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Remedial measures to facilitate the construction of stable bridge approach fills: A case study

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ABSTRACT: The proposed construction of a north-south log haul road from Secondary Road (SR 686) to North of the Little Cadotte River was to form part of the roadway infrastructure requirements for the proposed Daishowa Pulp Mill in Peace River, Alberta. This paper deals primarily with stability issues associated with the Cadotte River crossing. After the construction of 5 m of the proposed 20 m high bridge approach fill in No-vember 1988 deep seated movements were noted at a depth of 22 m below the original ground from monitoring of slope indicators on the north approach. Remedial measures consisting of wick drains, stone columns, flatter approach fill head and sideslopes, and interceptor drains were then implemented. However, as a result of the movements that were taking place at depth it was decided to abandon the proposed 3-span bridge and construct a one-span Bailey bridge. This paper addresses the geotechnical investigation, evaluation and assessment of the bridge approach fills and the remedial measures implemented to minimize the movements.

1 INTRODUCTION

Roadway infrastructure to facilitate log haul to the Peace River Pulp Mill, a Division of the Daishowa-Marubeni International Ltd (DMI), situated sixteen (16) kilometres north of the Town of Peace River, Alberta, required the construction of a north-south log haul roadway from SH 686 to the Little Cadotte River. This route, which covers a distance of 50 km, required a bridge at the Cadotte River crossing shown in Figure 1.

Due to the projected fast paced schedule of the pulp mill project a geotechnical assessment of the ground conditions at the proposed crossing was initiated in July–August 1987 to meet the requirements for the design and construction of the bridge approach fills. The bridge structure was conceived as a three-span bridge and scheduled for construction in November 1988.

The schedule appeared realistic if everything went well, but could not be accomplished as a result of unforeseen ground conditions at the site of the bridge crossing. This paper outlines and discusses the problems encountered during the construction of the bridge approach fills which resulted in the abandonment of the proposed bridge. To overcome these setbacks which resulted from slope instability problems, a Bailey bridge had to be constructed at this site on a shifted alignment.

2 SITE DESCIPTION AND CHARACTERISTICS

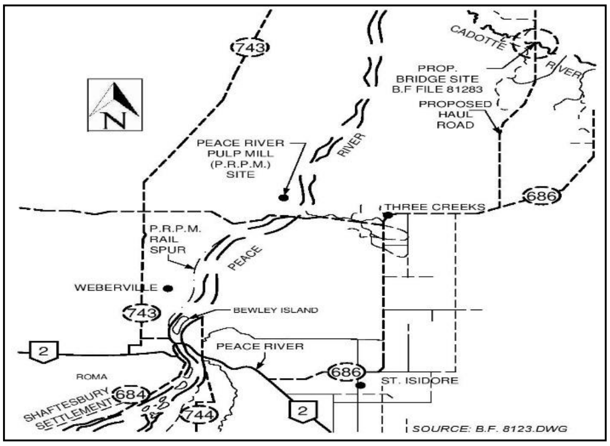

Figure 1. Location of Roadways, Bridge and Pulp Mil

At the proposed bridge location, the Cadotte River is a narrow, shallow east–west oriented stream contained in a deeply incised river valley. A width of 1 km between valley walls and an average elevation difference of about 65 m from streambed to the top of the valley walls are indicative of the extent of river activity and/or glacial activity on the topographic development of the Cadotte River in its geologic past. The topographic features of the north and south valley slopes appeared very similar except that the north valley slope was somewhat hummocky with evidence of slide scarps at the top of the valley immediately adjacent to the proposed alignment.

Limited aerial photography assessment during the alignment study did not indicate the presence of deep seated slide activity nor slide activity of recent origin. The aerial photos did, however, reveal some puzzling features which gave the impression that some ground displacement had occurred on the north side of the river or that a river channel once dissected the present flood plain. Similar river channel features were also present on the south side of the crossing.

