

STABILIZATION OF A HIGHWAY EMBANKMENT LANDSLIDE GEOHAZARD

Vishnu A. Diyaljee, Ph.D., P.Eng., GAEA Engineering Ltd, Edmonton, Alberta, Canada
Murthy Pariti, Ph.D., P.Eng., Stewart Weir Ltd, Edmonton, Alberta, Canada

Abstract: In 1985 the first signs of instability were observed along a 60 m section of SH 748:02, a two-lane paved Secondary Highway located approximately 8 km north of the Town of Edson, Alberta, Canada. This section of highway was inspected and investigated for the next 14 years without the implementation of any dedicated remedial measures. In 1999 an embankment landslide occurred in the distressed section of this highway resulting in its immediate closure. Stabilization of this highway landslide geohazard was effected through the use of drilled shaft piles in conjunction with subsurface drainage. Inspections, investigations, and historic development of this landslide geohazard during the 14-year period are presented along with details of the remedial measures, their implementation and associated costs. To date, good performance of the failed section of this highway has been reported based on yearly inspections undertaken following its stabilization in 1999. This case study illustrates the need to address stabilization of potential landslide areas at an early stage, since this could avoid costly remedial measures in some situations.

I INTRODUCTION

In 1999, approximately 14 years after signs of slope instability were reported along a 60 m stretch of SH 748:02, this two-lane paved Secondary Highway was rendered impassable by a landslide geohazard that resulted in the immediate closure of the roadway. A geotechnical investigation was undertaken in 1997 and remedial measures ranging from maintenance to reconstruction were recommended, but none were implemented. Following the road closure in 1999, alternate remedial measures were designed and implemented. These measures consisted of stabilization of the landslide using drilled shaft piles in conjunction with subsurface drainage measures.

Since the implementation of the remedial measures, this site has been inspected annually by Geotechnical Consultants to Alberta Infrastructure and Transportation (AIT). A risk assessment procedure is utilized in the inspection. This procedure evaluates risk as the product of a Probability Factor (PF) and a Consequence Factor (CF). The Probability Factor is ranked on a scale of 1 to 20 where 1 represents an inactive, very low probability of slide occurrence, and 20 a catastrophic slide is occurring. The Consequence Factor is ranked from 1 to 10 where 1 represents a shallow cut slope slide that does not impact the roadway, and 10 represents sites where public safety is at risk and mobilization of a large scale slide is possible.

An inspection in 2005, the most recent published by AIT, showed that while the rehabilitated section is performing well, minor pavement cracks noted during the 2002 inspection approximately 40 m south of the outermost drilled shaft piles places the risk level of the overall site at 24 (PF 8 and CF 3). This was the same risk level applied in years 2002 - 2004.

The primary objective of this paper is to provide a historical review of the stabilized slide area and details of the 1997 and 1999 remedial measures and their associated costs. The history of this landslide geohazard illustrates the need to undertake remedial action at an early stage in situations where site constraints and deterioration can possibly result in more expensive solutions as time progresses.

II SITE LOCATION AND GENERAL SETTING

Secondary Highway (SH) 748:02 forms part of the Secondary Highway Network of the Province of Alberta under the overall administration of AIT, a Ministry of the Government of the Province of Alberta, responsible for the planning, design, construction, rehabilitation, operation and maintenance of the Province's Highway Infrastructure and other Government owned/supported infrastructure.

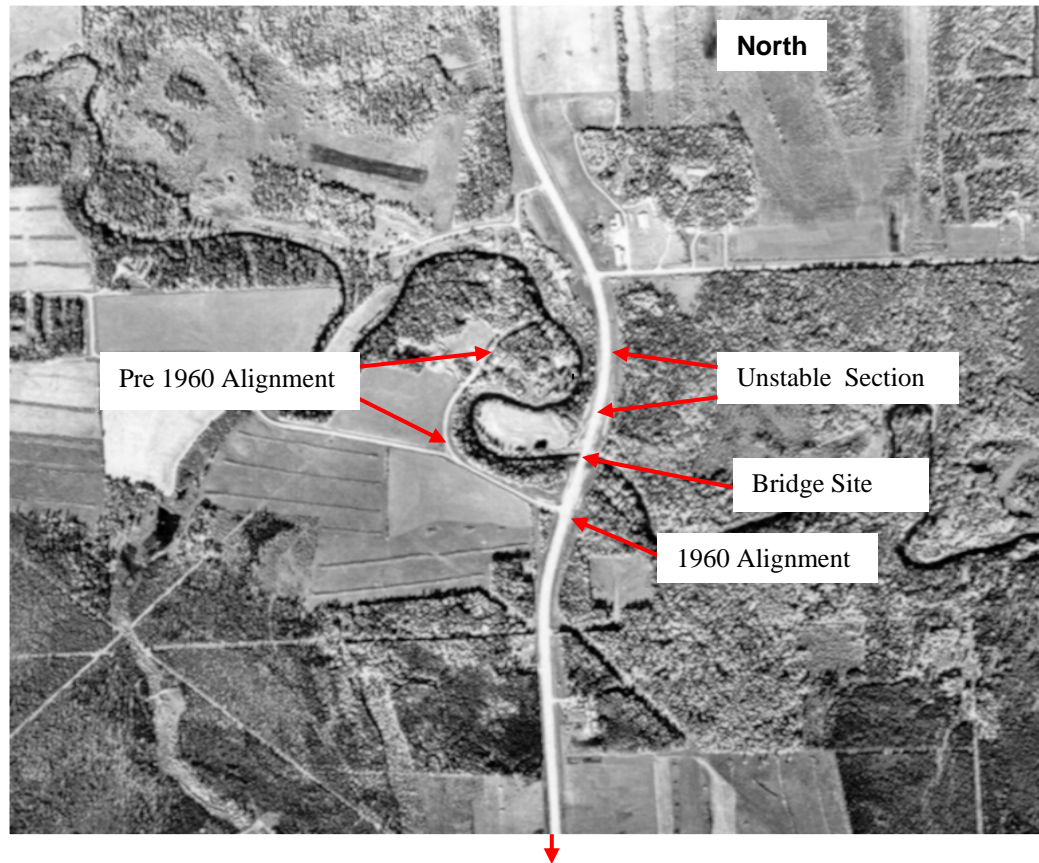
SH 748:02 is located within the North Central Region, one of the four Transportation Regions of the Province. This highway is under the jurisdiction of the Yellowhead County, one of the Municipalities within the North Central Region. SH 748:02 provides an important link to natural resource, recreational, and agricultural areas in the northern part of the region and to Hwy 16 (The Yellowhead Highway) and the Town of Edson. It is located about 8 km north of the Town of Edson and approximately 100 km west of Edmonton, the Capital of Alberta. Figure 1 shows the location of SH 748:02 within the Yellowhead County. The legal land description of the slide area is NW 10-54-17-W5M.



