EMBANKMENT SLOPE STABILIZATION USING SUBHORIZONTAL DRAINS AT HIGHWAY 39 NEAR DRAYTON VALLEY, ALBERTA

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RÉSUMÉ
Une section de l'autoroute 39 traverse un glissement de terrain d'environ 200m de long et 300m de large. L'autoroute à deux voies, est construite sur une pente et une rivière est située en aval, au bas de la pente. Le revêtement de béton asphaltique de la chaussée a dû être réparé plusieurs fois à la suite de mouvements de terrain. Un plan pour la construction d'une voie montante le long de la pente a nécessité une étude géotechnique pour stabiliser la pente. Les lectures d'inclinomètre ont mis en évidence une zone de mouvement latéral située à 26m de profondeur. Le niveau des eaux souterraines a été établi au dessus ou près de la surface du sol. Vu l'envergure du glissement de terrain et les contraintes du site, l'abaissement du niveau des eaux souterraines a été choisi comme la méthode de stabilisation des sols la plus économique. Afin de réduire la pression interstitielle tout au long du plan de glissement, plusieurs drains souterrains horizontaux ont été installés. Des lectures de niveau des eaux souterraines prises après la construction indiquent que le niveau d'eau souterrain a été abaissé d'environ 5m et que l'intensité du mouvement des terrains a été réduite de façon significative.

ABSTRACT
A section of Highway 39 crosses a landslide that is about 200 m long and 300 m wide. The site consists of a two-lane highway constructed on a sidehill, bounded by a river at the toe of the slope. The highway experienced recurring distress that required frequent pavement patching. Slope inclinometer readings indicated a shear plane at depths of up to 26 m and groundwater levels were measured above or near the ground surface. Plans for construction of a climbing lane prompted geotechnical intervention to stabilize the site. Given the size of the slide and the site constraints, dewatering was considered the most cost effective mitigation option. Multiple arrays of subhorizontal drains were installed to reduce the pore pressure along the slide plane. Post-construction monitoring showed that these drainage measures have successfully lowered the groundwater levels by about 5 m and have significantly reduced the rate of slide movements.

1. INTRODUCTION

This paper describes a case history including the results of a geotechnical investigation and design measures to remediate a slide area affecting a section of Highway 39:06, km 12.6, located east of the town of Drayton Valley, Alberta.

The site consisted of a two-lane highway constructed on a sidehill, bounded by the outside meander of a 12 m wide creek (Modest Creek) at the toe of the slope. The elevation difference between the road surface and the creek is approximately 46 m. This section of the highway was experiencing recurring roadway distress as a result of a large landslide that required annual or biannual pavement patching to maintain a safe road surface. The slide crest is uphill of the highway in some areas and approaches the uphill ditch.

As part of Alberta Transportation and Infrastructure (INFTRA) Highway 39 rehabilitation program a climbing lane was proposed through the slide area. Addition of the fill required to build the climbing lane across the slide area raised concerns about increased mobility of the slide. This initiated a geotechnical program to evaluate methods to improve the stability of the distressed slope.

Geotechnical instrumentation consisting of slope inclinometers, pneumatic and standpipe piezometers, and groundwater monitoring wells were installed in 2001. These instruments provided information about the site conditions and slide mechanism, and were the basis for the assessment of potential remediation measures.

This site was added in 2002 to INFTRA's Geohazard Monitoring Program under the North Central Region and was designated site NC23 – Greenwood Slide (INFTRA 2002). At that time the site was rated with a risk level of 40 according to INFTRA's qualitative risk analysis procedure.

2. BACKGROUND

According to Swan (2002), the first distress on the roadway surface was noted around 1978-79. The distressed area was patched twice yearly until the early 1980's. In 1982-84 a trench drain about 100 m long and 2 m deep was installed within the uphill ditch adjacent to the distressed area, and a trench drain was also installed crossing the highway. These efforts reduced the patching rate to about once every other year. With time, the patching rate increased to yearly.

In : J. Locat, D. Perret, D. Turmel, D. Demers et S. Leroueil, (2008).
Comptes rendus de la 4e Conférence canadienne sur les géorisques: des causes à la gestion.
Proceedings of the 4th Canadian Conference on Geohazards : From Causes to Management. Presse de l'Université Laval, Québec, 594 p.