Site reconnaissance of the alignment undertaken during the summer of 1988 showed waterlogged conditions from the river to almost 150 m on both north and south sides of the crossing. The water logged areas were observed to have resulted from beaver activity. From brief observations these stagnant sources were considered drainable towards the river. During this reconnaissance it was felt that the ground between the river and a beaver pond on the north side had undergone some previous displacement. This was deduced from the relief of the ground which showed an abrupt ridge 2 to 3 m in height merging into the beaver pond. No such abrupt relief was observed on the south side of the crossing. The puzzling nature of this ground relief along with adjacent ground features raised immediate geotechnical concerns of the site conditions.

3 SITE INVESTIGATION

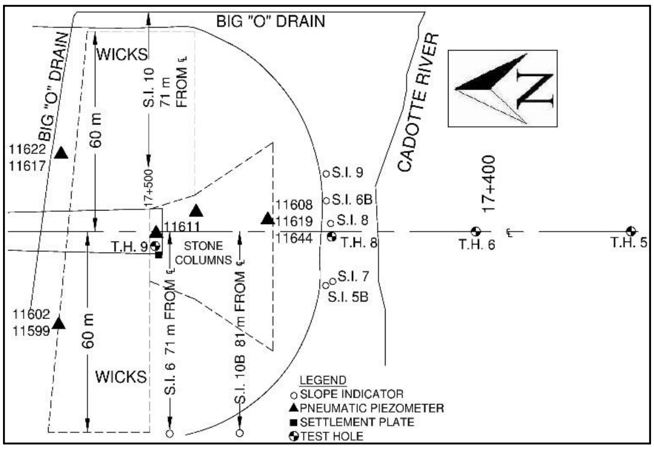

Figure 2. Testhole, Instrumentation & Trench Drain Loca-

Concerns of possible old slide activity coupled with the projected schedule for bridge construction in November 1988 and projected heights of approach fills in the order of 20 to 30 m above streambed prompted the undertaking of an early site investigation.

The site investigation was undertaken in July 1988. A total of twelve (12) testholes were drilled between Sta 16+650 and Sta 17+890 covering the river crossing and north and south valley walls.

For the bridge approach fills and foundation assessment four (4) testholes were drilled. These holes are referenced as TH 5, 6, 8, and 9 and located as shown in Figure2. Testhole drilling was undertaken using an auger rig with capabilities of coring using a wire line coring approach. Sampling by coring and standard penetration testing (SPT) was undertaken in the four (4) testholes.

4 SUBSURFACE CONDITIONS

In general, the soil stratigraphy in the vicinity of the approach fills consisted of 2 to 3 m of soft surficial alluvial deposits of sand, silt and clays overlying over consolidated clay shale varying in consistency from stiff to hard and extending to a depth of about 25 m where the drilling was terminated. Groundwater was observed within a metre of the ground surface at the testhole locations. In scrutinizing the testhole logs, it was observed that the SPT values were generally smaller at similar depths on the north side than on the south side. However, the SPT values showed the underlying shale to be reasonably competent with depth.

Laboratory visual description of the samples of clay shale revealed the material to be essentially silty clay containing silt varves, indicating a marine depositional environment. Laboratory testing for moisture content, index properties, and strength determinations indicated slickensided shear with strain softening behaviour at small strains. Variable unconfined compressive shear strengths were obtained from core samples within the clay shale. Moisture contents were generally below optimum moisture throughout with material classification being predominantly MH/CH. In one sample at a depth of 25 m in TH 8 a 50 to 100 mm thick piece of bentonite was reported in the laboratory visual description. This sample had a liquid limit of 135%, plastic limit of 31 % and a field moisture content of 22%.

5 ASSESSMENT OF SUBSURFACE CONDITIONS

Following the field testhole drilling and in consideration of the proposed fill heights along with soft and wet conditions in the top 2 to 3 m of ground it was decided to undertake stage construction of the approach fills. The construction of the fills was to be preceded by the excavation of soft surficial material.

A testpit investigation was undertaken to decide on the extent of removal of soft soil with depth throughout the site. This investigation was carried out in September 1988 and was concentrated on the south approach although it was originally proposed to undertake this investigation on both sides of the river. At this time the north side had only been cleared of treed vegetation and a drainage channel opened to the river to facilitate drainage of the beaver ponds/sloughs and no testpitting undertaken.

