

Stabilization of Roadway Landslide Using Anchored Drilled Shaft Piles Performance Evaluation over the Last 15 years

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ABSTRACT: The use of drilled shaft concrete piles is generally not a widespread method of stabilization of landslides along Alberta Highways as a result of perceived high costs and the uncertainty of performance of these structures in the long term. However, over the last thirty (30) years, the use of drilled shaft piles has been the preferred method of stabilization of various sites where conventional methods were not feasible either because of site constraints or not considered appropriate. This paper presents a case history of a roadway embankment slide for which the most appropriate remedial measure was determined to be the use of a tie-back drilled shaft concrete pile retaining wall as a result of the deep seated nature of the slide, inability to relocate the roadway, and concerns using a toe berm. Stabilized in 1997, the roadway and pile wall are still performing satisfactorily despite some observed roadway undulations and movements of the sideslope behind the pile wall. This paper addresses the details of the site and slide activity, the judgments exercised in deciding the locations and depth of the concrete piles, and the design and construction of the pile wall retaining system.

INTRODUCTION

Slope instability problems along a two (2) km stretch of Secondary Highway (SH744:04) known as the Judah Hill Road located in Peace River, Alberta (Fig.1) were first reported in 1984 following its regrading and asphalt paving. Since then, this highway has been the subject of detailed investigations and slope indicator monitoring, together with implementation of remedial measures in a staged approach.

This highway which provides an important link to the Town of Peace River, Alberta, has to be kept trafficable to avoid an almost fifty four (54) km increase in distance to access the Town. Over the years a number of unstable areas have developed along this two (2) km stretch of highway resulting in this highway being classed as slide prone. The topography and geologic history of the Peace River valley slope influences the behaviour of the Judah Hill Road. As shown in Fig. 1, this roadway is located near the crest of an ancient valley slope of the Peace River and is situated below the top of a narrow ridge that separates the valley slopes of the Peace and Heart Rivers. These valley slopes consist of ancient landslides created by the downcutting of the rivers in the geologic past (post glacial period) (Mollard 1977).



FIG.1. Location of Peace River

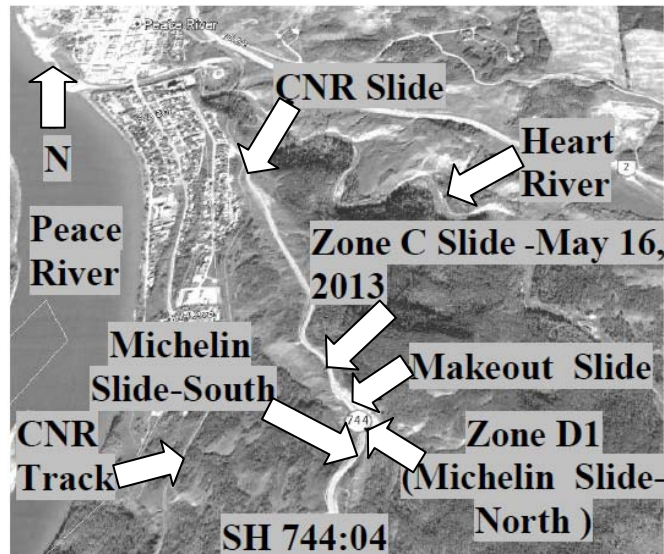


FIG.2. Location of Major Slides SH 744:04

It is well known that these ancient slides are readily reactivated by very minor construction activity or environmental effects such as rainfall and snowmelt precipitation. The latter effects appear to influence behaviour when run-off becomes concentrated as a result of the paved surface of the roadway. Without the presence of a roadway slope failures are still expected and do occur because of natural processes such as rainfall and river erosion.

This paper addresses the stabilization and performance of the Zone D1(also known as the Michelin Slide-North) slide along this stretch of highway located approximately 1.5 km south of the CNR at-grade crossing (Fig.2).

THE ZONE D1 (MICHELIN SLIDE-NORTH) LANDSLIDE

General

The Zone D1 (Michelin Slide-North) is located between Sta 57+760 and 58+000 along the Judah Hill road and was the second major landslide to occur along this highway. This slide area was investigated and studied from 1988 to 1994 by the Geotechnical Services Section (GSS) of Alberta Transportation (AT) under the leadership of the author, who was during that period Assistant Director or Head of the GSS.

The first major landslide along the Judah Hill Road was located at the CNR at-grade crossing of the Judah Hill Road which was considered the northern end of the two (2) km landslide prone stretch of the Judah Hill Road. The highway showed signs of distress about one week after asphalt paving in 1984 resulting in track settlement and development of tension cracks along the right-of-way in the vicinity of the track crossing. Details of this landslide and its remediation, which included stabilization by a tied-back pile wall, were reported by Diyaljee (1992).

