HIGHWAY 63:11 TWINNING NORTH OF ATHABASCA RIVER BRIDGE IN FORT MCMURRAY, ALBERTA: PINCH POINT WEST PILE WALL

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ABSTRACT
Highway 63:11 north of the Athabasca River, in the urban service area of Fort McMurray, Alberta is situated on a narrow river terrace. This creates a “Pinch Point” for transportation and utility corridors. The highway is constrained between the toe of a meta-stable colluvial valley slope to the west and the crest of a steep eroded river bank to the east. Since the highway twinning project through this area required a substantial increase in the roadway width, two tied-back pile walls were constructed parallel to the highway corridor to accommodate the widening plan. A 320 m long tangent pile wall was built along the east side to accommodate the widening fill and a 420 m long secant pile wall on the west side to retain up to a 9 m high cut at the toe of the valley slope. The west pile wall, which is the subject of this paper, is the longest of its kind in Alberta. The wall carries a factored lateral load of 200 MN, and consists of 270 infill piles, 271 rock socketed structural piles and 446 high capacity multi-strand anchors bonded into Devonian limestone. The paper provides a summary of the design and construction aspects of the wall, and includes a brief discussion of instrumentation monitoring results.

RÉSUMÉ
Dans l’agglomération de de Fort McMurray, Alberta, l’autoroute 63:11 est située sur une terrasse alluvionnaire étroite sur la rive nord de fleuve Athabasca, engendrant ainsi un rétrécissement local du corridor alloué aux voies de transport et aux services connexes. L’autoroute se trouve ainsi cintré entre le pied d’un talus métastable colluvial à l’ouest et par la crête abrupte de la berge du fleuve l’est. Deux murs de soutènement avec ancrages ont été construits à cet endroit afin d’accommoder un projet d’élargissement de l’autoroute. Afin d’accommoder le remblai additionnel dû à l’élargissement, un mur de 320 m de longueur composé de pieux tangents a été construit sur le côté est. Un second mur de 420 m de longueur composé de pieux sécants a été construit sur le côté ouest afin de soutenir une fouille de 9 m de haut au pied du talus de la vallée. Le sujet de cet article est le mur ouest, qui est le plus long de son genre en Alberta. Le mur soutient une charge latérale pondérée de 200 MN et est composé de 271 pieux-caissons à rainures structuraux principales, de 270 pieux intermédiaires et de 446 ancrages à câbles de haute capacité ancrés dans le socle castine Dévonien. Cet article présente un sommaire de la conception et des aspects de la construction du mur, ainsi qu’une brève discussion des données recueillis par l’instrumentation.

1 INTRODUCTION

Highway 63 is a 443 kilometre long provincial highway with an average annual daily traffic of more than 30,000 vehicles. The highway is the primary corridor to oil sand developments in and around the Urban Service Area of Fort McMurray in northern Alberta. In order to accommodate the industrial activity and population growth in the area, Alberta Transportation (AT) made a commitment to upgrade the corridor to improve traffic safety, reduce congestion, and accommodate over-dimensioned loads.

To the north of the Athabasca River Bridge within Fort McMurray, the highway was upgraded from 4 to 6 mainline lanes and 2 adjacent collector-distributor lanes were added. The highway at this location was constructed on a narrow river terrace situated between the toe of a meta-stable colluvial valley slope to the west and a steep eroded bank of the Athabasca River to the east. This existing geographic constraint created a physical “Pinch Point” that governed the highway upgrading design. Figure 1 presents a satellite image of the project site.

In order to accommodate the substantial increase in the out-to-out width of the roadway corridor from 33 m to 68 m a 420 m long secant pile wall, known as the West or Hillside Pile Wall (WPW), was constructed on the west side of the highway, and a 320 m long wall, known as the Riverside or East Pile Wall (EPW) was constructed on the east side of the highway. An artistic depiction of the east and west walls is shown on Figure 2. The construction of both walls was completed between June 2010 and December 2012 and the construction cost was in the range of CAN$21,000,000. Although both walls are equally important to the success of the overall widening scheme, the WPW is the focus of this paper. The design and construction details of the EPW were summarized by

Abdelaziz et al. (2016).

