Identifying Pre-Existing Shear Surfaces and Slope Stabilization - A Case History Spanning Almost 30 Years

Mahmoud Mahmoud, PhD, PEng, FEC
President, GES Geotech Inc., Vancouver, BC

Sadaf Sanii, EIT
Junior Engineer, GES Geotech Inc., Vancouver, BC

ABSTRACT The Bear Creek Village in Grande Prairie, Alberta is located adjacent to the east crest of Bear Creek valley. The apparent problem was related to the location of top-of-bank line and building foundation setbacks from the slope crest running along the western half of the development site and assessment of the risk of slope instability and its potential impact on the safety of the residents at Bear Creek Village. However, the main problem was later identified to be something much more critical. Closer look and review of some of the boreholes that were drilled for previous studies, prior to 1995, showed evidence of slickensides within the clay deposits. An extensive geotechnical borehole investigation program and CPT testing was initiated to confirm the extent of pre-existing shear zones and/or failure surfaces. The investigation program confirmed the presence of pre-existing shear zones and failure surfaces, which were later incorporated into slope stability analysis that indicated instability was along a non-circular slip surface. Setback distances corresponding to factors of safety of 1.5 were in the range of 35-45 m beyond the crest as compared to the initial disagreement between respective consulting engineers, which were debating about setback of 5 m versus 10 m. Given that the condominium units had already been built on site, establishing 35-45 m setbacks would have meant expropriation of homes, which was unacceptable to all parties including the homeowners. A shear key system was developed that enabled achieving factors of safety that were satisfactory to the City of Grande Prairie, as jurisdiction having authority for public safety.

Introduction

Site Location and History

Bear Creek Village is located in the southern part of Grande Prairie, Alberta. It is bordered by the crest of Bear Creek valley to the west, 68th Avenue to the south, and 100th Street to the east. Figure 1 is the key plan, which shows the location of the previously recorded landslides in the general area of the site. Figure 2 is a site location plan showing the location and topography of the subject slope and locations of relevant boreholes drilled during previous geotechnical investigations. The stability of the slope at cross-section A-A', located to the east of Building Unit 35, was the focus of the slope stability analysis.

Tension cracks were reported to have developed downslope from Unit 35. A primary focus of this study was to determine what had caused the subject tension cracks.

Fig. 1. Key Plan

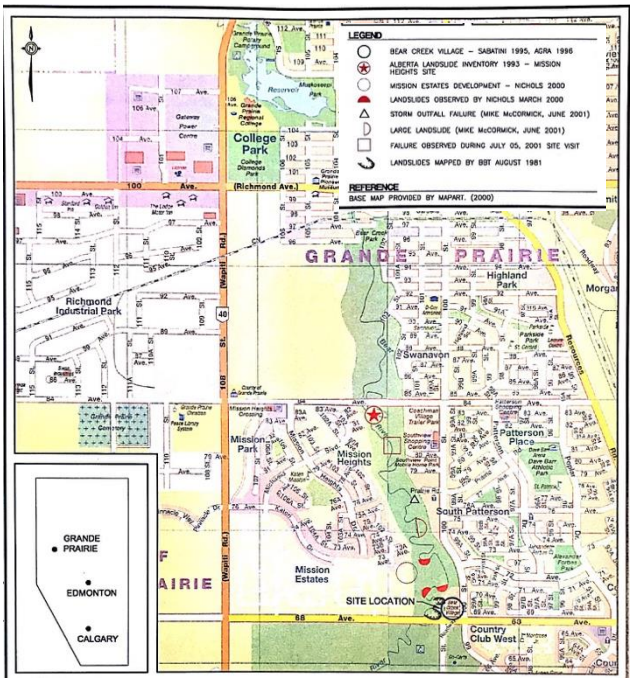
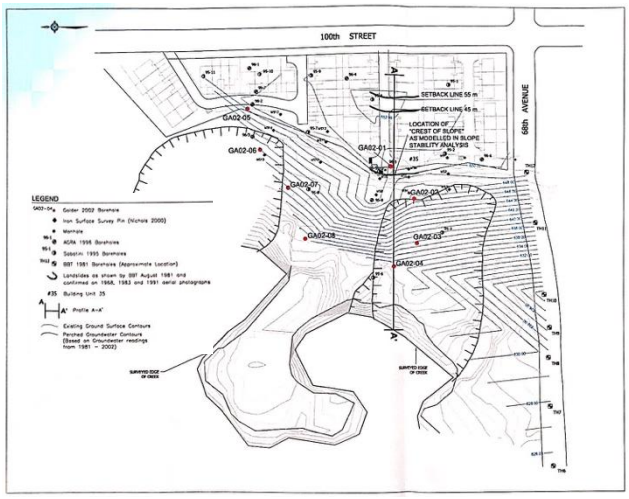


Fig. 2. Site Location Plan



Geology

Bedrock topographic maps indicate that the bedrock beneath the subject site is flat and sloping at approximately 0.8 percent towards the southwest, meaning that there is a slight dip towards Bear Creek. In the area of the site, bedrock lies approximately 30 m below the ground surface.

