Constructability of the North Saskatchewan River Bridge

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Abstract

The North Saskatchewan River Bridge (NSRB) is an important river crossing on Alberta's North South Trade Corridor. It forms part of the South West leg of the Anthony Henday Drive Ring Road around Edmonton. This paper examines the challenges faced and lessons learned relating to slope stability, foundations, girder design and erection, and environmental issues, during the construction of the NSRB. Scheduled for completion in 2005, these twin four span (80-100-100-80 meters) steel girder structures are the only new bridges of this type and length constructed in Alberta in recent years. Design standards and construction methods have changed since the last similar bridge was completed.

Design procedures and construction techniques used on this bridge, that are new for the Edmonton area and in addition to the effect of large bridge size on constructability, include tangent pile walls, caisson casings, large concrete pours, and girder erections. Challenges encountered on this project include approach fills that are prone to sliding, the effects of bridge curvature on final girder camber, and the stability of girders during erection. Several other challenges and lessons learnt are also covered in the paper. The writers conclude that constructability input can still affect project budget and schedule during the construction phase. Finally, recommendations and strategies for future projects are presented.

Introduction

In the last two decades, the construction industry suffered from the lack of constructability implementation. This lack caused many problems, such as increased cost and time required for constructing a project, reduced productivity of project personnel and equipment, and low quality construction [6]. Because of the size and complexity of projects and the fragmentation of the construction field into specialized roles and expertise, the construction industry has a great need to implement constructability. Researchers in developed countries, mainly in the United States, the United Kingdom, and Australia, realized the seriousness of this shortfall and suggested solutions to resolve it. In the United Kingdom, the Construction Industry Research Information Association (CIRIA) identified seven concepts. Those concepts were then broadened by CIRIA into 16 concepts. During this period, parallel but unrelated studies were undertaken in the United States. Based on the research of Tatum et al. in 1986 and O'Connor et al. in 1986, 14 concepts in its "Constructability Concepts file" were presented, followed by the research of O'Connor and Davis in 1988 who detailed them. Further elaboration in 1993, by Construction Industry Institute (CII), of these 14 concepts later resulted in the development of 17 concepts. [8], [9]. The Australian Construction Industry Institute, meanwhile, developed 12 concepts in 1993 [4]. Twenty-Three constructability concepts in the Malaysian Construction Industry have been researched by Nima in 2001 [5].

One of the major constructability concepts is maintaining evaluation, documentation, and feedback regarding the issues of the constructability throughout the project, to be used in later projects, as lessons learned [7]. This paper is written in support of this concept. It

comprises a case study and presents and discusses constructability lessons learned during the construction of the North Saskatchewan River Bridge. In the case study, the authors investigate a number of elements found at the bridge, including tangent pile walls, caisson casings, large concrete pours, girder erections, and the effects a large bridge size can have on constructability. With a bearing-to-bearing length of 360 metres, the NSRB is larger than most standard bridges, and the large size of most of the bridge elements caused additional constraints to what is normally encountered on smaller bridges. It is advisable for designers and constructors to be aware of some of these effects, as discussed in this paper.

Project Description and Background

The project selected for this case study is the North Saskatchewan River Bridge (NSRB) on Anthony Henday Drive in Edmonton, Alberta. The NSRB is an important river crossing on Alberta's North South Trade Corridor, and forms part of the South West leg of the Anthony Henday Drive Ring Road around the City of Edmonton.

The Anthony Henday roadway has been part of the City of Edmonton's and Province of Alberta's transportation and utility planning for approximately 30 years, and is an important link in the provincial North-South Trade Corridor. Upon completion, Anthony Henday Drive and the NSRB will have far reaching benefits to Edmontonians and thousands of travelers who will use this route for leisure, business and goods movement. **Figure 1** shows the location of the bridge in the Southwest corner of the City of Edmonton.

The North Saskatchewan River Bridge is a parallel, twin bridge crossing the North Saskatchewan River on Anthony Henday Drive. The span configuration is 80-100-100-80 meters. Each bridge deck carries two lanes and is 12.9 m wide. The bridges are on a 3500 m radius. A 4.0 m wide pedestrian bridge is suspended below the South Bound Lane. Alberta Infrastructure and Transportation commissioned two alternative designs, one with a steel plate girder superstructure and the other with a segmental precast concrete superstructure. Both designs were competitively tendered, with the steel plate girder option forming the low bid. The steel plate girder option was ultimately chosen for construction

The 4 lines per bridge of steel plate girders are 3.6 m deep. Each pier consists of two shafts and a capping beam and was constructed using variable width caissons (3.0 m and 4.6 m diameters). Adherence to a tight schedule, due to stringent environmental requirements for in stream work, was necessary. Based on the geotechnical findings, the North Abutment fills were expected to shift approximately 250 mm, either laterally or towards the river. To accommodate this, tangent pile walls were installed at the bases of head slopes, and the north abutments were designed with floating roof slabs that allowed the abutments to move with the fills without affecting the horizontal alignment of the roof slabs. The superstructure and bearings were designed so that the girder alignment could be adjusted as the movements progressed.

Head Slope Stability: Tangent Pile Walls

Background

Preliminary investigations of the subsurface soil conditions uncovered a slide-prone layer of soil sandwiched between the clay-shale bedrock and the overlying gravel layer. The slip plane was found to slope toward the river beneath both banks, and it was feared that the additional weight of the large approach fills would cause a slide beneath the approaches, as shown in **Figure 2**. To prevent the approaches from sliding, large tangent pile walls, which are shown in **Figure 3**, were constructed beneath the toe of both approach fills, effectively anchoring the overlying clay till and gravel into the bedrock below the slip plane.

The tangent pile walls constructed on the North and South banks of the river consisted of 88 and 49 piles respectively. Each pile was 1.5 m in diameter and between 16 m and 25 m in length. **Figure 4** shows a sketch of a typical tangent pile installation. This project represents one of the few instances in Alberta where large tangent pile walls are used to prevent potential slope instabilities prior to the occurrence of any failure. The tangent pile walls constructed in this project were designed assuming that the full strength of the underlying bedrock would be mobilized during a slide. There was concern that fractures or weakened layers in the bedrock would lead to a reduced residual strength in the bedrock, causing a significant increase in the load applied to the wall. Slope inclinometers for monitoring movement in both the piles and the adjoining slopes were installed to determine if the piles are adequate as constructed. **Figure 5** shows the layout of the instrumentation beneath the South approach, with slope inclinometers labelled SI-2, SI-8A, and SI-6. SI-6 is one of the slope inclinometers that were installed inside one of the piles. . If excessive slope movements, or excessive pile stresses are encountered, a contingency plan exists for the addition of another row of tangent piles on either bank.

The two walls were constructed at the same time by two different contractors, providing an excellent opportunity to compare the effectiveness of different construction methods and strategies. At the time of writing this paper, the walls have been complete and in service for about three years, and have so far performed as expected. The measured deflections in both the slopes and the piles are in line with what was anticipated. **Figure 6** shows the measured deflections of a pile in two directions. At present, analysis of displacement data from the slope inclinometers indicates that piles are currently resisting approximately 85% of their design moment resistance, and approximately 35% of their shear resistance design. To date there is no indication that a second row of piles will be required to restrain the slope on either bank.