Figure 1: Map of Yellowhead County Showing Location of SH 748

Prior to its construction as a Secondary Highway in 1960, SH 748:02 was originally a gravel-surfaced local roadway. At the time, this roadway alignment was to the west of a meander loop of the Edson River and terminated at the river. Reconstruction of this roadway to Secondary Highway standards was undertaken with the construction of a 3-span, 55 m long bridge across the Edson River on the

east of the meander loop. The 1988 aerial photograph, Figure 2, shows the pre 1960 and 1960 alignments.



To Hwy 16

Figure 2: 1998 Aerial Photo Showing pre 1960 and 1960 Alignments

As noted in Figure 2, the new alignment brought a section of the highway in close proximity to the outside of the bend of a meander loop of the river. The areas of instability that occurred in 1985 and minor cracks observed in the 2002 inspection by AIT are within this bend section of the highway.

III HISTORIC INFORMATION ON LANDSLIDE PRIOR TO YEAR 1997

A. General

The problems associated with instability of SH 748 will be provided chronologically from 1985 to 1999 incorporating the observations made, decisions and actions taken during these periods. Between 1985 and 1999, the Yellowhead County and Alberta Infrastructure and Transportation (AIT) as referenced, were at times referred to as Municipal District (MD) of Yellowhead, County of Yellowhead, Alberta Transportation, and Alberta Transportation and Utilities (AT&U). Where these appear in the historical information, they represent the same entities as the Yellowhead County and Alberta Infrastructure and Transportation as they are known today.

B. Year 1981

In 1981, while SH748:02 was still a gravel surfaced highway, a soils investigation was undertaken to obtain information on the subgrade soils, as is normally done, for base and surfacing design which was scheduled for 1982. Two test holes were drilled to a maximum depth of 12.5 m in the vicinity of the slide area. No specific problems were identified or reported at that time although the reason for the deep holes might have been an indication of concerns about the location at that time.

C. Year 1985

In 1985, the MD of Yellowhead contacted the District Transportation Engineer (DTE) of Alberta Transportation in Edson about likely slide problems associated with a section of this highway. This contact signaled the first official report of slide activity associated with this highway. The DTE subsequently informed the Geotechnical Services Section (GSS) of Alberta Transportation of the concern and in April 1985, the GSS undertook an inspection of the site. This inspection revealed a drop of 65 mm in the southbound lane along a 60 m section of the highway adjacent and parallel to the Edson River on the west side of the highway. Since the highway was scheduled for paving in the summer of that year, the GSS recommended a thin overlay through the slide area pending the results of a detailed geotechnical investigation.

In May 1985, the GSS conducted a detailed geotechnical investigation of the slide area. Six (6) testholes were drilled during this investigation. These consisted of holes for two (2) slope indicators (SI #1 and SI #2), two (2) standpipes and two (2) pneumatic piezometers. The standpipes and pneumatic piezometers were located within 6 m to the north of SI #1 and SI #2, respectively. The SI testhole locations are shown in Figure 3 and identified as ATU. A cross-section within the slide area is shown in Figure 4.

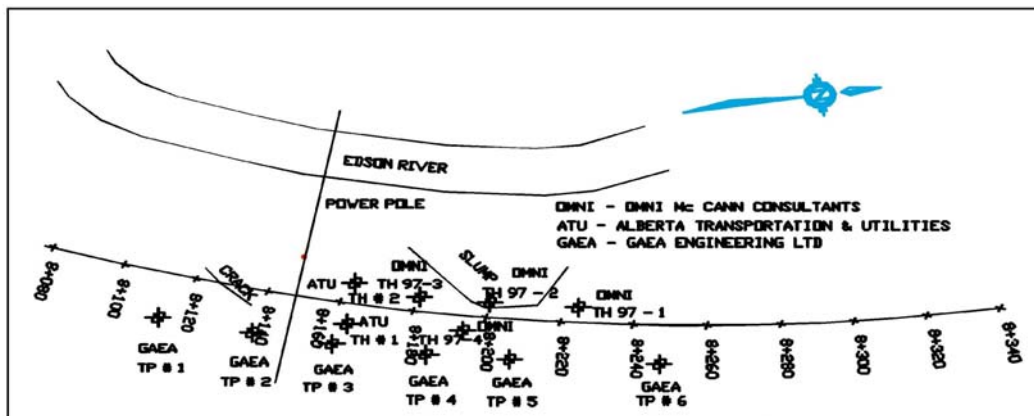


Figure 3: Location of Test holes and Some Testpits -1985 to 1999 Investigations

The slope indicators were monitored following installation on June 28, 1985 on four occasions up to September 1985, and in March 1986. The results showed a clear movement zone at 12 m below ground in SI # 2 located on the left of the highway centerline i.e., on the side toward the Edson River. SI # 1 on the opposite side showed no definitive movement location. The standpipe piezometers showed water levels in September 1985 to be on the average 5.8 m below ground level. The pneumatic piezometers did not show any water pressures.

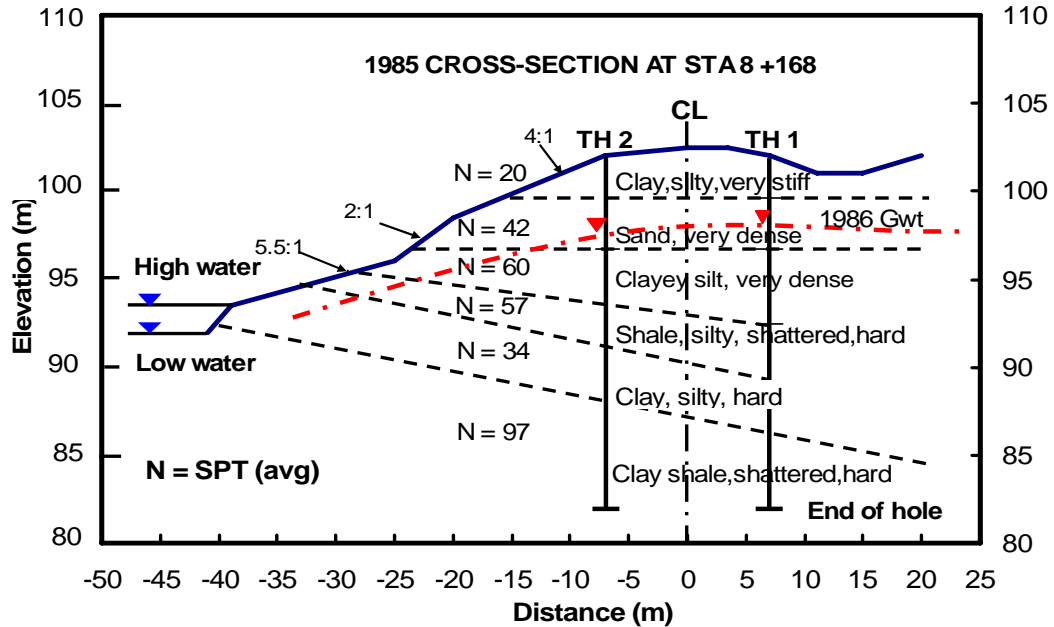


Figure 4: Cross-section Showing Stratigraphy and Testholes (ATU) within Slide Area

D. Year 1986

The site was inspected by the GSS on August 5, 1986 and photographs taken of several features of the slide area – road surface and river location. The cracks on the roadway showed a somewhat semi-circular pattern propagating into the roadway sideslope toward the Edson River. Along the bank of the river in the vicinity of the slide area showed evidence of bank slumping. This slump was not recent. The road had been recently paved, and except for the reflection of the past crack pattern, there was no discernible drop in the road surface as noted during the 1985 site inspection. Water levels in the standpipe piezometers on August 19 showed the levels to be 4 and 5.3 m below ground in Standpipes 1 and 2, respectively. These water levels were higher than those recorded in 1985.