Surficial geology of the site consists of varved lacustrine clays with intervals of clayey silts, and clayey sands underlain by clay till. The lacustrine clay is low to medium plastic, stiff to very stiff with some intervals of firm consistency. The lacustrine clay can be described as normally consolidated to lightly over-consolidated.

Historical Background and Review

Air Photo Interpretation

Review of aerial photographs includes the photos taken in 1968, 1976, 1980, 1983, 1991 and 1997. The following is a summary of visible activities, landform changes on the adjacent slope, and creek configuration adjustments:

- 1968

The subject site appears to be used for stockpiling construction material and equipment storage. Fill has been placed on top of the slope crest for the full length of the subject site. A 60-m wide landslide, which initiated at the crest of the slope, is visible near the south boundary of the property. The 1968 air photo is used as the baseline creek configuration for the comparison with the following air photos; see Figure 3.

- 1976

Stockpiling and storage of construction equipment are continued. A visible surface disturbance, possibly a landslide, is observed above the creek in the northern half of the site. Creek configuration is changed from that of the 1968 photo with two of the creek meanders showing more profound bends.

- 1980

Less construction material and equipment storing is observed. The quality and coverage of the air photo prevents further comments on creek configuration and slope state.

- 1983

Stockpiles and construction material are removed from the site but the two buildings remain on the northern portion of the site. The site appears re-graded and leveled. There is an approximately 30 m wide landslide in the southern portion of the site. Bank instability and a small landslide are observed on the east creek bend near the northern boundary of the site. 68 Avenue right-of-way and a strip of land parallel to 100 Street have been cleared; see Figure 4.

- 1991

The site is completely graded and leveled with few buildings remaining at the northern tip of the site. Two landslides covering approximately the full width of the site are visible on the slope below the site; see Figure 5. Creek configuration has been changed due to the landslide as a meander has been obscured and a new channel is formed.

- 1997

Phase I of Bear Creek Condominium Project is constructed with additional ongoing construction. Affected slopes are re-vegetated with grass and no further creek change is observed.

Figure 6 is a summary of major features shown on Figures 3-5. Figure 6 illustrates retrogressive behaviour of landslides over the period 1968-1991.

Fig. 3. Air Photo 1983

Fig. 4. Air Photo 1968

Fig. 5. Air Photo 1991

Fig. 6. Historical Landslides 1968-1991

Review of Previous Reports

In 2001 a review was completed of previous geotechnical investigations that were carried out at the site in 1995, 1996, 1999 and 2000. The previous borehole locations are shown on Figure 2. The stratigraphic cross-section (A-A) along Building Unit 35 (Phase 3 of Bear Creek Village) is shown on Figure 7. Included in Figure 7 is the following key geotechnical information:

- Location of slickensides identified at the location of boreholes.
- Depths at which the moisture content profile shows a significant increase and/or sudden drop in value. This profile was taken from the original 1968 borehole logs.

Borehole logs indicate the presence of slickensides within lacustrine clay deposits in a new development north of Bear Creek Village Condominiums (refer to Figure 1). Figure 7 is a stratigraphic cross-section of subsurface conditions at the site. The slickenside depths are included in Figure 7.

Site Investigation

Field and Laboratory Tests

The most recent geotechnical field tests (in 2002) included the cone penetration test (CPT), sampling by conventional geotechnical drilling, Shelby tube sampling, standard penetration testing (SPT), in-situ suction measurements, and geotechnical instrumentation installation and monitoring (including piezometers and slope indicators).

A drill rig was used for reaction to push the electric cone below the surface. A 10-tonne compression type cone was used with readings at every 2.5-centimetre. There were a total of eight CPTs to depths ranging from 9.8 to 20.7 m. All CPT locations are shown on Figure 2.