An 800 mm diameter centreline culvert was blocked, probably because of the slide, which caused water to pond in a low area adjacent to the uphill ditch. The pooled water was a continuous source of local groundwater recharge. Figures 1 and 2 show the site conditions.

Figure 1. Pooled water in south ditch at blocked culvert location.

Figure 2. Pavement distress, west flank of the slide, looking southeast.

3. GEOTECHNICAL INVESTIGATION

In 2001 a geotechnical site investigation was carried out. The results of the test holes indicated a soil stratigraphy consisting of a mantle of clay overlying medium plastic stiff to very stiff clay till, underlain by very hard, high plastic, slightly bentonitic clay shale/sandstone. During this program five slope inclinometers (SI), nine pneumatic piezometers (PN), one standpipe piezometer (SP) and two groundwater monitoring wells (GW) were installed at the site. The SI readings showed a shear plane at depths of 20 m to 26 m, extending into bedrock layers, with rates of movement up to 35 mm/year. Water levels above or near the ground surface were measured in most of the piezometers.

Figure 3 shows a cross section through the centre of the slide with the soil stratigraphy and inferred shear plane based on the depth of movement recorded in the SIs.

4. REMEDIATION MEASURES

Stability analyses were carried out using the limit equilibrium software GSlope. A back analysis indicated that the instability is primarily driven by high pore pressures, with artesian or near-artesian conditions occurring over the lower third of the slope. The back analysis also indicated that the bedrock was likely pre-sheared; and the shear strength of this material was likely near or at residual strength. Toe erosion at creek level was not found to be a significant factor in the slope stability. Additional stability analyses were performed to assess suitable remediation options to improve the slope factor of safety to a target of approximately 1.3.

The upslope pond formed by the centreline culvert blockage was a source of groundwater recharge. A new centreline culvert was considered most suitable for maintaining the upslope area dry. In addition to the pond dewatering, the following remediation options were considered to increase the factor of safety against slope failure:

a) toe berm;
b) pile wall;
c) slope unloading;
d) highway realignment; and
e) subhorizontal drains.

A toe berm was not considered feasible due to the presence of Modeste Creek at the toe of the slope. A pile wall would not be economically viable due to the significant depth of the shear plane. Slope unloading would not be effective due to the small change in driving force. Highway realignment would require a relatively long (2.5 km) stretch of highway to be constructed and was therefore not considered cost effective.

Reducing the pore pressure by installing subhorizontal drains was considered to be the most cost effective remediation measure. The stability analyses showed that by lowering the groundwater level by about 5 m the slope FS would increase by approximately 30 %.