The instability problem at the Michelin–North slide location was reported since 1985 following the construction of an asphalt paved roadway surface. As the years progressed cracks that had developed became progressively wider and depressions occurred in the wheel paths. The entire two (2) km Judah Hill Road, which includes the Zone D1 area, was monitored visually by the Maintenance Forces from AT's Peace River District Office, which was less than fifteen (15) minutes by road from the Judah Hill Road.

In contrast, seven (7) hours of road travel were required from the Head Office of AT in Edmonton, Alberta from where the GSS Engineering staff operated and addressed the geotechnical issues of the provincial roadway infrastructure of roads and bridges under the jurisdiction of AT. The initial stabilization measures using stone columns, slope indicator instrumentation installation and monitoring and the final stabilization measure using a tied back pile retaining wall are described in the sections that follow.

Stone Column Remedial Measure (1988)

Following a report in August 1988 from the District that the roadway in the vicinity of the Zone D1 slide was undergoing significant cracking, a site inspection and investigation was undertaken by the GSS of AT subsequently.

The geotechnical investigation was conducted by drilling twelve (12) test holes with a Texoma 700 piling rig. From the visual observations made of the pavement distress and testhole logs in 1988, the problem was diagnosed to be a loss of bearing capacity through degradation of the sideslope due to rainfall and snow melt runoff. The areas where this problem did not occur seemed to be associated with areas where runoff was directed towards the backslope ditch.

To prevent deterioration of the roadway stone columns were installed between Sta 57+760 and Sta 58+800 to improve the overall resistance of the subsoils in the top 3 m where the traffic stresses were the greatest. In addition, a series of curbs and catch basins were designed to prevent water running off the pavement surface onto the steep sideslope. Further details on the design and method of installation of stone columns at this and other locations in Alberta in highway construction related to settlement, bearing capacity and slope stability issues during the period 1980 to 1988 are provided by Diyaljee and Pariti (1990). The stone columns appeared to work satisfactorily until about August 1992 after which deterioration of the roadway surface was again evident.

Instrumentation Installation, Monitoring and Site Evaluation (1986 -1994)

Five (5) slope indicators were installed within the Zone 1 slide during the period 1986 and 1994 in testholes that were drilled to depths varying from 20 to 36 m below the ground surface. Of these, one (SI 1C) was installed in 1986, SI 13, 34 and 43 in 1992, and SI 42 in 1994. These slope indicators were installed by the GSS of AT. SI 1C installed to a depth of 30 m north of the scarp of the slide showed between 1991 and 1996 movements of 60 mm in a distinct shear zone at a depth of 7.5 m and 7 mm in a shear zone at 17 m depth.

Slope Indicators (SI 42 and 43) were located downslope of the slide to the north to monitor potential movement on a larger scale. SI 42 indicated shear movement at a depth of 28 m. The locations of the slope indicators installed are shown on plan in Fig.3. Around March 1994, the District reported that there was significant progressive deterioration of Zone D1 slide. This report prompted an immediate overview of the concerns of the instabilities both with the slide in question as well as other areas along the Judah Hill Road that were undergoing deterioration.

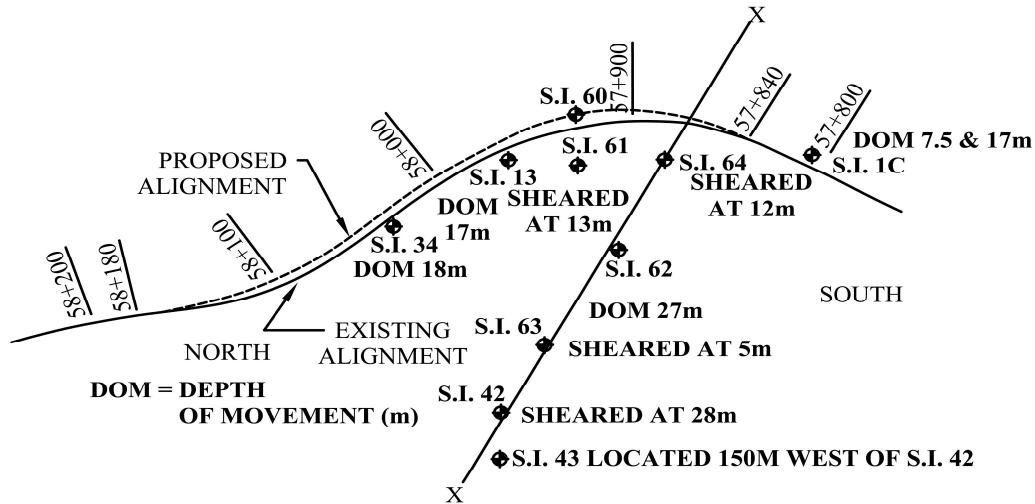


FIG. 3. Location of Slope Indicators and Depths of Movement

These concerns were discussed on April 19, 1994 at a meeting at the Office of the Municipal District (MD) of East Peace #131 in Peace River, with the MD, District, and Town of Peace River officials, and a GSS representative in attendance. This meeting also served to discuss with the MD the issues associated with the entire Judah Hill Road since responsibilities for the roadway's upkeep was transferred from AT to the MD on April 1, 1994.

