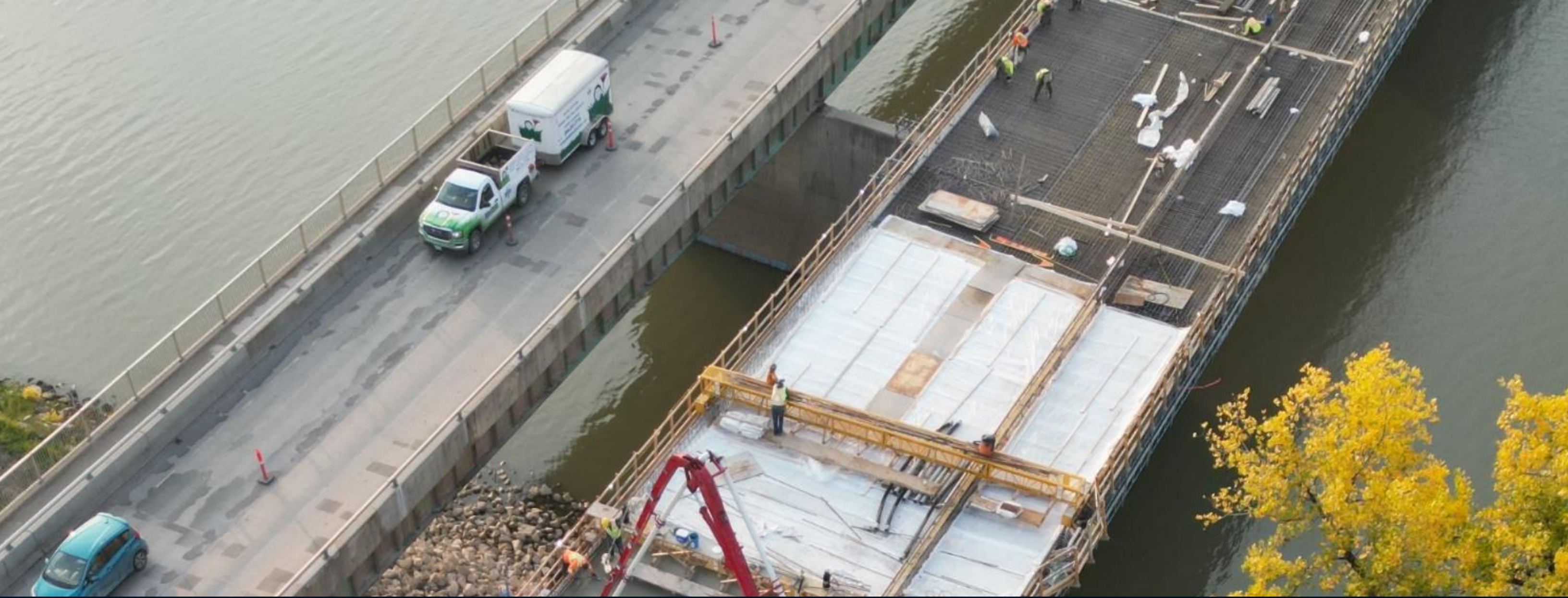

St Vital Bridge Riverbank Pier Stabilization

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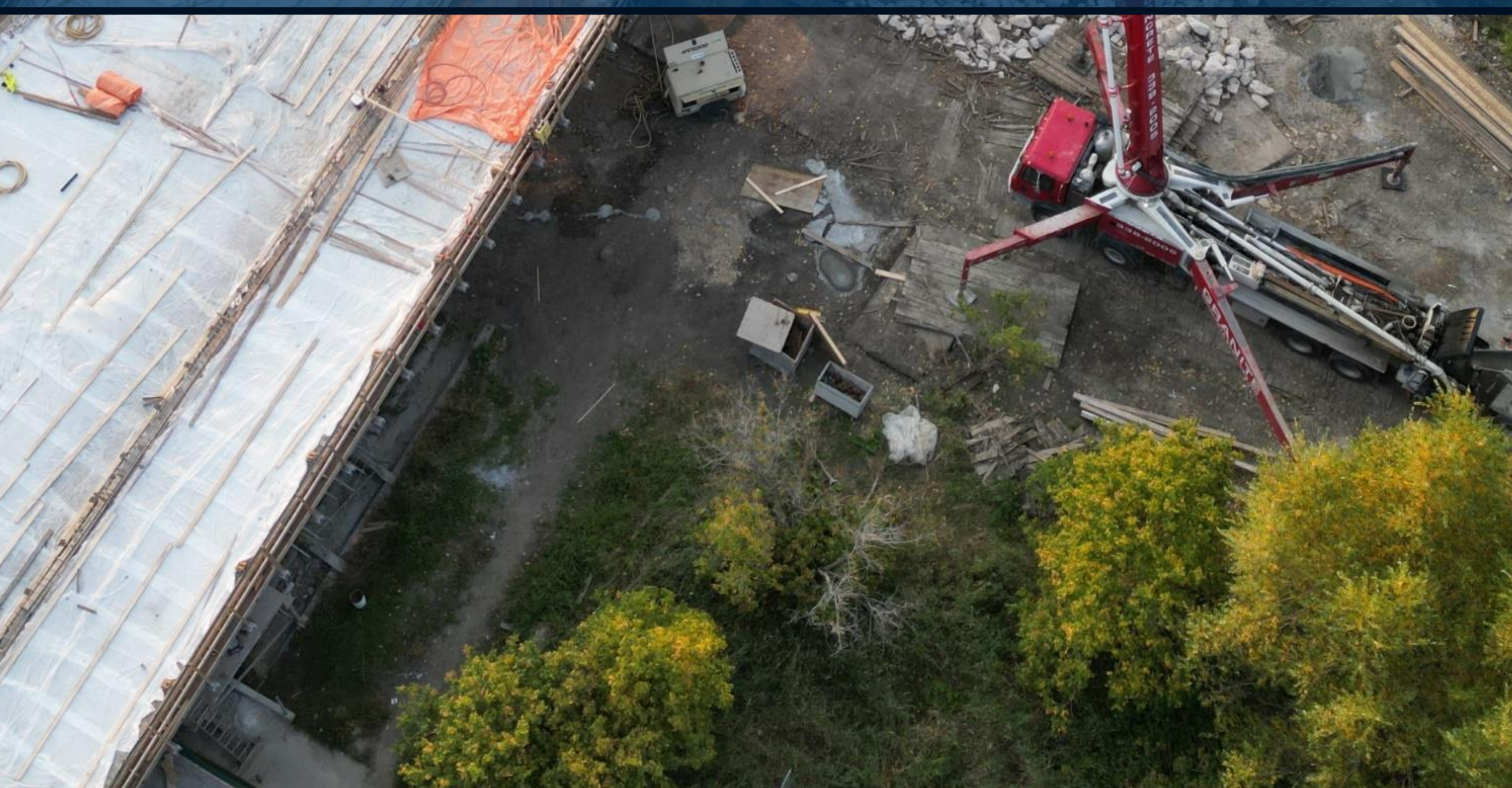


ABSTRACT

The St. Vital Bridge over the Red River in Winnipeg, Manitoba, built in 1964 and rehabilitated in 1988, is a vital link within the City of Winnipeg's transportation network, supporting the conveyance of approximately 44,000 vehicles per day. The nine-span continuous steel plate girder structure is 280 m long and supports two carriageways, each consisting of two traffic lanes and a 1.5 m wide sidewalk. The bridge superstructure is supported by vaulted cast in place abutments founded on piles and solid shaft bridge piers founded on a combination of piles on the riverbanks and spread footings within the river. In 2021, the City of Winnipeg retained Morrison Hershfield Limited and began planning the structure's rehabilitation to extend its service life by 50 years. The current rehabilitation includes the removal, widening and reconstruction of the concrete bridge deck, steel girder strengthening through cover plate installation, bearing replacement, and preservation works. The project also includes associated road works to accommodate widened bridge geometry, Active Transportation improvements, and intersection improvements.