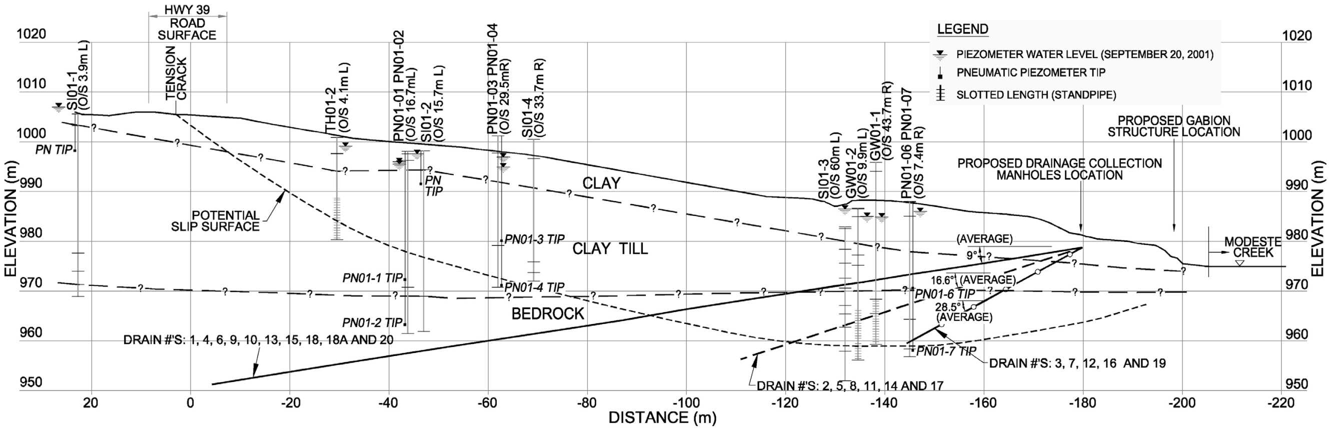


Figure 3. Cross section showing subsurface conditions, inferred shear plane, and subhorizontal drains.

Horizontal and subhorizontal drains have been used successfully on other projects to improve slope stability (Dharmawardene and Weimer 1988, and Proudfoot *et al.* 1998). Subhorizontal drains require periodic maintenance to keep the system in good operational condition and are often not relied upon as a sole remediation measure. Ideally they should be combined with some additional measure such as a toe berm to create some redundancy in case the performance of the drains deteriorates. However, due to the geometric restrictions a cost effective way to add redundancy measures to improve the slope stability was not available in this case.

There was a concern whether the ground permeability would be high enough to allow for effective dewatering due to the clayey nature of the overburden soil. To address this issue, two field permeability (slug) tests were carried out to allow for the estimation of the soil permeability through the bedrock/clay till interface zone. The results of the slug tests showed that the permeability of the formation was between 7.1×10^{-3} cm/s and 1.8×10^{-4} cm/s. This range was considered to be permeable enough to allow the subhorizontal drains to locally reduce the interface pore water pressure with time.

5. SUBHORIZONTAL DRAINAGE SYSTEM

The remediation work for this site started on May 3, 2005 with the installation of the subhorizontal drains. The construction was completed on June 14, 2005. The centreline culvert installation work for the dewatering of the upslope pond started on October 3, 2005 and was completed on October 24, 2005. A 750 mm diameter smooth wall steel pipe was pushed through the embankment and the uphill area was contoured to provide drainage to the new pipe.

The subhorizontal drain system consisted of two clusters of drains fanning vertically and horizontally as shown in Figures 3 and 4, respectively. Each drain cluster terminated in a drainage collection manhole consisting of a vertical corrugated steel pipe (CSP) culvert, 1.2 m in diameter, installed near the toe of the slope. Each manhole was constructed with a 300 mm diameter CSP outlet pipe that drained the groundwater through a gabion outfall structure installed along the river bank. A cross-connection between the two manholes was built to allow for continued drainage in the case that one of the manhole outlets became clogged.