The conventional geotechnical drilling program was completed using a truck-mounted auger rig. Boreholes were advanced 1.5m away from the CPT locations. Due to the soft ground conditions, borehole GA02-04 was advanced 6 m from CPT-4. Depth of each borehole ranged from 7.7 to 16.8 m. Standard penetration tests (SPT), Shelby tubes and grab samples were collected at minimum of one sample per metre. All borehole locations are shown on Figure 2.

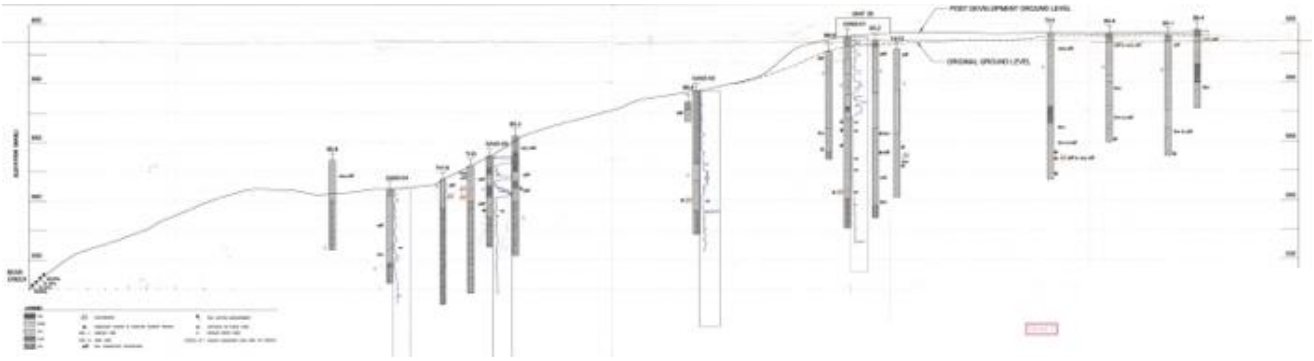
Pneumatic piezometers were installed within the till layer in boreholes GA02-02, 04 and 06. Standpipe piezometers were installed in boreholes GA02-02, 04, 06, 07 and 08 to assess the perched groundwater conditions.

Slope indicators were installed to depths of 16.3 and 7.7 m in boreholes GA02-01 and 03 to measure the slope movement due to subsurface displacements.

The laboratory tests included the following:

- Determination of natural moisture content on all grab samples and selected samples from Shelby tubes;

Fig. 7. Profile A-A'



- Atterberg limits on undisturbed specimens and clay samples corresponding to the shear zones;
- Grain size distribution by hydrometer method in silt samples from softened zones;
- Isotropically consolidated undrained triaxial test on one undisturbed clay sample; and
- Direct shear test on a sample containing a pre-sheared surface.

Subsurface Conditions

The site lithology is as follows based on the available borehole and CPT information:

- Fill

At borehole GA02-01, the fill consisted of 0.7m sandy gravel fill forming part of the pavement structure at that location. At borehole GA02-05, the fill consisted of organic clayey silt with trace sand and gravel. At other borehole locations fill varied from stiff to compact with 1.0 to 6.2 m of multiple layers (silty clay, silty sand and sandy silt).

- Lacustrine Deposits

At seven borehole locations lacustrine deposits were found just below the fill. At the crest, the thickness of the lacustrine deposits varied between 8.5 and 13.1 m; this is evident in the GA02-01 borehole log and CPT-5 results. On the slopes, the thickness of the lacustrine deposits varied between 1.8 and 7.1 m. Lacustrine deposits consisted of firm to stiff, moist silty clay with layers of clayey silt, silt, sand and soft clay with varved homogeneous and till-like texture. SPT N-values in this layer were between 9 and 15 with an average of 12. Moisture content was between 16.9% and 47.7% with an average of 28.3%. Liquid limit, plastic limit, and plasticity index for the lacustrine clay outside the softened/shear zones were 38%, 18%, and 20% respectively, whereas in the softened/ shear zones the values were 73%, 45%, and 28% respectively.

- Till

Very stiff silty clay till was encountered at depths of 5.2-13.8 m in six of the boreholes. Till was moist with trace of sand and gravel. SPT N-values of this layer were between 22 and 30 with an average of 26. Moisture content varied from 18.2% to 23.4% with an average of 19.5%.

