Design and Construction of the Saint Albert Bridge

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ABSTRACT:

The St Albert Bridge is one of 38 bridges in the Northwest Anthony Henday Drive (NWAHD) Design Built Finance Operate (DBFO) project. This bridge is one of the most technically challenging to design and construct among the 38 bridges of the NWAHD Project.

In 2008, the Government of Alberta entered into an agreement with a DBFO team, to construct this $1.42 Billion project. The team consisted of Bilfinger Berger BOT as the financier; Flatiron Construction, Graham Construction and Parsons as the project builders; AECOM and Parsons as prime engineering designers; and Carmacks Enterprises responsible for operations and maintenance. Detailed design began in June of 2008 with construction start-up following shortly thereafter.

The main challenge of this project is an aggressive design and construction schedule. Challenges at the St Albert Bridge site included poor soil conditions consisting of saturated silts and site constraints which dictated a Top-to-Bottom construction scheme to meet the required road profile design. These challenges resulted in a unique bridge type chosen to design and construct. The bridge consisted of cast-in-place post-tensioned concrete box girder superstructure, tall concrete secant pile wall abutments and a spaced concrete pile pier. The superstructure was cast on-grade on a sacrificial concrete slab which was removed after completion of deck placement & post-tensioning by excavating down to finished grade. Accessibility for materials delivery/handling was greatly improved by constructing the structure using this method.

This paper describes the initiatives undertaken to deliver the project within schedule and budget, including the Top-to-Bottom construction method of constructing the bridge. Factors that impacted the project costs are analyzed and the specific approach adopted for design and project delivery are summarized.

Challenges encountered on this project and lessons learned are discussed in this paper. The authors conclude that constructability input is accessible and available to designers and is an integral part of the success in DBFO projects. Collaboration between designers and builders results in a positive effect on project budget and schedule during the construction phase. Finally, recommendations and strategies for future projects are presented.

INTRODUCTION

This paper is written in support of the concept of constructability implementation. It comprises a case study and presents constructability lessons learned during the conceptual design and final design phases of the St Albert Bridge.

For the past two decades, the construction industry has suffered from the lack of constructability implementation into the design process. This has caused many problems, such as increased costs and construction time, reduced productivity of project labour and equipment, and low quality construction [1]. Because of the size and complexity of projects and the fragmentation of the construction field into specialized roles and expertise, the construction industry urgently needs to implement constructability consideration earlier in the project life, ideally in the design phase. One of the major constructability implementation concepts is maintaining evaluation, documentation, and feedback regarding constructability issues throughout the project, to be used in later projects, as lessons learned [2,3].

In this case study, the authors investigate a number of elements found at the St Albert Bridge including traffic management, selection of structure type, and associated detailing of structure components, and schedule. The St Albert Bridge has an overall length of approximately 96 meters and is the most technically unique bridge in the Northwest Anthony Henday Drive (NWAHD) Project.

The accessibility to the City of St Albert and maintaining the roadway elevation at current configuration added additional constraints to this bridge design and construction. It is advisable for designers and constructors to be aware of some of the issue encountered in this project, as discussed in this paper.

This paper also describes the challenges encountered by the design team, and describes the initiatives undertaken to maintain an aggressive construction schedule. The final design and approach to project delivery are summarized.

PROJECT DESCRIPTION AND BACKROUND

Highway 216, which is also known as Anthony Henday Drive Figure 1, is a ring road highway built to full freeway standards encircling the City of Edmonton, Alberta and named after Alberta explorer Anthony Henday. The northwest quadrant of the ring road is currently under construction as part of the P3 NWAHD project.

The southern half of the ring road is completed and in use, starting east of Edmonton at Yellowhead Trail (Highway 16), south to Highway 14, then west past Gateway Boulevard / Calgary Trail (Highway 2), to Cameron Heights, then north to 137 Avenue. The highway designation 216 denotes its bypass linkages to the two major crossroads of Edmonton, Highway 2 and Highway 16. A similar ring road, Highway 201, is also being constructed around the city of Calgary.

The Alberta government has signed a 30–year contract with Northwest–Connect General Partnership for the design, construction, operation, and maintenance (DBFO) of Anthony Henday Drive from Highway 16 in the west to Manning Drive in the east. The construction phase is scheduled to be complete in late fall 2011. Maintenance of this portion will be handled by Northwest–Connect until 2041.

Figure 01 - AHD Map

The $1.42 billion project includes the construction of:

- Approximately 21 kilometers of new four– and six–lane divided freeway;
- Additional basic and auxiliary lanes
- 38 Bridges (29 bridges to be built in 2011 and 9 bridges will be future bridges)
- Eight interchanges (entrances and exits to freeway)
- Five flyovers (no access to freeway)
- Two rail crossings
- Additional pre–grading for future interchanges

The component selected for this case study is the St Albert Bridge which is part of the NWAHD project shown in Figure 2. The bridge carries St Albert Trail over the new Anthony Henday Drive and provides a key link to existing and future traffic between the City of Edmonton and the bedroom community St Albert.

Figure 02 - NWAHD Map

The Average Annual Daily Traffic (AADT) on Anthony Henday Drive at the North Saskatchewan River crossing has increased from 30,400 vehicles per day in 2007 to 45,000 vehicles per day in May of 2009. The AADT at 87 Avenue has increased marginally from 51,500 vehicles per day to 53,000 vehicles per day. The AADT on Anthony Henday Drive west of 50 Street was measured at 42,000 vehicles per day in May of 2009. In contrast, the estimated 2040 average daily traffic on the St Albert Trail section is 94,000 vehicles per day.

Alberta Transportation used a three step process for evaluating project proponents in their project award procedure. Teams are shortlisted through the first Submission Requirement (SR1) based on their financial capabilities, then further shortlisted the second Submission Requirement (SR2) based on their technical submissions, and finally the third Submission Requirement (SR3) in which one team is selected based on the lowest price.

The St Albert Bridge configuration during SR3 is shown in Figure 3.

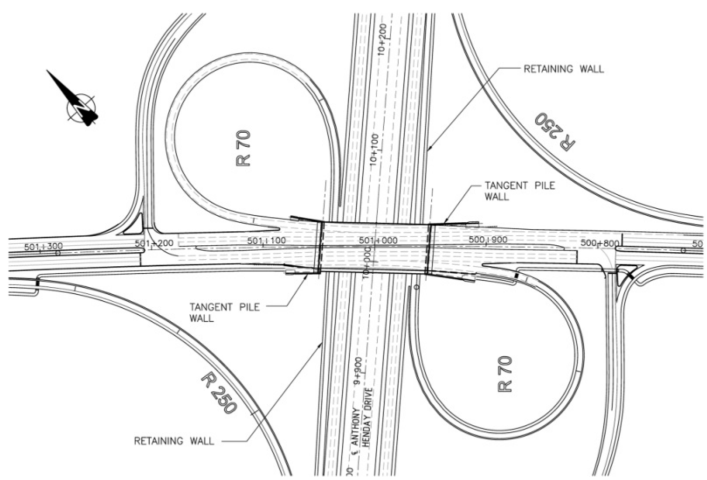

Figure 03 - Bridge Configuration During SR3

The final bridge configuration was changed from SR3 to the final design. The final design configuration is shown in Figure 4 and 5. The reasons for this change will be described in this paper.