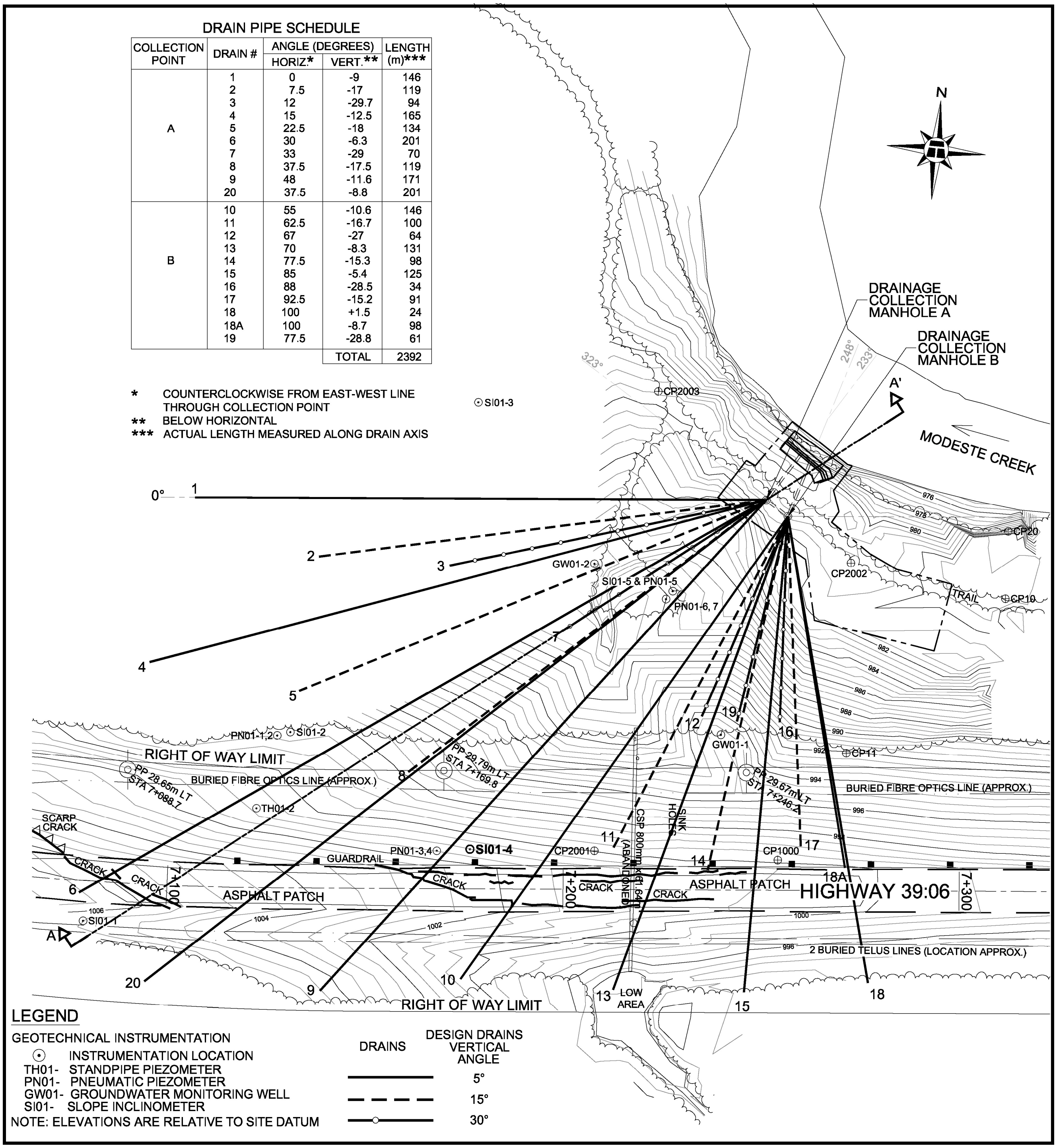
A total length of 2392 m of subhorizontal drains were installed, consisting of 21 drains with lengths varying from 24 m to 201 m. The drains consisted of 38 mm diameter schedule 80 PVC pipes, with 3 rows of 0.25 mm wide slots and 6.4 mm clear spacing between the slots within the rows. Each row of slots was centered at 120^{o} intervals around the pipe circumference.

The PVC pipe drains were installed using the Aardvark drilling system developed by Mobile Augers and Research Ltd., of Edmonton, AB, for the installation of subhorizontal drains. This system uses a disposable drilling bit that is left in the end of the drill hole once the design length is reached. The PVC pipe is inserted inside the drilling stem which provides casing for the hole. An anchor system installed at the end of the PVC pipe holds the pipe in place while the drill stem is retrieved.

The hole declination was monitored using a down hole survey tool that was put down the hole at desired drilling length intervals. The survey tool is only capable of tracking the vertical angle of the hole; previous experience with this equipment has shown that the hole does not deviate significantly in the horizontal plane.

All of the drains were installed without major problems. Some depth deviation in relation to the design target occurred, especially in the shallower angle drains (i.e., 5^{o} angle from the horizontal) however the intent of the design was achieved.

After completion of the 21 drains, seven were yielding water at a flow rate varying from 1 L/min to 60 L/min. Only two drains were yielding water several days after installation with an average flow rate of about 4 L/min.



