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Stabilization of Judah Hill landslide

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ABSTRACT: This paper addresses the approaches used during a 7-year period from 1984 to stabilize the Judah Hill landslide. Stability was maintained through shifts in road and rail alignments, use of retaining walls, and subsurface drainage using deep vertical wells.

1 INTRODUCTION

The Judah Hill landslide (Fig.1) is located in the vicinity of the at-grade crossing of the Canadian National Railway (CNR) and Alberta Transportation and Utilities (AT&U) Secondary Highway (SH) 744:04 known as the Judah Hill road.

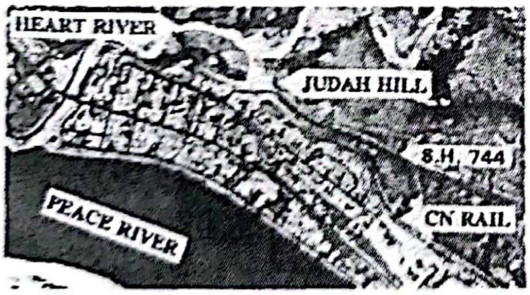

Fig.1 Location of landslide

The general location of the slide is near the confluence of the Peace and Heart rivers where the roadway grade climbs a narrow ridge between the two rivers. The slide is situated on the valley slope of the Heart river, approximately 40 m above river level.

The purpose of this paper is to provide background information into the developments of the Judah Hill slide from 1984 and the approaches used, in particular by AT&U , to maintain a stable highway grade.

2 BACKGROUND

SH 744.04 was maintained as a gravel roadway until 1984 when a conventional asphalt pavement was constructed. Prior to 1984, there were no reported slide problems along SH 744:04. However, approximately one week after completion of the asphalt pavement the CNR reported track settlement and the development of tension cracks along their right-of-way in the immediate vicinity of the track crossing. Shortly afterwards an attempt was made by CNR to support the grade using timber piles (Fig.2) but this did not prove successful.

Fig.2 Timber pile wall

The railway grade was realigned several metres to the west, and a new road crossing constructed immediately south of the original one. By July 12, 1984 cracks were visible in the new pavement on the east side of the tracks. The railway grade was shifted to the west once again before the beginning of Autumn 1984 and the road crossing shifted further south.

During this period, the slide encroached the roadway shoulder to the east of the crossing resulting in narrowing of the roadway. Cracks and depressions were also noticeable towards centreline of the roadway. The roadway was maintained during this period by patching with asphaltic concrete. On May 29, 1985, the slide was reported to progress into the northbound lane of SH744:04 causing cracks about 25 mm wide and some settlement of the lane (Fig.3).

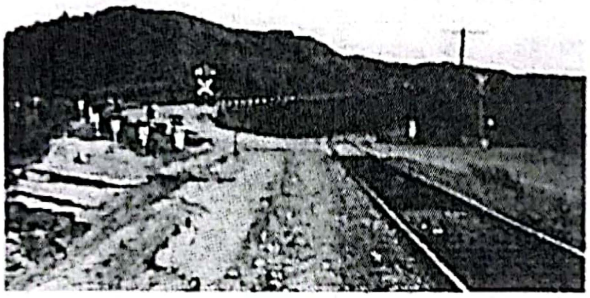

Fig.3 Roadway failure 1984-1985

Two slope inclinometers were installed in July 1985 outside the guardrail on the northbound lane. These slope inclinometers were anchored into shale bedrock about 30 metres below existing roadway level. Monitoring of the slope inclinometers showed movements at 13 to 20 m below ground level relative to the roadway surface elevation. The headscarp of the slide was also observed to be extending towards the centreline of the roadway.

Several meetings were held in 1985 and 1986 to discuss possible solutions to the instability problems as a joint effort. However, no definite decisions were made. Several possible solutions were discussed including an overpass grade separation structure or constructing a large diameter culvert in the Heart river to facilitate construction of a toe berm. Both of these solutions, while viable, were considered to be very costly. The approach used, thereafter, by AT&U was to improve surface run-off in the slide area and maintain the roadway through a maintenance operation.

Progressive movements of the railway grade were monitored over the next 10 months at which time accumulated settlement of the tracks and projections of continued movement prompted CNR to build a 25 m high stabilizing berm and reconstruct the failed sideslope up to track level. This work was completed early spring 1986.

On April 26, 1987, failure of a portion of the southeast flank of the berm occurred (Fig.4) resulting in the re-establishment of the headscarp closer to the tracks. The failure of the berm also resulted in movement progressing into the roadway grade.