In light of the presence of water and the hard bottom within the excavated areas in the vicinity of the stub approach fill on the south side, it was recommended that the excavation should be backfilled with granular material to original ground before placement of the

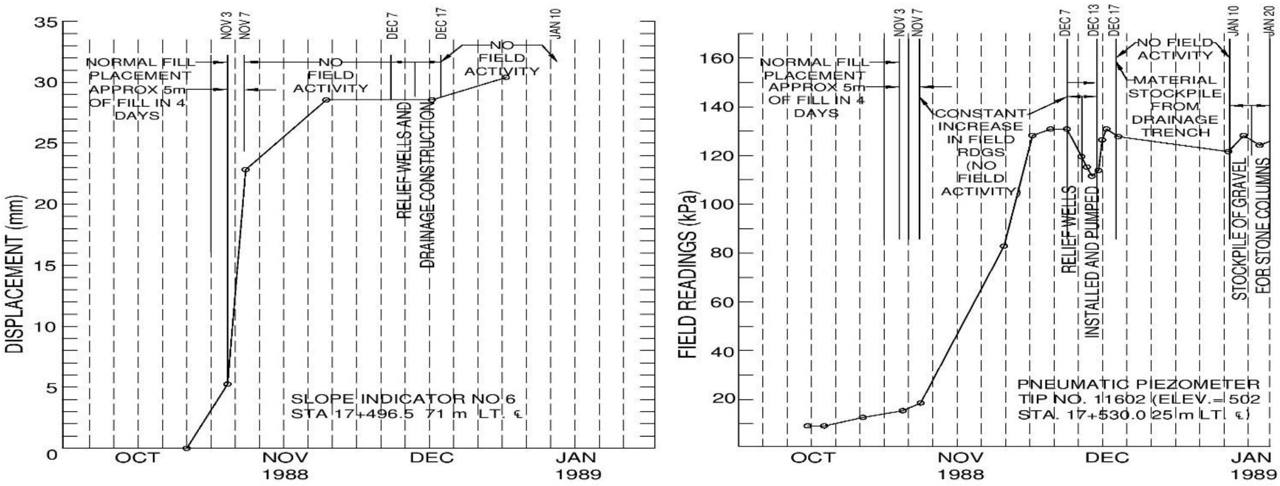

Figure 3. Porewater pressures and displacements versus time

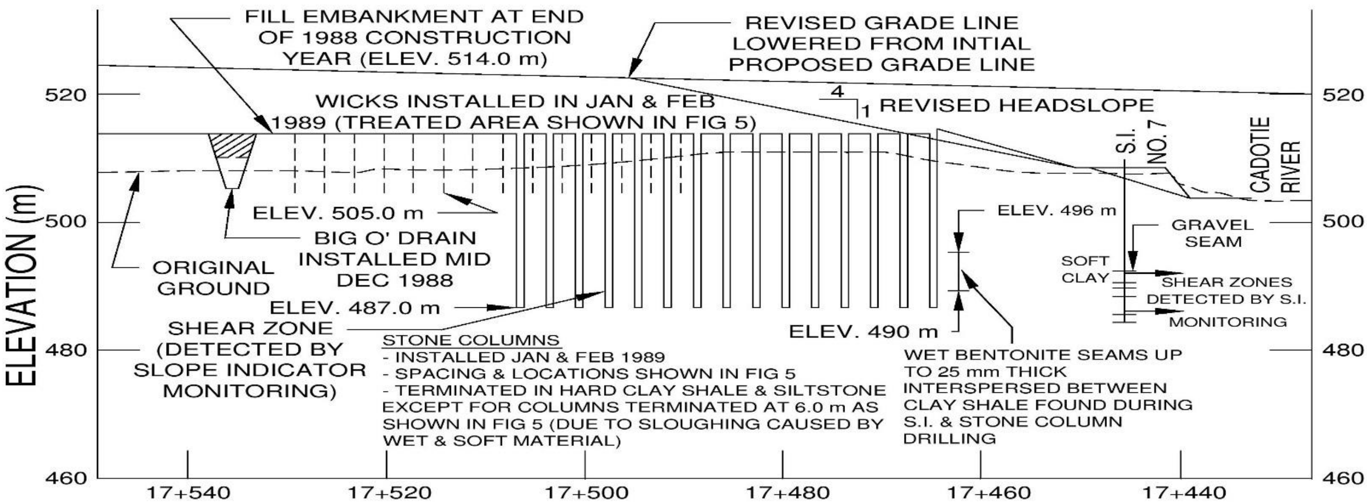

Figure 4. Cross section at North Approach Fill showing weak zones

approach fill. However, in consideration of the quantity of material required for both sides of the river, the non-availability of gravel within an economic haul distance and general restrictive site conditions at the time of truck hauling this proposal was abandoned in favour of backfilling with clay borrow and removing free water from the site through its displacement by the advancing fill.

The use of geotextiles beyond the stub fill area in a very silty and unstable section of the south side was also recommended. This was considered essential to facilitate earthworks hauling by scrapers. No geotextile was recommended for use on the north side although initial discussions considered its use in the slough areas if the bottoms of the sloughs were decidedly soft or weak. Testpitting was done subsequently on the north side and a similar sub-excavation was undertaken on this side as was done on the south side. In subsequent discussions of the project site characteristics with the Bridge Planning Engineers a 2.5:1 headslope was recommended for both the north and south approach fills instead of the traditional 2:1 headslopes used to minimize bridge length and hence overall structure costs.

6 INTRUMENTATION

Prior to the placement of the bridge approach fills slope indicators and pneumatic piezometers were installed to aid in the monitoring of fill construction. Piezometers were installed at locations shown in Figure 2 to a maximum depth of 9 m below existing ground. No deeper installation was undertaken at that time since it was not perceived that movements, if they were to occur, would be deep seated. Slope indicators were, however, installed to a depth of 26 m although the first installation (SI 2) on the south side was terminated around 13 m since very hard ground was encountered. The decision to install the inclinometers to 26 m was based on the observation of the variability of the clay shale with depth from the testhole logs.