DRAIN PIPE SCHEDULE

COLLECTION POINT	DRAIN #	ANGLE (DEGREES) HORIZ.*	ANGLE (DEGREES) VERT.**	LENGTH (m)***
A	1	0	-9	146
	2	7.5	-17	119
	3	12	-29.7	94
	4	15	-12.5	165
	5	22.5	-18	134
	6	30	-6.3	201
	7	33	-29	70
	8	37.5	-17.5	119
	9	48	-11.6	171
	20	37.5	-8.8	201
B	10	55	-10.6	146
	11	62.5	-16.7	100
	12	67	-27	64
	13	70	-8.3	131
	14	77.5	-15.3	98
	15	85	-5.4	125
	16	88	-28.5	34
	17	92.5	-15.2	91
	18	100	+1.5	24
	18A	100	-8.7	98
	19	77.5	-28.8	61
			TOTAL	2392

Figure 4. Site Plan showing subhorizontal drains layout.

Since installation these two drains have continuously yielded water with a few others seasonally producing water. Figures 5 and 6 show a general view of the inside of the manhole and the gabion outfall, respectively.

The total cost of the horizontal drain installation was approximately $380,000. The centreline culvert replacement cost was approximately $111.000. The total project costs, including engineering was about $ 597,000.

6. MONITORING

As part of INFTRA's Geohazard Monitoring Program the geotechnical instrumentation installed at this site is monitored twice a year. One set of readings is taken in early spring and the other in the fall.
Figure 7 shows the results of the piezometer readings before and after the remediation work. Of the twelve piezometers installed, seven measured a significant drop in groundwater levels. Three piezometers (PN01-2, PN01-4 and PN01-5) showed a relatively fast (5 month) response in lowering the groundwater elevation, and the others show a more gradual response with time. Since the end of the subhorizontal drain installation (May 29, 2005) the recorded drop in groundwater elevation varied from 4.6 m to 13.8 m. No significant rebound in groundwater elevation has been noted in the last 2 years of monitoring. In fact, piezometers PN01-7 and GW01-2 continue to show a slow but continuous drop in groundwater elevation.

Figure 5. Subhorizontal drain outlets.

Figure 6. Manhole outlets in gabion outfall structure.

Figure 8 shows cumulative and incremental movement readings from one of the site slope inclinometers (SI01-4) and Figure 9 shows the rate of movement plot from the same slope inclinometer. The rate plot shows that the slope movement measured in this SI prior to construction of the subhorizontal drains was approximately 13 mm/year, and dropped to near zero over the next three sets of SI readings following the implementation of the remediation measures. This result is typical of the slope inclinometer results obtained.

The instrumentation plots show a good correlation between the decrease in groundwater elevation and reduction in slope movement, confirming that the main destabilization factor for the slope instability was related to high pore pressures. The new climbing lane construction was completed in late fall of 2007. Stability analysis indicate that if the subhorizontal drains maintain the groundwater at its current elevation or lower, the addition of the new climbing lane should not cause a significant impact on the stability of the slope. The next instrumentation readings are scheduled for the spring and fall of 2008 which should quantify the impact of the climbing lane on the slope stability.

Based on the reduction in slope movement observed as a result of the implementation of the drainage measures, the INFTRA Geohazard risk level for this site was reduced to 28. Further reduction may be applied in the future pending on the assessment of the effect of the climbing lane on slope stability.

Ongoing monitoring and maintenance will be required to keep drainage system working and the slide from reoccurring.

7. CONCLUSION

A section of Highway 39 near Drayton Valley, Alberta was experiencing continuous pavement distress related to a deep-seated bedrock interface founded landslide. Frequent pavement patching over the years was required to keep the roadway surface in a reasonable condition. The proposed addition of a climbing lane at this section of the highway was the impetus for initiating the process for the design and construction of remediation measures to improve the stability of the slope.

Stability analyses indicated that the triggering failure mechanism was near artesian pore water pressure along the slide interface. A system of subhorizontal drains consisting of 21 drains up to about 200 m in length was installed in a fan configuration throughout the slide area. An upslope area of ponding water was also drained by augering a new 750 mm diam. smooth wall steel culvert.

Instrumentation consisting of piezometers and slope inclinometers was able to show an effective drop in the groundwater level in the slide area of up to 13.8 m. The drop in groundwater level and consequent reduction in pore pressure along the slide interface has successfully improved the stability of the slope as shown by the negligible rate of movement recorded in the SIs after construction.