Casing

A waterborne gravel layer, deposited by the river on a previous channel alignment, exists about 8 m below the ground surface on either bank of the river. This layer is hydraulically continuous with the existing river channel, and provides an easy path for the water to flow into any holes that are drilled through it. To maintain a dry condition and seal the drilled hole, a removable steel casing was driven through the gravel layer and into the underlying

bedrock. The two contractors employed markedly different strategies for dealing with the potential groundwater problem. One contractor assumed that every hole would have to be cased, and arrived on site prepared to do so. The other contractor, noting that not all test holes indicated ground water, gambled and assumed that only a small fraction of the holes would require casing, and brought only the bare minimum of equipment required to do so. In the end, the gamble did not pay off, as significant groundwater was encountered in about 60% of the second contractor's holes. Comparing the methods and results of the two contractors yields the following lessons:

- When drilling piles in close proximity to a river with a gravel bed, it should be assumed that all holes will have to be cased. Over geological time, river alignments can change quite dramatically, depositing a gravel bed over an area considerably wider than the existing channel. This porous gravel layer may remain hydraulically continuous with the existing riverbed, potentially leading to a high flow of water through the gravel layer. When a pile is drilled into the ground in close proximity to a river, there is a good chance that the hole will pass through this gravel layer and be flooded by river water, unless it is properly sealed. A watertight casing, well embedded into the soil layer underlying the gravel layer, is the only way to prevent river water from flowing into the drilled hole.

- Having two or more reusable casings of varying lengths available on site will allow the contractor to quickly adjust to varying sub-soil conditions. Soils investigations can provide the approximate depth and thickness of water carrying layers, but the actual depth and thickness of these layers across the construction site can vary, requiring different lengths of casing. Having two or more sections of casing of different lengths available on site can drastically improve efficiency by eliminating the need for cutting and welding a single casing section. The length of the casing sections brought to site should be based on a careful examination of the soils testing data.

- The largest suitable vibrating hammer capable of driving the required sections of casing should be used. Once the hole has been drilled to the depth of the wet layer, a vibrating hammer should be used to drive the casing through the wet layer and well into the underlying impermeable layer to provide an effective seal. An undersized vibrating hammer may not be capable of driving the casing deep enough into the impermeable layer, and considerable extra effort and time will be expended in an attempt to seal the hole. Using the end of the soil auger, instead of a suitable vibrating hammer, to ram the casing through the wet layer and into the underlying soil, is not effective in creating a seal, and may cause the casing to shift out of plumb.

- Having casing as a separate pay item in the tender documents can alert a contractor to the requirements for casing, and eliminate much of the contractor risk often associated with this item. A contractor may be more willing to employ casing if he knows that the cost to do so can be recovered. In the case of these two contracts, the contractors were paid a unit price for each metre of casing required below ground. This was easily calculated by measuring the length of the casing before it was inserted into the hole, and then measuring the length of the casing remaining above ground level before it was removed.

Lifting Rebar Cages

All of the cylindrical rebar cages used on these two projects were fabricated off site, and shipped, horizontally on their sides, to the site. The cages were up to 18 m long and were confined by a continuous spiral tie. As per Alberta Infrastructure and Transportation (AIT) guidelines, all bar intersections were tied with steel tie wire. No welding was permitted on the structural bars. Lifting these long, flexible cages from their horizontal position on the ground, to a vertical position in the holes presented an interesting challenge to the contractor. After some trial and error on the part of both contractors, the following lessons were learned about what worked and what did not work.

- The best results were achieved by lifting the cage from three points along its length using two cranes. The larger of the two cranes would lift the cage from its end by attaching to three or four points around the circumference of the end of the cage. The smaller of the two cranes would attach at the third points along the length of the cage. Both ends of a single cable draped over a pulley would be attached at the third points. The crane would lift at the pulley, ensuring that an equal force was applied to both third points and that the cage remained straight during lifting. If the forces applied to the third points were not kept relatively equal, the cage would bend, making it impossible to lower it into the hole.

- The contractor should be able to detach the crane from the end of the rebar cage without having to climb down into the hole or up the cage to untie the slings. One contractor used a sling system, with four separate straps that could each clip on to the bar ends at the end of the rebar cage. Once the rebar cage was in place and suspended in the concrete, the crane was slacked off, allowing the clips to be easily removed with the end of a shovel or a crow bar. No one ever had to climb into the hole to untie the slings, and no slings ever became tangled in the rebar and had to be cut off. The clips were simply thick steel plates with the strap connected to one end and a hole, slightly larger than the reinforcing bar diameter, on the other end.

Self-Compacting Concrete

Self-Compacting Concrete (SCC) is concrete that can be compacted into every corner of the formwork by means of its self-weight only. SCC was first developed in 1988 in Japan. Since then, various investigations have been carried out and this type of concrete has been used in practical structures in Japan, mainly by large companies [10]. Investigations have been carried out from the viewpoint of making it a standard concrete and recommendations and manuals for SCC have also been established [11]. Japan has used SCC in bridge, building and tunnel construction since the early 1990's. In the last five years, a number of SCC bridges have been constructed in Europe. In the United States, the application of SCC in highway bridge construction is very limited. However, the United States precast concrete industry is beginning to apply the technology to architectural concrete. SCC has high potential for wider structural applications in highway bridge construction [12].

In NSRB, SCC was used to eliminate the need to pump or consolidate the pile concrete. The specifications for the pile concrete were as follows: Minimum Compressive Strength

at 28 Days: 25 MPa, Size of Coarse Aggregate: 28 mm to 5 mm, Range of Slump: 150 mm to 175 mm. In contrast, Alberta Infrastructure and Transportation's standard pile concrete specifications call for a slump range between 50 and 70 mm.

The concrete could be dumped directly from the truck chute into the hole. A special trunk hose or other means had to be used to guide the concrete so that it fell straight down the middle of the reinforcing cage. To prevent segregation, care had to be taken to ensure that the concrete did not hit any support bars as it was falling to the base of the pile.

Safety Railing

One contractor had a simple safety railing that it put around all open holes during the drilling operations. The railing was fabricated from one-inch Hollow Steel Section (HSS) sections with two sides intersecting at 90 degrees and three posts. Anyone who was not secured with a safety harness was required to stand outside the railing on the opposite side from the hole that was being drilled. The railing was very simple, very inexpensive, and very effective in preventing fall injuries and increasing the comfort level of the labourers required to work around the holes. The other contractor did not employ any similar safety equipment, and the difference in comfort level and perceived safety of the site was noticeable.

Head Slope Stability: Abutments

The abutments consist of an abutment seat linked to two grade beams by a wing wall and roof slabs. A sketch of a typical abutment and the location of the tangent pile walls is shown in **Figure 7**. The geotechnical design predicted a maximum 250 mm long-term movement for the North head-slope, either laterally or towards the river or a combination of both. The Designers overcame this by providing a solution whereby the abutment girder diaphragm of the centre bay was strengthened and attached to restrainers on each abutment. The system relies on bridge inspectors to check if the abutments are moving. Should the abutments move, it allows adjustments of 50 mm increments at a time. The adjustment process involves jacking the girders to the correct alignment and shifting shims to keep the girders aligned. The contractor had to plan his construction procedure carefully to ensure that this portion of the project was understood and installed correctly.