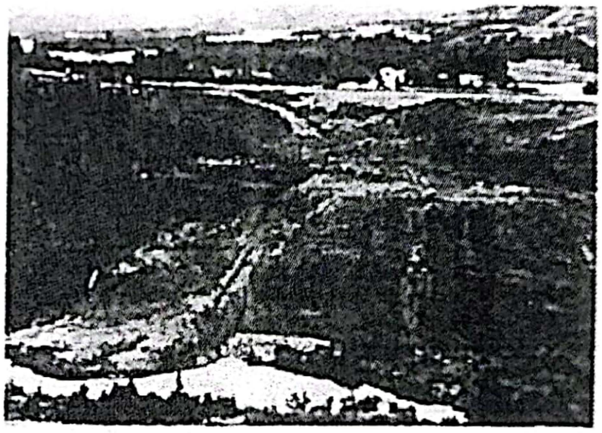

Fig.4 Failure of CNR berm

4 SITE GEOLOGICAL ASSESSMENT

Thurber Consultants Ltd., a local geotechnical consultant, was commissioned in 1986 by AT&U to undertake a study of the slide area. The following are some of the important geological features determined to be influencing stability of the area (Thurber, 1988).

4.1 Regional Geology

Bedrock in this region of the Peace River valley consists of flat-lying sandstones and shales. The topographic surface formed on this bedrock prior to glaciation was similar to the present surface with flat uplands and deep valleys. The valleys were comparable in depth but at least twice as wide with significantly lower slope angles. Glaciation caused the preglacial Peace and Heart River valleys to be infilled with sequences of clay till, lake clay and silt, and outwash sands and gravels. Following glaciation, rivers quickly cut down to their present levels, however the valleys are much narrower and have steeper slopes than the preglacial valleys.

Slopes evolved through landslide processes which continue to be locally active as the valleys slowly but progressively approach their preglacial width and slope angle. As a result of this geological history, slopes along the valley are mantled by re-worked glacial soils referred to as colluvium. In some places this colluvium overlies bedrock whereas at other locations it rests on undisturbed glacial soils which in turn rest on bedrock.

At the road-railroad crossing the erosion of the Peace and Heart rivers has resulted in a narrow spur between the two valleys.

4.2 Site Geology

At least eight (8) discrete surficial soil units are defined in the Judah Hill slide area. Most of these soils are listed, numbered and described in Fig.5.

Bedrock underlying slopes in the immediate vicinity of the site consists of two rock units, the Peace River and Shaftesbury Formations.

The Peace River Formation extends from below river level, approximate elevation 318 m, to about elevation 340 m. Exposures above river level consist of three massive coarse sandstones, each between 5 m and 7 m thick, alternating with two thinly interbedded zones of sandstone, shale and lignite, each about 1.5 m thick. The massive zones form steep cliffs and are separated by gently sloping sections formed in the thinly interbedded zones. Discontinuous terraces at approximately 323 m and 330 m are the result of enhanced river erosion during downcutting by Heart and/or Peace rivers at the levels of the thinly interbedded zones.

The Shaftesbury Formation overlies the Peace River Formation with the contact about elevation 338 m. The basal zone is exposed approximately 300 m east of the slide area where it consists of dark grey, high plastic, fissured, slickensided clayshale.

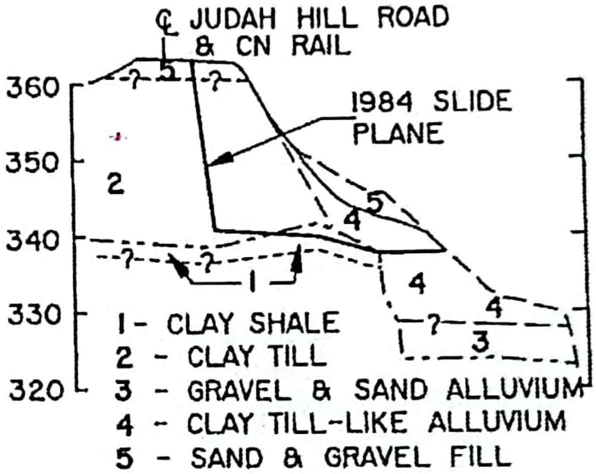

Fig.5 Section through slide

5 ENGINEERING ASSESSMENT OF INSTABILITY

In addition to the geological assessment, Thurber Consultants also undertook an engineering assessment of the instability occurring in 1984 and 1987 from which the following were determined.