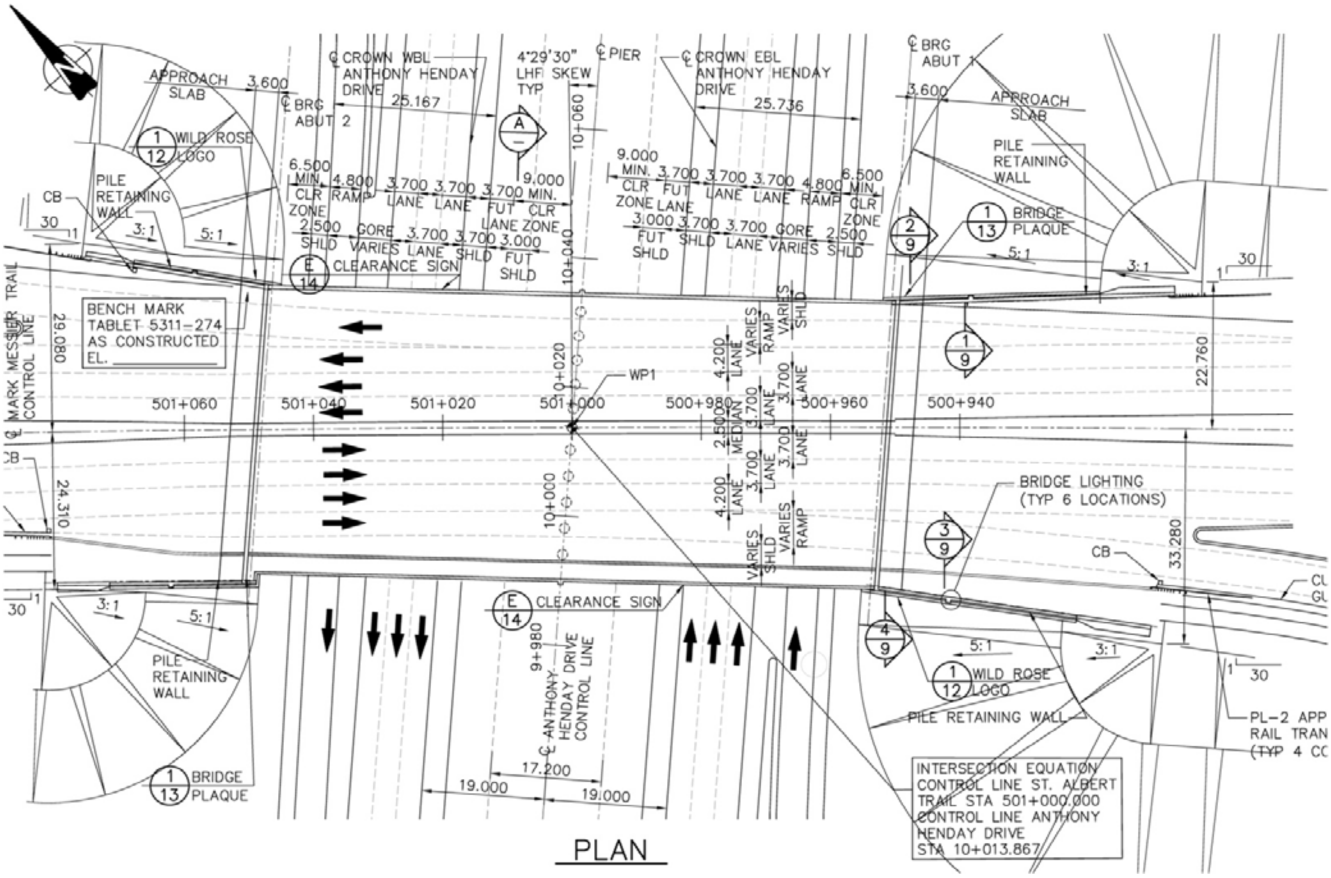

Figure 04 - Final Bridge Configuration

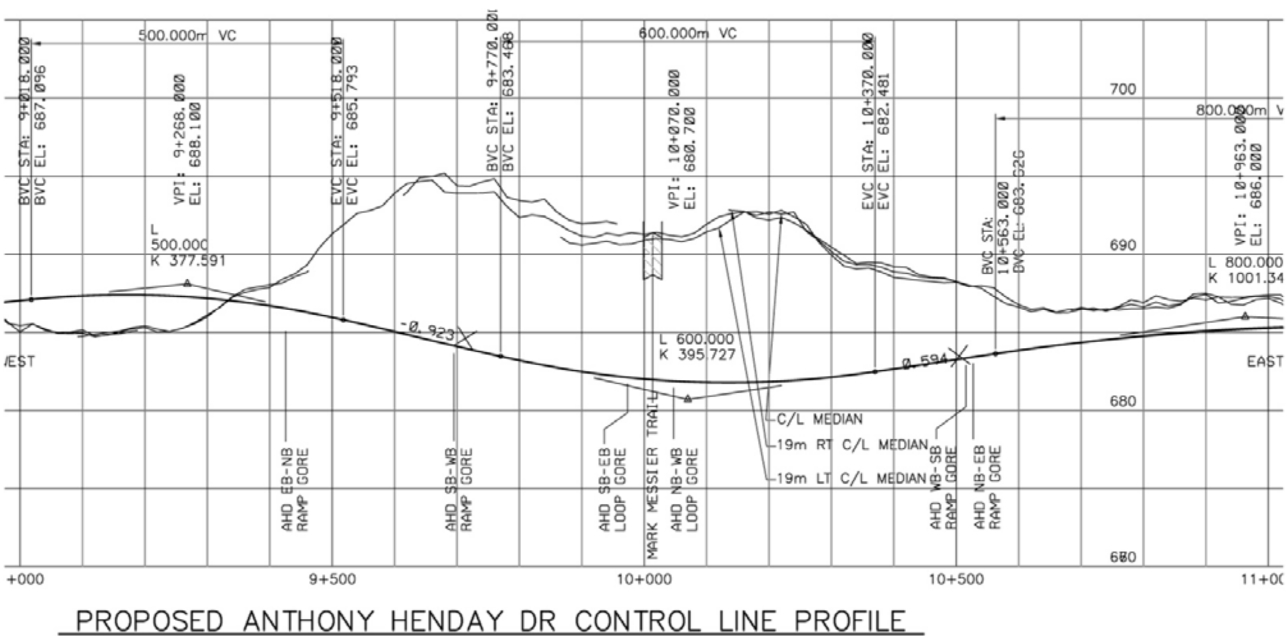

Figure 05 - Road Profile at Saint Albert Bridge

PROJECT TEAM

The project team consists of the following companies and as illustrated in Figure 6:

- Bilfinger Berger BOT is the financier for the project.
- Flatiron Construction Corp., Graham Construction and Engineering Inc., and Parsons is the contractor Joint–Venture that is teamed to build the project.
- Carmacks Enterprises Ltd. will be responsible for the project maintenance.
- AECOM and Parsons are the consultants that teamed together to deliver the project design and engineering services during construction.
- Numerous subcontractors and subconsultants.

Numerous subcontractors, vendors, and suppliers are supporting the prime contractors; those include; earth-work subcontractors, MSE wall suppliers, steel girder suppliers, precast concrete girder suppliers, machinery suppliers, cast-in-situ concrete suppliers, bridge bearing and expansion joint suppliers, many piling companies, and others.

Many sub-consultants supporting the main consultant; those include; environmental, geotechnical, electrical, Structure Independent Design Reviewers (IDR), and others.

Figure 06 - Project Team

SITE CONDITIONS AND PRELIMINARY DESIGN

The general subsurface conditions comprise a near surface lacustrine clay deposit of 4 to 6 m that is stiff to very stiff in consistency. With depth the clay becomes more silty and gradually grades into a silt deposit. The silt grades into sand, which is underlain by glacial clay till and bedrock. Based on the proposed Anthony Henday roadway alignment, it was anticipated that the lower half of the slopes would expose the soft, wet silt. Previous experience in west Edmonton had demonstrated that wet silt slopes were unstable at angles as flat as 7H:1V. As a result, there were significant concerns regarding long term stability of the bridge headslopes.

Historically headslopes for bridges in Alberta are typically designed with 2H:1V slopes. Due to the poor soil condition at the site, it was determined that 5H:1V headslopes would be required for a conventional bridge, which would significantly increase the overall bridge length. An option utilizing a mechanically stabilized earth (MSE) walls were not considered feasible due to the low bearing capacity of the soil. Therefore it was determined that a pile wall is the most suitable option to create the grade separation. To further economize the bridge it was decided to use the pile wall to support the bridge abutments and carry the vertical load from the bridge, as well providing lateral support for the soil.

Four options were considered for the pile wall:

<u>Alternative I:</u> Cantilever wall consisting of side-by-side or tangent concrete piles. The poor site soil conditions make this option not technically feasible.

<u>Alternative II:</u> Similar to alternative I with tangent piles however this option would use the superstructure as a compression strut between the two pile walls thus providing lateral support at the wall top. This option is similar

in concept to an integral abutment bridge.

Computer modeling was performed for two pile diameters, 900mm and 1500mm. The results of this analysis showed the following:

- 900mm caissons offer some flexibility for the superstructure thermal expansion, but are not sufficient to resist the lateral earth pressure.
- 1500mm caissons are too rigid to accommodate superstructure thermal expansion

The conclusion of the analysis was that the integral abutment concept is not technically feasible for this site and therefore this option was eliminated.

Alternative III: This option consists of piles that are overlapping to ensure no soil flow between the piles and is referred to as a secant pile wall. The piles would need to be tied-back using soil anchors or a similar method, and was found to be the most suitable alternative for this bridge.

Alternative IV: A soldier pile wall (providing gaps between the piles) was also evaluated. If subsoil conditions are favourable, it is possible in some instances to rely on soil arching between the main piles. This option is similar to the tangent pile wall option, but eliminates the infill piles. Due to the relatively soft soil and risk of groundwater seepage, it was anticipated that over time, the soil could potentially 'flow' between the caissons. As a result, this option was considered not feasible.

To expand on Alternative III, the option of using dead-man anchors instead of soil anchors was investigated. The advantage of the dead-man option is that the ties can be horizontal, thus their whole resistance can be utilized to resist the horizontal forces. In the soil anchor option the ties need to be inclined in order to be anchored into a deeper subsoil layer, thus for resistance only their horizontal component will be utilized while their vertical component will be added to the piles as vertical load. The disadvantage of the dead-man anchor option is safety risk, as any excavation or movement in the dead-man anchor can cause progressive collapse of the whole bridge structure.

CHANGING TIE BACKS CONFIGURATION

Upon investigation of the soil anchor requirements it was determined that two anchors per pile were necessary, thereby creating an upper row and a lower row of anchors. Figures 7 and 8 show the preliminary arrangement of the anchors.

Soils at the site from the Top-to-Bottom were classified as approximately 6m of a stiff clay, 10m of weak silt, 7m of clay till, and overlying bedrock. The upper layer of clay and the lower layer of clay till are competent material in which anchors could be founded however the large layer of weak silt is not.