Piers and Pier Foundations

Background

A sketch of the NSRB piers is presented in **Figure 8** and a photo during the construction of Pier 3 is shown in **Figure 9**. Each pier consists of two pier shafts joined at the top by a pier cap. The pier foundations consist of a 3.0 metre diameter caisson, embedded in bedrock for the bottom 16 to 20 metres of the foundations. A 4.6 metre diameter caisson, placed on the top of the 3.0 metre caisson, extends to the foundation to ground level. Oval shaped pier shafts are tied into the 4.6 metre diameter caisson with steel reinforcing dowels. The contractor's construction sequence consisted of pouring the caisson concrete

to final elevations and then setting the dowels in the wet concrete using a template as shown in **Figure 10**. Pier shafts were then poured in four segments each, followed by the pier cap and cross beam (SBL only).

Caisson Grooves

According to the design, the sides of the 3.0 metre diameter section of the caissons required 75 mm by 75 mm grooves, spaced 1.0 metre on centre to increase the shaft friction resistance. The grooving added to the complexity of the project and, at one stage, to ease the construction, the contractor considered increasing the shaft diameter in lieu of grooving the sides of the caisson. The extra concrete required for this modification would have increased the cost dramatically. Alternatively, the contractor decided to fabricate grooving tools that were designed to extend outwards to cut grooves if spun one way, and to retract if spun in the opposite direction. This tool was placed at the end of the auger shaft and was used to groove the holes as per the contract drawings.

Caissons Pouring Sequence

The contractor's proposed construction program for caissons was based on completing the concreting for a caisson, and immediately commencing with the placement of steel liner and excavating the next caisson hole. Vibrations from the steel casing driver for the caissons were substantial and raised concerns that micro cracking would occur in freshly poured concrete during their initial set in adjacent caissons. This was overcome by re-scheduling time off after pours or skipping adjacent caissons and returning to them at a later stage.

Concrete Hydration Heat in Mass Concrete

All concretes generate heat as the cementitious materials hydrate. Most of this heat generation occurs in the first days after placement. For thin items such as bridge decks, heat dissipates almost as quickly as it is generated. For thicker concrete, heat dissipates more slowly than it is generated. The net result is that mass concrete can get hot. It is necessary to manage these temperatures to prevent damage, minimize delays, and meet project specifications. Specifications generally limit temperatures in mass concrete to prevent cracking and durability problems [3].

According to ACI [1], mass concrete is defined as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change, to minimize cracking". Because this definition doesn't provide a specific measure, many agencies have developed their own definitions of mass concrete. For example, mass concrete is defined by many agencies in the United States as "any concrete element having a least dimension greater than 3 ft" [3]. In NSRB, the pier cap cross section is approximately 4.3 m by 4.0 m, which is considered mass concrete by any definition. The total pour was about 400 m^3, and during cold weather the internal concrete temperatures were monitored with thermo couples to check if thermal stresses were developing within the section due to the heat of hydration. The contractor generally monitored critical sections until the concrete

temperature was the same as the ambient temperature. During winter, it was found that the heat of hydration helped to maintain favourable temperatures in the curing concrete. The Class C concrete, using Type 10 cement, specified the inclusion of Fly Ash, which generally has a low heat of hydration. No cracking due to thermal stresses was observed.

Maximum allowable concrete temperatures and temperature differences are often specified to ensure that proper planning occurs prior to concrete placement. The CSA Standard A23.01-04 in §8.5.5 recommends that the internal temperature never exceeds 70°C and in §7.4.2.3 recommends that the difference in temperature between the inside and outside of the section is to be kept less than 20°C to prevent thermal stresses across the section [2]. This is a common concern when pouring mass concrete. As the size of mass concrete is not clearly defined, the engineer and contractor might disagree on the measures required to control concrete temperature. The authors recommend that the requirements for thermal monitoring be included in the contract Special Provisions when mass concrete is specified.

Caisson Dowels

Caisson concrete pours took 14 to 20 hours, depending on the concrete supply. On one occasion, the day the Pier 1 SBL West Caisson pour took place, it was hot, decreasing the concrete set time. The problem was exacerbated by slow concrete delivery. By the time the contractor was ready to position the dowel cage for the pier shaft, the concrete initial set had occurred and it was impossible to insert the dowel cages. Rather than remove the top 3.5 metres of concrete, the contractor elected to leave the concrete in place and explore methods of inserting the dowels at a later stage.

A number of methods were evaluated. The dowels were closely spaced (35M @ 91 mm) and, in the original design, embedded 3.5 metres into the caisson. The contractor focused on trying to increase the dowel spacing and reducing the embedment length. A number of high strength bars were considered but there were concerns that the increased strain on the bars, when required to reach their capacity, would result in transverse cracking at the base of the column, potentially allowing water ingress and corrosion. The use of an epoxy system was proposed, which required a nominal bond length, and it appeared that the embedment could be shortened; however, embedding all the dowels at a shallower depth raised concerns that a block pullout could occur.

The selected alternative consisted of drilling and epoxying dowels into the caissons to an optimized depth where a block pullout was unlikely to occur. The procedure took about two weeks and pull out tests confirmed the dowel capacity.

Pier Shafts

With 35M bars spaced at 91 mm on centre, the pier shaft reinforcement was congested, especially in areas of longitudinal lapping. The construction drawings showed lapped bars being "cranked" to allow lapping to the inside of the shafts. The final rebar cage was lapped to the sides of the starter bars, and care had to be taken during concrete placement to ensure that no honeycombing occured around the bars.

Bridge Girders

Background

The girders are 3.6 metres deep, with segment lengths ranging from 25 m to 40 m. The bending capacity of the girders about the horizontal axis is substantially higher than that about the vertical axis. As a result, there were concerns about the lateral stability of the girder before it was braced.

Girders Erection and Collapse

Girders were scheduled to be erected over winter. The erection firm proposed a method whereby the girders were stabilized with cables tied off to lock blocks on the ground and 2 cranes. This was somewhat unconventional compared to existing practice in Alberta, as no temporary erection towers were used, and there were concerns about the final girder profile. The girders were held in place by a system of lock blocks connected by wire cables to the boltholes in girders. The lock blocks were used to resist wind loads and maintain girder stability.

On 14 December 2003, the first girder line on span 1 of the SBL collapsed as shown in **Figure 11**. This occurred after all girders for the NBL span 1 had been erected successfully. It appeared that the girder rotated counter to the curvature of the bridge and collapsed due to bending failure about its minor axis. The cause of the collapse is still being investigated by the erection firm's insurer, and has not yet been revealed.

It appears that the proposed erection method, while stable, was new to the erection firm's ironworkers. This unfamiliarity could have led to a situation where changes made during the erection procedure were not fully evaluated, nor were the implications thereof understood.