Based on the overall nature of the instabilities, terrain constraints, associated projected costs for remediation, and lack of an immediate economic alternative route for this roadway the MD and Town of Peace River officials preferred that the present highway alignment should be maintained with a major shift to the west of the slide, and also reverting this paved roadway to a gravel road. This decision was also underscored by the Location Services, Planning Branch of AT, who were requested to evaluate the feasibility of alternative routes.

A subsequent meeting was held on July 29, 1994 in Peace River with the MD, District and the GSS representatives including the author. This meeting was centered on how the existing highway should be maintained.

The nature of instabilities were reviewed along the entire two (2) km stretch of the Judah Hill Road and it was advised that while some attempt could be made to stabilize the Zone D1 section other areas showing distress would require stabilization immediately or shortly afterwards.

Following the meeting a site review was undertaken at the roadway location and areas downhill. The ridge area at Zone D1 was first reviewed to assess whether a major shift of the highway was feasible. This review confirmed the initial suspicions

that the cracks uphill of the roadway would make the suggestion of a major shift of the alignment an undesirable choice since it would impact the valley slopes of the Heart River, and promote instability in that area compounding the problem. It was considered that a better choice would be to remain on the same general alignment and undertake remediation as suggested at the April 19, 1984 meeting.

A walk uphill across the rail tracks toward the Zone D1 slide along a line approximately perpendicular to the Judah Hill Road showed scarps and cracks uphill and downhill of the CNR tracks and distorted fences along the backyards of the residences in the vicinity. This observation further confirmed the general opinion that the slides that were occurring along the Judah Hill Road were likely part of a larger slide activity involving the Town and exiting at the Peace River banks. The tentative geotechnical recommendations for stabilizing the Zone D1 slide area included the installation of a pile retaining wall system. An earthwork solution incorporating a berm construction was not considered appropriate. However, because of the projected expenditures for this and other slide areas along this roadway and the shift in responsibilities of this highway from AT to the MD, no further work was pursued in relation to this slide area.

Site Inspection and Evaluation (1996)

Following significant movements of the highway during the heavy Province wide rainfall in the week of June 14, 1996, this site was again inspected by the author on June 21, 1996 at the request of the District on behalf of the MD. With the responsibilities for the highway under the jurisdiction of the MD and “Privatization and Downsizing of AT” being actively pursued by AT in Edmonton, GAEA Engineering Ltd, an Alberta Consulting Engineering Firm, under the management of the author, was commissioned by the MD in August 1996 to undertake a further evaluation of the slide area including the design, construction supervision and contract administration of the proposed remediation measures.

Observations along the roadway section of the slide area showed that the head of the slide had dropped a further 0.5 m to 1 m during the rainy period and that the headscarp cracks which are visible along the uphill ridge (hogsback) had opened up significantly, Fig.4. There was, therefore, a general concern by the MD that these cracks, if not sealed, would allow rainfall runoff to infiltrate the ground and cause further movement of the slide area.



FIG.4. Vertical Drop in Roadway and Backslope Michelin Slide - North

On site discussions at the headscarp location consisted of the following:

1. Displacement of the sliding mass by the addition of fill overtop of the roadway.
2. Sealing of cracks uphill and downhill of the roadway section.
3. Re-routing of ditch drainage away from the headscarp and downhill of the slide. A centreline pipe uphill of the slide area was observed to be discharging into the slide area which was badly cracked
4. The use of lightweight fill material within the roadway to bring the sunken portion of the road to grade through excavation of about 2 m within existing road and replacement with hog fuel or sawdust.

A concern was expressed by the author of the possibility of movement downhill of the slide area as had been previously indicated in AT's slide evaluation reports. The last report submitted in March 1996 indicated that movements were recorded by SI No. 42. This was consistent with predicted possible behaviour of this landslide area reported in the past evaluation reports which alluded to opinion that this slide was part of a larger slide area likely encompassing the slides within the Town of Peace River in this vicinity.

To confirm this recommendation slope indicator monitoring information for SI 42, 43 and 41 were examined. This information was provided by GAEA Engineering Ltd, Edmonton Office via fax the same day of the site inspection which showed that further movements were occurring at SI 42 at a depth of 28 m as previously recorded. No other perceptible movement zones were observed for the other SI's.