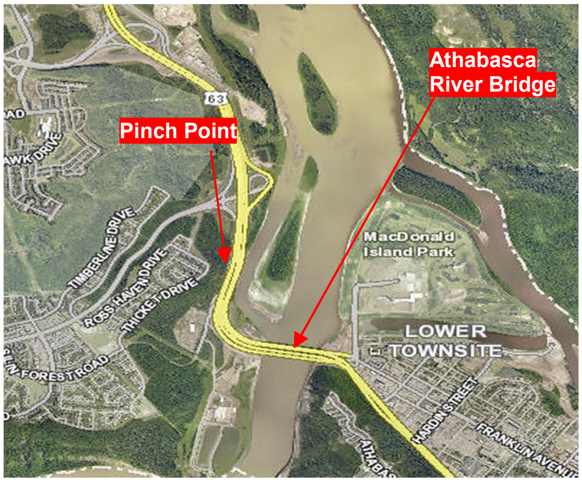


Figure 1. Satellite image showing site location

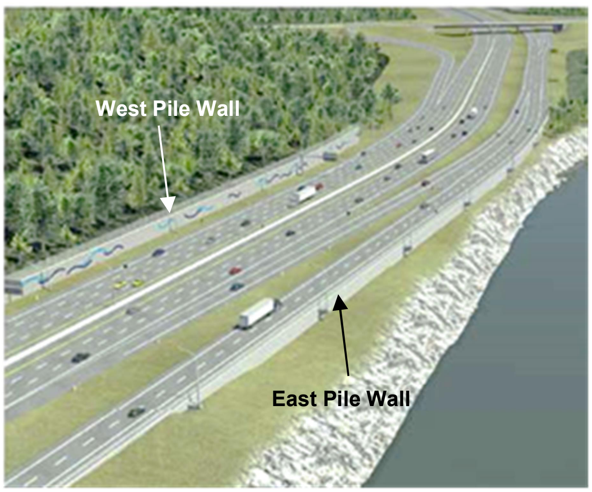


Figure 2. Artistic depiction of upgraded Hwy 63 and East and West Retaining Walls, looking north

2 DESCRIPTION

2.1 General

A geotechnical investigation, consisting of auger and core holes and installation of slope inclinometers and piezometers, was carried out by Thurber Engineering Ltd. between 2006 and 2008. The study characterized the sub-surface conditions and provided design recommendations for the walls. A shaded relief plan showing the old and upgraded highway alignments and the wall locations is presented in Figure 3. A simplified stratigraphic cross section at the pinch point location is also presented in Figure 4.

2.2 Geomorphological History

The broad basin north of Fort McMurray existed before the advance of the continental glacier. It was drained, probably to the north, by small streams and an unknown amount of material was removed by glacial scour. Regional recession of the continental glacier in a general north-eastward direction occurred approximately 9000 years ago. Drainage channels in the direction of regional topographical dip (to the northwest) were blocked by ice, and vast pre-glacial lakes ponded against the glacier front. The Clearwater River valley and much of the present course of the Athabasca River north of Fort McMurray once served as the channel for south eastward glacial meltwater and pre-glacial lake drainage.

After breaching of ice barriers and further glacial recession, a new regional drainage system was imposed upon the area. The Athabasca River displaced northward by disruption of drainage, occupied its present-day position and rapidly excavated its steep, deep valley west of Fort McMurray. The progressive down-cutting action by the river resulted in the development of landslides in the present valley slopes and the formation of the narrow river terrace and the eroded bank to the east of Highway 63 at the study area location. Although the valley slopes are currently moving at a very slow rate (i.e. creeping), accelerated movements have occurred due to disturbance of these sensitive valley slopes, most notably in response to previous highway twinning projects and urban developments.

2.3 Surficial and Bedrock Units

Three major surficial geology units are present within and in the general vicinity of the study area. These units include alluvial, colluvial, and glacial deposits (Fenton et al. 2013). The alluvial deposits from the river terrace are located below the highway and consists of sand, gravel and silt layers. Colluvial deposits are located on the valley slopes of the Athabasca River and originates from erosion and landslide events through surficial deposits and bedrock formations in valley settings. Most of the colluvium displays slow creep movement towards the base of the valleys. The glacial deposits consist of a mixture of clay, silt and sand, as well as minor pebbles, cobbles and boulders, and are generally present in the uplands plateau area.

Three major bedrock units are present within the study area (Prior et al. 2013). These units include the Clearwater, McMurray, and Devonian Formations. The Clearwater Formation underlies the plateau and the colluvial deposits in the upper portions of the valley slopes. It consists mainly of extremely weak to weak high plastic clay shale with occasional interbeds of weak to strong siltstone and sandstone. The stress relief due to the down-cutting of the river and valley rebound due glacial recession has weakened the Clearwater clay shale and made it highly susceptible to slumping and gullying. It is therefore considered to be the main source of the colluvium deposits and the landslides along Highway 63 valley slopes. The Clearwater Formation is underlain by the McMurray Formation. This formation outcrops extensively along the Athabasca River valley slopes. It typically consists of interbedded oil impregnated sand, siltstone and clay shale. The McMurray Formation contains three members: The upper McMurray - fine grained quartz sands, oil cemented; the Middle McMurray - medium grained quartz sand, oil cemented, lenses of siltstone, shale and coal, and; the Lower McMurray - conglomerate, detrital clays and shales, siltstone and coarse grained sands.

The Clearwater and McMurray Formations are of the Cretaceous Period, 70 to 130 million years old. Underlying these formations is limestone bedrock of the Devonian Period ,350 to 400 million years old. The Devonian bedrock consists primarily of limestone of the Waterways Formation. The limestone outcrops for several kilometers along the Athabasca River banks and is known to contain sporadic solution cavities and karst. The limestone ranges from weak clayey "argillaceous" limestone to moderately strong to very strong biomicritic and nodular limestone. Very weak, high plastic calcareous shale layers, are frequently present interbedded within the limestone. The upper surface of the Devonian Formation limestone is an erosional unconformity upon which the oilsand and the Clearwater shale were deposited. At depth the Devonian bedrock is comprised of a succession of carbonate rocks and evaporates which has been reported to be Precambrian granite of at least 600 million years old. The granite is found some 240 m below the present upland surface (Carrigy, 1959).

2.4 Surface and Subsurface Conditions

The highway at the study area was a four-lane divided roadway that was originally constructed on a 33 m wide terrace of the Athabasca River valley. The highway grade is located about 20 m above the river level and the river bank slopes to the east of the highway are steeply inclined at 45 to 60 degrees to horizontal. The limestone bedrock is typically visible along the steep river banks. The river bank slopes to the east of the highway have sparse vegetation cover with partially under-scoured hanging trees. An existing pipeline is present near the top of bank to the east of the highway location.

