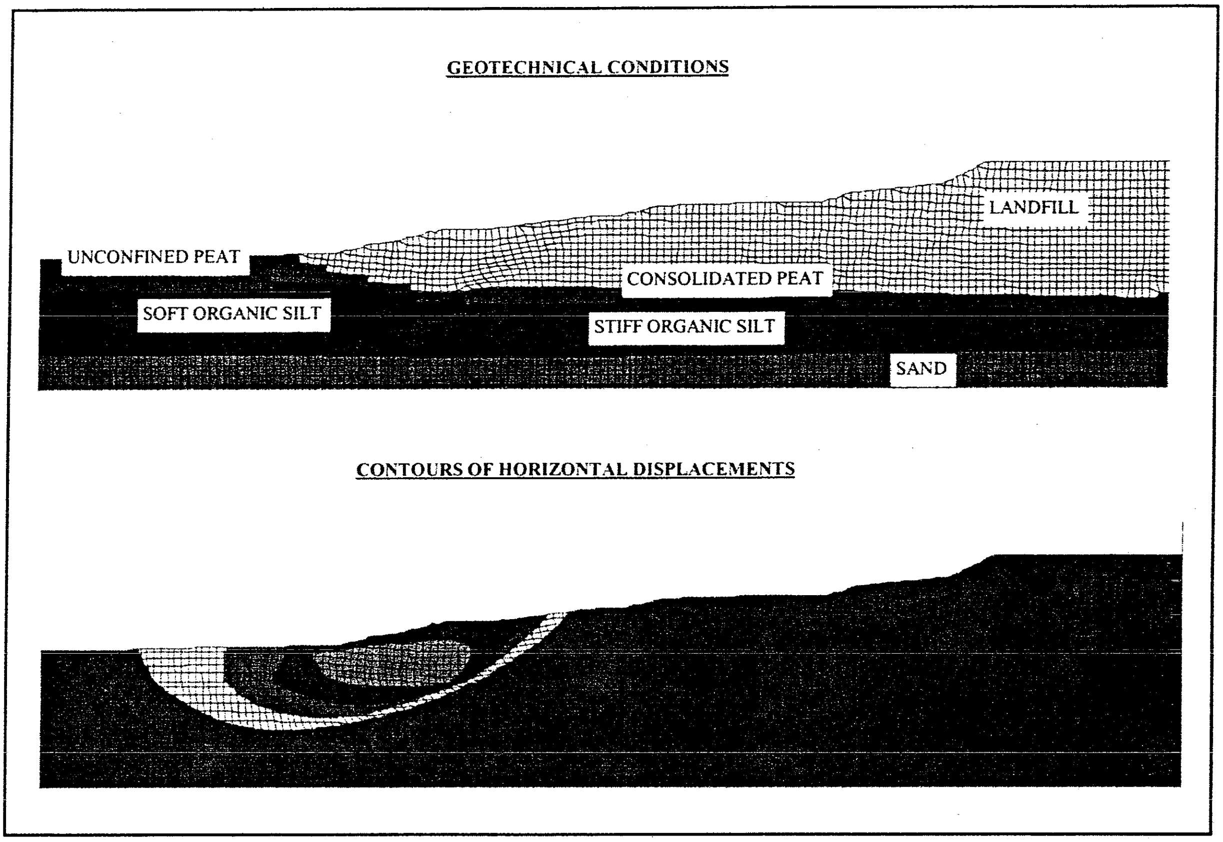

Figure 4. Zone of influence of movement block at the south toe.

Table 6. Deformation and trigger levels

Monitoring Point	Predicted Displacement	Actual Displacement
Soft foundation at south toe (AH99-A)	3 mm south	6 mm southwest
South crest (survey line S4)	4 mm south	1 to 7 mm random
Survey line S3	20 mm south	1 to 3 mm random
Soft foundation at inclinometer BK99-S	25 mm south	60 mm southwest*
Top of inclinometer BK99-F	10 mm south	3 mm north
Soft foundation at inclinometer BK99-F	60 mm south	12 mm southeast
North crest (middle of survey line N1, # 176)	5 mm south	1 mm north
Soft foundation at north toe (AH99-G)	30 mm north	15 mm north

* The displacement of the soft foundation at BK99-S was attributed to buckling of the inclinometer

In summary, monitoring indicated that addition of the new cold fill on top of the existing main landfill was causing deformations within the expected range of values and no remedial actions were triggered.