- Groundwater Conditions

Piezometeric groundwater conditions were found to be 7.4 to 10.5 m above the till horizon (i.e. corresponding to 5.3 – 9.1 m below ground surface). The perched groundwater conditions were found to be 0.2 to 9.7 m below the surface. Elevation of groundwater was found to vary seasonally and from year to year due to fluctuations of precipitation and changes in the regional groundwater regime, which was affected by nearby, upslope residential development.

Analysis

Softened Zones and Shear Zones

Prior to the geotechnical drilling program, depths to the softened shear zones were identified and used for targeting the geotechnical sampling program. CPT profiles were interpreted using the method developed by Mahmoud, Woeller, and Robertson (2000). Undisturbed samples were collected at depths indicated by the CPT soundings as shear/softened zones. Softened zones were identified in silt and clay layers while shear zones were identified in medium to high plastic clay layers along with slickensides.

A stratigraphic profile A-A' is shown on Figure 7. It includes previous borehole information along with the results of CPT soundings and borehole information gathered in May 2002. Significant changes in moisture content values, locations of slickensides, softened zones, shear zones, and groundwater profile are included on Figure 7.

Based on the information collected from the borehole logs, CPTs, and laboratory testing, softened zones and shear zones were found to exist at the crest and along the slope bordering the Bear Creek Village Condominium site.

Slope Stability Analysis

Stability analysis was carried out based on slope geometry, soil strength and groundwater conditions as discussed above. The purpose of the slope stability analysis was to obtain:

- Factors of safety under the conditions that were current at the time of the most recent (2002) geotechnical investigation program

- Setback distances (relative to the slope crest) that would correspond to a factor of safety of 1.5
- The effect of groundwater fluctuations
- Options that would ensure the sustainability and long-term performance of the existing buildings.

The 2-D limit equilibrium slope stability analysis software SLIDE was used for the purposes of analysis. Calculation for factors of safety was completed using the Bishop simplified method and the General Limit Equilibrium (GLE) method of slope stability analysis.

Slope Geometry and Slip surface

The slope along the cross-section A-A' is approximately 21 m high and 130 m long. There are two sections of the slope, one section runs from the creek for about 40 m and the other slope from borehole GA02-04 to the crest. There is a relatively flat bench between these two slopes. For the purposes of this report the overall slope geometry is considered for analysis. The slope grades are approximately 4.5 horizontal to 1 vertical (4.5H: 1V) for the Lower Slope, 5H: 1V (approximately 11° relative to the horizontal) for the Upper Slope, and 6H: 1V with slope angle of approximately 9.5^o relative to horizontal for the overall slope. Slope stability analyses were performed for the Upper, the Lower and the overall slopes. Slope model was presented using the subsurface conditions described in a preceding section.

Soil Properties

- Fill

Soil properties for the fill were assumed based on CPT and geotechnical drilling program:

Unit weight: 17 kN/m^3
Effective internal friction angle: 25°
Effective cohesion: 0

- Lacustrine Deposits

Peak shear strength parameters were obtained from triaxial testing:

Unit weight: 18 kN/m^3
Peak effective internal friction angle: 23°
Peak effective cohesion: 6 kPa

- Pre-sheared and/or softened zones

Residual strength parameters were obtained from a direct shear test performed on a sample containing a pre-sheared surface:

Unit weight: 18 kN/m^3
Residual effective internal friction angle: 8°
Residual effective cohesion: 0 kPa

- Till

Shear strength parameters for the till were estimated based on CPT and geotechnical drilling program:

Unit weight: 19 kN/m^3
Effective internal friction angle: 25°
Effective cohesion: 10 kPa

Groundwater Conditions

For the analysis both the existing (as at 2002) and perched groundwater elevations were used.

Existing (as at 2002) groundwater conditions based on perched water table and the regional groundwater table were gathered from previous measurements in previous reports. The "current" perched water table corresponds to profiles shown in Figure 7. This profile runs at depths 4.5-5 m below the development site.

High perched groundwater conditions were based on the highest water table recorded at depths of 2.7 m across the slope. Profile of the high perched water table is shown in Figure 7. This profile runs 2.3 m below the development site and continues towards the crest and downslope to the flat bench; perched water table then drops to a depth of about 7 m along the flat bench.

Due to limited available data on regional groundwater readings the same profile as that of the current groundwater conditions was used for high perched groundwater condition.

Results

Calculated Factors of Safety

The analysis results are summarized in Tables 1 and 2. Hypothetical setback distances corresponding to a factor of safety of 1.5 had been established from the analyses and are also included in Tables 2 and 3. Implementing a setback distance was one of the possible options that could provide a safety factor of 1.5 or greater along the slopes at the Bear Creek Village Condominiums.