Based on this experience, the authors suggest that contractors budget for designers of temporary works to visit the site to inspect initial work and to confirm the erection team's understanding of the proposed system and its limitations. The collapse resulted in a 6-week delay to an already tight construction program and substantial additional costs.

Girder Delivery

The girder design allowed for a maximum section length of 42 metres. The size limit was set to allow girder fabricators to handle the sections in their yards and to facilitate shipping. The sub contractor supplying the girders was from Quebec and the girders were shipped to site by rail, with the journey taking between one and two weeks.

Girder Splices

The road over the bridge is on a 3500 m radius. The girders were designed to be straight, inscribed chords of the horizontal curve, with each splice location positioned on the curve. The small (300 mm) eccentricity of the girders, at mid-spans due to the horizontal curve,

required extra consideration to ensure the first girder erected remained stable until adjacent girders were braced to it.

Girder splices were large, using bolts ranging from 428 to 568 bolts per splice. The erection sub contractor had to ensure that splices were completed, while ensuring that the ironworkers remained warm during cold days. Small rotations at girder splices could result in large deflections at the ends of girders.

Grouting of Bearings

For each 1°C change in temperature, the girder length measured from the centre pier (pier 2) to the abutment changes by approximately 2.2 mm. During the phase when the girders were in place, but not grouted, the contractor was concerned that the constant thermal movements of the girders, over time, would cause a ratcheting effect on the supporting shims, eventually resulting in the girder falling of the shims. To prevent this, he requested permission to grout them prior to erecting the complete line of girders

Survey Corrections

Due to the curvature of the earth, no single projection can cover the entire planet and still be a viable source for comparing vector data. As a result, vector data is often received in varying projections, which may be converted to a common projection for use. The Alberta Provincial Digital data is stored in either 3-degree Modified Transverse Mercator (3TM) or Universal Transverse Mercator (UTM) projections. The 3TM projection is used to represent data within three degrees of longitude zone with the 114th degree as a centre.

At the NSRB, the field benchmarks were laid out using the 3TM Grid coordinate system. In order to convert a distance based on a mapping projection to a horizontal distance measured on the ground a Combined Scale Factor (CSF) has to be used. The CSF is determined by the latitude and elevation of the area in question. Since these factors will change over a large site, an average combined scale factor is normally selected for the entire project. In NSRB, 0.999803 was selected as a CSF. This was not applied to the girder lengths, resulting in the girders being 71 mm shorter than they were supposed to be.

The authors suggest that the use of the CSF should be considered by designers and mentioned in AIT Bridge Structure Design Criteria and other similar design guidelines.

Cast In Place Concrete Deck

The concrete Class SF deck is 12.9 m wide and 245 mm deep. The contractor's main source of concrete supply for all spans was from the North end of the bridge. The design required the pours on the 4 spans to precede the pours above the 3 piers.

Initially, the contractor elected to set up a concrete pump truck below span 1 and use the boom to pump the concrete up onto the deck and into a straight 150 mm diameter pipe, to the location of the deck pour. The pipe was cleaned and lubricated prior to commencing the pour. The friction losses related to pumping from below the bridge, through the boom

and into the 200 m long 150 mm diameter delivery pipe, resulted in the concrete over heating and a zero slump cone at the pour location was obtained. The contractor wisely elected to review the planned arrangement with the concrete pump supplier and rescheduled the pour. For subsequent pours, the concrete pump truck was set up on the abutment, eliminating the need for the boom, and pumped directly into the straight 150 mm diameter pipe. A lesson learned here is to balance mix design, pump capacity, and delivery pipe diameter to achieve the required slump before pouring concrete in order to avoid interruptions and delays.

Preliminary concrete testing (slump, temperature & air content) was conducted at the concrete pump and final testing took place at the end of the concrete hose. In general, slump and air contents decreased from the pump to point of discharge. Challenges were encountered when substandard concrete was delivered and had to be removed from the bridge deck. The concrete had to be moved manually back to the disposal site on the head slopes.

Another lesson learned here is to schedule the pouring sequence so that workers get easier access to the fresh concrete. It required less effort on the labourers' part once additional sections were poured and one could travel over the completed deck sections instead of the walkway planks.

Cast In Place Roof Slabs

The roof slabs were 13.4 metres wide by 5.0 metres long by 0.45 metres deep. The contractor employed a system of 3 levelling beams spanning from abutment to grade beam, dividing the pour into 4 bays across the width of the road for screeding to the final elevation.

After the first pour it was noted that the slab top surface had settled visibly, by approximately 50 mm, as shown in **Figure 12**. This was not picked up during the pour as the levelling screed rails were removed immediately after the relevant bay was complete.

Initially it was assumed that the formwork had settled, however the contractor found that he had to wait until concrete filled three adjacent bays before he could screed the first bay to its final elevation. The vibrating activities during concrete placement in the adjacent bays caused the completed bays to slump, resulting in additional finishing work; and the distance (more than 6 meters) affected by this vibration was greater than expected. Accordingly, the authors recommend that when using screed rails in large slabs, the rails be kept in place as long as possible to ensure that the concrete does not slump when adjacent bays are poured.

Berm Construction and Removal

The Department of Fisheries and Oceans (DFO) approved construction windows and allowable stream widths to be adhered to during construction. DFO specified that no in stream work could occur between April 1 and August 1 of any given year to allow for fish spawning. The minimum opening width was to be 40% of the river width at all times. With this in mind, the contractor constructed a berm for Pier 3 on the south bank and a separate berm for Pier 2 on the north bank. The Pier 3 berm constricted the river by approximately 10%, and the combination of Pier 2 and Pier 3 berms constricted the river by approximately 60%. Some siltation from the riprap was observed in the river, and a lesson learned by the contractor was to ensure that rock material supplied to armour the berms is clean of deleterious material that can enter the river. Failure to do so may result in the protection works becoming part of the problem, resulting in unnecessary work such as the provision of silt curtains to remedy the situation.

The contract stipulated that the riverbank was to be armoured adjacent to pier 3. The contractor had created a berm to construct Pier 3 which extended 15 metres into the river. The contractor reduced the environmental impact by placing the permanent riverbank riprap in its final position, behind the temporary berm, before removing the temporary berm used to construct Pier 3. A lesson learned here is that the sequence of construction activities plays a major role in optimizing time, minimizing cost, and reducing environmental impact.