During the initial phase of construction in 2023, a shift in the position of Pier 3, the north riverbank pier, towards the river was identified, prompting a swift response to assess, design, and stabilize the pier while construction of the current rehabilitation was ongoing. This paper details the innovative analysis, design, and construction of the pier stabilization. Rigorous soil-structure interaction analysis, using Midas Civil to develop a three-dimensional analysis model of the pier, was used, which replicated the mode of movement within the pier. Nonlinear soil springs representing the detailed soil stratigraphy at the site were modeled. The stabilization methodology involved offloading the riverbank, underpinning the pier using 28 steel H-piles, which were installed where ongoing bridge construction and the existing pier footprint would allow, and casting a reinforced concrete encasement pile cap around the existing foundation and additional piles.

The unique repair approach provided a robust pier stabilization and increased the foundation capacity while being constructable within the required project timelines and dimensional constraints. This paper discusses the analysis and construction details, and challenges associated with stabilizing the pier under the constraints of ongoing traffic on one unrehabilitated carriageway and construction on the other. The presented case highlights the importance of adaptive engineering solutions in addressing unforeseen issues and ensuring the longevity and safety of critical infrastructure. The project was executed by M.D. Steele Construction Limited, a General Contractor for this project based in Winnipeg.

1. INTRODUCTION

The St. Vital Bridge over the Red River in Winnipeg, Manitoba, was built in 1964, and rehabilitated in 1988. The bridge is a vital link within the City of Winnipeg's transportation network, supporting the conveyance of approximately 44,000 vehicles per day. The nine-span continuous steel plate girder structure is 280 m long and supports two carriageways, each consisting of two traffic lanes and a 1.5 m wide sidewalk. The bridge superstructure is supported by vaulted, cast-in-place concrete abutments founded on piles and cast-in-place concrete solid shaft bridge piers, founded on piles on the riverbanks and spread footings within the river. Figure *1* shows the existing bridge half cross-section prior to the current rehabilitation, and Figure *2* shows the bridge half cross-section of the rehabilitated bridge.

In 2021, the City of Winnipeg retained Morrison Hershfield Limited (MH) to develop a major rehabilitation plan to extend the bridge service life by 50 years. As part of this assignment, MH undertook a condition survey of the existing structure, performed preliminary and detailed design, and is currently performing contract administration on the project. As of January 2024, the southbound bridge carriageway rehabilitation is substantively completed, and the northbound bridge carriageway rehabilitation is ongoing. Overall, the rehabilitation scope includes the removal, widening, and reconstruction of the concrete bridge deck, steel girder strengthening through cover plate installation, bearing replacement, abutment resurfacing, abutment column refacing and corrosion protection, abutment upper backwall reconstruction, and other general preservation works.