Table 1: Factors of Safety Using Peak Strength Parameter for Lacustrine Deposits

Groundwater Table	Slope Section	Minimum Factor of Safety	Setback from the Crest for a Factor of Safety of 1.5
Current	Overall Slope	1.29	45 m
	Upper Slope	1.40	
	Lower Slope	1.30	
High Perched	Overall Slope	1.28	50 m
	Upper Slope	1.38	
	Lower Slope	1.30	

Table 2: Factors of Safety Using Residual Strength Parameter for Lacustrine Deposits

Groundwater Table	Slope Section	Minimum Factor of Safety	Setback from the Crest for a Factor of Safety of 1.5
Current	Overall Slope	1.18	50 m
	Upper Slope	1.17	
	Lower Slope	1.04	
High Perched	Overall Slope	1.17	55 m
	Upper Slope	1.16	
	Lower Slope	1.00	

The analyses yielded values of factor of safety ranging from 1.00 to 1.40.

Factors of safety of 1.17 to 1.40 were obtained for the Upper and the Overall slopes, indicating that these slopes are stable under the current groundwater conditions.

Values of factor of safety were found to range from 1.04 to 1.30 for the Lower slope, indicating that the Lower slope is also at a stable condition under the current groundwater conditions.

Effect of Groundwater Fluctuations

As demonstrated in Tables 2 and 3, high perched groundwater conditions, which were based on the highest groundwater table recorded at the site, were also used for analysis. Maximum fluctuations in the perched conditions were found to correspond to approximately 2 m to 3.5 m below the surface.

The analysis results indicate that fluctuations of the perched water level do not significantly affect the current stability of the slopes, with values of factor of safety reducing from 1.29-1.40 to 1.28-1.38, showing little or no change as a consequence of the maximum recorded fluctuations in the perched conditions.

The sensitivity of the analysis was also evaluated for possible fluctuations in the regional groundwater conditions. In particular, back-analysis was carried out in order to determine what amount of change in the regional groundwater conditions would lead to the onset of slope instability. This back-analysis was warranted because of the history of landslides in the area; see Figures 1 and 2 for the extent of previous landslides at the site.

The two old landslides shown on Figure 2 were depicted from the information on the 1968, 1983 and 1991 air photos. A review of the air photos illustrated that landslides have been retrogressive over the period 1968-1991.

Rise of the regional groundwater table increases the pore-water pressure, reducing the strength of the clay within the pre-sheared zones, and thus reducing the resisting forces against slope movement. Analyses indicated that the factor of safety of the overall slope and the Upper slope would be reduced to 1.0 in the event of a 2 m rise in the regional groundwater conditions. For the sake of brevity, the back-analysis for the above-calculated results are not included in this paper.

In order to assess the probability of a 2 m rise in the regional groundwater table, the available precipitation data from 1943 to 2000 were reviewed. The precipitation data suggested annual precipitation in the range of approximately 240-680 mm with an average value of 447 mm. Over the period from 1943 to 2000, the months of June, July and August typically correspond to the highest precipitation. The mean annual precipitation in the Grande Prairie region is approximately 140 mm higher than in the Calgary region. Given that groundwater fluctuations in the Calgary region are in the order of 1.5-2 m, it is reasonable to expect groundwater fluctuations in the Grande Prairie region to be of a similar order of magnitude. Having said that, it should be noted that groundwater also fluctuates as a result of other factors such as nearby development, changes to aquifers, etc. In the circumstances, at the time of the 2002 geotechnical investigation program and subsequently over the period 2002-2010, groundwater conditions at the site were monitored on a quarterly basis, including the period covering the highest precipitation months of June to August. A more frequent monitoring program was unwarranted given that the most likely slope movements, were they to occur, would follow a slow, creep type, mechanism. Furthermore, the probability of occurrence of a 2 m rise in the regional groundwater conditions was carried out as part of a probabilistic assessment intended to help the decision makers (the developer and the City of Grande Prairie) regarding the extent of mitigation measures required in the short, medium and long term. Again, for the sake of brevity, this paper does not describe the probabilistic assessment results.

Mitigation Options

A number of options were considered in order to provide safety factors of 1.5 (or greater) for slope stability at cross-section A-A'. The mitigation measures included setback from the crest, groundwater lowering, and a berm at the toe of the Upper slope. These mitigation options are discussed in the following sections. Mitigation measures for the Lower slope are discussed afterwards.