The highway alignment is bounded on the west by meta-stable valley slopes. The valley slopes are about 55 m high and inclined at 15 to 26 degrees to horizontal. Oil sand is visible on the steeply inclined valley slopes. The valley slopes are typically covered with mature tilting trees and deadfall. An ancient landslide block is visible in the valley slopes within the study area, as shown in Figure 3.

The subsurface conditions along the west pile wall alignment generally consist of 3 to 11 m of sand and clay colluvium over 2 to 6.5 m of dense to very dense oil sand with clay shale interbeds overlying interbedded weak to extremely weak argillaceous limestone and strong to medium strong nodular and biomicritic limestone bedrock. The limestone bedrock top surface appears to dip towards south and varies in elevation from 247 m to 251 m. The colluvium cover thins out towards north and the oilsand outcrops are more visible within the north side of the wall. The upper 1 m of the limestone bedrock is closely jointed and weathered. The uniaxial compression strength of the oilsand formation ranges from 2 to 3 MPa (i.e. weak to extremely weak bedrock). For the Devonian Formation, the uniaxial compression strength ranges from less than 1 MPa (i.e. extremely weak argillaceous limestone bedrock) to more than 50 MPa (i.e. strong nodular and biomicritic limestone). Groundwater table is about 5 to 10 m below ground surface at the wall location.

3 WEST PILE WALL (WPW)

3.1 Pile Wall Design Details

A plan and design profile of the West Pile Wall (WPW) alignment is presented in Figure 5. The WPW is about 420 m long , which is the longest of its kind in Alberta, and retains up to 9 m of vertical cut at the toe of a meta stable valley slope. The purpose of the wall is to minimize the impact of the widening plan on the stability of the slope and existing residential subdivision located directly above the Pinch Point in the uplands area. To facilitate optimization of the design, the wall was divided into eight design sections. The differentiation between the design segments (A1 to A2, B to C, D1 to D3, and E) was based on the loading condition, wall height, and available passive resistance on the downslope side of the wall location. In design segment C, the wall was designed to retain the highest cut within the toe of an ancient landslide and hence it was prone to the highest lateral load among the wall segments.

Slope stability and structural analyses were completed for each of the design segments to come up with the final configuration presented in Figure 5. In general, the variability in the number of anchors per pile, anchor lengths (variable bond and free zones) and loads, and the pile tip elevation complicated the wall design. The wall was designed to meet the SLS and ULS loading conditions and to sustain a range of factored lateral loads ranging from 400 kN/m to 1300 kN/m. Additional analyses were completed to check the stability of the wall during various stages of construction.

Typical cross sections of the wall are presented in Figure 6. The WPW consists of a secant pile wall with multiple levels of grouted ground anchors in the majority of the design segments. The wall consists of 273 cast-in-place concrete structural piles (PS) socketed 4 to 6 m into the strong limestone and 272 infill piles (PF) . The structural piles are 1200 m in diameter with lengths varying between 12 and 21 m. The infill piles are 760 mm in diameter with no reinforcing steel and have lengths varying between 2 and 10 m. A 1.2 m deep pile cap was constructed that connects the pile tops to increase the rigidity of the retaining system. The system also includes a removable pre-cast concrete panels between the roadway and the front face of the secant wall. A sub-drainage system is included between the secant wall front face and the concrete panel to collect seepage from 50 mm diameter weeping tiles drilled into the infill piles to relieve hydrostatic pressure against the back of the wall.

The original design included one to three rows of 36 to 46 mm diameter high capacity Double Corrosion Protection (DCP) permanent grouted anchors in Segments A2, B to C, and D1 to D3 to limit the wall deflection. Segments A1 and E had no ground anchors. The anchors were to be inclined at 30^o and had variable bond zone length into the oil sand and limestone bedrock. Selected vertical bars of the structural pile reinforcement cage were bundled to provide constructability tolerances and to avoid damage to vertical rebars while coring through the structural piles.