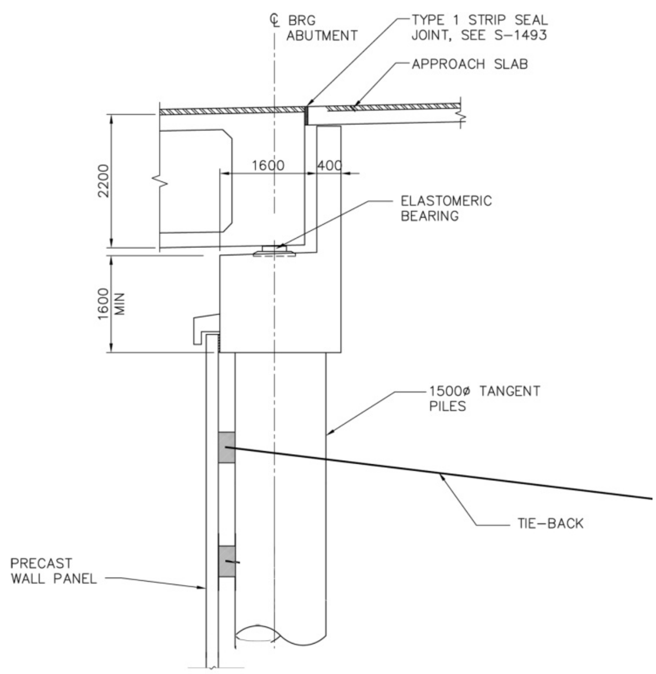

Figure 07 - Tangent Pile Wall Tie-Backs in SR3

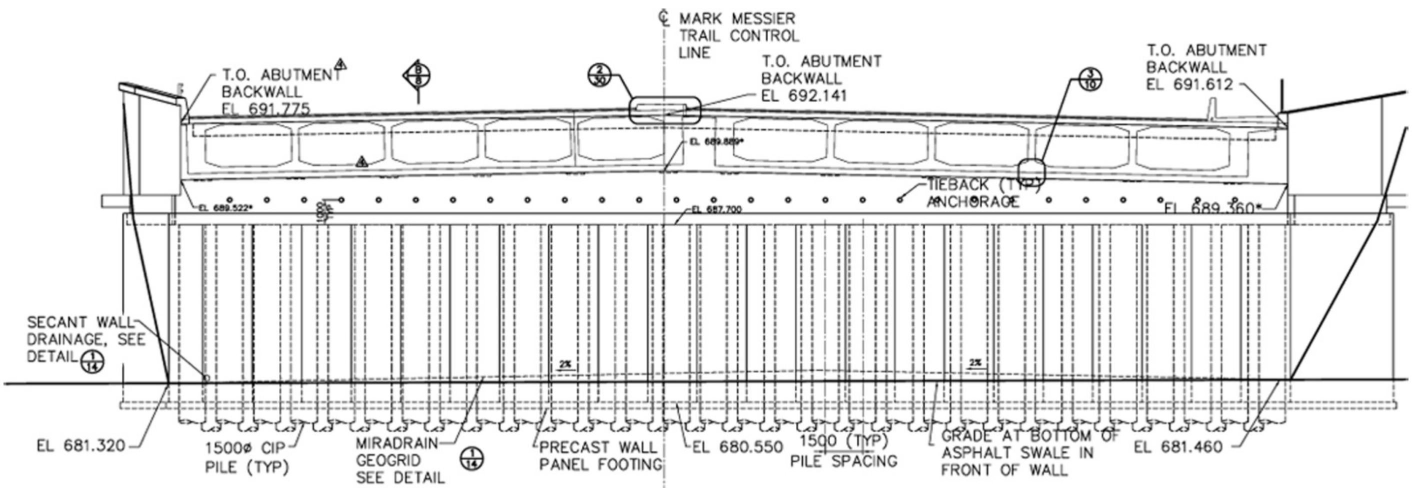

Figure 08 - Tangent Pile Wall in SR3

The resulting geometry of the two rows of tie back anchors is shown in the figure and it can be seen that the lower row would require an acute angle with an excessive length. Figures 9 and 10 shows two options studied later on the project for the tie-backs.

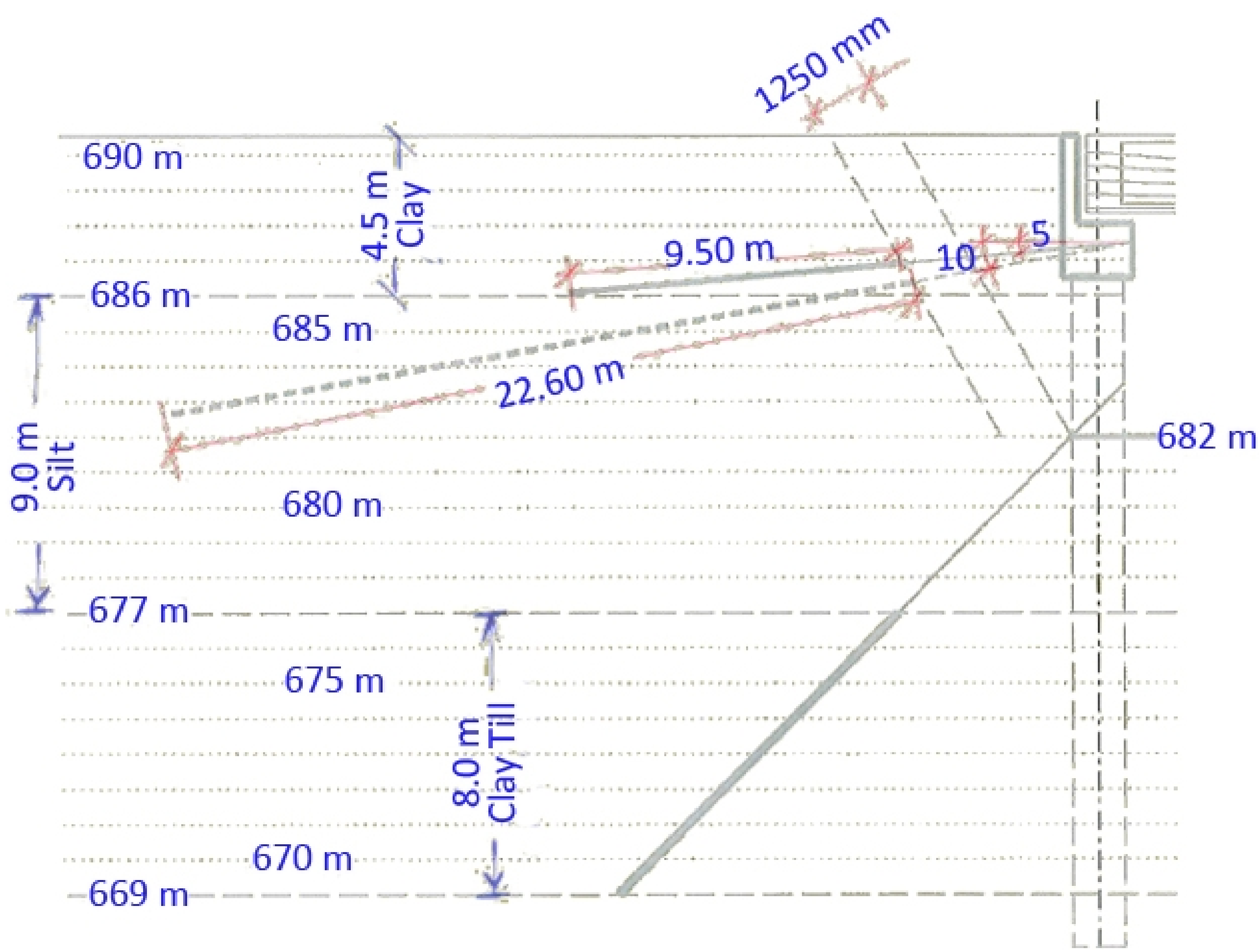

Figure 09 - Tie-Back Modified Option I

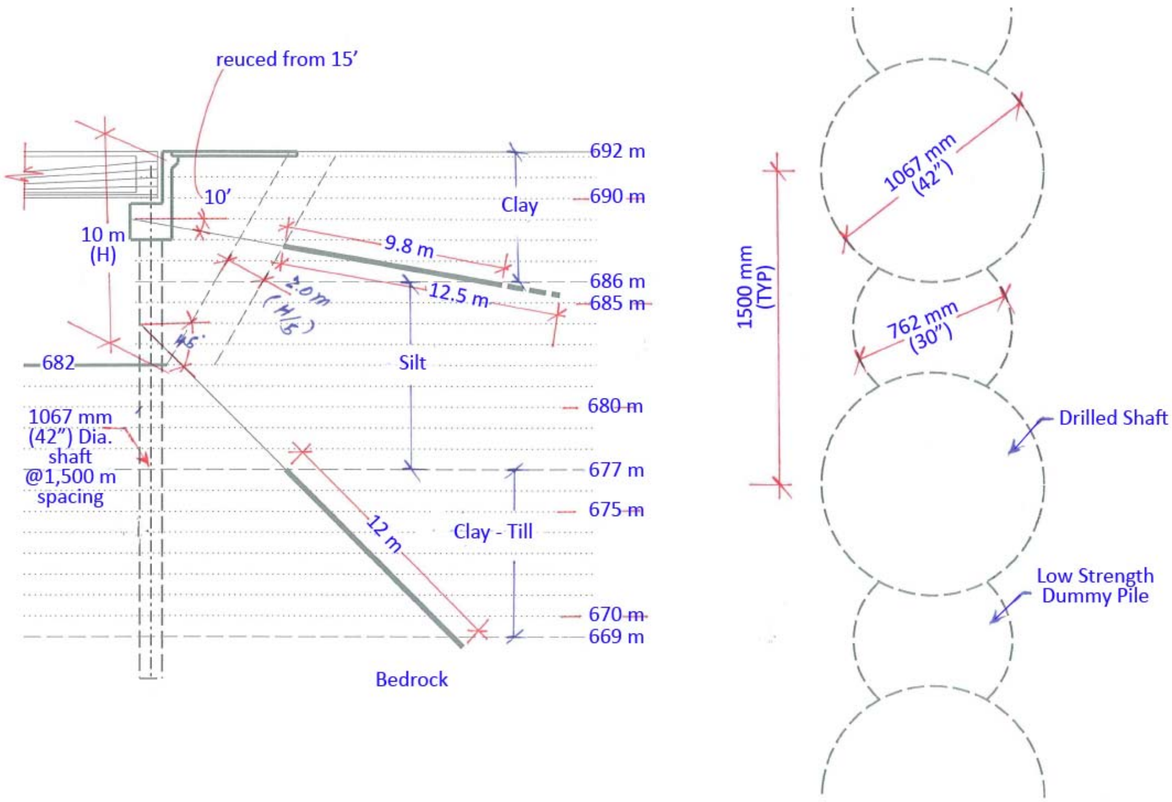

Figure 10 - Tie-Back Modified Option II

Due to the relatively high cost of the lower anchor row compared to its efficiency, later on in the project, a dead-man anchor system was re-visited. To address the safety risk associated with a gravity-resisted dead-man anchor, Figures 11 shows an option for a pile supported dead-man anchor system. For economic reasons the configuration shown in Figure 11 was chosen. This option was further refined to incorporate the use of high strength stressing strand as the tie material with a proprietary "Double Corrosion Protection System" for longevity.