Figure 1: Bridge Half Cross Section Prior to Rehabilitation

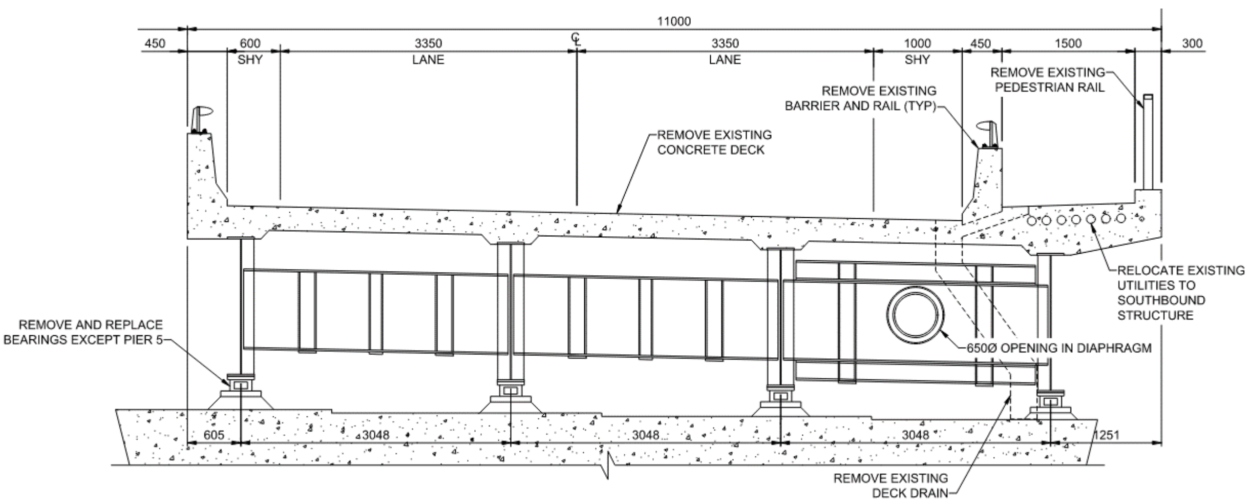