Following the examination of the headscarp area a review was made of the downhill side of the slide area by walking uphill from the CNR tracks. During this site review the SI locations were examined and surrounding ground observed. The toe of the slide was a short distance uphill of SI 42. This distance appeared to be closer than previously observed when the slide had first occurred.

It is important to note here that as the moving mass progresses downhill it would add load to other areas and possibly headscarps of other dormant slides. The resulting effect would be to create instability of another section of ground. This then progresses in a somewhat 'domino' effect. Hence, it is important not to load the downslope indiscriminately above and beyond what the natural process was currently doing.

Seepage was noted to be pronounced at the toe of the slide area in an area which was judged to be a previous drainage gully, which might have been the watercourse created by the centreline pipe discharge over the years. It was suggested that in any regrading of the slide area this seepage would be required to be maintained rather than be blocked. The use of a gravel trench drain daylighting outside of the regraded toe would be desirable.

Additional Slope Indicator Installation (1996)

Five (5) slope indicators in addition to the ones previously installed at the Zone D1 site by the GSS were installed by GAEA Engineering Ltd between August 12 and August 24, 1996. The locations of these additional slope indicators are shown

in Fig 3. These were installed with the aim of defining properly the slide depths and to assist in evaluating the remedial options. Before the installation of these slope indicators were undertaken the slide area was cleared of all vegetation between the roadway and SI 42 over a width of 200 m so that the site could be inspected.

In addition, the drainage uphill (south) of the slide area previously directed to the slide area by a centreline culvert across the roadway was redirected to flow along the back slope ditch via a 600 mm diameter “Big-O-Pipe” to a location outside of the slide area. This measure was undertaken to avoid runoff entering the large cracks that had opened within the slide area along the backslope and side slope of the slide area.

The slope indicators were read three times after their installation and confirmed the depths of movements and the active nature of the slide. SI 61 and 62 sheared within a few days of installation showing movements at 13 and 5 m depth respectively while SI 63 and 64 showed shear zones at 27 and 12 m respectively.

Fig.5 shows some of the soil stratigraphy at SI locations. The regional and site geology of the Judah Hill road have been reported previously (Diyaljee, 1992). Also shown in Fig.5 are the inferred planes of movement from the Judah Hill Road to the Peace River some 700 m to the west of the Judah Hill Road.

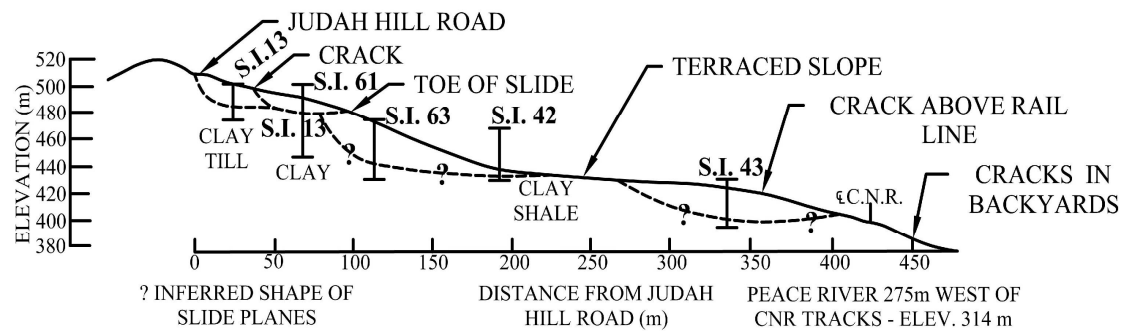


FIG.5. Cross-Section XX (Fig.3) through Slide Area

SLIDE REMEDIATION DESIGN CONSIDERATIONS

General

All SI's within Zone D1 slide were used to evaluate and assess the behaviour of the Zone D1 slide. The following inferences were made:

- The slope indicator plots generally showed deep-seated movements occurring at the contact between clay shale and the soil matrix above. The interpretation from this and previous observations is that a series of slides are occurring along the entire valley face.
- From the slope indicator data and a visual inspection of the site the area requiring immediate attention clearly spans a distance of 180 m from approximately Sta 57+870 to 58+050.

Various forms of remedial measures were possible independent of each other or in combination to minimize the distress on the roadway performance. However, for this site considerable engineering judgment was needed to arrive at a most logical and cost effective solution. Two scenarios were examined as indicated below:

- Maintaining the existing alignment by stabilizing the slide area
- Shifting the roadway alignment slightly to the backslope toward the top of the ridge between the Heart and Peace Rivers.

Normally a toe berm constructed at the toe of the slide is a common remedial measure used to stabilize the slide. However, this option was not considered desirable since any placement of an additional load on the valley slope would have triggered more complex slide activity by further activating ancient landslides.

