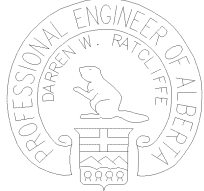


SITE NUMBER AND NAME C07 H16:30 Kenilworth Lake		HIGHWAY & KM	PREVIOUS INSPECTION DATE June 19, 2012	INSPECTION DATE May 15, 2013
LEGAL DESCRIPTION NE 28-50-4-4	NAD 83 COORDINATES N 5910800 E 532788		RISK ASSESMENT PF: 9 CF: 2 TOTAL: 18	

SUMMARY OF SITE INSTRUMENTATION: 4 Pneumatic piezometers and 1 standpipe operational as of 2002 (No instrument readings have been taken since 2002 when the slope was repaired.) LAST READING DATE: May 23, 2002	INSPECTED BY: 
PRIMARY SITE ISSUE: Cracking and settlement of shoulder requiring patching	
APPROXIMATE DIMENSIONS: 50 m long	
DATE OF ANY REMEDIAL ACTION: 2001 - granular berm placed at the toe of the slope keyed through the potential weak shear zone in combination with an overall flattening of the slope to about 6H:1V.	

ITEM	CONDITION EXISTS		DESCRIPTION AND LOCATION	NOTICABLE CHANGE FROM LAST INSPECTION	
	YES	NO		YES	NO
Pavement Distress	X		Cracking of the pavement and settlement of the shoulder was observed.		X
Slope Movement					
Erosion					
Seepage					
Culvert Distress					
COMMENTS					
Refer to previous reports and photographs.					
Install and inclinometer to monitor movement rate and elevation.					

September 13, 2013

Alberta Transportation
Central Region
#401, 4902 – 51 Street
Red Deer, Alberta
T4N 6K8

Mr. Dennis Grace, P.Eng.
Construction Engineer

Dear Mr. Grace:

**Central Region GeoHazard Assessment
Site C7 H16:30 Kenilworth Lake Slide
Site Inspection & Instrumentation Report**

The above site was inspected on May 15, 2013 by Mr. Darren Ratcliffe, P.Eng. of Klohn Crippen Berger Ltd. (KCB). A drilling investigation and instrumentation installation was performed by KCB between June 17 and 19, 2013. This report was prepared by KCB for Alberta Transportation Central Region under Contract No. CON0013499.

1 PROJECT BACKGROUND

1.1 Site History

The slide is located south of Highway 16 about 2 km East of Islay Junction along the north shore of Kenilworth Lake. The shoreline is located about 60 m from the highway and has a vertical elevation difference of about 15 m (approx. original slope of 4H:1V). The slide was first observed in 1977 and had no effect on the highway except for some cracks along the shoulder. The slide appeared to be a deep-seated rotational slide into the lake. Remedial works constructed in 1980 included a dumped pit run gravel toe berm about 25 m wide and 50 m long.

The slide was investigated in 1985/86 with the installation of slope inclinometers, pneumatic piezometers and standpipe piezometers. Horizontal drains up to 94 m long were also installed at this time. Twinning of the Yellowhead Highway past Kenilworth Lake was completed in 1990 or 1991 and followed the original highway alignment. In 1998, Highway 16 was re-paved, which included a nominal amount of additional fill at the crest of the slope to accommodate an increase in highway width. Patching on the shoulder was subsequently required in the fall of 1999, indicating that the slide was still active. It is understood that the addition of asphalt was required in the area from 1990 to 1999, with about one load of asphalt placed every 2 years over a 50 m long section of highway.

Patching on the shoulder at the slide location was regularly required in 1999, 2000 and 2001. Following the last patching work in August 2001, significant displacement was occurring in the driving lane adjacent to the lake. This lane was subsequently closed. Slide repair work at this site started on September 25, 2001 based on a design provided by KCB. The selected remediation design included a granular berm at the toe of the slope keyed through the potential weak shear zone in combination with an overall flattening of the slope to about 6H:1V.

The base for the granular berm was excavated to elevation 604 m to 605 m, as shown on Figure 2. The berm was constructed with an external slope of 3H:1V, a crest width of 2 m, and a back slope of 1.5H:1V. Five of the six horizontal drains were extended through the granular fill as it was raised. Rip rap was collected from the lake bed and placed on the face of the berm to protect the toe from possible future erosion. Using the granular toe berm as a starting point, the overall slope was flattened from 4H:1V to 6H:1V using compacted impervious fill. The slope was reclaimed with topsoil and seeded.

1.2 Existing Instrumentation

The existing instrumentation at Kenilworth Lake was last read in May 2002. The slope inclinometers were last read in April 1988. A summary of the status of the existing instrumentation on site is provided in Table 1. Operational instruments in 2002 were limited to 4 pneumatic piezometers and 1 standpipe piezometer. In 2013, these instruments were not functional.