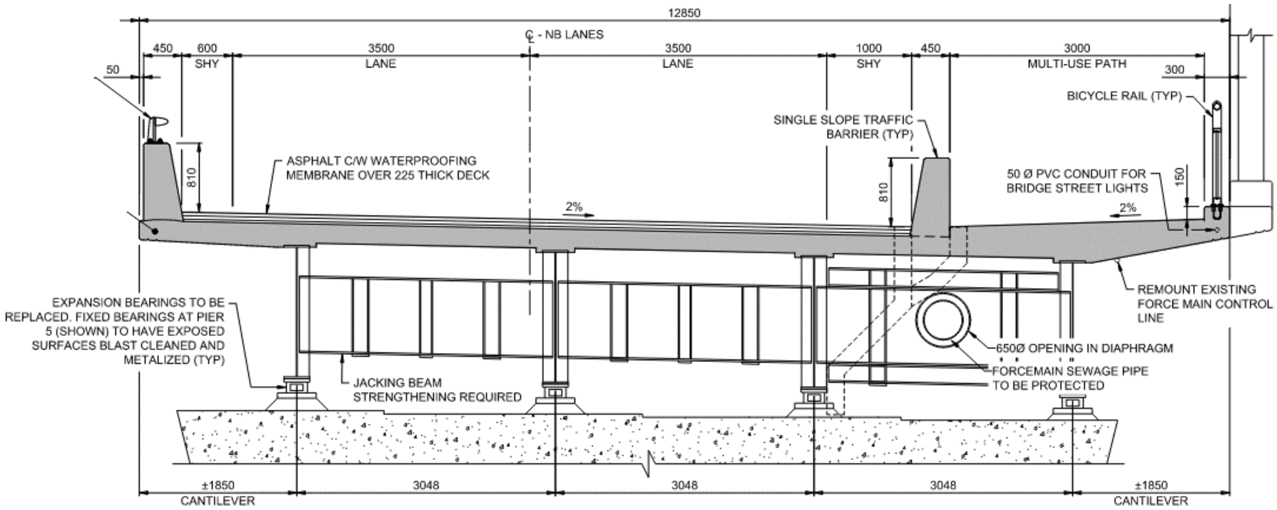
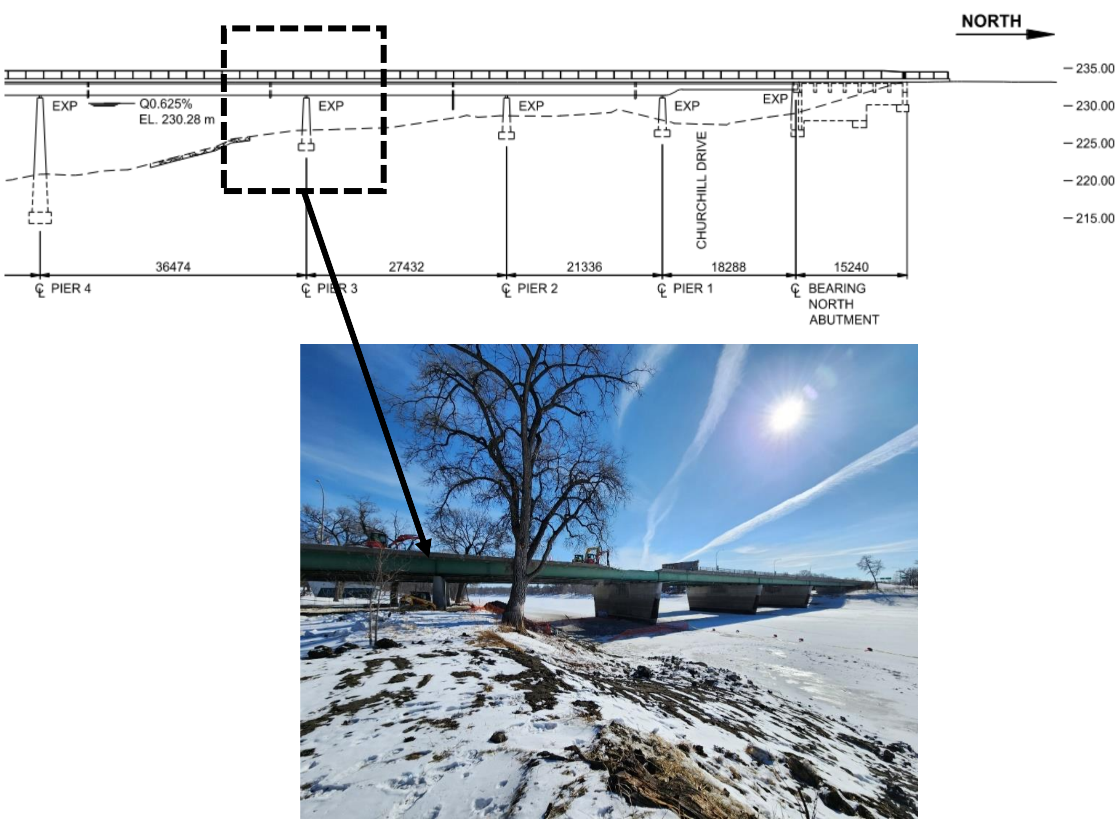