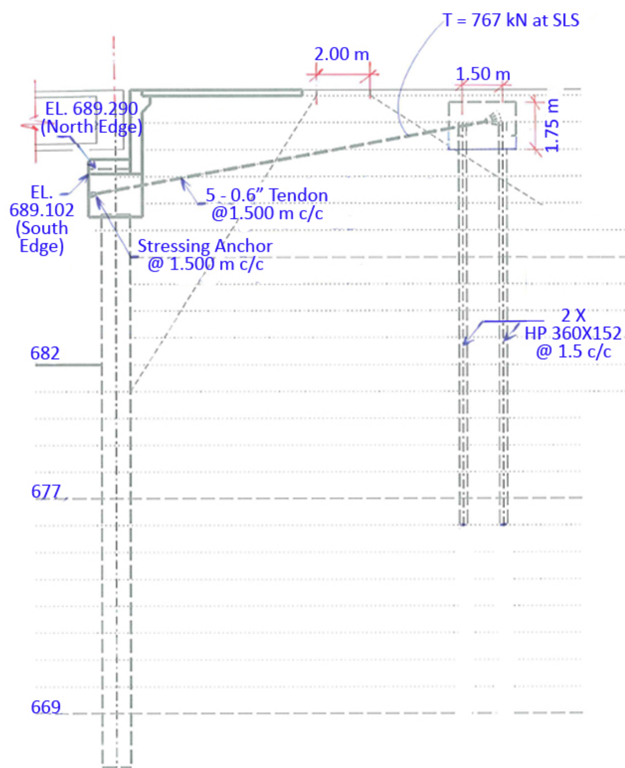

Figure 11 - Modified Tie-Back with Deadman

Figure 12 shows the active and passive zones behind the proposed tangent pile wall. The active plane would begin at the deepest cut adjacent to the front face of the wall as shown. The region where we can apply passive pressure to the soil is behind the passive plane (defined by alpha p).

As a minimum requirement; the deadman block would be placed behind and above the passive plane. For the case where a pile support with the deadman is incorporated, the active and passive planes have been separated by 2 meters to deal with the rotation of the deadman below the passive plane. For the location of the anchor strand on the tangent pile wall, it would ideally be within the upper stiff clay at approximate elevation 686 or higher. In order to rely on a single tieback, if the anchor needs to be deeper, there is a risk of poor ground conditions to install the tieback and whaler system at the face of the wall. It might be possible but it could be a mess requiring dewatering and gravel or a mud slab to facilitate construction.

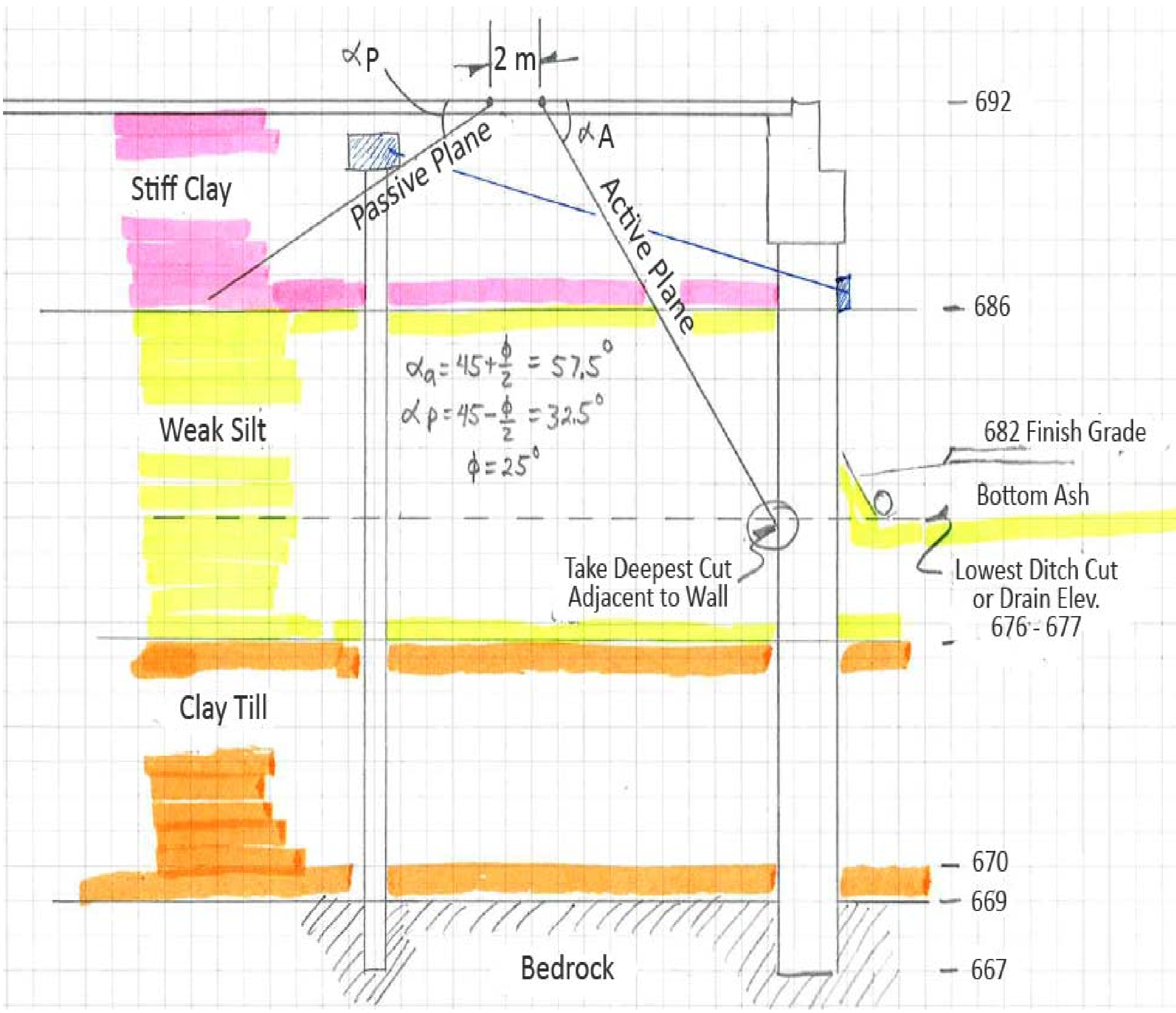

Figure 12 - Passive Plane

CHANGING PILE CONFIGURATION AT ABUTMENTS

To further optimize the economy of the pile system at the abutments, an alternative pile arrangement was investigated. The arrangement in the bid phase of the project used tangent or side-by-side piles to form the retaining wall and support the bridge abutments, as shown in Figure 8. A secant pile system was investigated and determined to be viable. The arrangement ultimately adopted used 1067mm diameter concrete structural piles spaced at 1500mm on center with 760mm low strength unreinforced concrete infill piles between the structural piles. The advantages of this system are that the quantity and thus cost of the structural piles is minimized and a greater amount of construction tolerance is achieved. The system was constructed by installing the infill piles first using a low strength concrete mix, then installing the structural piles by partially drilling into and thus overlapping the infill piles. The infill pile concrete was designed to achieve 5 MPa compressive strength at the time of structural pile installation. Details of the pile system are shown in Figure 13. The Tie-back configurations are modified shown in Figure 13.