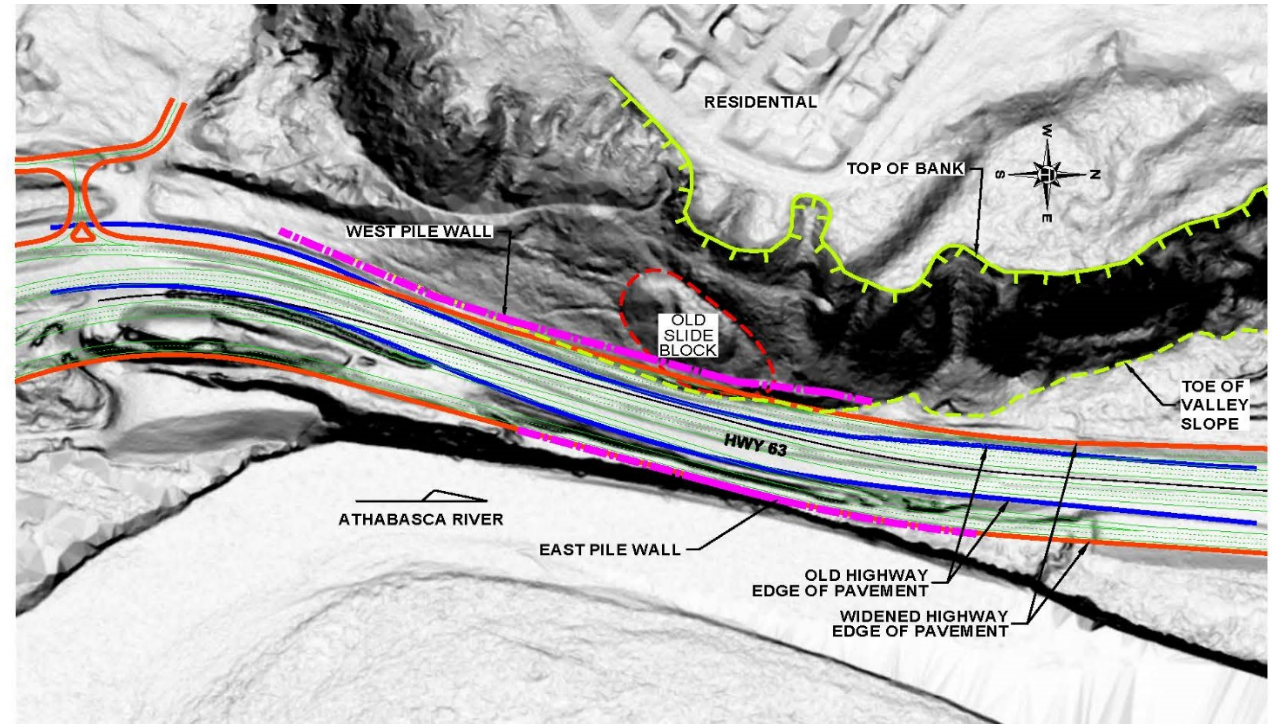


Figure 3. Shaded relief plan showing pile walls at the Pinch Point location

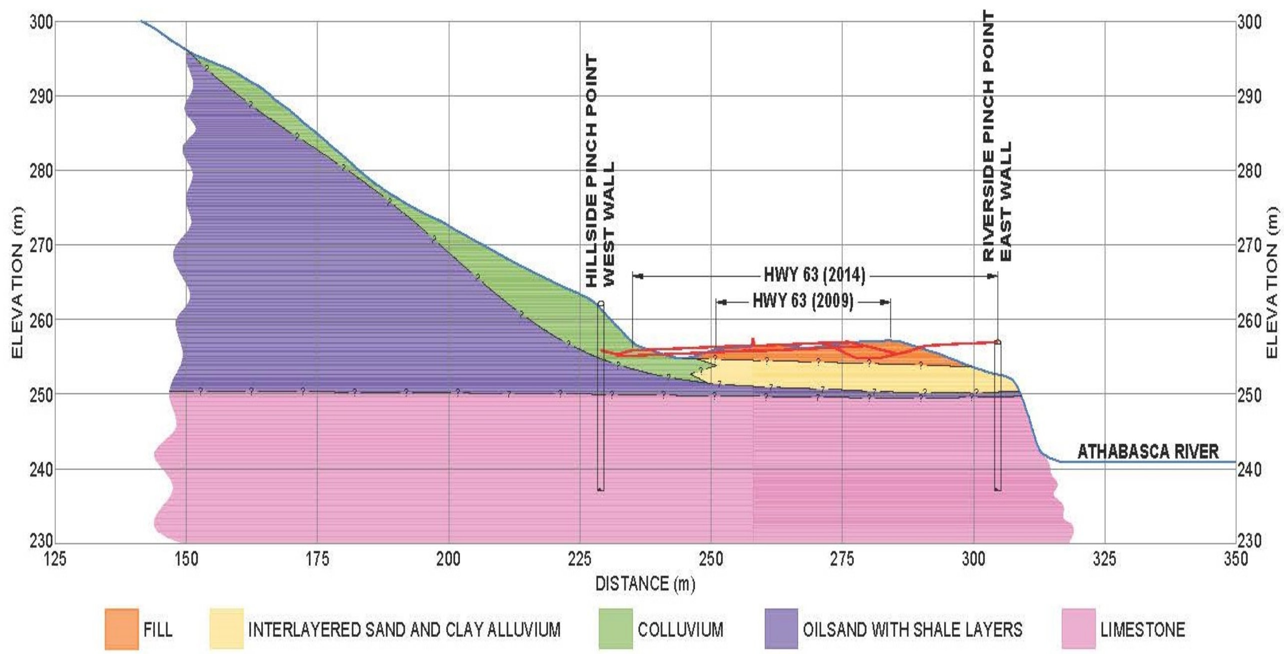


Figure 4. Simplified stratigraphic cross section at the Pinch Point location

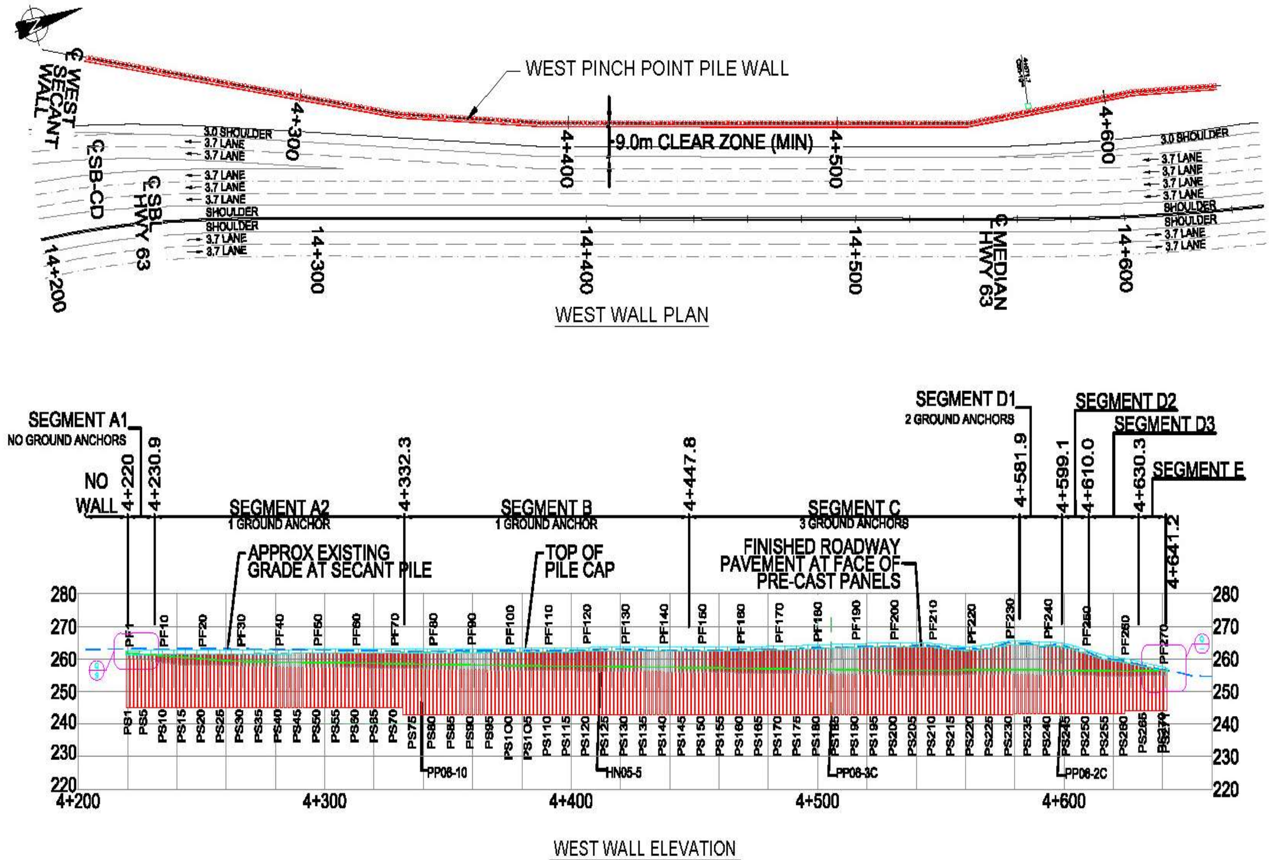


Figure 5. Plan and design profile of the WPW

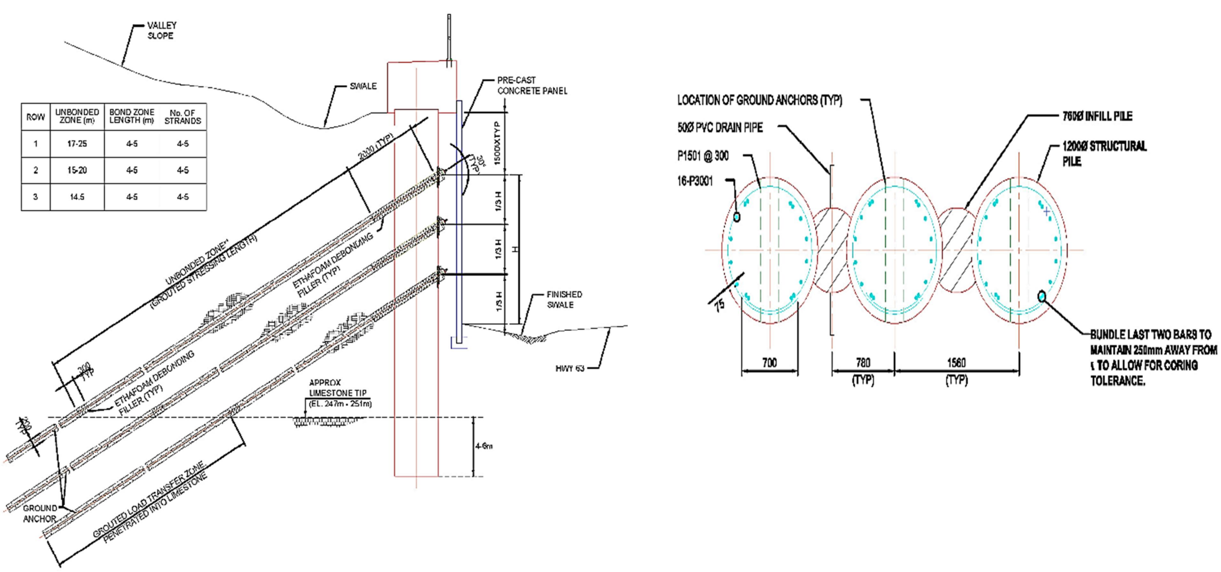


Figure 6. Typical Cross sections of the WPW

3.2 Construction

3.2.1. General

The construction of the wall was completed between October 2010 and December 2012 with a total construction cost of about $15 Million. Figure 7 shows the primary construction stages of the wall. A top-down construction technique was used to construct the full height of the wall. At the onset of the project, a side hill access bench configuration was constructed in short sections to install the piles prior to undertaking any excavations. The cut slopes of the access bench were retained by lock blocks to minimize the footprint of excavated mass from the hill side.