Table 1 Kenilworth Lake Slide Instrumentation (May 2002)

ID	Old ID	Ground Elevation (m)	Tip Depth/Response Zone (m)	Stick-up (m)	Date Installed	May 2002 Piezometric Elevation (m)	Comments
Slope Inclinometers							
Ken01	1A	614.62	24 (?)	0.9	25-Apr-85	-	Sheared @ 4.9 m
Ken02	2	617.52	24.4	0.9	17-Oct-85	-	Destroyed
Ken03	3	611.88	20.7	1.0	18-Oct-85	-	Sheared @ 3.0 m
Ken04	1	607.40	15.2	0.8	24-Apr-86	-	Blocked @ 2.4 m
Ken05	2	609.18	15.2	0.8	25-Apr-86	-	Blocked @ 2.7 m
Ken06	3	612.61	21.3	1.0	28-Apr-86	-	Blocked @ 3.4 m
Pneumatic Piezometers							
P1		607.40	10.7	-	24-Apr-86	603.6	
P2		607.40	4.6	-	24-Apr-86	603.9	
P3		608.84	10.3	-	25-Apr-86	604.8	
P4		608.84	3.4	-	25-Apr-86	-	No return
P4A	P4	612.06	16.8	-	28-Apr-86	-	No return
P5		612.06	6.1	-	28-Apr-86	-	No return
P6		612.06	3.1	-	28-Apr-86	610.0	
Standpipe Piezometers							
SP 1A		614.65	19.8	1.13	26-Apr-85	610.8	
SP 2		617.59	1.5 - 21.6	0.31	17-Oct-85	-	Removed
SP 3		612.08	0.5 - 16.8	1.40	18-Oct-85	-	Blocked

1.3 Site Stratigraphy from Previous Investigations

Based on the results from previous investigations, the slope comprises about 10 m to 20 m of medium to high plasticity silty clay till overlying bedrock. Typically there is a 3 m thick layer of soft saturated silty clay (till) followed by a 9 m thick layer of medium plasticity sandy clay (till). Some logs indicated the presence of clay fill overlying the native till, but the thickness is not well defined. The clay fill likely resembles the clay till in terms of material type and properties. An estimate of the fill thickness is about 5 m below the highway and about 3 m thick at the mid-slope. The clay deposits are underlain by a very dense uniform sand or sandstone of the Upper Cretaceous Belly River Group.

Moisture contents of the clay till ranged from 15% to 25% with an average of about 20%. Liquid limits ranged from 45% to 75% with corresponding plastic limits between 18% and 25%. The logs noted that the till contained low strength high plasticity bentonitic and reworked shale layers.

Laboratory testing was conducted in 1980 and 1985 to determine the strength properties of the clay till. The peak effective friction angles for the till varied from about 25° to 36°, with corresponding effective residual friction angles varying from about 15° to 20°. Due to the displacement that has

occurred at the site, it is considered that the failure was occurring on a layer at which the residual friction angle has been attained.

2 SITE OBSERVATIONS

Prior to 2010, the site was last inspected in May 2002, the spring after construction was complete. Vegetation had established and the site appeared to be performing relatively well. However, cracking and settlement was observed on the shoulder above the slope in 2010. One crack extended into the driving lane and a semicircular area on the shoulder had dropped by a few centimetres.

By June 2012, the cracking extended almost to the centerline of the eastbound portion of the highway. Additional settlement, necessitating repaving, continued to occur in 2011, 2012 and 2013. However, no visible distress was observed on the slope.



Photograph 1 **Pavement Condition May 2013**

3 JUNE 2013 SITE INVESTIGATION

3.1 General

Two slope inclinometers and two standpipes were installed in the slide area as shown on Figure 1 between June 17 and 19, 2013. One hole was located at the road shoulder approximately in the centre of the semi-circular failure surface, (SI13-01) and one hole was located mid-slope near the fence line (SI13-02). Slope inclinometer casing and 25 mm diameter standpipes were installed at each location.

Drilling was undertaken with a tracked auger rig provided by Mobile Augers and Research Ltd. (MARL) of Edmonton, Alberta. All utilities were located through Alberta 1-Call protocol and companies were notified of drilling work taking place. A private utility locate was conducted by ProLine Locators of Lloydminster, Alberta, at the drilling locations.

During drilling, Standard Penetration Test (SPT) and grab samples were obtained at approximately 1.5 m intervals. All samples were bagged and transported to the Stewart, Weir & Co. Ltd. (SWG) soil laboratory for testing.