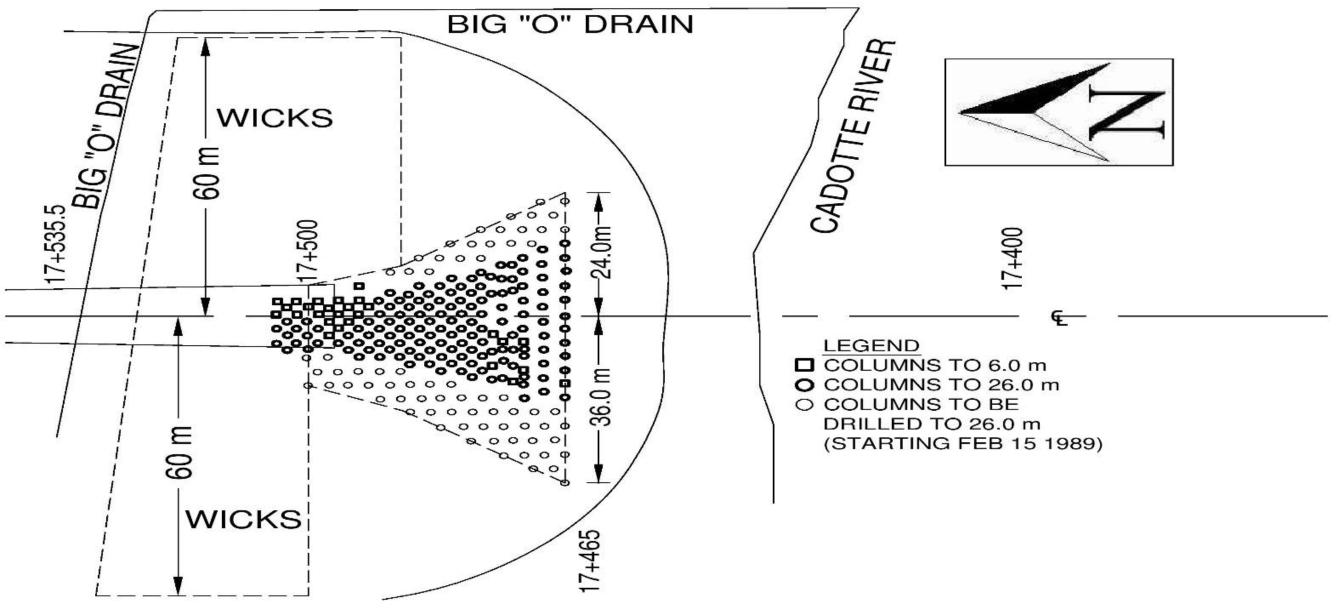

Figure 5. Layout of Stone columns and Wick Drains – North Approach zones

7 FILL CONSTRUCTION

Approach fill construction began around September 15, 1988. Slope indicators and pneumatic piezometers were monitored regularly as the fill was placed. The weather condition was generally good until October 20 when a temporary halt in construction was made due to snowfall. Heavy snowfall on November 7 signaled the onset of winter and resulted in a permanent halt to construction activity.

At this stage fill construction attained a height of about 5 to 6m on both sides of the river crossing. A third set of slope indicator readings was taken on November 7. The results of this monitoring showed movement to be occurring in all slope indicators on the north side to a depth of about 22 m. Except for SI 2 on the south side which was installed to a depth of 13 m and which showed movement around 12 m, the remaining SIs on the south side did not show any well defined zone of movement.

The observed depth of movement at 22 m led to many questions concerning the validity of the field measurements in relation to whether instrument error, poor installation techniques, etc., were responsible for the results attained. After much evaluation it was decided that movement was indeed occurring at that depth and that there was some correlation with the weak zones noted in TH 8 where blow counts had been observed to be variable. A decision was made to re-read the SIs with a different inclinometer probe to ensure that the depth of movement was indeed factual. The slope indicators were re-read on November 24, 1988 using a different probe. During this re-reading it was reported that the probe became stuck at the depth of 25 m recorded previously in SI 8, hence confirming the previous monitoring results. As well, the deformations recorded to the depth at which the probe became stuck were comparable to those obtained previously.

Piezometer readings showed substantial increases in porewater pressure than previously recorded during fill construction. On the basis of these field observations it was concluded that deep seated movements were indeed occurring at the site.

8 INVESTIGATION OF GROUND MOVEMENT