The excavation downslope of the wall was carried out in controlled stages after piling was completed to maintain the stability of the slopes and the wall. Within each level of the excavation stages, the anchors were installed and locked off before proceeding with excavations to the lower level. To further ensure stability of the valley slope each of the vertical excavation stages, excavations perpendicular to the slope direction in front of the wall were completed in 25 m wide slots separated by 25 m of unexcavated slots. The anchors were installed and locked off in each of the open slots before excavating the adjacent 25 m wide slots.

3.2.2. Pile and Anchor Construction

The piles were installed using a SoilMec R-625 hydraulic rotary rig. Rock augers and core barrels were used to advance the pile holes into the limestone. The rock sockets of the structural piles were cleaned using a cleanup bucket equipped with a wire brush. A surveyed control peg and string lines were used to maintain the pile reinforcement cage alignment during installation. A lockable temporary steel casing was used to prevent sloughing and ingress of water into a few of the pile holes. A low strength concrete was used to construct the infill piles. The structural piles were installed between the infill piles 48 hours after the infill piles were cast.

The anchor holes were advanced through the overburden by means of air rotary and a pneumatic guide device. The anchor holes in the bedrock were completed using a down-hole pneumatic hammer and a tricone roller bit. The anchor tendons were assembled on a construction bench downslope of the wall before being lowered into the drilled holes.

3.2.3. Design Changes and Construction Challenges

At the onset of the project, the contractor proposed to replace bar tendons with multi-strand tendons to enhance traffic and crew safety. In fact, strand anchors are considered favourable in areas with limited work space since they do not require large crane or lifting apparatus; require less space for storage, equipment, and workers onsite when compared to bar tendons.

Strand tendons are not commonly used in pile walls in Alberta and are considered less attractive in other aspects such as the requirement of field grouting the inside of the free length of the corrugated sheathing; inability to couple

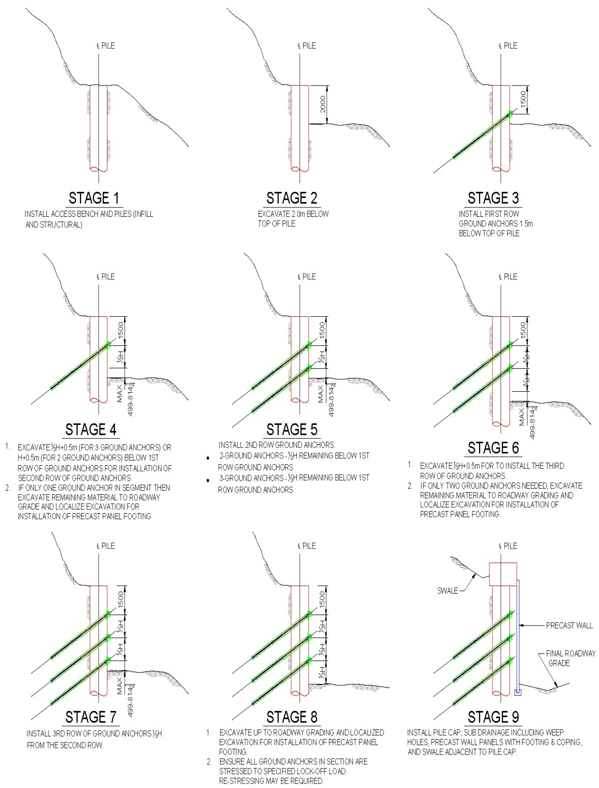


Figure 7. WPW general construction sequence

the bond zone and distress anchors once tails are cut; and complex lock off procedures, particularly when lock off loads are less than 50 percent of the ultimate strength of the strands. Despite the limitations of strand tendons, they were approved to enhance project safety aspects in addition to potential cost savings. For this project, the bar tendons were replaced 0.6-inch diameter, 270 KSI, 7 wire DCP permanent low relaxation strand tendons. At the early stages of construction, the structural team re-designed the wall to account for: a) reduction of the anchor hole from 225 mm to 200 mm as requested by the Contractor to improve the drilling quality and production rate; b) increasing the free length to establish the bond zone completely in the limestone formation since the oilsand thickness varied significantly along the wall alignment, and; c) reduction in wall stiffness in response to switching from bar tendons to stand tendons.

Table 1 provides the revised ground anchor schedule of the west pile wall. All anchors were designed and installed as per the Post Tension Institute's (PTI) Recommendations for Pre-stressed Rock and Soil Anchors (2008) and Canadian Highway Bridge Design Code (2006). As Table 1 shows, the anchor design loads vary from 610 to 900 kN with free lengths ranging from 17 to 30 m and 4 to 5 m bond zones in the intact limestone bedrock. Four to five multi-stand tendons were used in the design segments of the wall.