Conclusions and Recommendations

This paper illustrates a case study of one of the constructability concepts related to the construction of the NSRB. 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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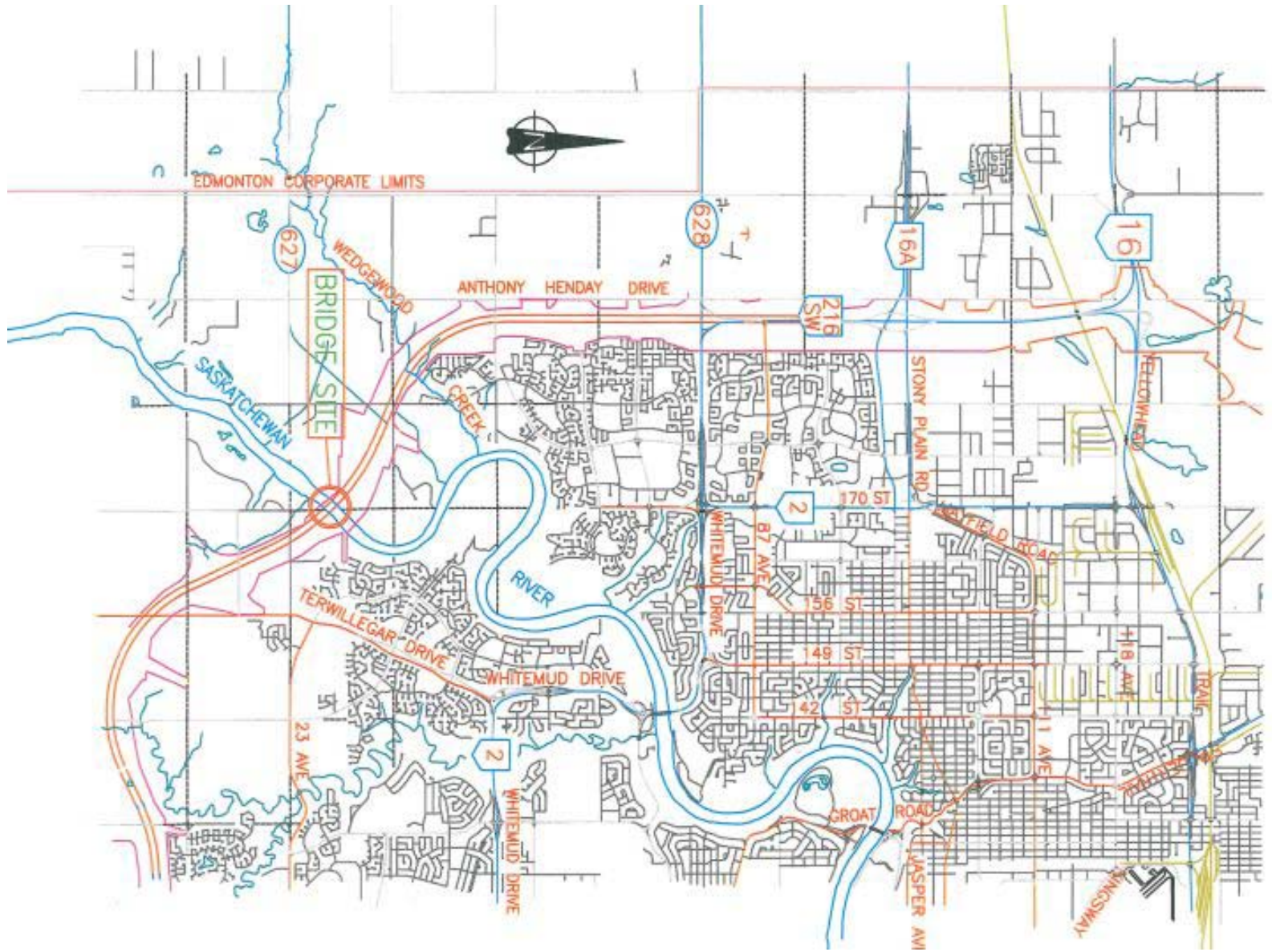
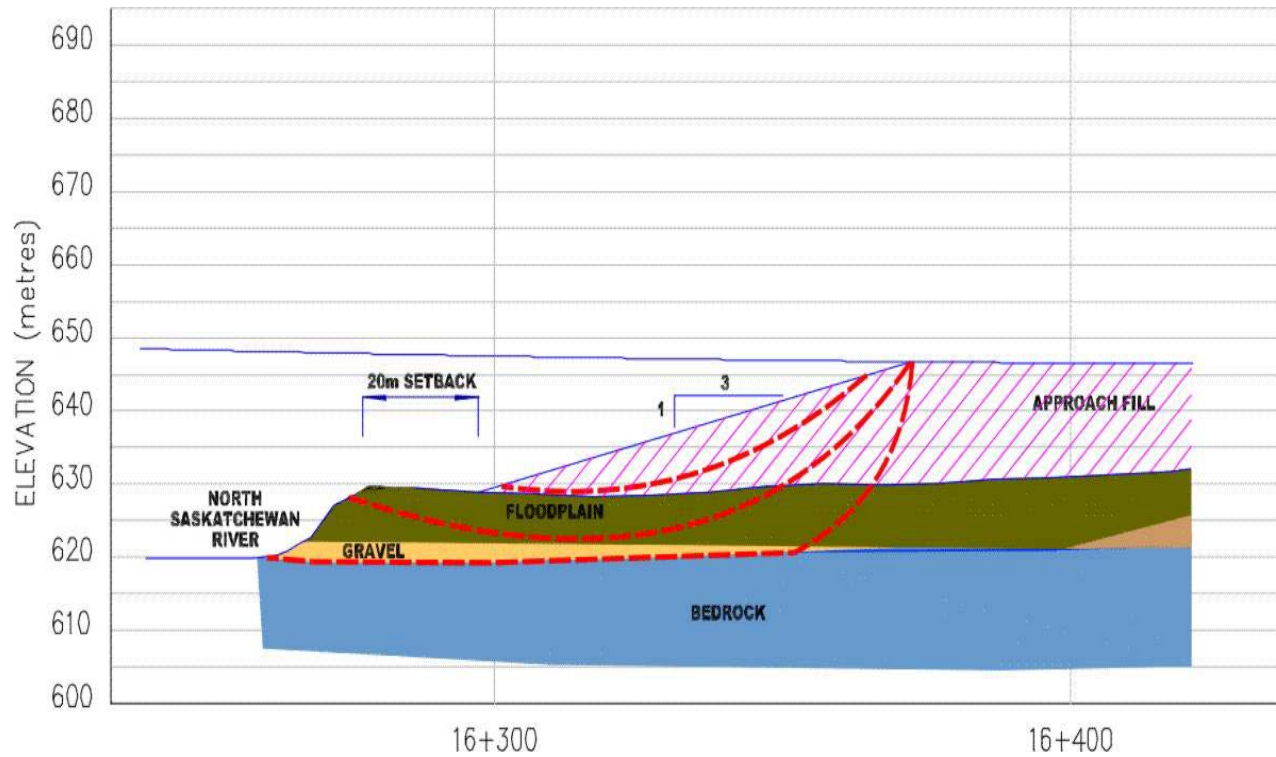Figure 1: Location of NSRB Site