Setback at the Crest

In Alberta (cities of Calgary and Edmonton) it is common to locate structures at the crest of slopes beyond a setback line defined by a factor of safety of 1.5. The setback for a factor of safety of 1.5 is the distance from the crest of the slope to the point where the most 'critical' slip surface - one with a minimum factor of safety of 1.5 - intersects the ground surface.

The information in Tables 1 and 2 is re-presented again; Tables 3 and 4 give the setback distances of between 45 to 55 m for the groundwater conditions recorded at the site.

Figures 2 and 3 show the areal extent of a 45 - 55 m setback from the slope crest.

Table 3: Setback Distance Under Current Groundwater Conditions

Strength Parameters for Lacustrine Deposits	Setback Distances
Peak	45 m
Residual	50 m

Table 4: Setback Distances Under High Perched Groundwater Conditions

Strength Parameters for Lacustrine Deposits	Setback Distances
Peak	50 m
Residual	55 m

Groundwater Lowering

Groundwater lowering can consist of a series of vertical drains that would help increase the factor of safety. The low groundwater table used ¡n the analyses was assumed to be near the top the till unit, and was used to simulate the groundwater conditions by lowering the groundwater within the lacustrine deposits. The factors of safety for a lowered groundwater conditions are summarized in Table 5.

Table 5: Values of Factor of Safety for Lowered Groundwater Conditions

Strength Parameters for Lacustrine Deposits	Slope Section	Minimum Factor of Safety	Setback from the Crest for a Factor of Safety of 1.5
Peak	Overall slope	1.41	20 m
	Upper slope	1.60	
	Lower slope	1.30	
Residual	Overall slope	1.27	30 m
	Upper slope	1.29	
	Lower slope	1.01	

The results indicate that the factors of safety increase to 1.29 to 1.60 for the Upper and Overall slopes by lowering the groundwater table to the top of the till layer. These show that groundwater lowering will help render the Upper slope with a safety factor greater than 1.5, provided that peak strength parameters prevail within the lacustrine deposits at the site. However, groundwater lowering would have to be combined with setback distances of about 20 m in order to achieve safety factors of 1.5 for the Upper and overall slopes. In the event that residual strength parameters prevail within the lacustrine deposits, groundwater lowering leads to factors of safety of 1.01, 1.29 and 1.27 for the Lower, Upper and overall slopes respectively. To achieve a factor of safety of 1.5, groundwater lowering would have to be in combination with setback distances of about 30 m from the crest. It is anticipated that dewatering at Bear Creek Village Condominiums would be marginally successful given the relatively low permeability of the lacustrine deposits. Thus, the likelihood of effectively dewatering the slope is low to moderate.

Other Mitigation Measures

Erosion protection measures (creek training) was recommended at the toe of the Lower slope where there was a potential for erosion by Bear Creek. However, further consideration regarding creek training required input from the City and other jurisdictions responsible for Bear Creek.

A berm at the toe of the Upper slope would have increased the factor of safety for the Upper Slope, but would lower the factor of safety of the Lower Slope, and possibly result in movement (and instability) of the Lower slope. Therefore, this mitigation option was not further considered.

Lower Slope

Mitigation of the Lower slope was considered a priority. Possible mitigation measures included construction of a granular berm, slope reinforcement using geogrid, and granular trench construction. Since these options involved encroachment onto Bear Creek, it was important to engage jurisdictions responsible for the creek in discussions that would have helped select the optimum remedial work.

Shear Key Concept

The initial assessment of the shear key option suggested that the concept could achieve the desired stabilization of the Lower slope in a cost-effective manner. The proposed shear key mitigation option consisted of a continuous gravel trench excavated and constructed from ground surface through the sliding surface. The granular fill used to backfill the trench would be placed in lifts and compacted to mobilize the full strength of the fill. The principle was that the higher strength of the granular backfill would effectively “pin” the overlying lacustrine sediments to the silty clay till thereby increasing the resistance to sliding. In addition, the gravel-filled trench would act as an interceptor drain and would control the level of groundwater in the vicinity of the trench. The main advantage of using a shear key included its simplicity of construction, use of local materials and no environmental impact on the creek.

Detailed evaluation of the shear key concept was carried out in 2003. The evaluation was carried out to determine the feasibility of the shear key concept in terms of increase in the factor of safety for stability of the Lower slope.