On August 19, 1986, the GSS discussed with the DTE in Edson observations and monitoring results up to that time. It was agreed that the proximity of the river, treed sideslope, geometric constraints, and jurisdictional considerations regarding funding for remedial work posed too many restraints at the time for any rational solutions to be provided. The DTE agreed that he would keep maintaining the highway by filling up the settling areas on the pavement with asphalt concrete mix. This approach is often adopted in slide areas at the inception until a geotechnical investigation is undertaken and a recommended remedial measure receives funding for implementation. Very often, despite the remedies that may be developed, action is not taken unless serious problems manifest rendering the highway impassable or when complaints are received from the traveling public.

E. Year 1987

On October 13, 1987, a site inspection of the site was made by the GSS as a follow up to a further request by the DTE for a review of the site conditions. The following were recorded during this inspection:

1. Pavement cracking seemed similar to past crack patterns.
2. Sections along the bottom and backslope of the east ditch appeared damp. This dampness was thought to be associated with groundwater movement from higher ground to the east.
3. The topography of the immediate area suggests that the slide is situated in fill between two ridges of higher ground elevations and there may be a confluence of groundwater movements through the heart of the slide toward the Edson River which, if true, would likely be the cause of the ongoing slope instability.
4. A test pit investigation was recommended to determine groundwater patterns and assist in the design of a possible subsurface drainage facility.
5. Known utilities in the area include Alberta Government Telephones and Northwestern Utilities. These would have to be located and flagged prior to undertaking any testpitting.
6. The GSS would like to undertake extensive test pit investigation within the next 1- 2 weeks.
7. Given the history of this slide, it was anticipated that the cracking will worsen with time and further patching will be required next year if no remedial measures are taken.
8. If the groundwater confluence theory is correct, it was believed that the slide could be stabilized with subsurface drainage improvement measures.

F. Years 1988 – 1991

Visual inspection of the slide area was undertaken during these years and no major distress was observed as the slide area was maintained by periodic patching of the driving lanes as required to maintain a reasonable smooth highway surface. As a result, the investigations required for designing appropriate remedial measures were not undertaken.

G. Year 1992

The DTE made a request to the GSS on October 5, 1992 for assistance in solving a few outstanding slide problems in District 8. This District was under the jurisdiction of the North Central Region of AT&U and incorporated the Municipal District of Yellowhead. The GSS provided an internal memorandum on November 16, 1992 to the DTE with the following response.

1. The 748:02 slide area was investigated and scheduled for testpitting in 1988. This was not undertaken since the roadway seemed to be performing satisfactory following an overlay treatment at that time.
2. It was known that the problems would re-occur but not everyone saw the urgency of undertaking remedial works at this site.
3. Over the years, periodic checks were made by District personnel and since 1988, no problems associated with this slide area were reported to the GSS.
4. A Geotechnical Engineer from the GSS has been assigned to review past data and undertake a testpit program in the uphill ditch since the evaluation in 1988 showed that seepage was a factor that could influence stability of this area. As the slide area is at the bend of the river, the probability of toe erosion is also present.
5. Additional slope indicators are to be installed to cover the distressed area in more detail and monitoring is to be done before a final solution is developed.

The review by the assigned Geotechnical Engineer included undertaking a slope stability analysis using the G-Slope Software program (www.mitresoftware.com) and Bishop's Modified Method of analysis. A Factor of Safety (FOS) of 1.68 was obtained using a fixed location of the

failure at the highway location and piezometric level described by water levels of 4 and 5 m below ground on the right and left of the roadway. Increasing the piezometric levels to the clay sand interface provided a FOS of 1.22 and for a similar circle a FOS of 1.10 with a weak layer introduced at the toe of the side slope.

A review of contour mapping of the site and adjacent areas showed general concurrence with the groundwater flow hypothesis as postulated in the 1987 GSS review. On the basis of the overall review of the past data and results of the slope stability analysis it was concluded that there were likely three probable factors influencing the slide area:

1. High pore pressures in the silty sand and clayey silt materials.
2. Seepage toward the river causing a weakening of the clay shale
3. Active erosion of the base of the slope by the river.

Tentative remedial measures in the order of likelihood were:

1. A buried interceptor ditch in the east roadside ditch to intercept seepage and drain it south to the river in the vicinity of the bridge.
2. A cut-off trench backfilled with gravel to act as a secondary water interceptor, and to strengthen the toe of alluvial material.
3. Erosion protection for the riverbank.

H. Years 1994 - 1996

Secondary Highway 748:02 was handed over to the MD of Yellowhead in 1994 for upkeep. This required the MD to acquire Grants from the Alberta Government through AT&U for undertaking design, construction, and maintenance of transportation infrastructure under their jurisdiction. At this time, as well, AT&U was undertaking privatization initiatives, which would see the devolution of some of its activities to the Private Sector. During this period, none of the proposed work outlined in previous years was undertaken. The file was closed in May of 1994 with a note that further work would depend on the MD of Yellowhead contacting the GSS. However, as the mandate at the time was for work to be privatized any funding to the MD of Yellowhead through grants would have to be undertaken by Consultants rather than by Alberta Transportation forces.

V 1997 GEOTECHNICAL INVESTIGATION

A. General

In June/July 1997, a side slope failure occurred on the west side of the highway in the same general vicinity of the 1985 slide area resulting in extensive cracking of the asphalt pavement in the southbound lane. This failure required patching of the pavement surface and additional granular fill to maintain the west side slope. At this time, Omni-McCann Consultants Ltd (OMCL), an Alberta Consulting Engineering firm based in Edmonton, was solicited by the MD of Yellowhead to conduct a detailed investigation to re-assess the site conditions and suggest a long-term solution (OMCL, 1998). The objectives of this investigation were as follows:

1. To determine the nature and causes of reoccurring slides at the above mentioned location.
2. To provide recommendations for various slope stabilization measures including the costs as well as an overall recommendation for the most practical, and cost effective measure.

B. Existing Site Conditions

A brief site reconnaissance was conducted by OMCL in July 1997 to assess site conditions and to determine the type and nature of investigation required. Based on this site review a proposal was prepared from observations made during this site visit and presented to the MD of Yellowhead. The scope of work included the review of the files of the GSS, review of historical air photos, further testhole drilling, laboratory testing, and slope stability analysis, determination of slope stabilization measures, construction requirements and associated costs.