Observations and monitoring results

An estimated 14,400m^3 of hot waste and 43,200m^3 of cold waste was removed from the Horseshoe area and moved to the Hot and Cold pads, as appropriate during the firefighting activities which were on-going 24 hours per day and 7 days per week.

An Active Monitoring Program of the geotechnical instruments took place from the date of respective installation to the time that the fire fighting efforts were completed (effectively, Friday, January 14, 2000). Subsequently, an Ongoing Monitoring Program of the instrumentation was carried out as part of the restoration program the details of which are beyond the scope of this paper. Due to the high intensity of filling and moving operations, the instruments were monitored at a fairly high frequency (generally twice a week) during the Active Monitoring Program, to ensure that significant displacements which could indicate an imminent failure would be detected in time for remedial action.

Survey lines located along the south side of Pad 2 moved in the range of 0 mm to 25mm in a variety of directions with a range of vertical movement 40 mm upward to 50 mm downward. The western portion of the south side of the landfill had the greatest displacements.

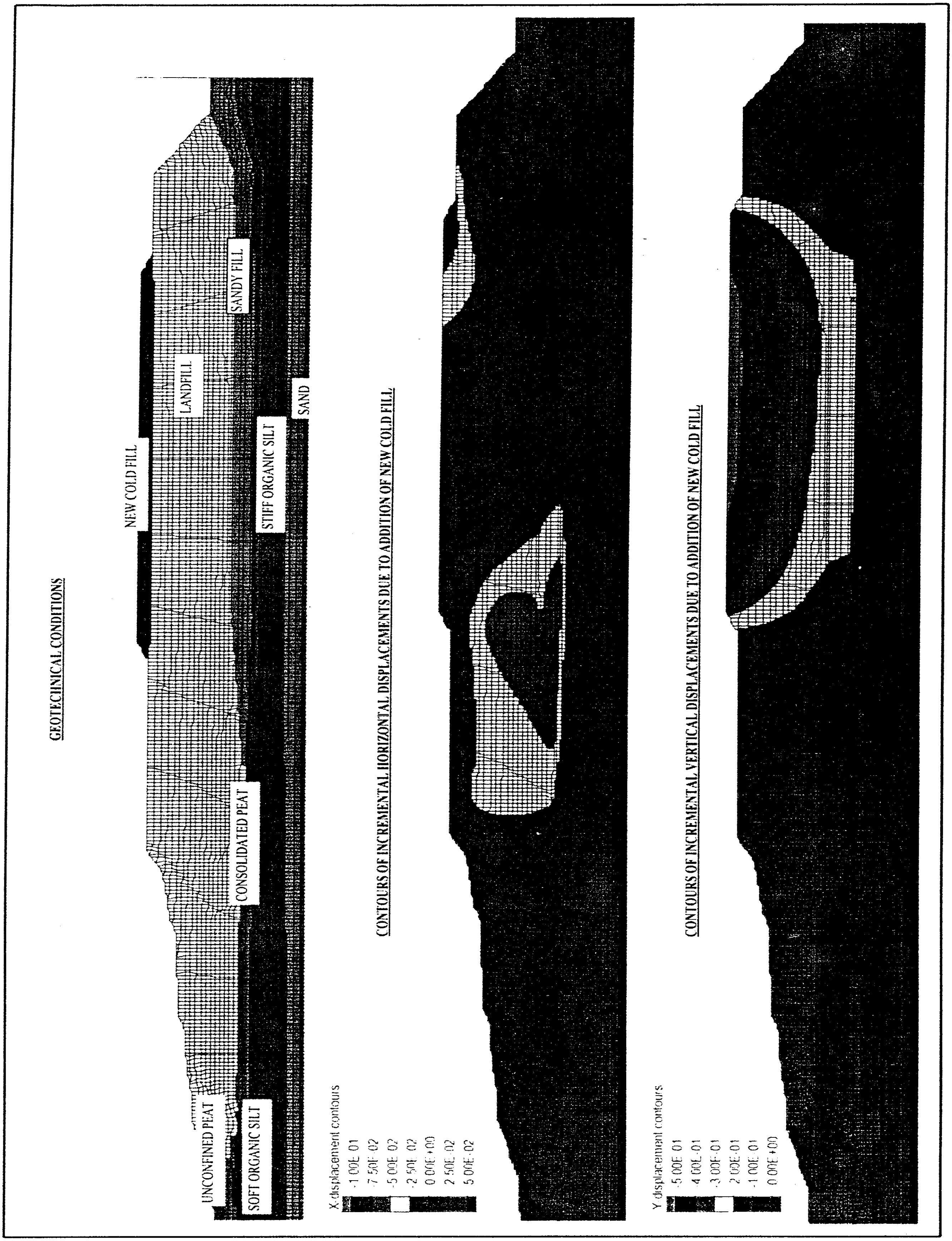

Figure 5. Additional movements due to addition of Pad 2

Survey lines along the north and east sides of Pad 2 underwent displacements of 10 to 45 mm to the southwest and 15 to 20 mm to the southeast along the north and east side respectively. The change in elevation was between 20mm to 70mm downward and 5mm to 30mm downwards along the north and east side, respectively.

Survey lines near the water and sewer mains (north of the landfill) near Hot Pads 1 and 3 underwent 5mm to 25 mm in a variety of directions and 10 mm upward to 10 mm downward. Survey lines near the Contingency Pad underwent lateral displacements of 5 mm to 30 mm southeast and 47 mm upward and 15 mm downward movement.

Movement measurements generally indicate that settlement and outward movement resulted from the filling at Pad 2. It appears that removal of the burning material caused displacements towards the Horseshoe area due to stress relief.