Calculation of Factors of Safety

The stability of the Lower Slope with the proposed shear key was analyzed using the GSLOPE computer software, in order to determine a factor of safety for slope stability. The following assumptions were made for analyses:

The potential slip surfaces that would be generated within the slope were restricted from penetrating the till stratum. This was based on the earlier determination, from the most recent (2002) geotechnical field tests, which concluded failure was along the pre-sheared/softened zones within the lacustrine deposits.

The analyses included checking the stability of the Lower slope without any remedial measures in place (i.e., no shear key). Next, a 1.5 m wide shear key, with an internal angle of friction of 38º, was inserted into the model. The failure surface was restricted to the Lower slope area. The model was adjusted by changing the value of the friction angle to 36º and 40º to present the lower and higher limits of material variability, respectively.

The relative benefit of the Lower slope shear key on the factor of safety for the overall slope was evaluated by running the analysis and 'searching' for potential slip surfaces that encompassed the overall slope.

As part of the design for the shear key, permanent drainage along the key was provided. Based on this, the current groundwater condition (i.e., groundwater at the till surface) was used for all the analyses, since "high" groundwater condition would not occur within the Lower slope as long as adequate drainage was operational.

Analyses Results

The results of the stability analyses indicated that a 1.5 m wide shear key, embedded 1.5 m into the till deposits, would increase the factor of safety for stability of the Lower slope to about 1.5 from the 'existing' factor of safety of about 1.3. The results of the analyses are shown in Table 6. Included are the factors of safety for the Lower slope with no shear key, and with a 1.5 m wide shear key. The factor of safety for the Lower slope with no shear key was estimated to be 1.33, and it increased to 1.49-1.54 with the proposed shear key system.

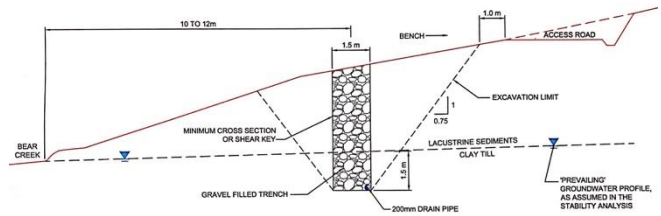
In order to illustrate the effect of the shear key on the stability of the overall slope, global stability was also checked with shear key in place. As can be seen in Table 6, by stabilizing the Lower slope by means of a shear key system the stability of the overall slope is improved. The analyses of the overall slope were based on the current groundwater condition being present within and beneath the overall slope, corresponding to the elevation demonstrated in Figure 8. The calculated factor of safety for the overall slope is 1.43 increased from 1.385 without shear key stabilization.

Table 6: Factors of Safety

Description	Lower Slope	Overall Slope
No Shear Key*	1.330	1.385
1.5 m wide Shear Key key mat'l ϕ=40º (ie + 1σ) key mat'l ϕ=38º (ie mean) key mat'l ϕ=36º (ie - 1σ)	 1.539 1.516 1.494	1.433

*Based upon the prevailing groundwater conditions.

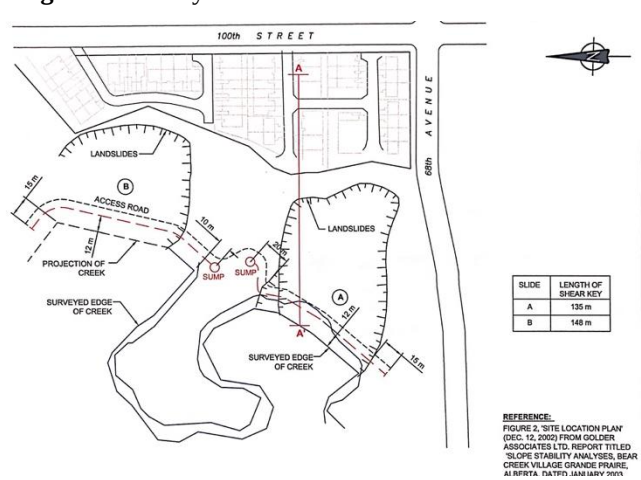
Fig. 8. Detailed Shear Key



Shear Key Design

Based on the above analyses results, the shear key design required the stabilization system to be constructed with a minimum width of 1.5 m, and embedded into the till deposits at least 1.5 m. In terms of extent, the shear key would protrude about 15 m beyond the edges of the scars of Slide A and B as shown on Figure 9. The resulting lengths of the shear key were then estimated be about 135 m for the southern shear key (Slide Area A) and about 148 m for the northern shear key (Slide Area B).