A total of four pre-production anchor tests, with at least one test in each of the anchor rows, were completed for this project. The pre-production anchor testing was completed

Table 1. Ground Anchor Schedule

Anchor Mark	Design Segment	Number of Strands	Factored ULS Load (kN)	Proof Load (kN)	Lock off Load (kN)	Free Length (m)	BOND LENGTH (m)
G8A to G72A	A2	4	610	650	430	21.9-25.0	4
G73A to G76A G93A to G110A	B	5	740	790	530	23.0-26.0	5
G77AtoG92A G111AtoG146A	B	5	740	990	660	23.0-26.3	5
G147A, B&C to G159A, B&C, G177A, B&C to G192A, B&C, G210A, B&C to G225A, B&C	C	A-5 B-4 C-4	A-900 B-550 C-550	A-960 B-590 C-590	A-660 B-320 C-320	A-24.2 to 28.1 B-23.0 to 23.3 C-19.0 to 22.0	5 5 5
G160AB&Cto G176A, B&C G193A, B&Cto G209A, B&C G226A,B&C to G232A,B&C	C	A-5 B-5 C-5	A-730 B-730 C-730	A-780 B-780 C-780	A-430 B-430 C-430	A-25.4 to 27.0 B-22.1 to 27.0 C-20.0 to 22.0	5 5 5
G233A&Bto G243A&B	D1	A-5 B-5	A-770 B-740	A-820 B-790	A-530 B-530	A-25.4 to 30.0 B-22.0 to 23.0	5 5
G244A&Bto G250A&B	D2	A-4 B-4	A-640 B-640	A-680 B-680	A-430 B-430	A-21.3 to 26.5 B-19.0 to 22.0	4 4
G251AtoG263A	D3	4	670	720	430	17.6 to 20.3	4

Notes: A, B, and C denotes upper, middle, and lower row of anchors, respectively. Anchors were not installed in design segments A1 and E.

to confirm the ultimate capacity and the deformation characteristics of the strand anchors. The anchors were loaded to double the design load with no signs of failure. Based on the testing results, the ULS factored adhesion value of the limestone was increased from 200 kPa to 400 kPa.

Since the limestone elevation was noted to vary by at least 3 m during the installation of the piles, a decision was made to drill inclined pilot holes in the infill piles to determine the actual depth of the limestone. This was crucial to confirm the length of the strand tendons and avoid the need for coupling.

Prior to drilling the anchor holes, the front face of each of the structural piles was exposed to drill the anchor holes. However, a few of the vertical bars of the strutural piles were intercepted while advancing the upper row of anchor holes. This has resulted in the reduction of the bending moment capacity of the affected piles. The reduced capacity had to be assessed and it was determinedthat the maximum bending moment towards the retained soil mass (i.e. under ULS conditions) did not correspond to the location of the reduced capacity. Later on, an allowance was given for cutting one bar at the pile back face and two rebars at the pile front face since the bending moment was mainly towards the back of the retaining structure. In addition, the lock off loads were slightly reduced to account for the possibility of coring through two vertical rebars at the pile back face. For the second and third rows, the Contractor exposed the vertical rebars prior to coring through the piles to avoid cutting more than one rebar at the front face of the pile.

The anchor assembly was lowered into the drilled holes as shown in Figure 8. The production anchors were grouted in two stages. The first stage included grouting the anchor corrugated sheathing in the free zone to about 2 m below the face of the pile, followed by cutting the individual Polyethylene smooth sheathing for each of the strands to 25 mm below the pile face. After completing the lock off, the jack was removed and the second stage of grouting was completed through the grout hole in the bearing plate to fill the annulus between the trumpet and the anchor hole. This process was followed by injecting grease into the trumpet and the installation of a plastic protective cap.

Steel shims had to be used to lock off the majority of the anchors since the proof loads were less than 50 percent of the ultimate strength of the stand tendons (521 kN and 652 kN for 4 and 5 strands respectively). The requirement was to avoid slippage of wedges. This resulted in multiple lock off stages, which complicated the construction procedures. For instance, for anchors in segment A2 with a final lock off load of 430 kN, the following steps were completed : a) Use a Tensar 200 tons jack to proof load the anchor to 430 kN, b) Use a Tensar 1500 jack with automatic seater to seat the wedges at an interim load of 552 kN using a stack of 19 mm thick shims below the re-stressable wedge plate, c) Lift off the anchor using a stressing bell to confirm the interim lock off load, which should not be less than 521 kN (wedge seating load), d) remove the shims to establish the 430 kN final lock off load and perform a final lift off to confirm the lock off load. The above steps were repeated for each of the design segments of the wall. The difference between the interim lock off load and the wedges seating load was mainly the 6.5 mm seating loss and the thickness of the shims was calculated based on the difference between the final lock off load and the seating load based on elasticity theory. Selected photographs of the wall construction are presented in Figure 9.

Figure 8. Lowering the stand tendon assembly into the anchor hole

3.3 Instrumentation Monitoring

Five slope inclinometers (SI11-W1 to SI11-W5) were installed in structural piles PS40, PS110, PS185, PS210, and PS265 to monitor the pile wall deflection. Figure 12 shows the incremental and cumulative deflection plots with depth for SI11-W2. The plot also shows the stratigraphy along the pile length, excavation depth, and the locations of the three ground anchors (G185A to 185C) installed within the pile. A negative deflection indicates that the pile wall has deflected towards the hill (uphill) in response to the anchor tensioning. A positive deflection indicates that wall moved towards the highway (downhill).

The pile wall moved downhill at this location by 3 mm in September 2011 in response to excavating the upper bench to install G185A. Afterwards, the wall moved towards the hill upon locking off anchor G185A and adjacent anchors. A rebound occurred in response to excavating the second bench to install G185B followed by further movement of the wall into the hill side upon locking off the second row of anchors. The same pattern occurred in response to excavating and installing the third row of anchors.