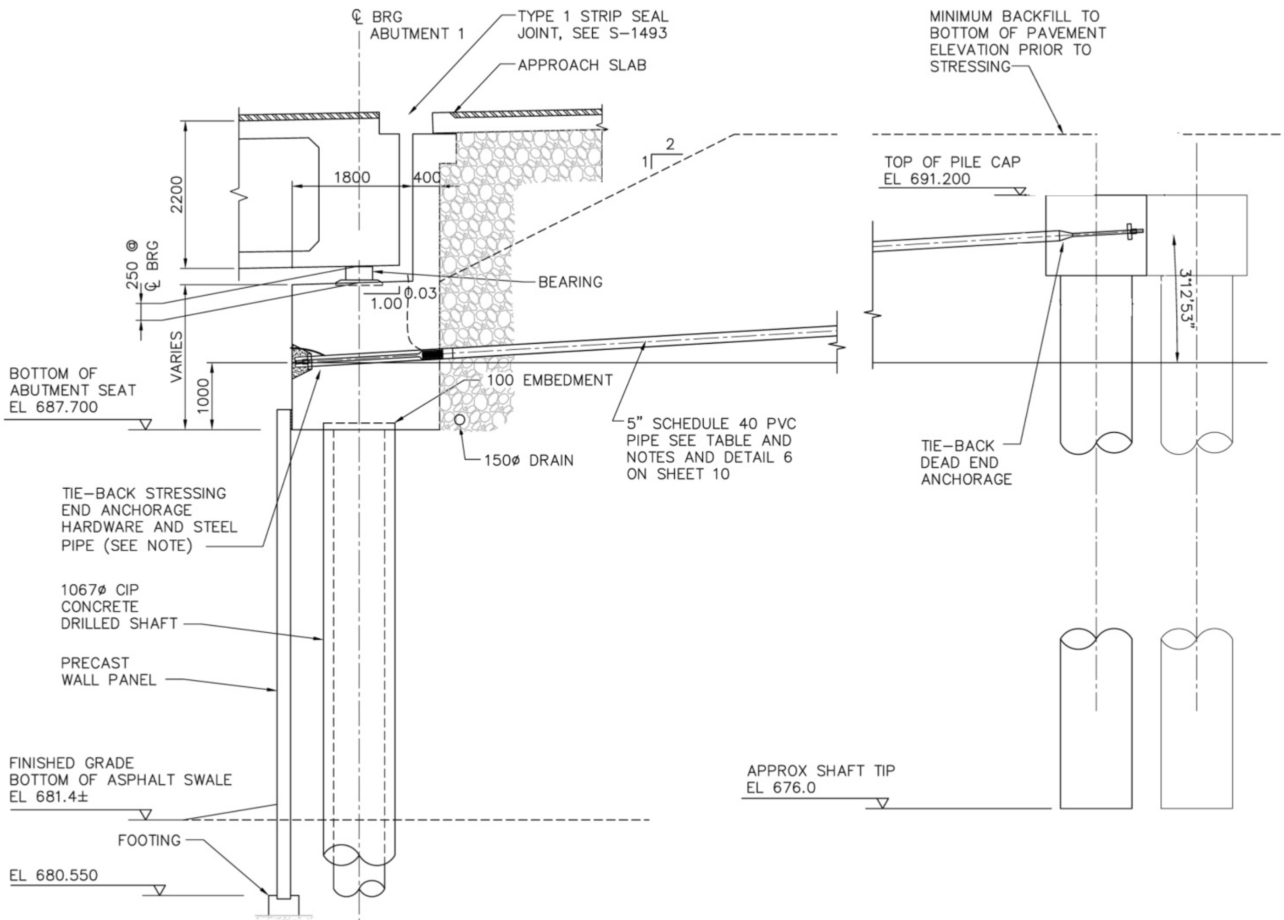

Figure 13 - Tangent Pile Wall Tie-Backs in the Final Design

Figures 9 and 10 show 2 alternatives studied to be compared to the dead-man tie-back option cost-wise.

Figure 15 shows an additional abutment configuration option that was studied during the design phase and was selected for the final design.

The final layout of the pile wall is shown in Figures 13 and 14. The two wingwalls on each abutment were tied together using the same tieback system and were thus self anchoring. The pile retaining wall under the abutment seat, pile walls under the wingwalls and tieback deadman piles created a box of reinforced soil for each abutment. At the ends of the pile supported wingwalls once the headslopes reached the elevation of the abutment seat, a conventional gravity-designed retaining wall on a spread footing was used until the headslopes reached the upper roadway grade. Once the excavation is completed for the lower roadway, Anthony Henday Drive, the secant pile walls are to be covered with precast panels for aesthetics.

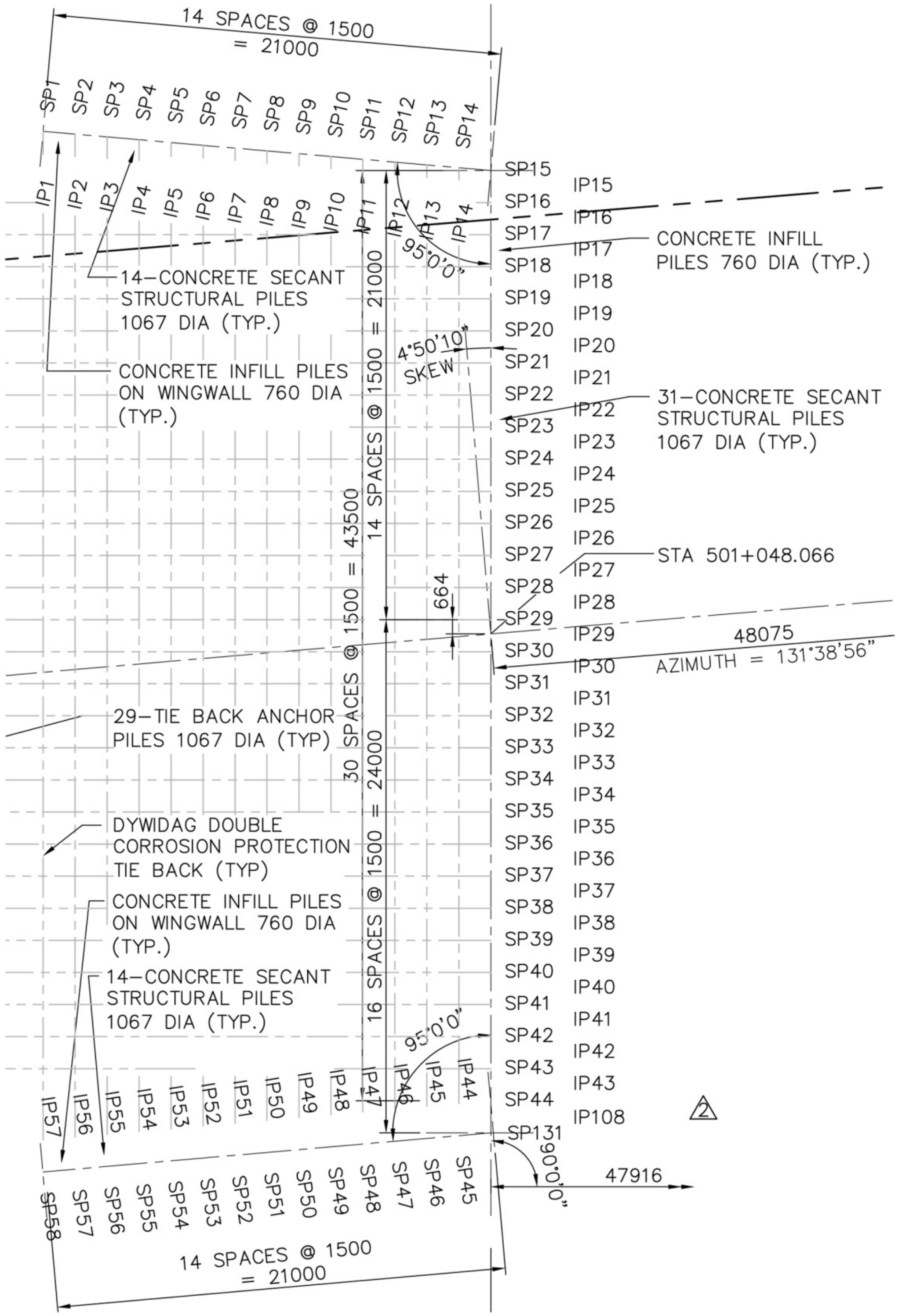

Figure 14 - Tangent Pile Wall in the Final Design

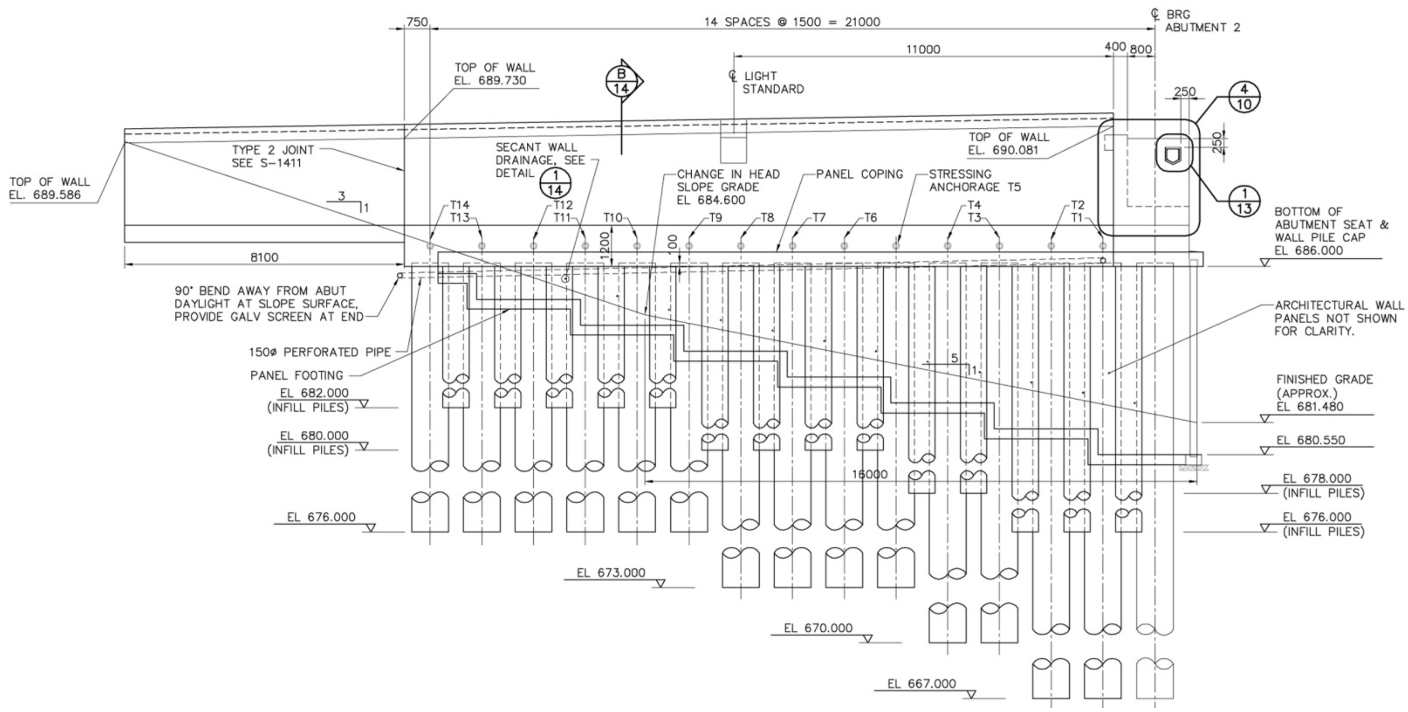

Figure 15 - Abutment Configuration

CHANGING PILE CONFIGURATION AT PIERS

The design at the bid phase of the project for the pier piles comprised a straight shaft rock socket pile cast-in-place concrete piles 1500mm in diameter spaced at approximately 4m on center into the underlying weak clay shale bedrock resulting in piles that were approximately 30 m long. Options for shortening the pier piles included the possibility of belled cast-in-place concrete piles in the bedrock. The original design had assumed that the bedrock was too hard to construct a bell within the bedrock. Discussions with the piling contractor confirmed that belling within the bedrock was possible. Consequently the design was economized with the use of significantly shorter belled piles founded within the bedrock layer

BACKFILL AND POST-TENSIONING INSTALLATION SEQUENCE:

The backfill and post-tensioning installation sequence for the secant pile wall include the following:

1. Backfill with sand up to underside of transverse wingwall ducts.
2. Install wingwall duct and post-tensioning tendons.
3. Stress tendon to 25% from one end for strengthening and alignment.
4. Add sand up to underside of post-tensioning that extends from abutment seat to anchor caps. Backfill to be compacted between lifts, install duct and tendon.
5. Stress abutment to anchor cap tendon to 25% (same as in step #3) from one end.
6. Backfill with more sand above abutment to anchor cap duct 24 inches above duct. Backfill to be compacted between lifts.
7. Continue backfill up to road level (as shown in Figures 16, 17, and 18).
8. Stressing to be done in conjunction with the stages on DSI shop drawings and the design drawings. Stress post-tensioning in the following sequence:

a. Stress all odd number post-tensioning (1, 3, 5, 7, 9, 11, 13, etc.).

b. Stress all even number post-tensioning (2, 4, 6, 8, 10, 12, etc.).