SI's 42 and 43 installed at 180 and 420 m downslope of the roadway centreline slide were showing movements at depths of 28 m El 456 and 30 m El 400 below ground level. These movements suggested the general fragile stability of the valley slopes that as mentioned previously can be disturbed quite readily by minor construction or other activity. The merits and demerits of scenarios identified above and others required to improve the stability of the roadway are discussed below.

Slight Uphill Shift in Alignment

To improve the stability of the slide area along the existing alignment given the fragile nature of the site was to use a pile-wall system with tie-backs. This was based also on the satisfactory performance of a similar wall installed at the CNR crossing in 1988. Since ground movements were occurring between 14 and 20 m below the existing roadway elevation, 20 - 25 m long concrete piles spaced 2 m on centres and two rows of tie-backs were evaluated to be required.

Shift in Alignment

A realignment option would involve moving the roadway slightly into the side slope of the narrow ridge. This relocation would cover 340 m length of the roadway from Sta 57+840 to Sta 58+180. However, three (3) constraints had to be considered for this shift in alignment:

- Since the roadway is following a ridge, the shift in alignment could not be too far away from the existing alignment because of the proximity of the steep valley face of the Heart River. This slope can also exhibit slope instability problems since the geologic process forming this slope is similar to that of the Peace River and Heart River valley slopes.
- The side slope of the roadway along the new alignment would still be within the zone of instability and therefore would still need a pile retaining wall system
- A Northern Utilities (NUL) gas pipe line near the top of the ridge would have to be relocated.

The main advantage of the shift in alignment is that it would enable the design of a less expensive pile-wall system within the critical area. Preliminary design showed a 20 m deep pile-wall system with two levels of tie-backs. For a shift in alignment additional costs would have to be accommodated as follows:

- Grading costs associated with the realignment
- Cost of relocating the gas pipeline at the top of the backslope

Lightweight Fill

The use of lightweight fill material was initially considered for use in stabilizing this slide area. Ideally, a sufficient depth of existing soil is required to be replaced with a lighter material to reduce the driving forces contributing to the instability of the slide area. However, because of the geometry of the slide, general topography of the area and risks that would be associated in undertaking very deep excavation in the roadway area a depth of 5 m was considered reasonable. This depth of material would be removed and replaced with a lightweight material such as sawdust, used whole tires, used tire shreds or a combination of these materials.

This procedure could be applied to either the existing or realigned roadway. Since the roadway will be retained by a pile wall system for both scenarios, it was determined that the application of the lightweight fill material would be the most cost effective option. A direct advantage of using a lightweight material in the roadway subgrade was the use of a more economical pile wall system by reducing the driving forces contributing to the slide movement. A 30% reduction in the driving forces and moments have been estimated with the use of this lightweight fill. A precaution necessary for the execution of this work was to undertake the construction in short sections of not greater than 25 m in length. This was necessary to prevent possible slumping of the ridge uphill of the roadway and impairing the overall stability of the area. This will also require closing the roadway for the duration of the work operations.

The lightweight material chosen was shredded tires, which was approved by the Tire Recycling Management Association of Alberta (TRMA) with financial assistance of \$45 per tonne to assist in offsetting transportation, engineering and installation costs. One of the stipulations of the TRMA was that drainage of the shredded tire be undertaken. This was implemented in the final design shown in Fig.6.

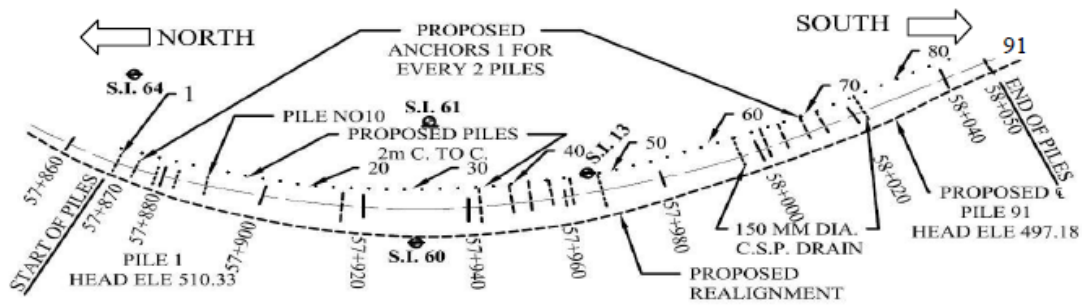
Removal of Slide Material from the Side slope of the Roadway

Ideally the removal of the slide material to the depth of the zone of movement would allow for rebuilding the excavated area with better material. This operation would, however, result in a very large depth of excavation - at least 15 m at the road shoulder and increase the risk of instability of the ridge area considerably.