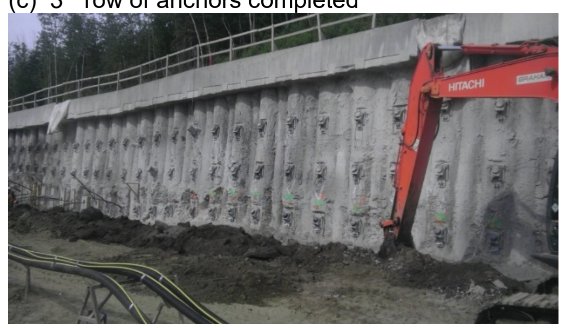
The pile wall moved uphill at this location by at least 7 mm in response to locking off the three levels of anchors. However, the majority of the uphill and downhill movements were in response to excavating and locking off the upper row of anchors. The observed movement of the wall during construction occurred mainly within the colluvium deposit overlying the competent oilsand and limestone formations. Between 2012 and 2014, and after all anchors were locked off, the wall moved downhill by 6 mm. Fourteen vibrating wire load cells were also installed in ground anchors G42A, G110A, G130A, G185A to C, G210 A to C, G230 A to C, and G246 A and B to monitor anchor loads.

Figure 13 shows the variation of G185 A to C loads with time. The graph shows that the three levels of anchors were not installed simultaneously. G185A was installed in late 2011 while G185B and C were installed in mid 2012.

(a) 1st row of anchor completed

(b) 1st and 2nd rows of anchor completed

(c) 3rd row of anchors completed



(d) Pre-cast panels installed

Figure 9. Selected photographs during wall construction

G185A displayed a reduction of the anchor load by up to 6 percent prior to locking off G185B. However, the initial reduction in anchor loads has been observed in similar pile wall projects. After locking off all anchors, the anchor loads increased by 30 to 50 kPa between 2012 and 2014. This indicated that the ground anchors restrained the wall from further downslope movement. Since the lock off loads were about 60 percent of the design loads, there was no concern with regard to the increases in the anchor loads.

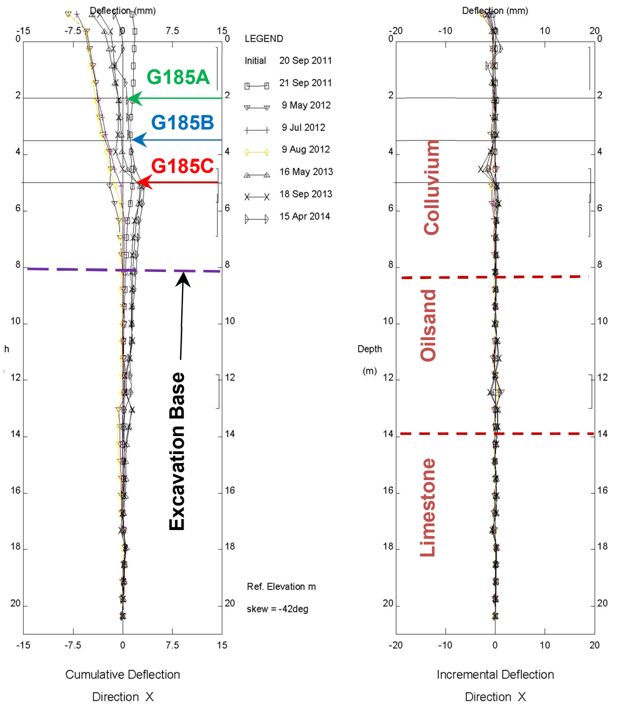


Figure12. SI11-W2: Incremental and cumulative deflections versus depth plots for Pile PS185

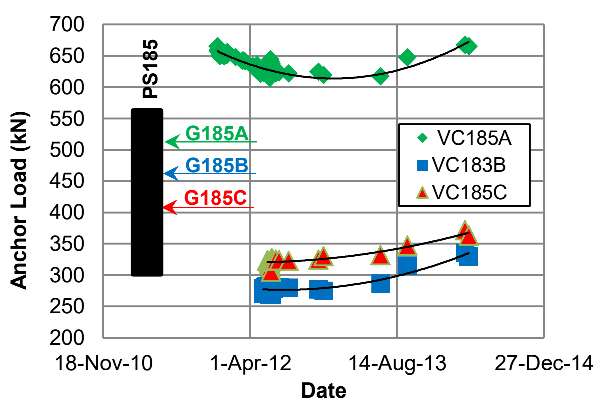


Figure13. Variation in G185A to C load versus time

4 CONCLUSIONS

These following provides conclusions and lessons learnt from this project.

a) The west pile wall has proven to be an effective measure to accommodate the Hwy 63 widening project.

b) Strand anchors are considered advantageous when it comes to project sites with limited space. However, strand anchors are deemed more complicated than bar anchors If strand anchors are used in pile wall projects, the design locked off loads of strand anchors should not be less than 50 percent the yield strength of the strands to facilitate the locking off procedure.

c) Instrumentation monitoring has been valuable to assess anchor loads and wall movements. Continued monitoring of existing instrumentation is essential to assess the long-term performance of the wall.

d) The construction sequence of the west wall reduced the risk of triggering a movement in the valley slope and distressing the wall during construction.

e) For pile walls with anchors installed through the piles, redundant rebar should be included to account for the possibility of coring into the rebar cage.

f) Effective communication between the design and construction teams has been essential for the success of this project.

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