9. Grout post-tensioning duct with approved grouting product.

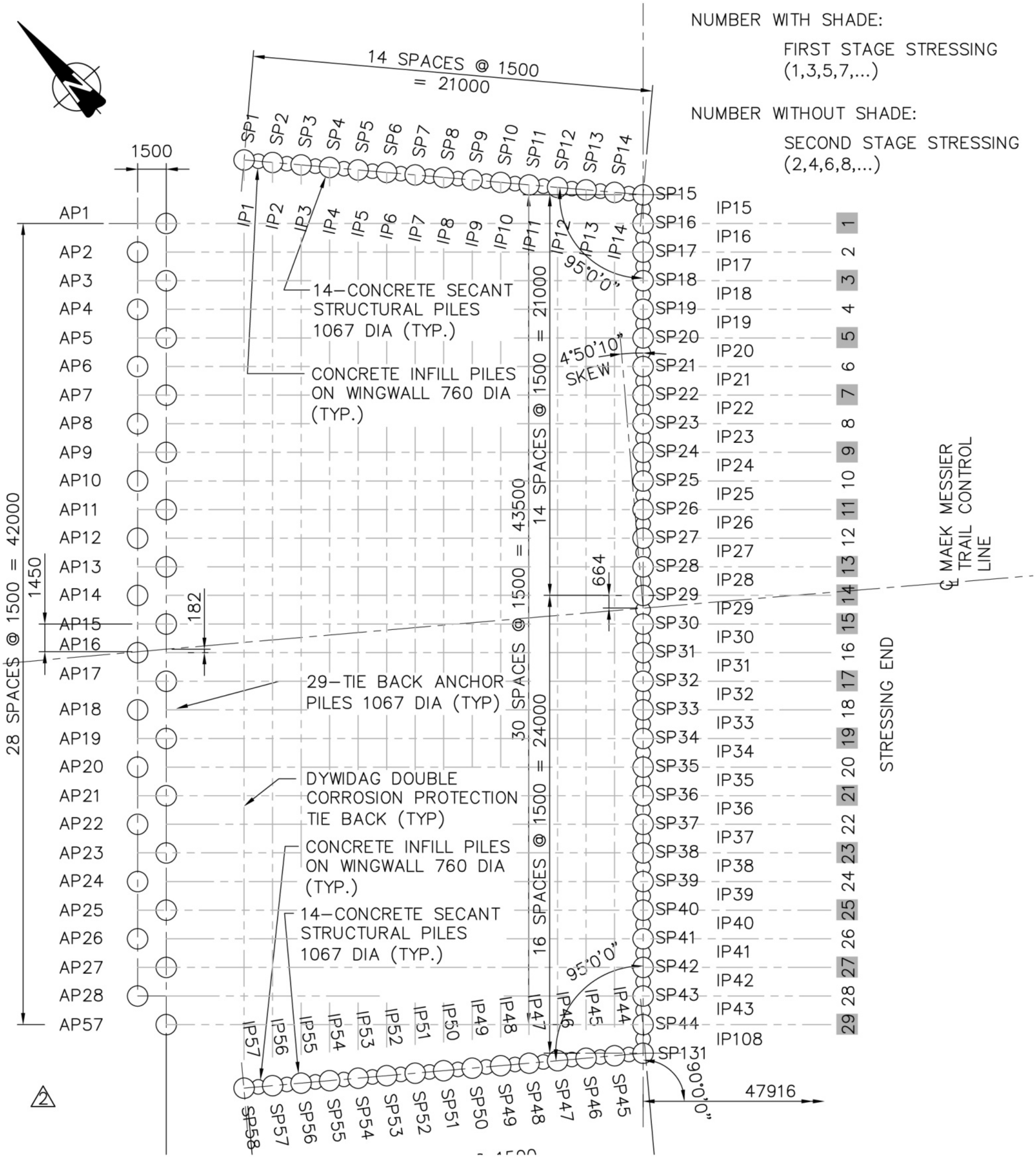

Figure 16 - Stressing Sequence

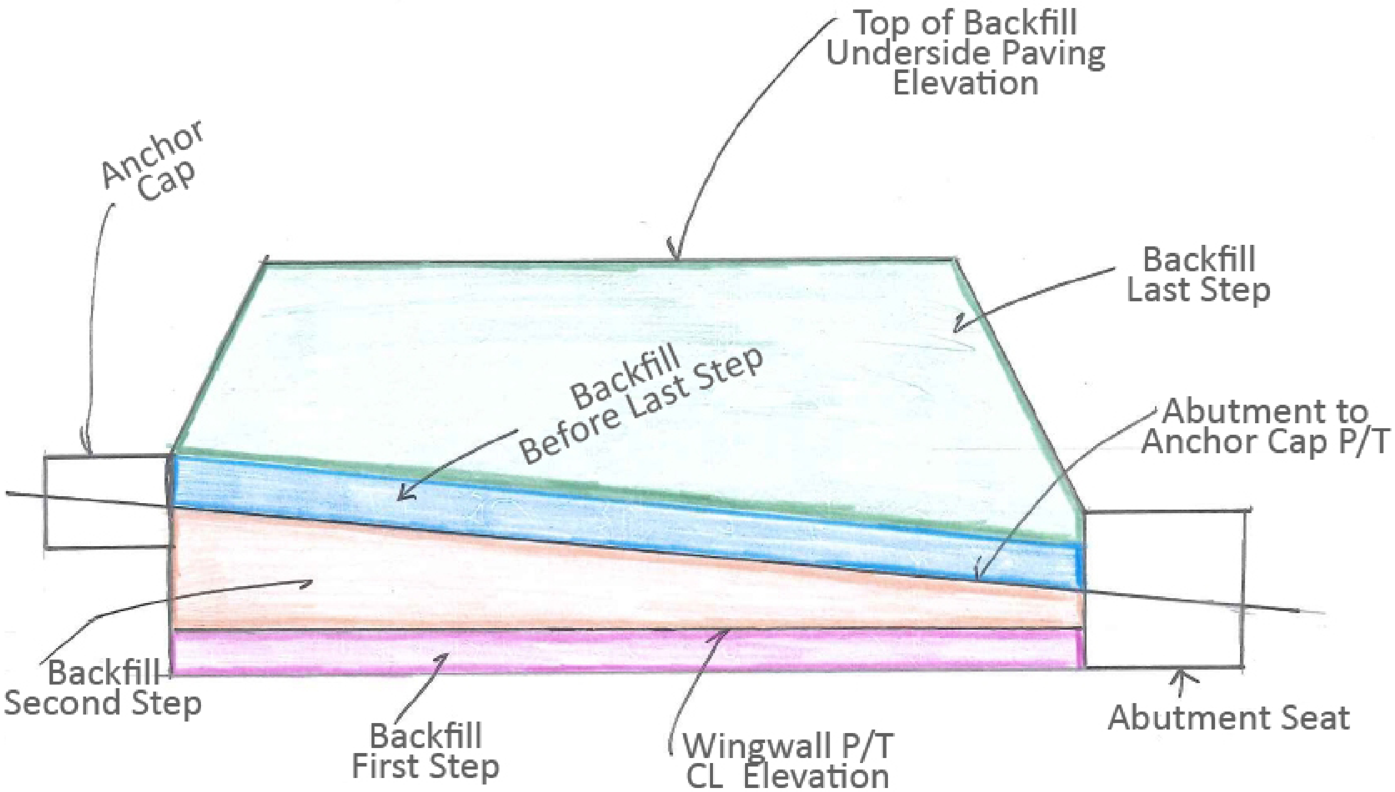

Figure 17 - Backfill Sequence Longitudinal

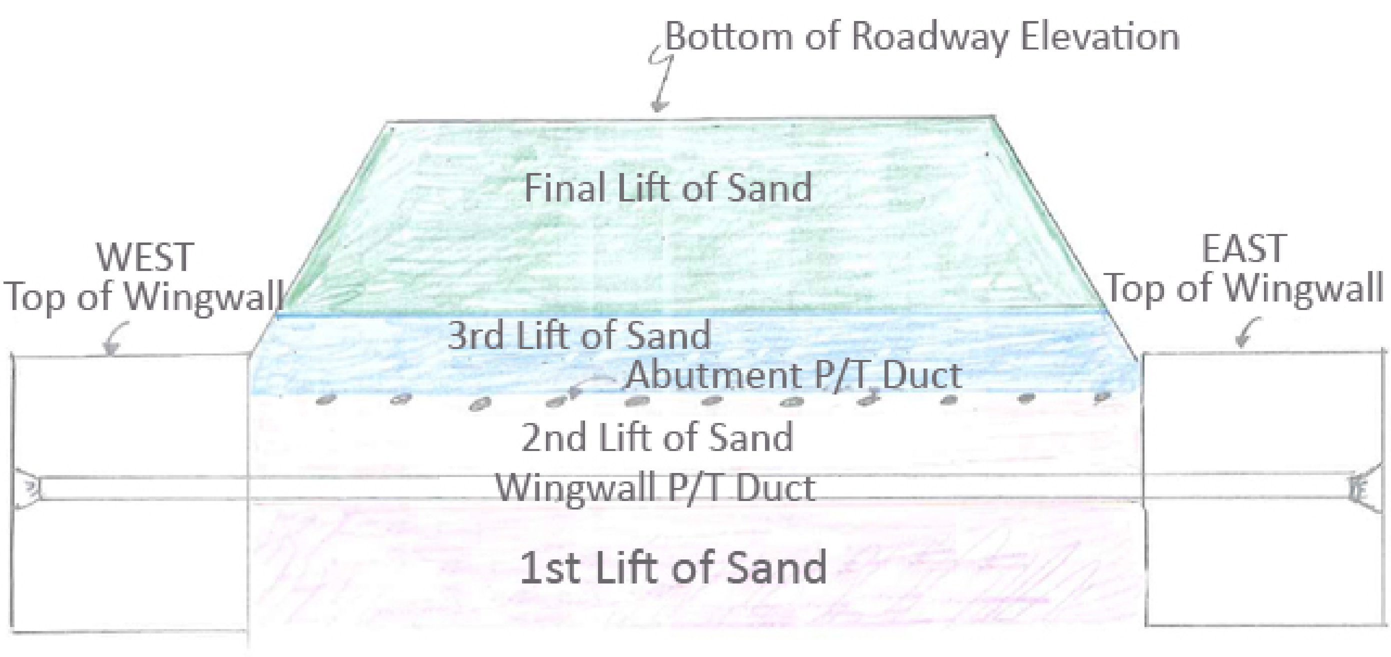

Figure 18 - Backfill Sequence Transverse

SUPERSTRUCTURE CONSTRUCTION SEQUENCE

Unique to a “Top-to-Bottom” construction method the superstructure concrete box girders were constructed as fully supported on-grade and later post-tensioned.

In the “Top-to-Bottom” method of construction; once the soil below the bridge is excavated there is no way to prevent the piles from moving forward. This movement is beneficial for the structure to distribute the loads evenly on the tangent pile wall, as the load will be redistributed due to this movement.

Tie backs are installed when the excavation reaches the designated level.

During the SR3, it was decided to use tangent piles due to the road profile shown in Figure 5 and the soil conditions at Location 5. During the SR3, the proposed tangent pile walls at St Albert Trail functions as the bridge abutments and extend approximately 77 m across St Albert Trail. This structural portion requires a form of tie-back as generally illustrated on Figure 7. The tangent pile wall continues as the wing wall extending beyond each side of the main structure abutment, for approximately 15 to 35 m to retain the slopes transitioning between St Albert Trail on ramp or St Albert Trail and the Northwest Anthony Henday Drive mainline. Where the wing wall portion exceeds approximately 4.5 meters of retained earth, tie–backs are required, the remainder will be cantilevered. An additional portion of retaining wall with retained fill heights of approximately 1 to 2.5 meters is required beyond the tangent pile wall area for a total retaining wall length of approximately 250 meters at both the north and south bridge abutment areas. The particular retaining wall system beyond the tangent pile wall portion will be designed as a part of the final detailed design effort.

The vertical structural loading of the bridge is carried by the tangent pile wall. The horizontal component of the bridge loading and retained soil loading is to be carried by permanent tie–back anchors and partially by cantilever action. The axial capacity of the piles is determined similar to a bored cast–in–place shaft, similar to a traditional drilled shaft. Only a portion of the skin friction would be utilized given the