Fig. 9. Shear Key Location Plan



The cross-section shown on Figure 8 shows the minimum width and penetration of the shear key into the underlying till deposits. After removal of the lacustrine silt deposits, the trench would be backfilled with compacted sand and gravel. A 200 mm perforated plastic drain pipe was also proposed to be placed at the base of the trench.

The purpose of the shear key was to improve the stability of a potentially unstable slope. Construction procedures were

then proposed to be implemented such that they minimize temporary destabilization of the slope. It was recommended that the shear key for each slide area be constructed in sections to minimize the impact of undercutting the toe of the slope during excavation. Sections/slots of no longer than 10 m were proposed to be excavated and backfilled completely before excavation commenced for contiguous sections. Summary of the recommended construction procedures is outlined in the following section.

Construction Considerations

Construction of an access road

This road would provide access for equipment and material during the construction, and would be available for monitoring and maintenance of the entire shear key system. As shown on Figure 8, the access road was proposed to be located on the upslope side of the shear key and located at least 5 m away from the edge of the shear key. A ditch was also proposed in the access road to intercept and control runoff. The length of the road was estimated to be about 363 m, as shown on Figure 9.

Stripping of Topsoil

The vegetation and topsoil along the length of the shear keys were also required to be stripped off down to an estimated depth of about 300 mm. Stripping was proposed to be completed to at least 2 m on either side of the shear key.

Figure 8 shows the shear key with vertical sides. This represented the minimum configuration of the shear key cross-section. From a safety standpoint, the sides with 5 m deep trench were proposed to be sloped. Figure 8 shows the proposed slope with ¾ : 1 (H:V).

Backfill

Figure 8 shows the minimum width required for the sand and gravel backfill. The backfill for the construction of the shear key was proposed to consist of well-graded sand and gravel with a maximum size not exceeding 150 mm, compacted to at least 98% standard Proctor maximum dry density (SPMDD) before placement of the next lift.

Drain Pipe

Since the shear key backfill would then be more permeable than the lacustrine deposits (silt and clay), it was expected that the shear key would therefore act as a collection system for groundwater flow. In order to prevent accumulation of water in the shear key and resulting build-up of water pressure, a drain pipe along the base of the shear key was proposed. The proposed drain pipe consisted of 200 mm perforated plastic pipe installed with perforations facing up. The pipe was requested to be installed with a slight, positive gradient toward the area between Slide Areas A and B. This was proposed as it would allow the flow of groundwater from Slide Areas A and B to the two sumps shown on Figure 9.

Sumps

At the terminal end of each of the drain pipes, a sump was proposed in order to collect the accumulated water for discharge to the creek. Each proposed sump consisted of nominal 1m diameter prefabricated reinforced concrete catch basin rings with a standard frame and grated cover. Float-actuated pumps were also recommended for each sump with 2 l/min (10 gpm) capacity each.

Power Poles and Cables

Because the operation of the pumps required power, construction of an overhead power line from the hydro-electric grid along the road was requested.

Concluding Remarks

A review of historical information dating back to 1968 enabled the identification of previous landslides and existence of slickensides, which were then methodically and carefully verified through a detailed investigation by means of CPTs, quasi-continuous Shelby tube sampling, laboratory testing including triaxial and shear box tests, and instrumentation installation and monitoring at regular intervals over an extended period of time. This carefully planned and executed geotechnical investigation program proved the existence of shear surfaces and softened zones within the subsurface lacustrine deposits, which were subsequently utilised for accurately modelling slope stability. The slope stability analyses results demonstrated that groundwater conditions played an important role in slope instability. The shear key concept was developed to provide a means of ‘naturally’ lowering the groundwater table and increasing the factors and safety for the Lower slope to 1.5 without having to establish setback distances which would have otherwise resulted in expropriation of homes.

In order to avoid compromising the safety margins (factors of safety) for slope stability, monitoring of the May 2002 installed piezometers and slope indicators continued for at least 3 years on a quarterly basis. In fact, geotechnical instrumentation monitoring continued until 2007, at which time there was full confirmation that slope movement had been within the “extremely slow” category of the Landslide Velocity Scale by Cruden & Varnes (1996), signifying evidence of the satisfactory performance of the shear key system that was built in 2003.

Reference

Cruden, D.M. and Varnes, D.J., 1996. Landslide types and processes. In Landslides, Investigation and Mitigation. Special Report 247, Transportation Research Board, Washington, pp. 36-75.

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