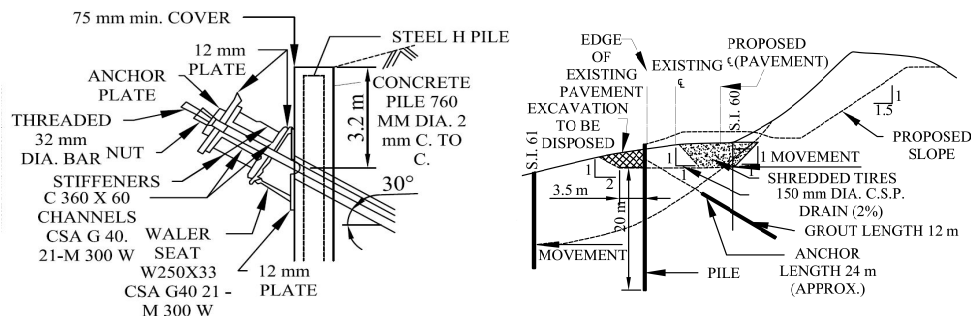
Since impairing the instability of the ridge area would have serious long term consequences on the roadway, this option, while a possible solution, was not recommended. Instead, partial excavation of the slide material was recommended. The removal of the slide material downslope of the slide was considered important to prevent areas downslope from being loaded further by further movement of the slide material.

FINAL RECOMMENDATIONS

The final recommendations for stabilizing the site consisted of the following in the order outlined. (1) Installation of tie-back pile wall. (2) Relocation of the NUL gas line. (3). Shift alignment toward backslope. (4) Excavate the roadbed and replace with lightweight fill. (5) Excavation of slide material along backslope. (6) Installation of ditch lining. (7) Application of gravel surfacing on completed grade. (8) Surfacing the roadway with asphalt concrete.



Plan Layout of Piles, Anchors and Drains



Anchor Head Details

Cross section at Sta 57+920

FIG.6. Plan and Cross-Section of Pile Wall

DESIGN AND CONSTRUCTION OF PILE WALL RETAINING

General

The forces to be retained by pile wall system was undertaken using the Free Earth Support Approach, an assumed pile spacing of 2m on centres, concrete pile size of 760 mm diameter each reinforced with an HP 310 x 94 kg/m designation CSA G40.21-M 300 W steel section as shown in Fig. 6. Both the structural and geotechnical design of the piles, and tiebacks were undertaken by GAEA Engineering Ltd while the walers and connections to lock the tie-rods to the piles were undertaken by the Contractor and cross-checked by GAEA Engineering Ltd. The installation of the pile wall retention system was undertaken in two stages (Stage 1 which consisted of the vertical pile installation) and (Stage 2 which consisted of the tie-back installation). The construction entire pile retaining wall system began on March 6, 1997 and was completed on July 22, 1997.

Design and Construction of Piles (Stage 1) of Pile Retaining Wall

Ninety one (91) piles were designed to cover a distance of 180 m. Based on the S.I movements sixty (60) of these piles (Nos 1 to 60) were installed from Sta 57+870 to Sta 57+990 each to a depth of 20 m while the remaining thirty one (31) piles were each installed to a depth of 24 m. Pile concrete was designed using Type 10 cement for a 28-day compressive strength of 30 MPa, slump of 120-150 mm and air content of 5.5 % and piles installed during March and April of 1997 as the first stage of the remediation design. Compressive strength cylinders tests were taken during each pile installation and tested after 7 and 28 days. All 7 day results averaged round 25 MPa while the 28-day strength averaged around 32 MPa. Fig.7. shows the completed piles. Twenty (20) selected spliced H-Pile steel reinforcement were subject to Magnetic Particle Testing of the welds to acceptance standard of CSA W59 Clause 11. In general the steel piles subjected to visual inspection and wet colour contrast showed no defects. In summary all piles were constructed to meet the design requirements. The pile holes were generally dry requiring no temporary casing to be used except for holes 5 through 19 where some wet areas were encountered between 4 and 14 m requiring the use of casing to prevent sloughing. The casing was removed after concreting.



FIG.7. Installed Vertical Piles

Design and Construction of Tie-back Anchors (Stage 2) of Pile Retaining Wall

As a result of cost considerations one level of tie-back anchors was designed rather than the two levels that were initially projected. Two levels of anchors would have required a deeper excavation and a much higher construction cost. The anchor force per tie-back was determined to be 300 kN with each tie-back located at a depth of 3.2 m below the top of pile and inclined at an angle of 30 degrees from the horizontal. Each tie-back installed midway between two piles consisted of 32 mm diameter double corrosion protected Dywidag thread bar anchors conforming to CSA G 279 prestressing steel Grade 1035 MPa, and Modulus of Elasticity of 205,000 MPa. The double corrosion system consisted of 56 mm diameter corrugated PVC sheathing with a minimum thickness of 1.2 mm, compressive strength of 102 MPa and tensile strength of 48 MPa. The annular space between the bar and the corrugated sheathing was shop grouted prior to shipment to site. The debonding materials for the free stressing length of the anchor consisted of 59

mm diameter polyethylene sheathing which was assembled over the corrugated PVC sheathing for the portion of the anchor located in the free stressing length of the anchor. Each anchor was approximately 24 m long with a grout length of 12 m.