Figure 1

Figure 2: Typical Failure Surfaces at South Approach

Figure 2

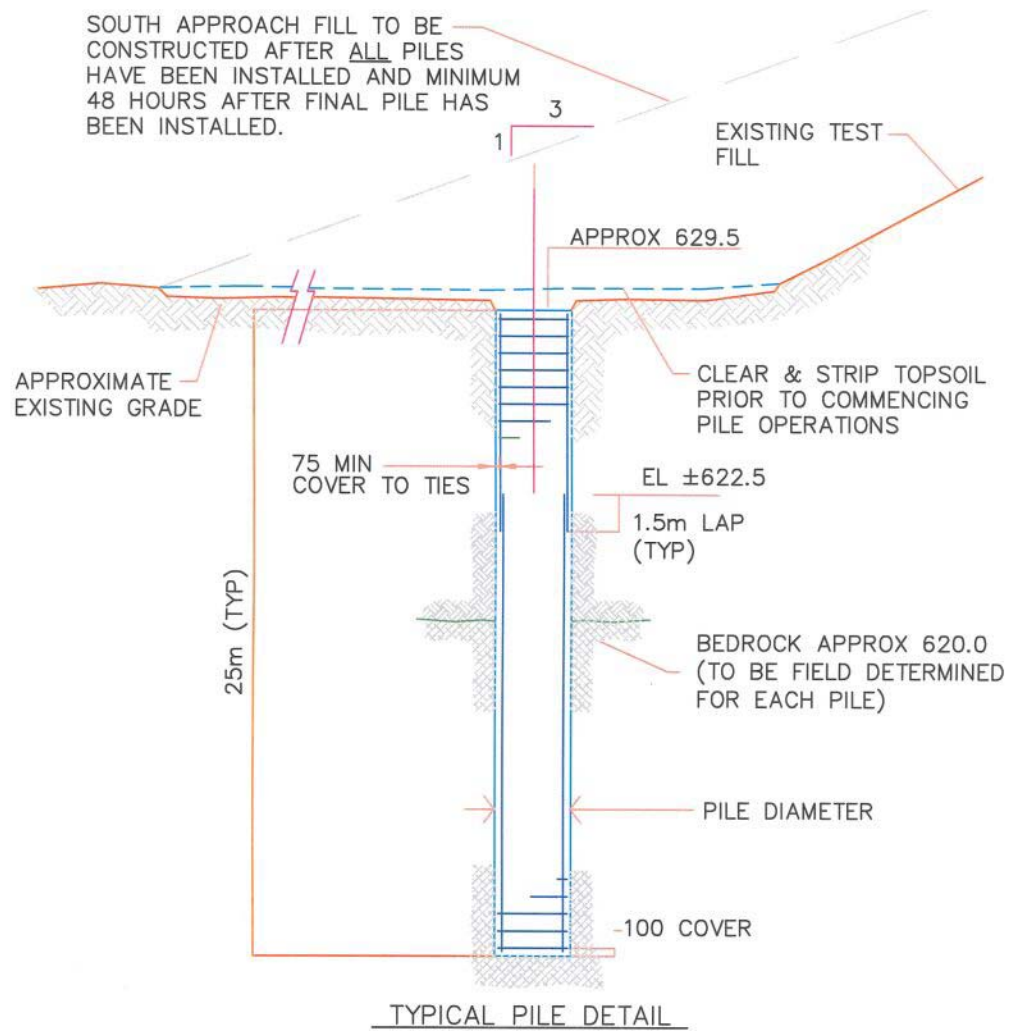Figure 3: SCC is Being Poured into The Tangent Pile

Figure 3

SOUTH APPROACH FILL TO BE CONSTRUCTED AFTER ALL PILES HAVE BEEN INSTALLED AND MINIMUM 48 HOURS AFTER FINAL PILE HAS BEEN INSTALLED.

EXISTING TEST FILL

APPROX 629.5

APPROXIMATE EXISTING GRADE

CLEAR & STRIP TOPSOIL PRIOR TO COMMENCING PILE OPERATIONS

75 MIN COVER TO TIES

EL ±622.5

1.5m LAP (TYP)

25m (TYP)

BEDROCK APPROX 620.0 (TO BE FIELD DETERMINED FOR EACH PILE)

PILE DIAMETER

100 COVER

TYPICAL PILE DETAIL

Figure 4: Typical Tangent Pile Detail

Figure 4
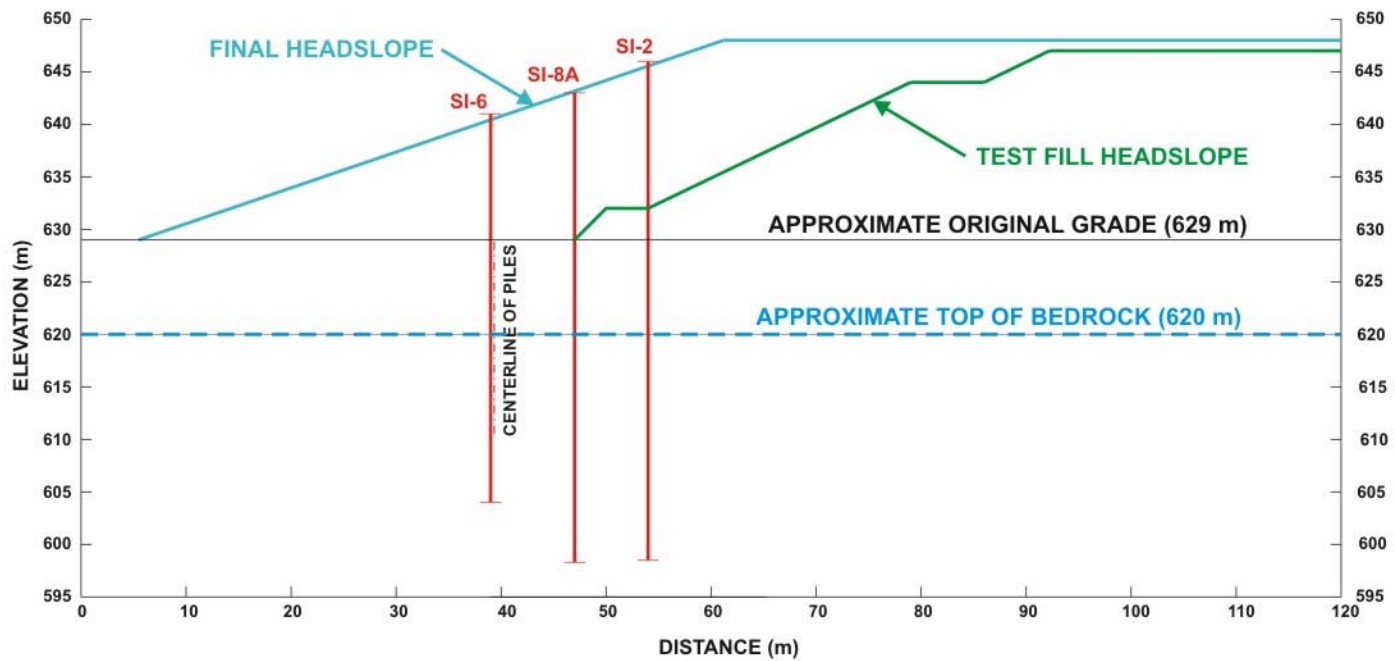
Figure 5: Layout of Slope Inclinometers Beneath the South Headslope