1. No clear evidence of an active landslide scarp could be found in any of the pre 1984 air photographs examined.
2. No artesian pressures were likely on account of the underlying massive sandstone formation acting as a sub- drain.
3. Precipitation in the spring of 1984 was unusually high. Rainfall records over a 30 year period showed 150 mm of precipitation in late June and early July 1984 .
4. The failure surface inferred from slope indicator monitoring (Fig.5) was marginally stable.
5. Failure of the CNR berm in 1987

was secondary to the 1984 landslide. However, evidence from 1950 to 1952 airphoto coverage suggest a pre-existing failure scarp in the same area as the 1987 failure.

5.1 Conclusions

It was concluded that unusually heavy and sustained precipitation was the main cause of the 1984 landslide. This precipitation, it was felt, infiltrated the ground through pervious zones at higher elevation and created near saturated conditions within the slope. On this basis, it was recommended that improvements of stability to the area could be enhanced by limiting surface run-off infiltration to an absolute minimum. To accomplish this, it was recommended that (a) the run-off from the inner ditch along the Judah Hill road should be controlled between the road and a discharge point at the Heart river, (b) the lower 50 m to 100 m of the ditch be inspected to ensure that permeable layers are not recharged in areas immediately above the landslide, and (c) an impervious clay blanket should be used on the CNR berm and run-off controlled and collected over the 2:1 slope face.

6 INVESTIGATION AND STABILIZATION MEASURES BY AT&U 1987-1990

6.1 1987 Investigation

During the spring of 1987, renewed activity in the area prompted AT&U to implement some positive measures to stabilize the slide area since closure of the Judah Hill roadway was feared. An investigation of the slide area was undertaken through borings and test pitting. The following is a summary of the findings from this investigation.

1. The slide was occurring at the bedrock interface (Shaftesbury Formation) which was being charged with water.
2. Unless water pressures could be minimized the slide activity would continue at a slow but progressive rate.
3. There was a need to relocate the roadway from the established headscarp of the slide.

Recommendations following this investigation were to install horizontal drains within the slide area to intercept the zones where groundwater was observed and to realign the roadway from the headscarp placing one lane of the roadway in a total fill situation.

The construction of a bored pile retaining wall to assist in stabilizing the slide movements was considered as a last resort.

6.2 1987 Remediation

Installation of horizontal drains was undertaken in the fall of 1987. A total of 5 drains were installed. Further drains were curtailed due to the presence of a high pressure gas line just outside of the active slide zone. The horizontal drains yielded water at the rate of about 1 litre in 10 minutes initially, but later ended in small trickles.

6.3 1988 Remediation

The 1988 remediation consisted of controlling surface runoff infiltration.

In June 1988, the roadway ditch over a length of about 600 m uphill of the slide area was cleaned and lined with a geomembrane on top of which was placed a flexible erosion control mat. In addition, a 900 mm flexible corrugated plastic downdrain pipe was installed outside of the zone of visible slide activity. Nevertheless, despite the lining of the ditch, the roadway distress continued to occur.

In June-July 1988, it was decided that the retaining wall option would be considered utilizing piles terminating at or above the slide plane for the following reasons.

1. Piles in excess of 40 m would be required to retain the slide and hence would be too costly.
2. There was, however, no need to design long piles to retain the slide since the roadway was to be widened away from the well defined headscarp.
3. Shorter piles would be anchored by tiebacks since further loss of slope would result in failure of piles through rotation.

The pile wall retaining structure was installed during the period July

20 to August 12, 1988. A total of 36, 762 mm diameter, reinforced concrete cast-in-place piles were installed to an average depth of 20 m below existing ground elevation. This wall system was installed at existing roadway centreline (Fig.6).

Fig.6 Location of pile wall

Following pile installation, a preformed crack was constructed within the sideslope on the slide side of the roadway. This crack, was constructed by digging a ditch to about 6 m and infilling it with strawbales. The purpose of this crack was to provide a mechanism to prevent the headscarp of the slide from propogating too close to the free standing pile wall.

Immediately following construction of this preformed crack, the slide mass was noted to propagate to the ditch area, hence indicating that the installation was fulfilling its purpose. After construction of the wall, the additional lane of roadway was constructed.

6.4 1989 Remediation

In April-May, 1989 a deep well system was installed behind the piles to pump an aquifer that was identified during the installation of the pile wall system.

A total of 14 vertical holes were drilled to an average depth of 15 m to intercept seepage zones. These holes were belled at the bottoms to provide a continuous seepage zone and the holes backfilled with clean gravel aggregate. Provisions were made for installation of two submersible pumps within this subsurface system.

The two pumps were installed in August 1989 and set to run automatically following the build-up of a certain head of water.