OMCL conducted a second site inspection in October 1997. At that time a thorough visual inspection and topographic survey of the slide and adjacent areas were conducted. The conditions observed were as follows:

1. A recent slide occurred on the west side of SH 748:02 in relatively the same area as the as the previous slides.
2. Extensive cracking of the asphalt pavement road surface particularly in the southbound lane (west side of roadway) was observed. Evidence of recent asphalt patching/overlays and additional granular fill on the west side of the road was noted in the slide area.
3. The sideslope on the west side of Hwy. 748:02 leading down to the river in and around the slide area is steep and near vertical adjacent to the roadway. The sideslope appears to contain undesirable organic rich soil layers and or seepage zones in places.
4. The height of the slope in the area of the slide is approximately 12 metres above the river water level.
5. Apparent older failure scarps are visible at various elevations/locations on the sideslope.
6. Some undercutting of the slope toe by the Edson River appears to be in progress. The existing bank erosion does not appear to be serious enough, however, to significantly affect the road embankment.

C. Historic Aerial Photograph Review

A review of the available historical air photos was conducted. Most of the available air photos were of a relatively large scale, 1:15,000 or larger. The following air photos were reviewed by OMCL - Year 1950 (Scale1:15,840), Year 1961 (Scale 1:15,840), Year 1980 – (Scale 1:5,000), and Year 1988 – (Scale1:15,000).

Photos from 1950 showed that the area of roadway in question had not yet been constructed and the old roadway was located on the west side of the river, west of the area under investigation. The current bridge crossing the Edson River south of the area in question had not been constructed and the river was crossed farther north. These photos also revealed that prior to embankment construction the area under investigation was a heavily treed sideslope leading down to the Edson River.

The 1961 photos showed the highway much as it appears today, indicating that the embankment in question was constructed prior to August 11, 1961. Numerous trees were removed from the pre-development sideslope for the highway right-of-way. Photos from 1980 showed recent maintenance or upgrading of the sideslope on the west side of the highway north of the bridge and south of the area currently under investigation. This may have been a previous slide repair. The 1988 photos showed conditions as they were seen during the current investigation.

D. Testhole Drilling and Piezometer Installation

A drilling and piezometer installation program was carried out by OMCL on December 1, 1997, using a truck mounted B61 drill rig. The program included four test holes; Test hole TH 97-2, on the west shoulder of SH 748:02 within the slide area, TH 97-1 approximately 20 metres north of the slide area, TH 97-4 approximately 25 metres south of the slide area and TH 97-3 approximately 12 metres east of the east shoulder of SH 748:02. The locations of the testholes are shown in Figures 3 and 7, and identified as OMNI.

The test holes varied in depth from 9.2 metres to 12.7 metres below ground level (bgl) and all four test holes were completed as standpipe piezometers. Three of the test holes were advanced using continuous flight solid stem augers. The test hole within the slide area was advanced using a hollow stem auger with continuous core sampler. During drilling, standard penetration tests were conducted at approximately 1.5 m intervals. Pocket penetrometer measurements and disturbed soil samples were taken at 0.75 m intervals. It is to be noted that soil descriptions are somewhat different from that reported for the 1985 drilling because of changed site conditions caused by maintenance and difference in field classification by different testhole loggers.

Soil conditions observed in testholes 97-1, 97-2 and 97-3 on the west side of the road were as follows:

1. Granular fill (sand and gravel) with clay inclusions/seams ranging in depth from 0.9 metres below ground level (bgl) (TH 97-3) to 2.6 metres bgl (TH 97-2 in the slumped area).
2. Fill soils comprised of a mixture of clay/clay till soils with numerous weathered sandstone/clay shale bedrock inclusions, some of which appear to be boulder or larger in size. The geotechnical characteristics of these soils were highly variable but generally the materials were stiff, moist and had a plasticity ranging from low to very high. Some of the high plastic clay soils were slickensided. These soils extended to depths ranging from 5.2 metres bgl (TH 97-1) to 6.1 metres bgl (TH 97-3).
3. Organic rich soils were encountered at the base of the fill soils. These soils were dark brown, clayey and contained numerous wood pieces, roots and tree limbs/trunks. This organic soil horizon ranged in thickness from 0.35 metres to 0.6 metres.
4. Highly weathered ice thrust and/or rafted bedrock composed of clay shale and clayey sandstone. These soils were moist to saturated and soft to firm with occasional stiff zones. Slickensides were observed in the clay shale material. A hard rock zone was encountered in two test holes leading to auger refusal at depths of 9.2 metres in TH 97-1 and 10.7 metres in TH 97-2.
5. Glacial till soils were encountered below the ice thrust and/or rafted, highly weathered bedrock in TH 97-3 at a depth of 8.2 metres bgl. The clay till soils were silty, sandy, moist, firm to stiff and medium plastic. Occasional gravel pieces and coal chips along with numerous local bedrock inclusions were observed.

Conditions on the east side of the road determined from testhole 97- 4 were as follows:

1. Granular fill (sand and gravel) associated with the roadway base was encountered to a depth of 0.15 metres bgl.
2. Ice thrust/rafted bedrock composed of weathered clay shale was encountered below the granular fill. This material was damp, hard, and medium to high plastic and extended to a depth of 2.0 metres bgl.

3. Glacial clay till soils were encountered below the ice thrust or rafted bedrock. These soils were silty, sandy, moist, stiff, medium plastic and contained pebble to gravel size rocks and coal chips. These soils extended to a depth of 2.3 metres bgl.
4. Ice thrust/rafted bedrock composed of a weathered sandstone/siltstone below the clay till zone. These materials were clayey, damp to moist and stiff to firm with depth and extended to a depth of 8.2 metres bgl.
5. Glacial clay till soils were again encountered below the ice thrust/rafted bedrock. These soils were silty, sandy, moist, stiff, medium plastic and contained occasional pebbles, coal chips and local bedrock inclusions. These soils extended to a depth of 9.75 metres bgl.
6. Sandstone bedrock was encountered below the glacial till. This material was very clayey, very stiff to hard, damp, and slightly plastic and fine grained.

E. Slope Stability Analysis

A slope stability analysis was conducted using the G-Slope Computer program. Initial analyses were performed using strength parameters determined during the drilling programs. These analyses indicated that the slope was relatively stable and that failure should not occur. This was consistent with the 1992 findings of the GSS. The highly variable nature of the soils observed during the drilling program indicated that pockets or zones of very soft, wet soils may be present at some locations leading to the small localized failures observed on the existing embankment sideslope. Based on this observation, an analysis was run using much weaker strength parameters in the near surface clay layer. This analysis produced a FOS of 1.07 indicating that slope failure could take place.

Based on the highly variable nature of the subsurface soil conditions observed during the most recent (OMCL) and previous (AT&U) drilling programs, the G-Slope program analysis was considered inadequate to fully address existing conditions. Visual site evidence observed by both OMCL and the GSS indicated that at least two different types of failures have occurred – shallow and large scale.