Figure 2: Bridge Half Cross Section Following Rehabilitation



During the initial phase of bridge deck demolition, movement of the north riverbank pier (Pier 3) towards the river was noted, which initiated the requirement to stabilize the pier. Figure 3 shows a cross-section of the north riverbank as surveyed prior to construction; Pier 3 is the pier that required stabilization. This paper outlines the overall pier stabilization works process, including identification of pier movement, structural analysis, design of remedial works, and construction.

Figure 3: North Riverbank Pier Location



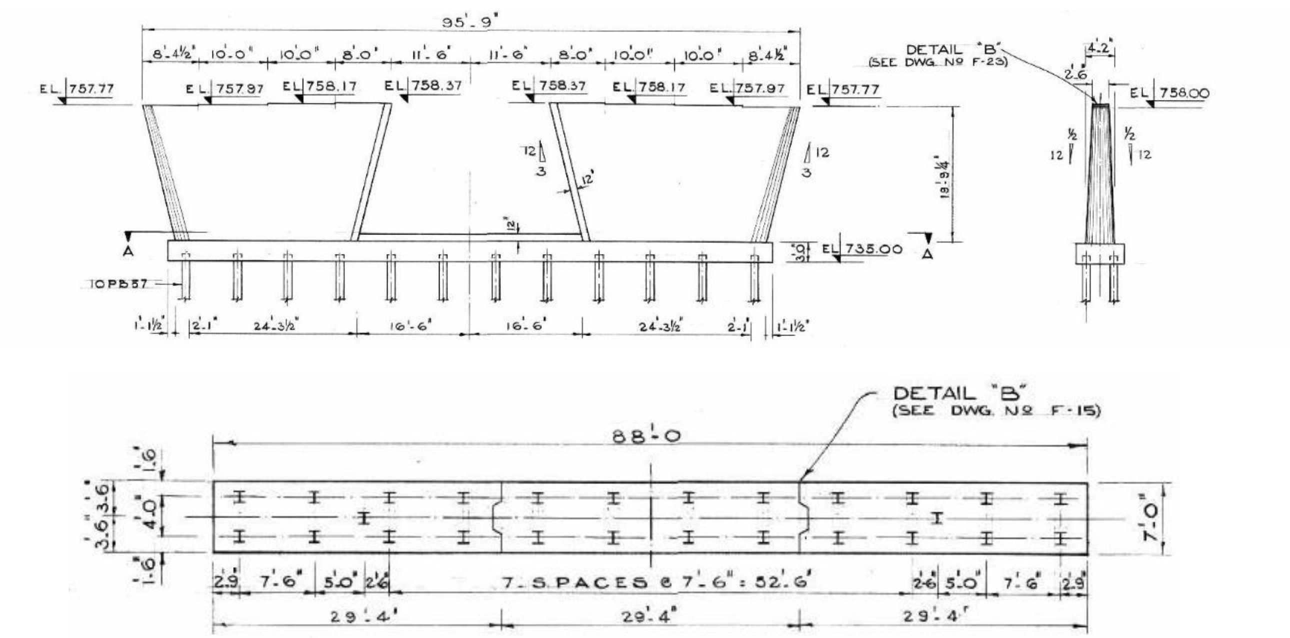
2. INITIAL PIER ASSESSMENT

During the initial phase of the project, during deck demolition, it was identified that the bearing stiffeners at the north riverbank pier were notably offset from the centreline of the bearings. This observation instigated investigation into potential causation of the bearing stiffener eccentricity. Pier modifications and foundation analysis were not included in the major rehabilitation scope; therefore, a detailed file review of pier design and construction records and desktop study was completed by the design team.

2.1 PIER GEOMETRY AND FOUNDATION

Figure 4 shows the Pier 3 concrete outline and pile layout. The pier features three separate pile caps connected by a concrete shear key between the segments. The pile caps are founded on a total of 26 HP250x85 piles with nine (9) under each pier shaft and eight (8) in the middle pile cap segment between the carriageways. The original construction drawings indicate that the pile cap shear keys do not appear to have continuous reinforcement crossing between the segments. The piles are embedded 300 mm into the concrete pile caps. Pier 3 had piling placed between the carriageways to accommodate potential deck widening by construction of additional girders lines, however neither the 1988 or current rehabilitation plans required additional girder lines to be installed.

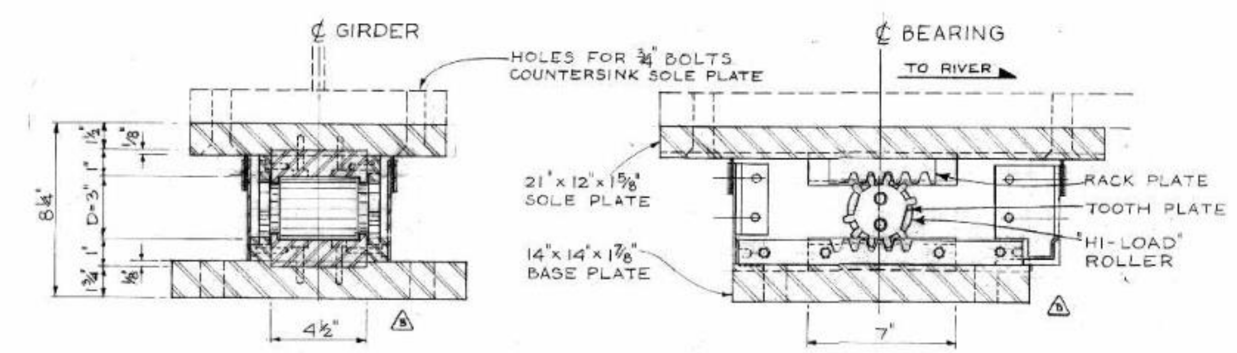
Figure 4: Pier 3 Concrete Geometry and Pile Layout



2.2 HISTORIC BEARING MOVEMENT RECORD AND RESET

The file records contained information on a bearing reset undertaken at Pier 3 in 2019, under a separate construction contract, due to the bearings reaching the limit of their thermal expansion range. Review of bearing offset measurements from the 1980s to present showed that the bearing offset measurements were increasing year over year, which indicated movement of the pier towards the river at a rate between 1 mm and 2 mm per year. As part of the 2019 work, the bearing top plate and original "toothed gear" bearing roller were recentered within the bearing bottom plate to provide additional expansion room. Figure *5* shows the existing as-built bearing detail. The amount that each bearing top plate for the southbound lanes carriageway was shifted at Pier 3 in 2019 are provided in Table *1*.