The determination of the cause of movement led to further drilling and site inspections during the period November 29 to December 18, 1988. Deep drilling using auger and rotary drilling were undertaken to ascertain the reasons for the high pore pressures as well as to ascertain whether weak material existed at the zone of movement. Auger drilling revealed water to be associated with the old beaver ponds/slough areas which was attributed to possible seepage from uphill of the flood plain or from water trapped during embankment construction. Testpits dug to a depth of 8 m uphill of Sta 17+585 proved to be dry and suggested that the slough areas resulted from beaver dam activity. However, the high piezometric pressures at a depth of 7 m below original ground suggested that downward seepage was possible and that piezometric pressures were caused by the weight of fill. A check on the piezometric levels confirmed that these levels were as high as and becoming higher than the fill elevation. The steadily increasing pore pressures observed following fill construction were attributed to the poor drainability of the site.

Further field testing to reduce pore water pressures included installation of 300 mm diameter wells to about 9 m below fill elevation. These wells were located in the vicinity of Sta 17+585 to intercept the old slough areas and were pumped for 2 days. A total of

2200 gallons of water were pumped. During this period porewater pressures were monitored and from Figure 3 it can be seen that a drop of 27 kPa in piezometric pressure occurred. This observation along with the observation of large underground flow during testpitting opposite SI 6 outside of the fill slope led to the idea of installation of trench drains to aid in removal of trapped or seepage water.

Trench (French) drains with Big-O drainage pipe were installed as shown in Figure 2. During excavation of these trench drains seepage zones were observed along the walls of the trenches from gravel and sand layers. The majority of flow, however, was associated with a lower gravel layer which appeared to be co-incident with the base of the beaver ponds.

Almost opposite SI 6 a classic structural folding due to a geologic process was observed showing bentonite rich shale thrust into the gravel stratum resulting in an upward displacement of the gravel. This bentonite rich shale was very friable and crumpled instantly when handled without hardly any pressure.