The shallow slumps are daylighting predominantly within the west (southbound) driving lane of the roadway. These small scale failures were believed to be caused by soft wet zones within the fill soils below the roadway and sideslope.

A much larger scale failure of the entire embankment area may also be in progress. Large cracks were observed in the asphalt road surface at two locations approximately 100 metres apart, in the area of the most recent small scale slope failure. Both of these cracks extend across the roadway in a near perpendicular direction indicating the crest of this failure is well to the east of the roadway. These cracks indicate that some movement along this failure plane has occurred. It was concluded that it would be impossible to predict when further movement or a major slope failure will occur. As well, additional slope failure planes may exist in the embankment fill area and underlying soils.

It has also been suggested that the Edson River may be undercutting the toe of the sideslope and contributing to the failure of the slope. Visual inspection of the riverbank erosion in the area of the slope failures indicated that erosion has been or is taking place in this area. Although the river bank erosion is a very slow process, it may be contributing to the suspected large scale failure in the highway embankment.

F. Stabilization Options and Associated Costs

Various stabilization and remediation options were determined to be feasible. Some of these are as follows:

1. *Monitor and repair* - Monitor slope and embankment conditions by thorough visual inspection on a regular basis, once or twice a year and/or if problems occur or are reported. Install three additional slope indicator monitoring devices in the affected area and carry out regular monitoring of these installations. This would incur an initial cost of \$10,000 and a yearly monitoring cost of about \$2,500.

Repair embankment slopes as necessary when slips occur that undermine the roadway. This would include removal of the slide materials where the failure has occurred, including the roadway shoulder and sideslope as required depending on the extent of the slip. The excavated area would then be backfilled with good quality granular fill (road crush), placed and compacted under engineering supervision. If paved areas are affected, asphalt overlay and/or replacement would be performed following the fill placement operation. This option appears to have been the one adopted by AT &U in the past for this area.

- 2a. *Replace sideslope and monitor* - Remove sideslope of embankment fill for a length of approximately 100 to 110 metres along the west side of the roadway in the affected area. The excavation would start at the west shoulder of the roadway and would extend to a depth of approximately 4 metres below the existing grade at a 1:1 slope. Good quality granular fill (road crush) would then be placed and compacted under engineering supervision. The higher friction value associated with the granular fill would increase the stability of the sideslope and reduce potential slope failure. The initial cost for this option was estimated at \$140,000 and a yearly monitoring cost of \$1,500.
- 2b. This was similar to 2a, but with increased size and depth of sideslope replacement. The excavation would start approximately one metre east of the west shoulder of the roadway including part of the existing paved surface. This excavation would extend to a depth of approximately 6 to 8 metres below existing grade at a 1:1 slope. The area would then be filled with granular material as above. The replacement of additional undesirable material would further reduce potential slope failure. Both of these methods should be used in conjunction with ongoing regular visual inspection and performance assessment. The estimated cost for this option was \$ 300,000.00 and a yearly monitoring cost of \$1,500.00.
3. *Replace embankment* - Excavate and remove all undesirable embankment fill and the organic rich zone observed immediately below the fill. Soft wet areas which may be present at the top of the native soils would also have to be removed. The exposed slope underlying the embankment fill should be benched to enable proper fill placement. The length of the embankment to be replaced is approximately 100 to 110 metres. The depth of the excavation would be variable, but should be generally 6 to 8 metres bgl. Upon completion of the excavation, the embankment would be reconstructed using granular fill materials placed and compacted under engineering supervision. Good quality clean pit run gravel could be used for the majority of the fill the upper zone (top 0.3 metres) should be constructed using a road crush type gravel. This option was estimated to cost \$600,000.

G. Discussion and Recommendations

Based on information obtained during this investigation, OMCL offered the following to the MD of Yellowhead for their consideration:

1. The most recent and previous small scale slope failures which have been undermining the roadway are very likely due to the poor quality of embankment fill and improper construction practice utilized during the embankment construction. Based on air photo evidence the embankment was constructed prior to August 1961 at which time construction practice may not have been to current standards. Soils observed during the subsurface investigation indicate that the embankment fill was placed directly over the existing ground without proper stripping of organic and/or undesirable soils. To compound the problem, it appears that the embankment fill was placed on an existing slope.
2. The apparent movement of a larger scale slope failure would require a long term and more thorough investigation to determine if a large scale slope failure would occur. Many factors may be contributing to the movement along the deeper seated failure plane some of which may include deforestation of natural sideslope, erosion of the toe of the Edson River, intermittent upward groundwater movement during periods of heavy precipitation coupled with high seasonal high groundwater levels, loads imposed on the sideslope by the embankment, highly variable local geology, and numerous undermined factors.
3. This deeper failure plane may have been a pre-existing condition when the roadway was constructed.
4. From an engineering standpoint OMCL recommends that remedial measures outlined in Option 3 should be utilized. However, this option may not be the most feasible or appropriate for the MD of Yellowhead due to the high costs and road closure. More extensive consultation and discussion of the various options may be required to determine the MD of Yellowhead's best course of action.
5. The likelihood of the MD of Yellowhead obtaining a cost recovery from the Alberta Water Management and Control Program was very low since no direct correlation between the erosion caused by the Edson River and the slope failures could be provided.

VI 1999 GEOTECHNICAL INVESTIGATIONS AND DESIGN

A. General

The recommended Option 3 by OMCL was not undertaken by the MD of Yellowhead, presumably as a result of the high cost, and the uncertainty of funding. In the meantime, the slide area became progressively worse by May 1999 resulted in the closure of the southbound lane of the highway. Photographs of the slide area at the time of inspection and prior to construction are shown in Figures 5 and 6.

The MD of Yellowhead solicited GAEA Engineering Ltd (GAEA), an Alberta Consulting Engineering firm based in Edmonton in May 1999 to investigate and provide a solution for remedial action. This was an urgent request since one lane of the roadway was closed. In

response, GAEA's geotechnical staff made two site visits. On May 6, 1999 the day after the request was made by the MD of Yellowhead a site review was undertaken. This review was undertaken along with technical staff of the MD of Yellowhead. The second site visit was made on May 17, 1999.

On the May 6 visit, reading of 1985 slope indicators were attempted. Only SI #2 was undamaged at the surface but could not be read as the probe would not go further than a depth of 12 m. This confirmed the depth of slide plane inferred from the 1985 monitoring. During the period between the two site visits relevant airphotos of the area were reviewed. The second visit was done to confirm some of the observations made from the reviews of the airphotos. It was noted that the highway alignment might have been further to the east of its present location. This was confirmed during the field review. During this visit the opportunity was also taken to discuss the history of the slide area with nearby residents. Of interest, was the observation by the residents of springs in the general area where the slide was occurring, and that the highway was experiencing problems since its construction.



Figure 5: Southbound lane Closure



Figure 6: Closure of both lanes

A letter report was provided to the MD of Yellowhead on May 18 outlining recommendations for remedial measures. These were based on a review of the OMCL report, the aerial photos, the historic information from 1985 to 1992, along with the intimate understanding of the slide area, since some of GAEA's staff was formerly part of the staff of the GSS of AT&U at the time.