The pore water pressures at all piezometers generally decreased slightly from the start of monitoring to the end of the Active Monitoring Program. The standpipe readings within the landfill showed that the water levels fluctuated by about 0.25m 0.5m between December 4, 1999 and January 12, 2000.

During the Active Monitoring Program, most inclinometers showed no significant movement (ie measured movements were less than respective trigger levels defined as a result of the geotechnical analyses). For readings taken between the initial reading to January 10, 2000, two inclinometers located within the landfill closest to the Horseshoe, showed the greatest movement. Although the observed movements were small in magnitude, monitoring of these instruments continued during the Ongoing Monitor Program.

Inclinometer P showed movement toward the Contingency Pad prior to January 10, 2000. This movement was most likely due to the excavation of the drainage trench on the south side of that pad. There were no movements after January 10 when excavation and fill activities in the vicinity ceased.

Closure

In closure, it is concluded that the fire fighting efforts did not adversely affect the safety of the landfill, the neighbouring properties and utility corridors. This conclusion was predicted by the limit equilibrium and finite difference models and later confirmed with an intensive monitoring program.

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The work carried out as described in this paper was an efficient emergency response to a critical and complex geotechnical problem. The authors would like to thank our colleagues who deployed and rescheduled resources on behalf of this project. We would like to acknowledge the commitment of the individuals involved, many of whom sacrificed much personal time to the demands of the project schedule. Due to the urgency of this project, engineering personnel and resources from Horizon Engineering Inc, Knight Piesold Ltd, Pacific Geodynamics Ltd and Promatech were enlisted to assist Horizon Engineering with this project. In addition, Horizon called upon Foundex Exploration Ltd, SDS Drilling, Mud Bay Drilling Co Ltd, RS Technical Instrumentation, and McElhanney Associates to assist the project team with installation and supply of the geotechnical instrumentation, monitoring devices and surveying.

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