In August 1989, approximately one year following the pile wall installation, the slope behind the pile wall slipped and exposed some of the piles (Fig.7). At the same time the slide opposite the CNR tracks was also activated and resulted in the headscarp progressing within the track location. This activity resulted in the CNR having to once again relocate their tracks further away from the slide.

Fig.7 Failure of slope

Immediate action was taken to tieback the wall system as had been envisaged at the time of design.

Tiebacks were installed from both sides of the roadway (Fig.8). A horizontal drilling machine was used to install the tieback system which consisted of high yield deformed structural steel bars installed at

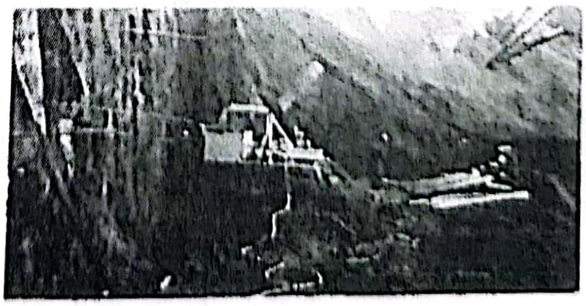

Fig.8 Tieback installation

an angle into the underlying bedrock and horizontal bars anchored to a deadman anchor on the opposite side of the slide. The anchor system was further tied into two large diameter concrete piles.
In addition to the tiebacks, a further 17 vertical piles were installed between the roadway and tracks to terminate at the right-of-way boundary of the two facilities to prevent cracks from the track side propagating behind the existing roadway piles. These piles were drilled to depths varying from 20 to 29 m and reinforced with rail sections donated by CNR.

The CNR also installed a pile wall system opposite their tracks (Fig.9). This system consisted of bored piles reinforced with H-piles and timber lagging, and anchored on the opposite side of the tracks using a deadman system and tierods placed just below track level. This was part of a two stage operation which was to consist of tieback installation the following year.

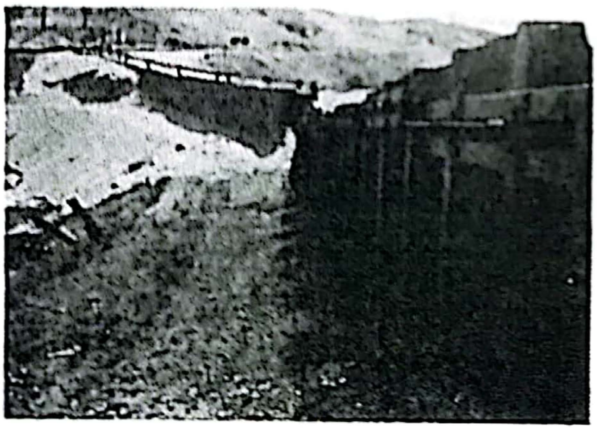

Fig.9 AT&U and CNR pile walls

6.5 1990 Remediation

Backfilling behind the pile wall system was done using lightweight material consisting of trees and sawdust as embankment material. The sawdust was utilized when the fill reached close to the top of the piles and was capped with clay to prevent any incidence of fires.

The downdrain system was rerouted from the active slide area, to discharge roadway surface runoff directly into the Heart river. The downdrain pipe was anchored to the slope by cables and an anchor pile system.

7 PERFORMANCE MONITORING

Five slope inclinometers were installed parallel to the pile wall. These inclinometers have been monitored regularly and do not show any significant movements of the wall. However, some movement was noticed since the CNR wall began undergoing distress in the spring of 1990 caused by rotation of the pile wall system.

From periodic inspections AT&U's wall system is intact and the roadway is performing well. Some cracking is occurring in the rebuilt slope area. This is to be expected since this area is very unstable. The pump system is still working and since installation has produced about 1 megalitre of water.

8 CONCLUSIONS

The investigation of the Judah Hill landslide over the last 7 years has clearly demonstrated the significance of regional and site geology on the stability of natural slopes in a complex geological and geotechnical environment. Although the true mechanism causing instability at the Judah Hill site may not have been clearly determined, it is evident that the changes in the physical environment caused by roadway and rail constructions also influenced the limiting stability of this area. The success so far of the stabilization remedial measures has resulted from an appreciation of site behaviour based on a combination of the observational and theoretical approaches to geotechnical problem resolution.

Because of the sensitive nature of this site and its importance to commuter traffic frequent monitoring and visual inspections have been scheduled to assess its performance with time.

9 REFERENCES

Thurber Consultants Ltd. 1988. Geological assessment of the Judah Hill landslide.