The MD of Yellowhead was advised by GAEA in this May 18 report that several remediation alternatives were addressed such as realignment, lowering of the gradeline, shifting a lane to the east, incorporating a sag in the roadway over the affected area, providing a restraint at the toe in the form of a gabion retaining wall, pile wall, shear key, and the scheme of partial and full removal of material as proposed by OMCL. On the basis of experience with slides through Alberta, and the proximity of the Edson River it was the opinion that a restraint in the form of a pile wall would be most suited for the site. The other alternatives were subject to geometric constraints or were otherwise too expensive to combat the combined effect of subsurface movement of water, erosion of the river and slide movement at depth.

The preliminary proposal was accepted by the MD of Yellowhead and approval granted to GAEA for further investigation, design of appropriate remedial works, tender preparation, and construction supervision.

B. Testpitting Investigation

A testpitting investigation was undertaken to determine the depth of possible seepage zones and ground water movement in the backslope ditch since this was not done in the previous investigations. Few test pits were also excavated on the sideslope to identify the general nature of soil in the slide area and to identify any seepage zones.

Using a John Deere 260 LC backhoe, six test pits (TP # 1-TP # 6) were dug in the backslope ditch starting at the south end of the crack close to the bridge and was continued towards higher elevations along the ditch. Predominantly silty to sandy clay was encountered in these test pits followed by thin clay shale ledges which could be easily broken by the backhoe. The depth of clay varied between 2 and 3 metres in all the test pits. The moisture condition of the silty clay varied from moist to very wet. Fairly fast seepage was noticed at the contact between the silty clay and broken clay shale. GAEA provided details of the testpits in the Geotechnical Report in May 1999 (GAEA 1999). Testpits (TP #1 - TP # 6) are shown in Figures 3 and 7, and identified GAEA.

Test pits TP # 7 and TP # 8 were dug in the middle of the slide area while test pit TP # 11 was dug close to the tree line. The material observed in these pits was generally silty clay but with more clay content. Seepage was visible within this material. Test pits # 9 and # 10 were dug closer to the centreline of the road. These two test-pits revealed the existence of very wet pitrun gravel and coldmix which were used to maintain the roadway during the past years. Heavy and fairly consistent seepage was also noticed in the pitrun. Generally, the direction of the seepage appeared to be from the higher ground that exists on the east side of the highway. Based on the cleanliness of the water, flow rate and depth of movement, it was felt that the source of the water would be the subsurface flow from the east (backslope area). Testpits (TP #7-TP #11) are shown in Figure 10 and identified as GAEA.

C. Recommended Remedial Measures

From the observations of seepage locations from the testpits dug in the backslope ditch and on the side slope, it was quite evident that drainage measures would be needed to assist in stabilizing the slide area by installing a subsurface drain in the backslope ditch. This drain would intercept seepage traveling in an east to west direction and discharge it in the Edson River near the north end of the bridge where the highway ditch discharges its surface runoff. Another important measure also recommended was to retain the side slope fill material and arrest river erosion at the toe of the side slope by installing a pile wall close to the tree line. The length of this pile wall would be about 120 metres with offset from the highway centreline varying 12 m at the south end to 27 m at the north end to avoid clearing trees along the riverbank.

D. Detailed Design

Following approval of the recommended remedial measures in principle by the MD of Yellowhead, the detailed design and tender document were completed on July 8, 1999. The design of the pile wall was undertaken using, in general, the approach for drilled shaft walls by Nethero (1982), local experience, overall observations of site behaviour over the years, and engineering judgment. In the analysis, passive resistance was taken into consideration below the top 4 m of ground. Factors of Safety varying from 1.5 – 1.8 in relation to overall stability of the system were realized and considered acceptable. Design details of the recommended remedial measures are shown in Figures 8 to 11.

The important features of the proposed design were:

1. Excavating an approximately 250 metre long trench drain in the backslope ditch on the east side of the roadway and installing a perforated pipe subsurface drain at about 3 metre depth below the ditch bottom. The perforated pipe would be daylighted through a non-perforated 800 mm corrugated steel pipe to flow into the Edson River at the north end of the bridge. See Figures 7-9.
2. Installing a pile retaining wall at two levels of the side slope, one close to the tree line close to the toe of the sideslope on the west side of the highway and another within the slumped area. The pile wall close to the tree line on the west would consist of sixty five (65) – 600 mm diameter straight shaft piles drilled to a depth of 15m and spaced at 2 m on centres. These piles would be reinforced with 250 mm x 62 kg steel H-piles and backfilled all around with 25 MPa concrete. The other pile wall would consist of similar sized and reinforced drilled shaft piles located at 4 m centres within the slumped area. The lengths of the piles were such that they would extend below the riverbed and would be anchored in hard clay shale about 8 – 10 m below the slide plane. The pile wall schemes are shown in Figures 10 and 11.
3. Installation of gabion basket walls at two levels on the side slope in the main slump area. The objective of these gabion baskets was to assist in retaining the constructed side slope fill and accommodate any fill settlement. The upper level gabion wall was designed to retain the pitrun gravel fill which was to be used in the main slump area to bring back the highway and side slope to its original grade. The gabions would be constructed to bear against the inside face of the upper piles. The projected length of the upper gabion basket wall was about 50 metres. A similar gabion wall approximately 62 m long would bear against the lower level piles to assist in retaining the fill between the upper and lower level piles. Details of the gabion walls and their location are shown in Figures 10 and 11.

Based on the proposed design, the Engineer's estimate for the pile wall and drainage measures was \$286,000, inclusive of a 10 % contingency amount.