This observation added credence to the belief that this site had undergone prior movements. The folding led to the belief that glacial thrusting, for lack of a better explanation had occurred. The presence of bentonite near the surface was also of valuable significance since it confirmed the laboratory visual observations. As well, the movements recorded by the slope indicators were becoming more reasonable to understand. In short there was now more evidence from field observations to conclude that this site presented some very complex ground conditions. A number of probe testholes were subsequently done to determine whether bentonitic zones could be defined and also to define the wet areas and seepage zones. As shown in Figure 4 soft wet zones about 1 m in thickness were observed to be sandwiched between very intact and competent shale material. At first it was felt that poor drilling methods might have caused this to occur but careful drilling and scrutiny of cores showed that these zones resulted from softening due to shearing action. The presheared zone was very soft with pocket penetrometer readings varying from 0.25 to 0.5 Tons/ft^2. Also shown in Figure 4 is the depth range at which bentonite was observed.

On the basis of the field observations and stability assessments several aspects were debated for the continuing construction of the fills. A culvert option was considered in lieu of a bridge but was scrapped a result of instability problems that would result from excavation along with associated costs for a proposed large reinforced concrete structure. In addition, environmental concerns would be a major problem. The idea of ground modification by deep soil mixing was pursued but this equipment could only be obtained from a Contractor in the USA, with a price tag of about $US1.5 million. This was considered too expensive and not pursued. Site relocation was also considered but not pursued since this idea had limitations in terms of overall roadway network logistics and the fact that there may not be a more desirable site. Instead it was finally decided that a combination of wick drains, stone columns along with flatter fill and head slopes would be the preferred approach to use to stabilize the site.

9 REMEDIAL MEASURES AND FIELD IMPLEMENTATION

The use of wick drains and stone columns in combination would provide a reduction of the pore water pressures by the wicks while the stone columns to be installed by augering and replacement of the excavated material with compacted gravel aggregate would provide for removal of the weak material and an increase in the shearing resistance of the ground. From stability assessments the combination of reduced porewater pressures, increased shearing resistance and flatter head and side slopes would result in the overall factor of safety of 1.2 to 1.3.

Field implementation of remedial measures began on January 10, 1989 with wicks and stone columns. The stone columns were installed using two Texoma Drill rigs equipped with a 1m diameter auger. Compaction of the stone columns was undertaken with the aid of a 750 mm diameter plate tamper attached to the end of the Kelly bar of the drill rig. The compactive effort was applied to small lifts of gravel backfill via the crowd force applied by the drill rig on the Kelly bar. Figure 5 shows the layout of stone columns and wick drain installations.

On January 16, 1989 stone column installations revealed the zones of bentonite to be contained between El 490 and El 496. A test drilling was done near to SI 8 to ascertain the material characteristics at the zone of movement. It was rewarding to find bentonite in a very wet and greasy condition in addition to free water between clay shale that portrayed a fissured appearance. The reason that water was present at great depths either via downward penetration through fissures or underground seepage was therefore substantiated and that instability resulted from excess porewater pressure at very low shearing resistance of the bentonitic seams. The fissured nature of the clay shale was attributed to valley rebound following glaciation.

10 SUMMARY AND CONCULUSIONS

The implementation of the remedial measures did not fully arrest the ground movements. As a result, the approach fill slope on the north headslope was modified from 2.5:1 to 4:1 with side slopes of 4:1. On the south side the initial design of the approach fill slopes of 2.5:1 headslope and 3:1 sideslopes were maintained. In addition, as a result of the movements it was decided to utilize a Bailey Bridge at the crossing location. This bridge is shown in Fig 6. Monitoring of this bridge by survey hubs between June 2 and October 11, 1990 showed initial movements of 7 mm per day to July 4 and thereafter 1.2 mm per day. A recent site observation in August 2013 (Figure 7) shows that the bridge is still serviceable after 23 years.

Figure 6. Aerial Photo Showing Bridge

Figure 7. Site photo taken in August 2013

11 ACKNOWLEDGEMENTS

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