proximity and influence of adjacent piles. Tie back anchors are limited to within the upper stiff clay soils above approximate elevation 686.0 meter or they must extend into the deeper clay till or bedrock anticipated below approximate elevation 677.0 meter. Additional field exploration and soil and bedrock strength data is collected to complete the design as a part of the final detailed design effort.

Bridge Approaches have 3:1 sideslope at bridge locations with guardrail and with subgrade to be widened by 1.0 m on each side. The subgrade width will be tapered back to the un-widened subgrade width at a ratio of 30:1.

The sequence of this construction is outlined below.

1. Install "rat slab" - 75mm thick sacrificial slab below the level of the bottom of the bridge, this slab will be shaped to provide proper camber to account for the future DL deflection of the bridge
2. Install bearings
3. Install a false level over the abutment seats with sand to the level of the bottom of the superstructure
4. Install rebar for the bottom slab
5. Cast bottom slab, embedding bearings
6. Install mild reinforcement, post-tensioning ducts, and form and pour web walls
7. Form roof slabs (bridge deck) and install reinforcement
8. Post-tension superstructure
9. Form and pour abutment backwalls
10. Remove sand from between the abutment seat and the superstructure with compressed air
11. Install gravity walls
12. Backfill abutment tieback system to the level of the underside of the roadway structure and install

approach slabs

13. Install bridge deck waterproofing system, bridgerails, and wearing surface
14. Open bridge to traffic
15. Excavate under bridge including "rat slab"
16. Install aesthetic precast facia panels over pile walls and pier columns
17. Construct lower roadway (Anthony Henday Drive)

Figures 19 illustrated the “single” formwork that was used to form both the top and bottom of the box girders.

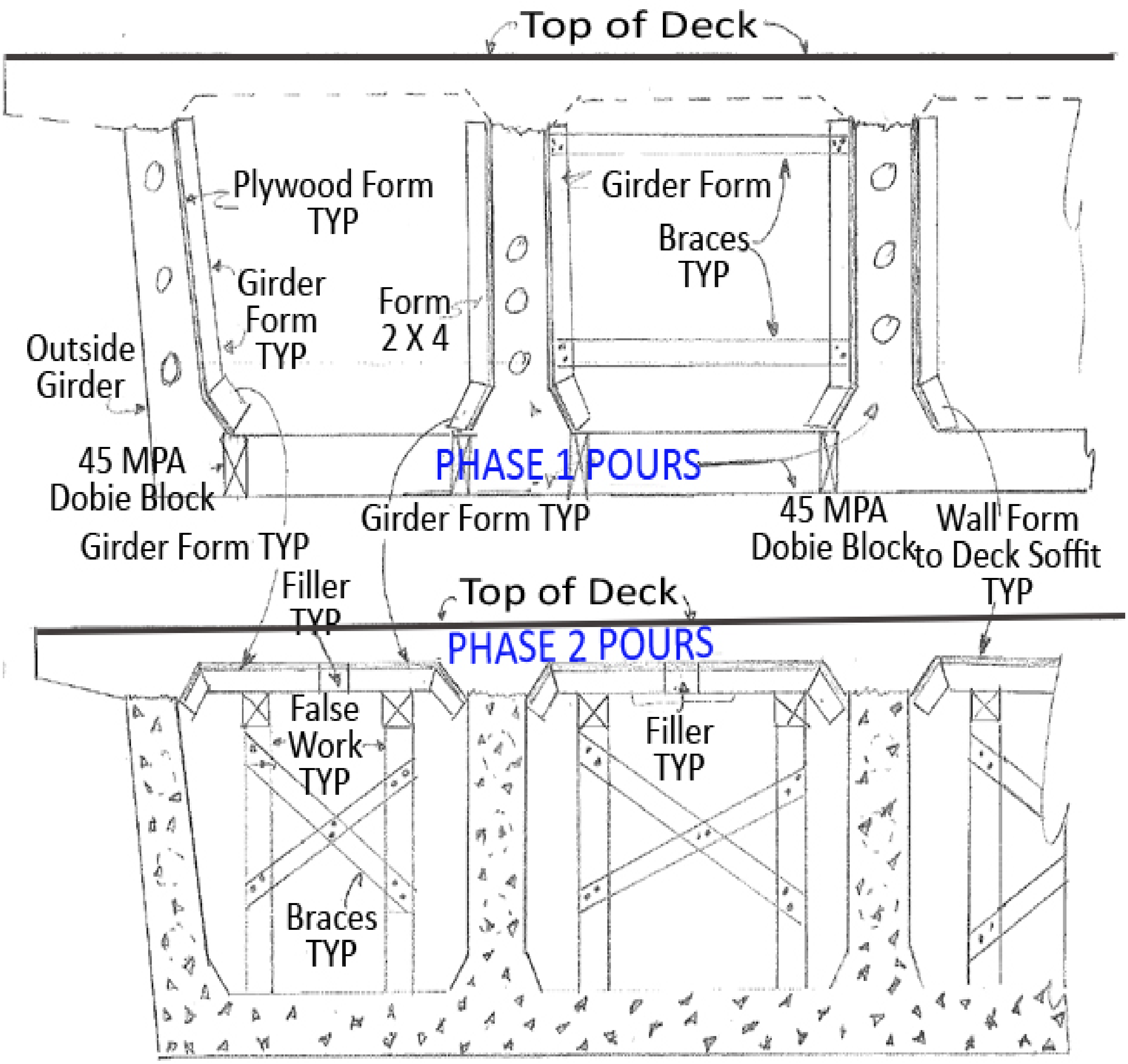

Figure 19 - Box Girder Form

PRECAST PANEL DETAILS

Precast Panel Details as well was drainage details for the secant pile wall are shown in Figure 20. The precast panel serves as a facial aesthetic wall to cover the secant pile wall. Panels are connected by anchorage assembly to the piles. After excavating in front of the secant pile wall, anchors are installed into the piles by epoxy. The precast panels are supported on a concrete pad which is founded on granular fill Des 2 Class 25 as per Alberta Transportation Specifications.