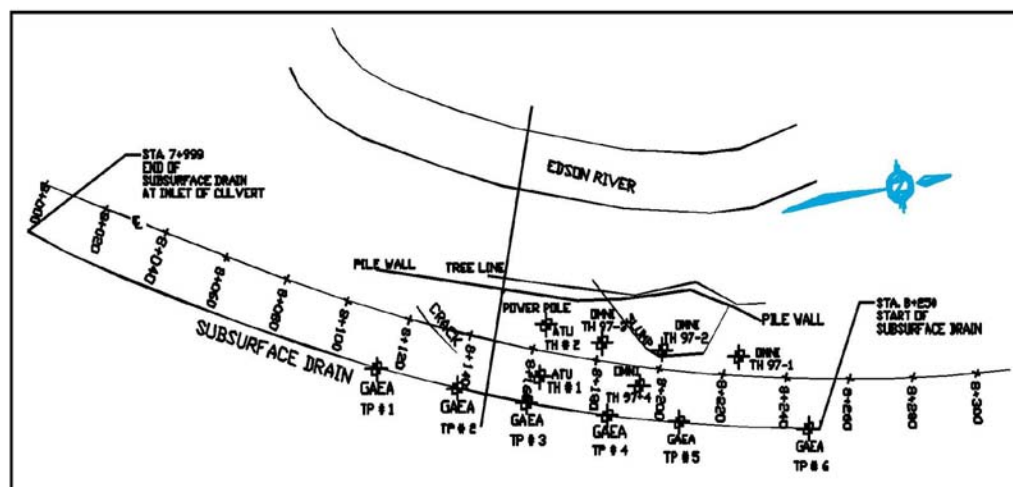


Figure 7: Plan Showing Location of Subsurface Drain

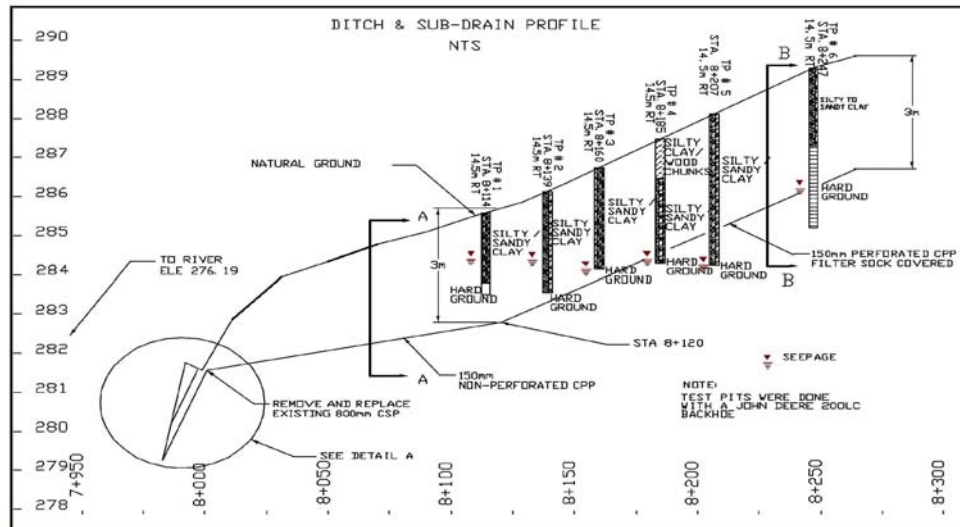


Figure 8: Profile of Ditch and Sub-surface Drain

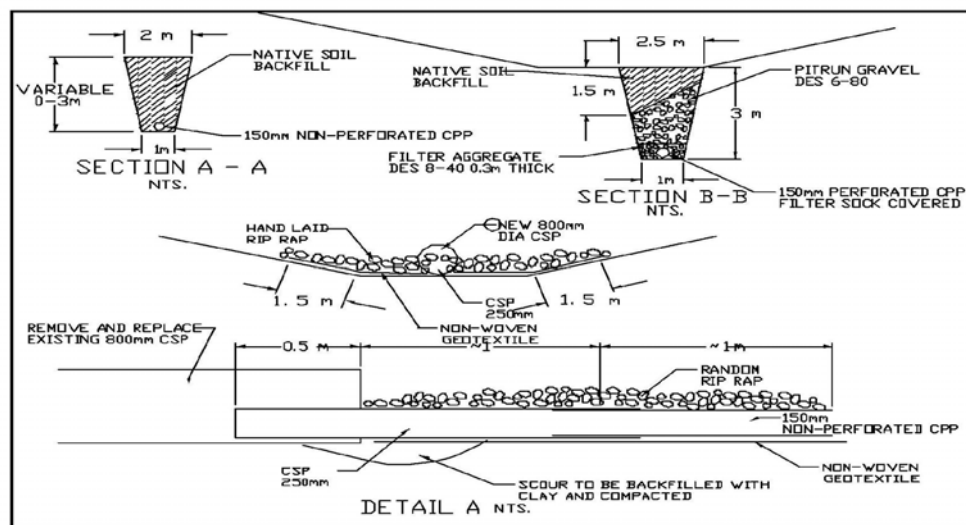


Figure 9: Details of the Sub-surface Drains

E. PROJECT TENDER

During the time of detailed design and tender preparation, the slide activity continued and a larger area of the roadway slumped within the limits of the proposed construction. This led to the closure of both lanes of the roadway until the construction was completed. This further slumping resulted in revised quantities for asphalt concrete from 36 to 550 t to resurface the roadway and increase in pile quantity from the design quantity of 780 m to 975 m as the depth of was increased from 12 m to 15 m. This resulted in an increase in the initial design estimate from \$285,000.00 to \$352,000 inclusive of 10 % contingency. The project was tendered by open competition in the first week of July and bids received on July 20 for review.

Bids obtained from three Contractors ranged from \$445,000 to \$605,000. The lowest bid price was about 27% above the design estimate and resulted primarily from the increase in piling cost from an estimated price of \$175 per metre, at the design stage to \$225 per metre, by the lowest bidder. Costs for this item by other bidders were \$288 and \$350 per metre. The higher unit rate for this item resulted mainly from the lack of availability of drilled shaft piling rigs due to an increase in construction activity at the time. The project was awarded on July 29 to the low bidder at a price of \$445,071.85 for an estimated forty four days of construction starting August 3 with completion date of September 15, 1999.