3.2 Soil Stratigraphy

Solid-stem auger drilling was completed to depths of 25.4 m and 21.2 m at the shoulder (SI13-01) and fence line (SI13-02), respectively. The soil conditions encountered can be generalized as follows:

- 0 – 1.2 m FILL: Clay, some silt to silty, trace fine sand, trace gravel, medium to high plasticity, soft to firm, brown, moist, iron oxide staining;
- 1.2 m – 13.6 m CLAY TILL (CI), silty, trace fine to coarse sand, trace gravel, medium plasticity, very stiff to hard (increasing with depth), brown to grey, moist (moisture content 20% to 30%). SPT N values about 10 blows per 300 mm;
- 13.6 m – 18.20 m CLAY TILL (CI-CH), silty, trace fine to coarse sand, trace gravel, medium to high plasticity (liquid limit about 60%), hard, clay shale fragments, grey, damp. SPT N values 20 to 30 blows per 300 mm;
- >18.20 m CLAY SHALE, silty, hard, grey, damp to dry (weathered bedrock).

Similar soil conditions were encountered at SI13-02; however, bedrock was encountered at a depth of 15.70 m

Test hole logs of SI13-01 and SI13-02 are provided in Appendix I. The descriptions applied to the various soil units as shown on the logs follow the Unified Soil Classification System. Such descriptions are judgmental in nature and may differ in detail from that actually encountered in the field. The descriptions noted in the test hole logs are based solely on inspections of soil samples recovered or cuttings observed on the auger flights. The actual nature of the materials between samples may vary.

3.3 Laboratory Testing

Laboratory testing was performed on selected samples by SWG in Sherwood Park, Alberta. The testing included natural moisture content and Atterberg limits. The results of the laboratory testing are provided on the test hole logs. The laboratory tests were performed following standard soil testing procedures or protocol, unless otherwise noted. The test results are to provide a general indication of some of the engineering properties of the material.

4 INSTRUMENTATION RESULTS

Baseline readings for the slope inclinometers were recorded on June 26, 2013 with a further reading set taken on August 16, 2013 (48 days later). As shown on the movement plots in Appendix II, SI13-01 indicates a shear movement of about 11 mm at a depth of about 8 m. SI13-02 indicates a shear movement of about 16 mm at a depth of about 7 m. The current movement rates are of the order of 80 mm/year to 120 mm/year. The inclinometers and estimate of the failure plane location are shown in Figure 2. SP13-01 recorded a water level of 6.4 m below ground surface and SP13-02 recorded a water level of 4.8 m below ground surface.

5 SITE ASSESSMENT

Movements appear to be occurring on a shear plane in the clay till located at a depth of about 7 m to 8 m below the existing ground surface. As noted previously, the till contains high plasticity bentonitic and reworked shale layers and the failure is likely located in a low strength zone. A stability analysis was undertaken on section shown in Figure 2 to determine the estimated friction angle of the material in the shear zone. Using the latest groundwater data, the stability results determined a clay friction angle of about 10°. This value is consistent with a high plasticity clay material at a residual strength.

Based on the risk level criteria provided by Alberta Transportation relating to safety, a risk rating of 18 was assigned to this site. This is based on a probability factor of 9 for an active slide and a consequence factor of 2 due to the potential partial closure of the road.

6 RECOMMENDATIONS

The slide movements have likely become more noticeable at this site due to the higher precipitation levels in recent years. Kenilworth Lake is now full of water while in 2001, at the time of the repair, the lake was dry. Stability analyses indicate to achieve a factor of safety of 1.5, a slope angle of about 10H:1V is required. This would require a fill volume of about 6,000 m³ and an estimated project cost of about \$150,000 to \$200,000.

Alternatively, a line of driven steel H piles could be provided along the shoulder for a distance of about 20 m (about 25 piles). Each pile would be about 15 m to 20 m long. The estimated cost of this

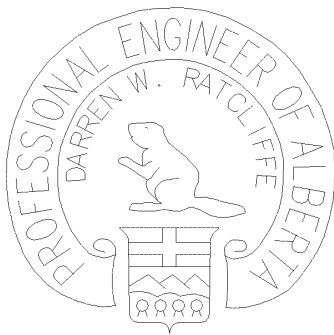
option is about \$100,000. The pile option offers a number of advantages over an earthworks approach including a lower cost, rapid construction, and minimal environmental impact.

7 CLOSURE

The analyses, conclusions and recommendations contained in this report are based on data derived from a limited number of test holes obtained from widely spaced subsurface explorations. The methods used indicate subsurface conditions only at the specific locations where samples were obtained or where in-situ tests would infer, only at the time they were obtained, and only to the depths penetrated. The samples and tests cannot be relied on to accurately reflect the nature and extent of strata variations that usually exist between sampling or testing locations.

This report is an instrument of service of Klohn Crippen Berger Ltd. The report has been prepared for the exclusive use of Alberta Transportation for the specific application to the Highway 16 slide repair. The report's contents may not be relied upon by any other party without the express written permission of KCB. In this report, KCB has endeavoured to comply with generally accepted geotechnical practice common to the local area. KCB makes no other warranty, express or implied.

Yours truly,
KLOHN CRIPPEN BERGER LTD.



Darren W. Ratcliffe, P.Eng.
Project Manager

APEGA Permit to Practice No. 9196