One tie-back was installed between every two piles and was given a designation as follows. Between pile 1 and 2 the tie was designated tie-back 1.5, between piles 4 and 5, tie-back 4.5 and so on. A total of forty six (46) tie-backs were installed. The tie-back anchor grout mix design used consisted of Type 10 cement water cement ratio of 0.45, design strength of 35 MPa at 7 days. Six mortar cubes of the grout mix tested before field installation of anchors showed an average 7-day compressive strength of 60 MPa. Grout testing was undertaken for each tie-back installed. Tests during installation showed 7 day strengths varying from 34 to 72 MPa and 28 day strengths from 31 to 76 MPa, satisfying the design requirements.

Four different types of stress testing were undertaken on the tie-backs. The type of tests and the tie-backs on which they were undertaken are shown in Table 1.

Table 1. Types of Tests for Quality Assurance of Tie-backs

Test	Tie Back No
Creep	22.5 and 42.5
Performance Test	4.5, 62.5 and 84.5
Proof Test	All Remaining Tiebacks

The initial lift off test, the fourth test was supposed to be done on four tie-backs at the same time as the creep and performance test prior to the sequence of proof testing, but was not done at the desirable timing. These tests were done in the month of July on seven (7) tie-backs as follows - 12.5 , 14.5, 24.5, 36.5, 40.5 76.5 and 78.5. Before the proof testing was undertaken and based on a review of the creep test results it was agreed with the Contractor that the tie-backs would be locked off at 375 kN instead of 300 kN as had been originally designed. All tests on tie-backs were taken to a load of 450 kN. During the proof testing, five of the tie-backs showed elastic movements that were to be within acceptable limits or marginally above the acceptable limits. The evaluation of the results of each test was based on the general guidelines set out in the Canadian Foundation Engineering Manual, 3rd Edition 1992. Figs.7 to 10 show the installation of the tie-back and waler system while Fig.11 shows the tie-back test setup.



FIG.8. Installation of Tie-Backs



FIG.9. Installation of Waler Seat

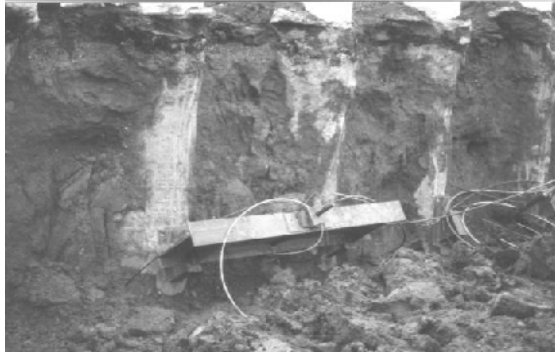


FIG.10 . Installation of Waler



FIG.11. Tie-back test Set-up

VISUAL INSPECTION AND SLOPE INDICATOR MONITORING

Since the completion of installation of the pile wall retention system and the excavation and replacement of the contorted grade uphill of the wall with lightweight fill, the stabilized system has been monitored on a yearly basis initially by GAEA Engineering and since around 2005 by other Engineering Consultants. The yearly results of these evaluations are reported in the Alberta Transportation Web site (www.transportation.alberta.ca) under Technical Resources-Geotechnical and Erosion Control – Annual Landslides Assessment-Peace River/High Level.

Over the years up to the last site review in 2012, some 16 years since stabilization, and a further review by Roy Callioux, P.Eng (Private Communication) on May 13,2013 a noticeable drop of the sideslope downhill of the pile retaining wall has been observed. Approximately 3-4 m of the wall is now exposed on the downhill side, Fig. 12. As well, there is the development of a slide (known as the Michelin Slide–South) which was stabilized by others in 1999 that is to the south of the Michelin Slide–North and the Makeout Slide, Fig.2. The Makeout Slide occurred in 2005 and was stabilized the same year using an earthworks solution consisting of a toe berm with geogrid reinforcement. This slide is also showing some distress as noted in Fig.14.