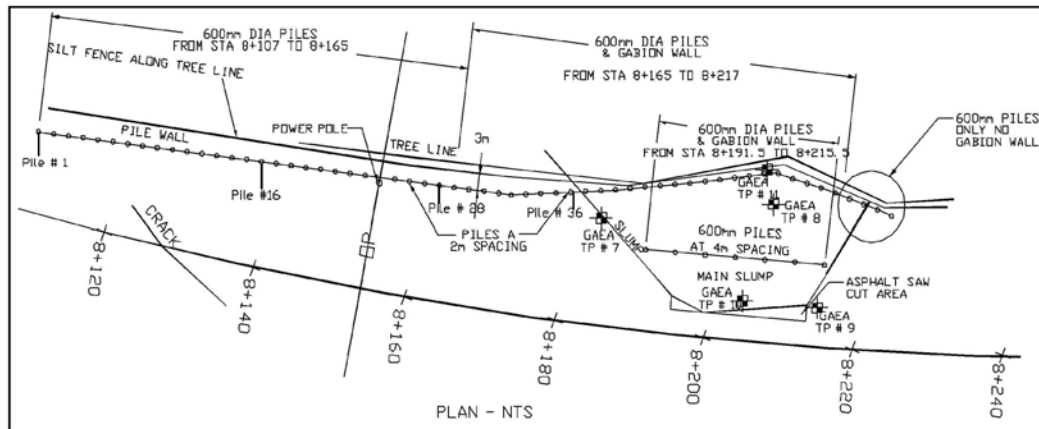


Figure 10: Plan Showing Layout of Pile Walls and Testpits in Slumped Section

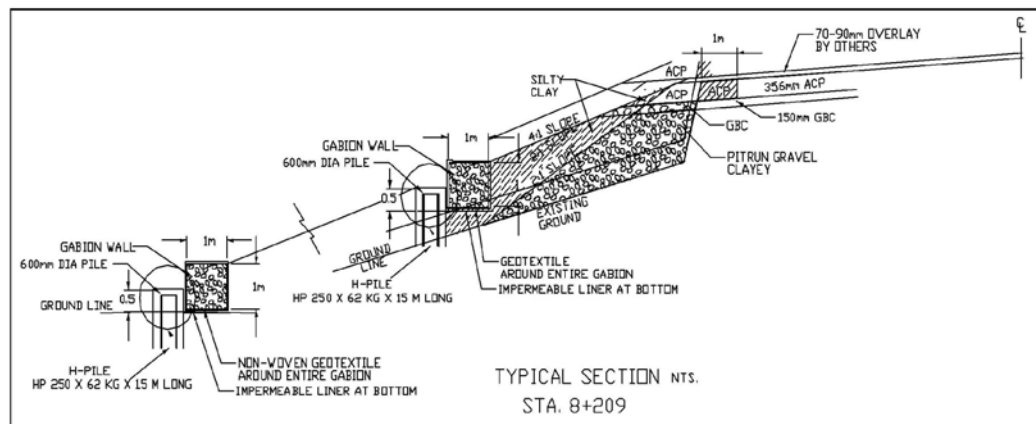


Figure 11: Cross-Section Showing Pile Walls and Slope Rehabilitation

F. DETAILS OF PROJECT CONSTRUCTION

The preconstruction meeting was held on August 5 and actual field work initiated on August 9 with the installation of the subsurface drain. This was followed by the installation of the drilled shaft piles, placement of gabion baskets, reshaping of the failed slope, application of asphalt concrete by grader and paver to repair the roadway surface, and topsoiling and seeding. All work was completed on September 10. Piling was started at the south end of the project limits. For the first 16 piles the fill varied from 1.5 to 4m and seepage noted around 4 to 5 m. From pile #16

to pile #28 there was noticeable more water and more sloughing. This water and sloughing was not serious enough to prevent installation of these piles. However, from pile # 27 to # 65, a 6m length of temporary casing had to be used in each hole to prevent sloughing.

From pile # 36 onwards, large amounts of water were observed to be coming from the bottom of the drilled holes at a depth of 15 – 15.4 m. While installing pile # 37, water was observed to seep in very quickly after reaching the desired depth resulting in 2.5 to 3 m of water accumulating in the hole before the concrete was poured. For the remainder of holes the strategy used was to drill to a depth of 12 m, wait until the concrete truck was on site and then drill to the desired depth. This approach was reported to have worked well.

A final inspection of the works was undertaken on September 16 with the Contractor and MD Yellowhead and corrective work identified during this inspection, consisting mainly of surface cosmetics, was completed by the Contractor on September 19, 1999. The final cost of the project was \$448,018.01 which was 7.71% over the tender sum and resulted from an increase in the quantities of topsoiling. Photographs of some aspects of the construction and the highway condition in May 2005 are shown in Figures 12 to 17.

VII STATUS OF SLIDE AREA

As mentioned in the “Introduction” section of this paper, the site was inspected in December 2005 and conditions found to be satisfactory in the area that was subject to the remedial measures. However, a short crack was observed at the roadway shoulder edge in 2002. There is uncertainty of the cause of the crack and recommendations have been made by others responsible for monitoring this site to undertake installation of slope indicators for determining the nature of the problem. The crack occurring some 40 m to the south of pile #1 places the distressed area within the influence of the bend of the river. There is, therefore, a possibility that some erosion activity may be causing this problem.

On the basis of the history of this site and the proximity of the bridge, the need for evaluating and addressing the problem at an early stage needs to be given a higher priority than the risk level of 24 that has been assigned to this area. The benefit of stabilizing the area before the roadway falls apart is perhaps one of the most effective means of dealing with many roadway embankment slides.



Figure 12: Sub-Drain Installation – Solid Pipe



Figure 13: Piling Installation - Lower Level



Figure 14: Piling Installation – Upper Level



Figure 15: Upper level Gabion Basket Wall



Figure 16: Highway Section-September 1999



Figure 17: Highway Section - May 2005

VIII GENERAL DISCUSSION

The 14-year period during which this slide area was maintained is often typical for many slide areas along highways. Maintenance by patching using asphalt concrete mix is generally the first course of action used when distress is observed on a paved highway. This asphalt concrete mix provides a smooth surface for safe traffic movements. If the highway is unpaved, the distress may go unnoticed for years as periodic blading and shaping does not allow initial signs of instability to be readily noticed. In fact, in areas with known slide activity, leaving highways without the application of an asphalt concrete pavement is often a cheaper maintenance approach.

A visual site inspection often follows the initial observation of slide activity. Based on observations from this inspection, decisions are made whether to investigate and monitor. Generally instrumentation would be the next recourse, while gathering information for possible design of remedial measures. Remedial action may be delayed if the roadway does not show progressive deterioration or instrumentation shows small rates of movements. This decision is often based on the importance of the highway, assessment by the geotechnical engineer, and availability of funding. The cost of immediate remediation versus the cost for maintenance cannot be properly evaluated as the maintenance costs will depend on the time over which the highway will be maintained until it is deemed necessary for remedial action. The only certainty for action is when the highway collapses or when there is a complaint made to Politicians by the traveling public. However, the result of this

waiting process could seriously affect the types and costs of remedial measures that are eventually required to be used in comparison with costs and solutions that may be required before the roadway reaches a state of collapse or is deemed unfit for service. The risk assessment procedure being currently employed by Alberta Infrastructure and Transportation for evaluating highway landslide geohazards does not incorporate the influence of early versus late application of remedial measures. These need to be addressed for effective decision making.

In hindsight, in this particular case study, the slide should have been addressed by undertaking the recommended investigations since 1986. This would have resulted in the installation of the subsurface drain as the first course of action. Regular slope indicator monitoring should have been undertaken to evaluate the performance of the site for the implementation of further remedial measures, if warranted. Given the site constraints and proximity of the river, it is possible that the pile retaining wall would have been the desirable solution. Had this solution been implemented before 1997, the lower level pile wall and elaborate rehabilitation of the highway and side slopes may not have been required. These measures would have resulted in reduced overall costs since construction rates and material costs would have been much lower at that time.

IX CONCLUSION

This case history has demonstrated that the need for stabilizing a highway landslide geohazard at an early stage is often necessary to avoid deterioration of the highway which would result in less costly remedial measures. Very often, if breaking up of the ground can be prevented then implementation of solutions can be undertaken much easier. Despite the consequences that led to the delay in the implementation of remedial measures, the outcome has been successful with the stabilization measures used in this landslide geohazard over the last 8 years. However, as with any stabilized landslide areas, monitoring both visually and by instrumentation must continue in future years.

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ACKNOWLEDGEMENT

The work presented has resulted from the accumulation of information in memos produced over the years by the GSS of Alberta Transportation. The OMCL report was made available by the MD of Yellowhead to GAEA Engineering Ltd in 1999 for the final design of the remedial measures. Information on the slide area from 1985-1992 was obtained from past AT&U files while the status of the slide area since its stabilization was obtained from the AIT website (www.Infratrans.gov.ab.ca).