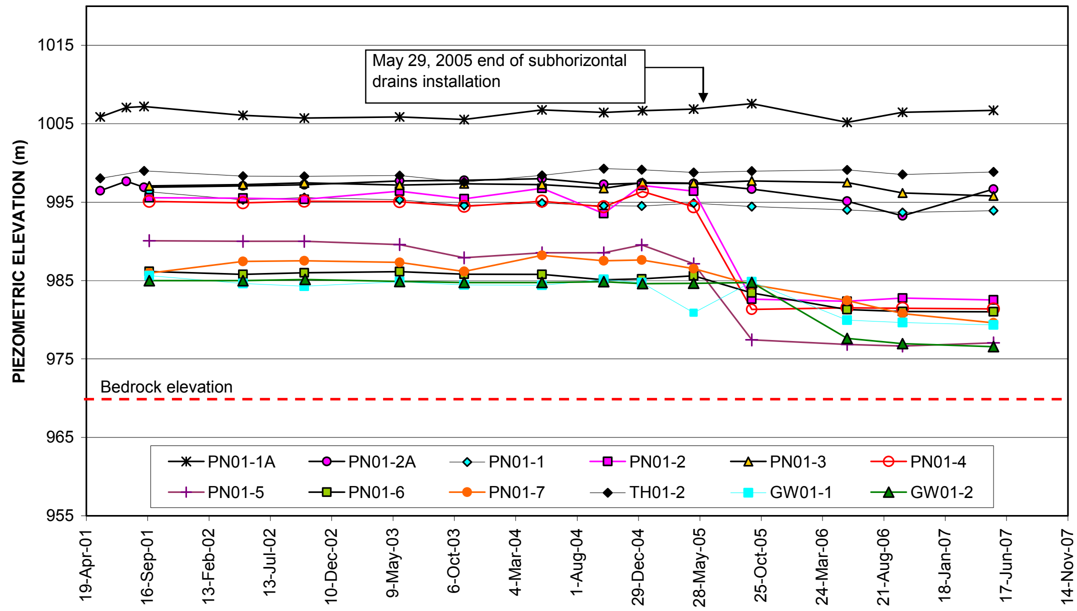


Figure 7. Piezometer readings before and after subhorizontal drains installation.