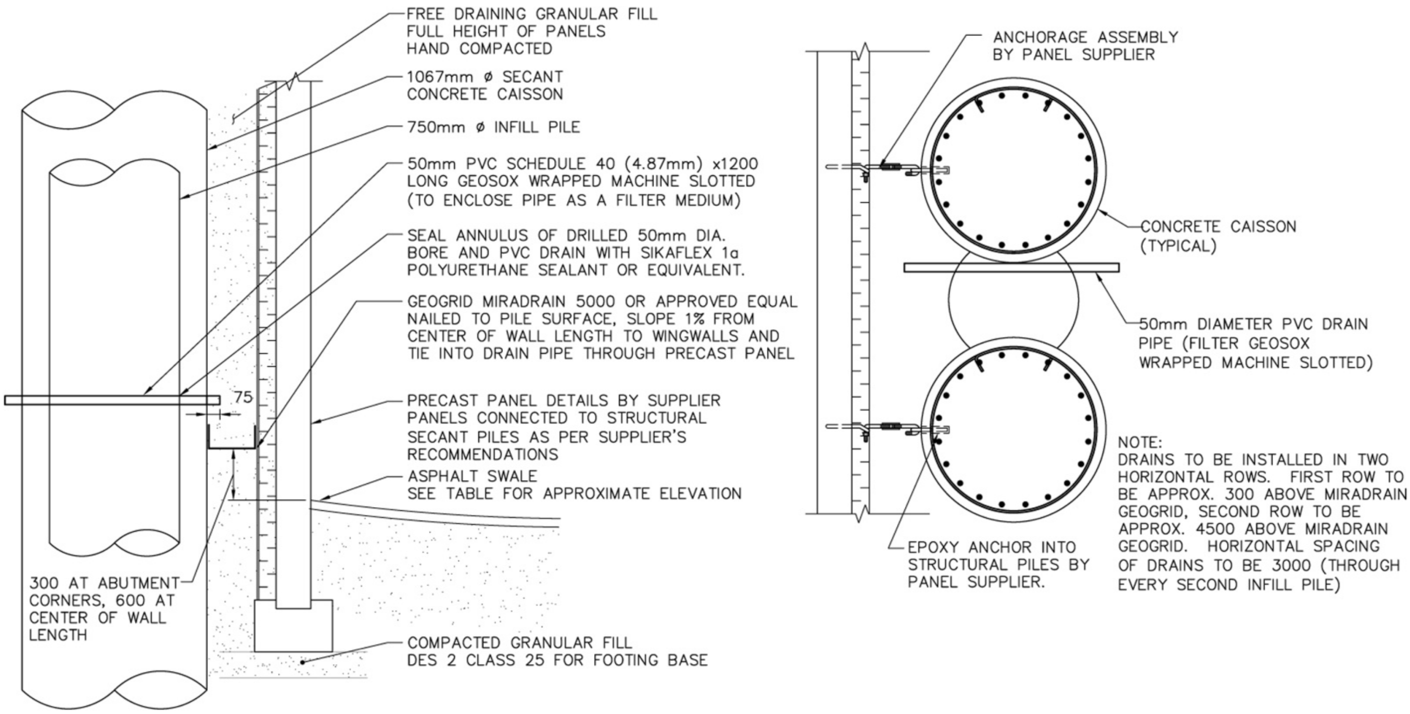

Figure 20 - Precast Panel Detail

SURFACE CRACKS IN SHAFTS

Surface cracks in abutment shafts were noticed as shown in Figure 21. An inspection was performed on April 21, 2009 of the pile tops for the piles on Abutment 2 (north abutment).

The inspection revealed that cracks were visible on most of the structural pile tops for the abutment seat piles and the piles for both wingwalls. These cracks generally extended in a circumferential direction between the projecting reinforcement bars with some piles having a crack around the entire circumference and some piles having a crack around a portion of the circumference. The cracks varied in width from approximately two to three millimetres down to less than a millimetre.

A few random piles were selected to do further investigation. It was decided to use localized chipping of the top concrete surface to determine the depth of the cracks. The investigation revealed that these cracks did extend deeper than approximately 50mm. Based on the inspection and the further investigation of the pile tops it was concluded that these surface cracks will not compromise the structural capacity of the piles. However a definitive cause of the cracks was not determined.

It is known that some construction equipment was operated in close proximity to the piles shortly after they were cast and did damage some of the protruding dowels, this may have been a contributing factor to some of the cracks. It is also possible that a temperature gradient may have existed due to low ambient temperatures combined with cement hydration heat, or that local drying of the top surface may have occurred due to improper curing. At the time of writing of this paper the underside of the bridge has not been excavated to expose the piles but when this occurs the piles will be more thoroughly inspected.

Figure 21 - Surface Cracks in Shafts

WALL DRAINAGE

It is paramount on any bridge that drainage is provided behind the abutments to allow the approach fills to drain properly. The St Albert Bridge is unique in that there is no access behind the pile walls to easily provide conventional weeping tile type drains. On this bridge two rows of small drain tubes will be drilled through the infill piles to relieve water pressures behind the secant pile wall. The tubes will be placed through each infill pile and will be protected from siltation with the use of a filter sock. One row will be installed near the base of the wall and another row will be installed approximately 5m higher. Once the water is through the pile wall it will drain down through free-draining granular fill placed behind the precast finishing panels. A horizontal Miradrain at the bottom then collects the water and channels it transversely out the corners of the abutment and into the drainage swale of Anthony Henday Drive. The drainage system used is shown in Figure 20.

FORMWORK REMOVAL AND ACCESS HATCHES

In order to facilitate future inspections of the interior of the box girder voids permanent access hatches were installed into the girder soffit slabs. Code requirements necessitated the use of intermediate diaphragms for the box girders, therefore to minimize the quantity of access hatches internal steel cross bracing shown in Figure 22 was used which would allow for inspectors to pass through. It was determined however that the pier diaphragm and abutment diaphragms should be solid concrete, thus one access hatch per span per girder was needed. These were placed as close as possible to the abutment diaphragm to allow for the easiest access not over the traveled lanes of Anthony Henday Drive.

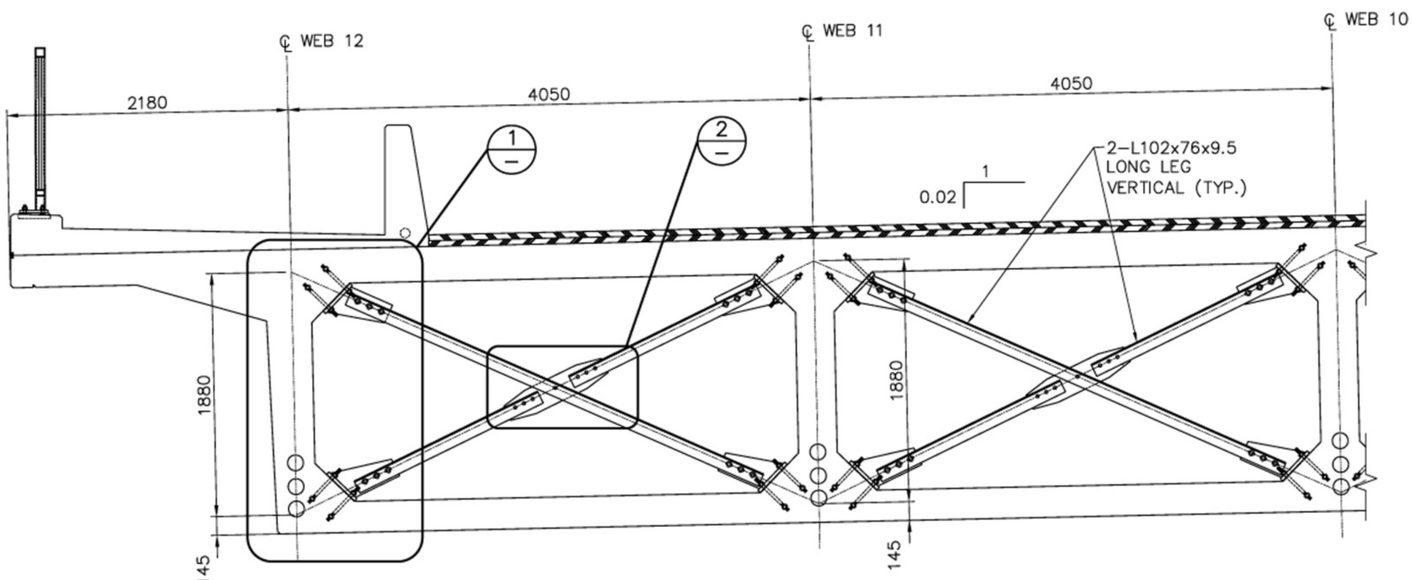

Figure 22 - Superstructure Cross-Section

Another requirement was for the removal of the formwork after casting the top deck slab. It was considered to use temporary access holes in the deck at each girder which would be filled in later. This option was deemed undesirable by the client due to concerns of leakage and deterioration of the cold joints in the future. To avoid this concern the deck was poured up to a line approximately 3.5m from the inside face of the abutment diaphragm, the majority of the formwork was removed, then the last strip was formed and cast. Later, the small amount of formwork and falsework from the last pour will be removed through the soffit access hatches. It was surmised that the single transverse cold joint across the full width of the bridge at each end should be held closed by the girder post-tensioning and should not be prone to future leaking and subsequent deterioration.

CONCLUSIONS AND RECOMMENDATIONS

As a result of the specific design and project delivery initiatives undertaken, the construction cost estimate for the St Albert Bridge was reduced substantially and is now in line with the budget.

This paper illustrates a case study of one of the constructability concepts related to the construction of the St Albert Bridge. It demonstrates that both designers and contractors can enhance constructability by maintaining evaluation, documentation, and feedback regarding the issues of constructability throughout the project to use in later projects. Many lessons learned that could benefit similar large bridge projects were explored in this paper.

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