Figure 5

EBA Engineering - Edmonton
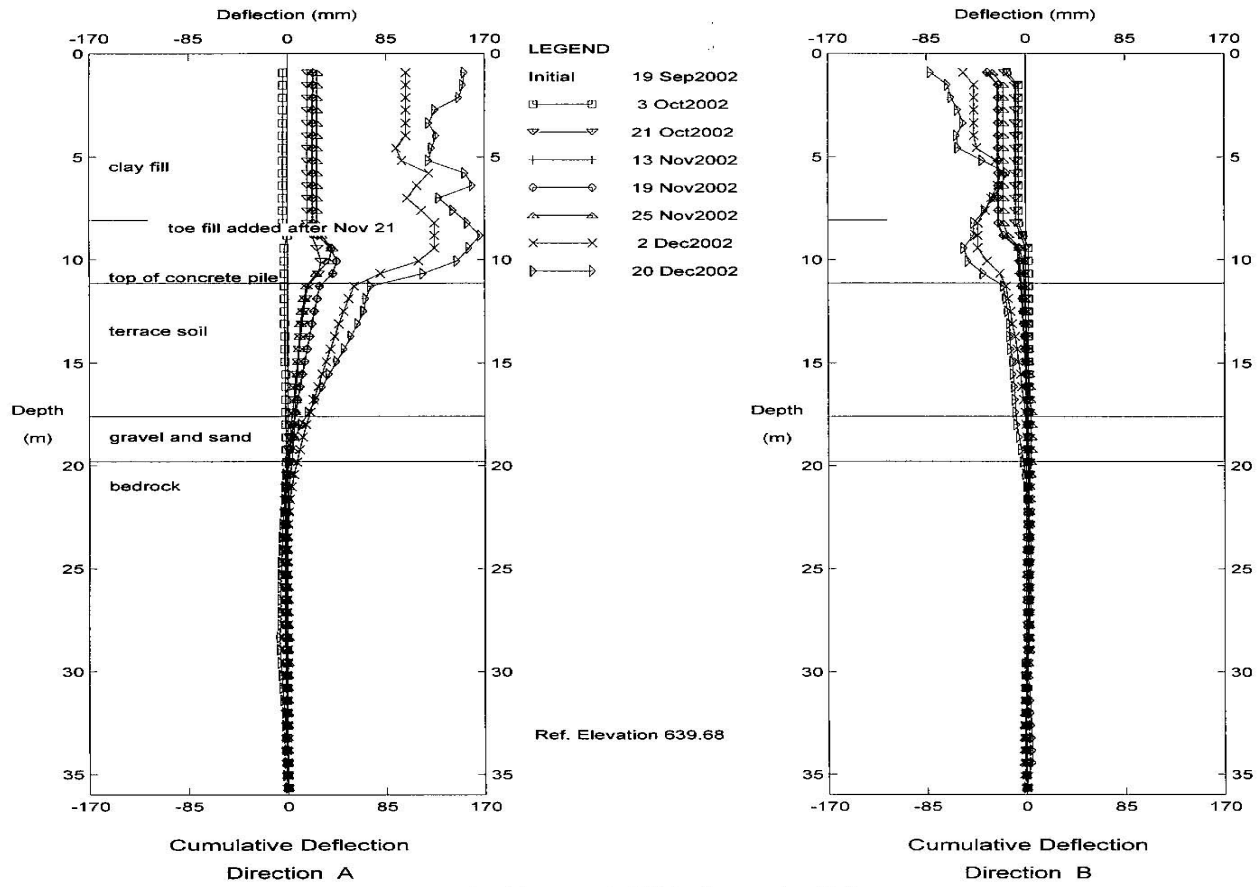
East Approach Fill, Inclinometer SI-6

Alberta Transportation

North Saskatchewan River Bridge - South Bank

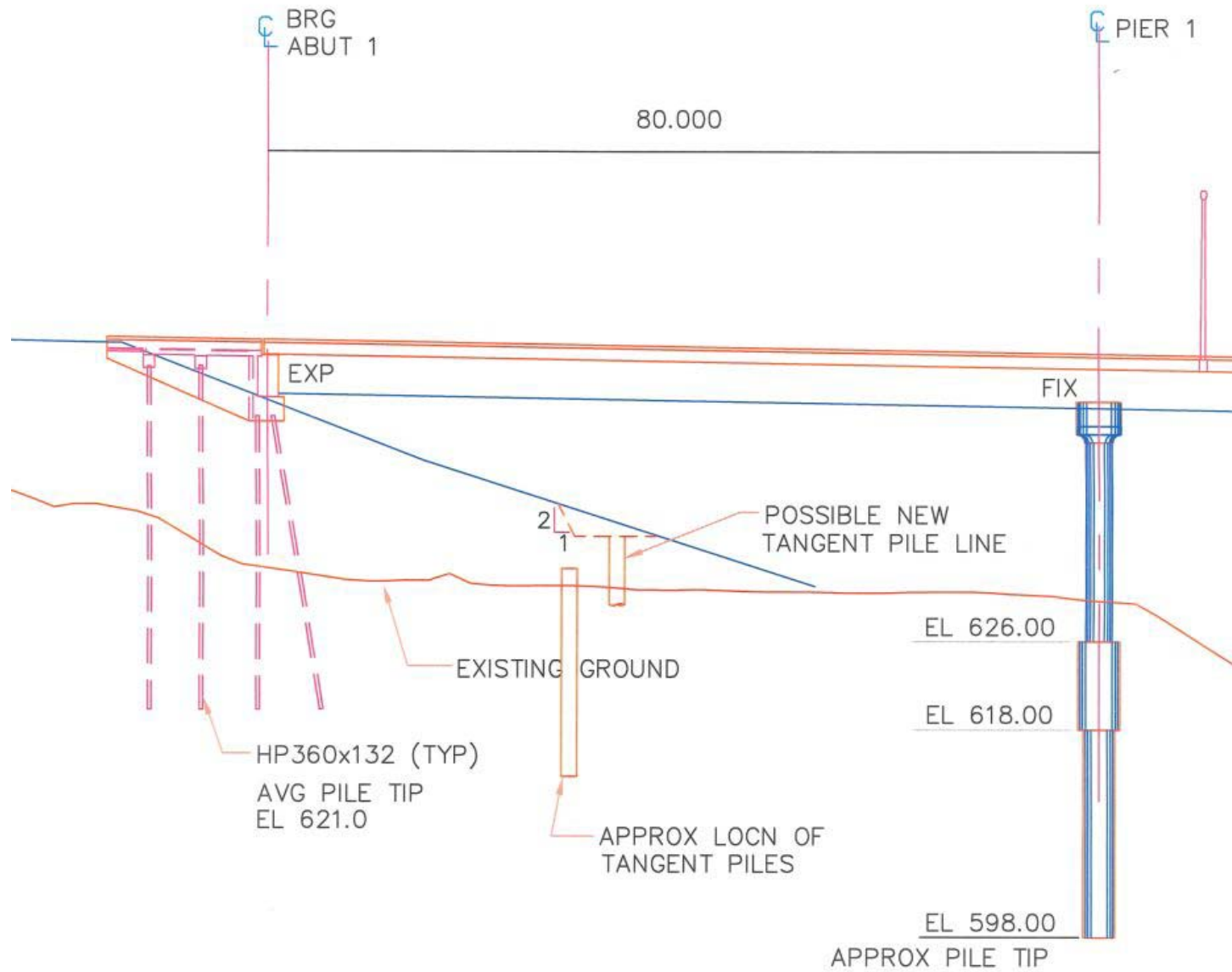Figure 6: Displacements in SI-6 (Jan 20/03 – Mar 19/04)