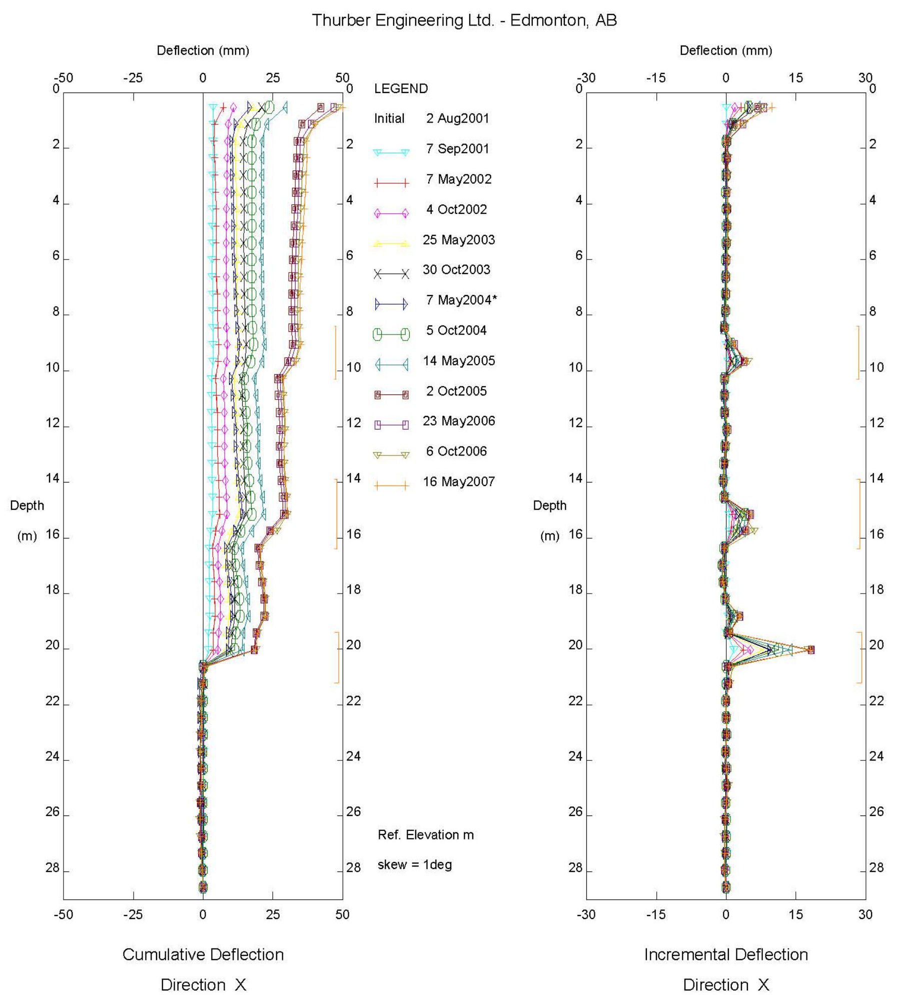


Figure 8. Typical slope inclinometer (SI01-4) cumulative and incremental deflection.

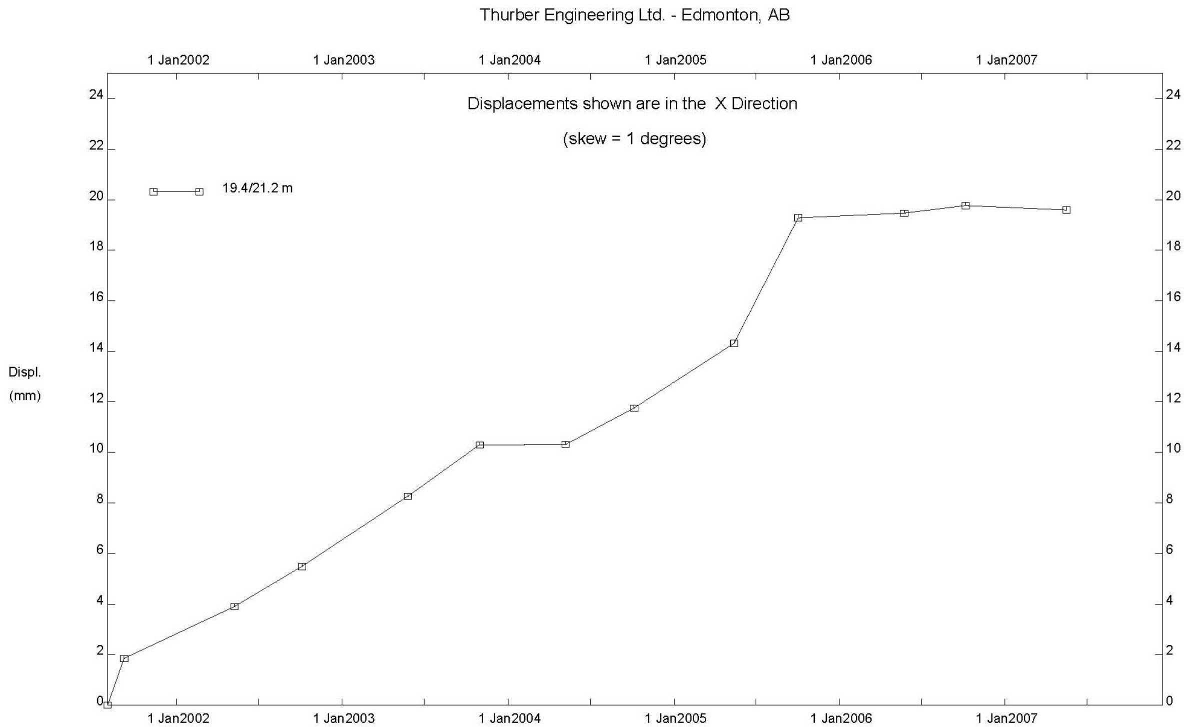


Figure 9. Typical slope inclinometer (SI01-4) rate of movement plot

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