FIG.12. Michelin Slide - 2012



**FIG. 13. CNR Slide, May 13, 2013
25 years after Stabilization**

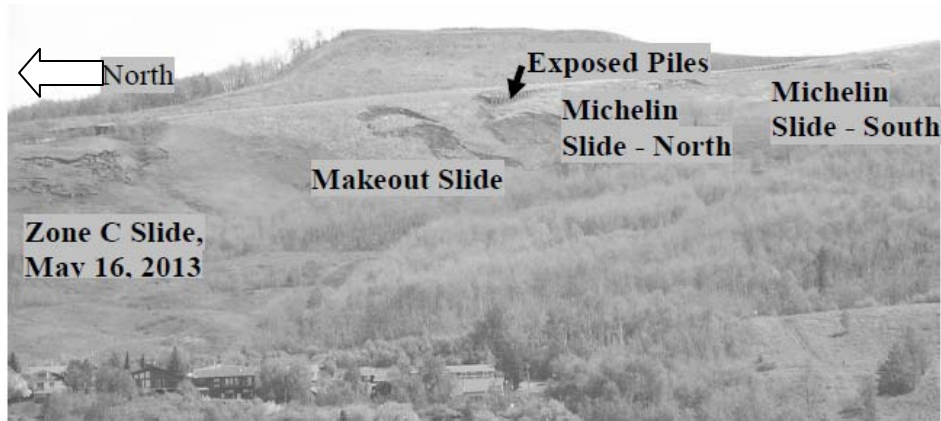


FIG.14. Looking Uphill at Slides on Judah Hill Road, May 22, 2013

It is the prevailing opinion that the recent development of the Michelin-South slide is part of a larger slide area encompassing the Michelin-South slides and Makeout slides. It has been noted that at the Michelin South slide location the slide movement has now encompassed the Heart River slope and the tentative recommendation by others is the construction of a double retaining wall with an estimated projected expenditure of C\$2.5 to C\$5 million dollars.

In terms of the Michelin-North slide the loss of support downhill of the piles as shown in Fig. 12 is not considered critical as this loss was taken into consideration during the design of the pile wall. Nature has now provided the development to allow additional tiebacks to be installed at higher and lower levels of the piles. This same situation occurred at the CNR slide which after the loss of the passive support allowed tiebacks to be installed, Fig 13. However, the timing of this additional remedial work is critical and needs to be done before the soil completely moves off and downward to fully expose the toes of the piles. The time span to implement this additional work can be judged from the slope indicator monitoring. So far, slope indicator SI 43 is still intact and has shown 15 mm of cumulative movement in the A direction and 30 mm in the B direction since 2008.



FIG.15. Slide at Zone C May 16, 2013

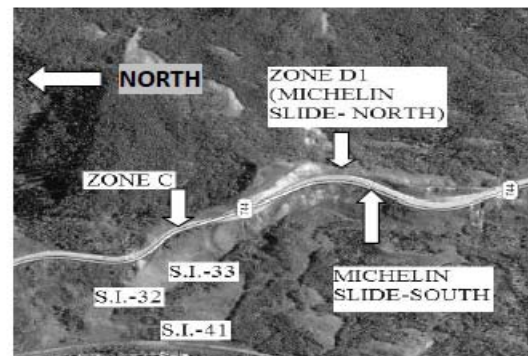


FIG.16. Location of SI 32, 33 and 41

Recently on May 16, 2013 a major landslide occurred just north of the Makeout slide (Fig.15) resulting in the closure of the entire Judah Hill Road. The approximate location of this slide is shown in Figs.2 and 14. This section of

roadway had been identified in 1994 as a potential slide area and slope indicators SI 32, 33 and 41 were installed in September 1994. These were read by the GSS six (6) times from October 1994 up to and including May 1996 and showed subtle movement zones at 12, 16 and 18 m (SI 32), 20, 25 and 36 m (SI 33), and 20, 30 and 34 m (SI 41). It is not known whether the SIs were read in subsequent years.

SUMMARY AND CONCLUSIONS

A tied-back drilled shaft retaining wall was used to stabilize a section of the two (2) km stretch of the Judah Hill roadway which has been undergoing major landsliding activity during the last 28 years. Since its stabilization in 1997 this section of roadway has been performing satisfactorily by maintaining unhindered traffic flow despite recent observations over the last year of isolated roadway undulations and continued movements of the sideslope behind the wall. The experience gained with slide stabilization along the Judah Hill roadway over the years has shown that traditional slope stabilization measures such as shear keys and toe berms are not as effective as the use of tied-back pile retaining walls. These walls can be used as in this and a previous case (Fig.13) to retain only the roadway prism rather than the entire sliding mass thereby being somewhat cheaper to implement.

As more slides occur (Fig.15) there will be a time when a decision may have to be made to undertake a full scale stabilization of this 2 km section of the Judah Hill Road. Tied-back retaining walls show much promise and are recommended. However, for such future walls it is imperative that a detailed instrumentation program be implemented to monitor the displacements, loads and strain in the structure with time. Such a program was not implemented as a result of jurisdictional changes in responsibility for the roadway coupled with the privatization of the GSS, and funding issues. These factors led to a lapse in much needed proactive monitoring.

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