Figure 6

BRG
ABUT 1

PIER 1

80.000

EXP

FIX

2
1

POSSIBLE NEW
TANGENT PILE LINE

EL 626.00

EXISTING GROUND

EL 618.00

HP360x132 (TYP)
AVG PILE TIP
EL 621.0

APPROX LOCN OF
TANGENT PILES

EL 598.00
APPROX PILE TIP

Figure 7: Typical Abutment and Tangent Piles Location

Figure 7
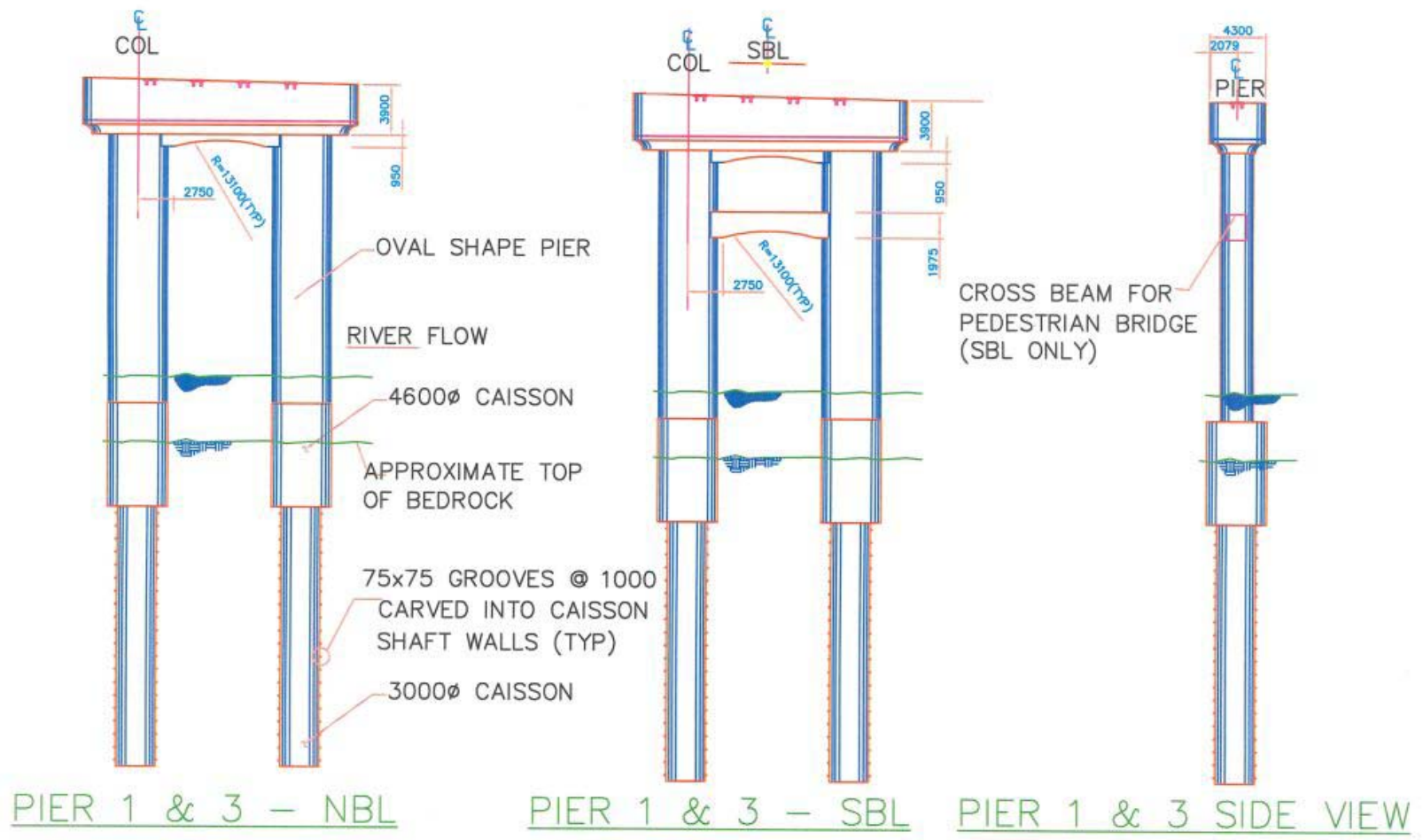
Figure 8: North Saskatchewan River Bridge Piers

Figure 8

Figure 9: NBL Pier Shafts Completed and Preparing to Pour Second
Lift on the SBL Pier Shafts at Pier 3

Figure 9

Figure 10: Dowels are Put in After The Pour When The Concrete
is Still Fresh.
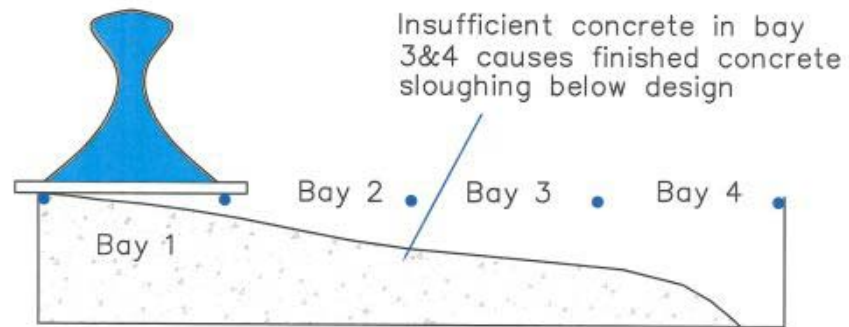
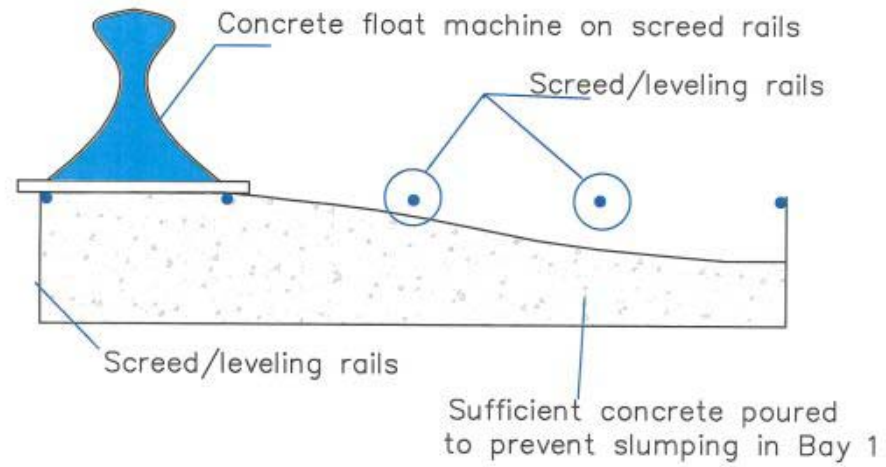Figure 10

Figure 11: Girder Collapse

Figure 11

Figure 12. Pouring Actual and proposed sequence

Figure 12