Figure 5: Typical Bearing Detail from Original Construction



2.3 Field Measurements

Bearing stiffener offset measurement were taken on April 17, 2023, following the desktop study of previous Pier 3 bearing measurements. The measurements were taken at -2°C when the bearings should be near their design centred position. Figure *6* shows the bearing stiffener offset measurement location and the offset amount.

Figure 6: Bearing Stiffener Eccentricity (a) Overall Pier (b) Close up of Bearing

(a) (b)

Table *1* provides a summary of key bearing measurements reviewed for assessment of the bearing position relative to potential substructure movement. The measurements indicated the bearing stiffeners were, on average, offset 104 mm with respect to the centre of the pier. Based on assessment of the April 2023 bearing offset measurements and comparison to the 2019 bearing top plate shift file records, 21 mm of movement, on average at the four girder lines measured, was noted at the Pier 3 bearings between the 2019 bearing reset and the April 2023 bearing measurements.

Table 1: Pier 3 Southbound Lanes - Bearing Stiffener Eccentricity to Pier Centreline and Movement

Location	April 2023 Bearing Stiffener Offset to Centre of Pier (mm)	2019 Bearing Top Plate Shift to Recentre Bearing (mm)	2019 to April 2023 Delta Average Movement (mm)
West Girder	116	100	16
Centre West Girder	109	85	24
Centre East Girder	96	75	21
East Girder	97	75	22

Rotation measurements were also taken by placing a level on the girder bottom flanges and bottom plates of the bearings on April 24, 2023. These measurements found an average pier top rotation of 0.3 degrees (0.0054 radians) away from the river.

2.4 Riverbank Slope Inclinometer Readings

Slope inclinometers (SI) on the north bank, located upstream and downstream of Pier 3, were monitored to review slope movement adjacent to the pier. The SI monitoring indicated that the riverbank had shifted approximately 15 mm to 30 mm since the monitoring began on August 31, 2022 to April 2023, with the majority of the movement experienced between August and the end of November 2022. Additional measurements taken at regular intervals after April 2023 found negligible additional slope movement has occurred.

2.5 Field Investigation Findings

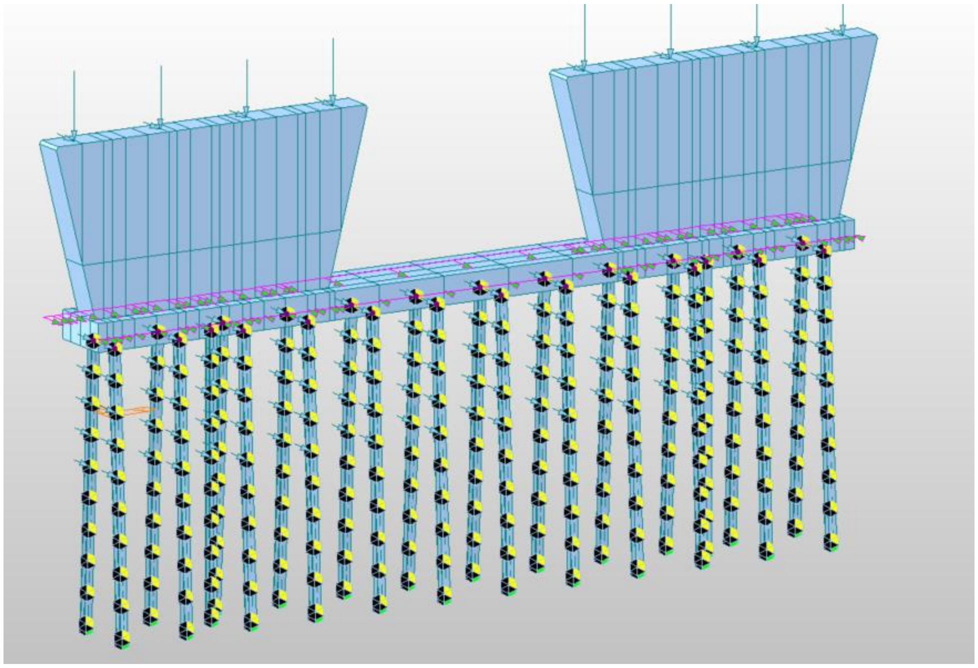
Overall, the initial investigation into bearing and pier movements concluded that Pier 3 had been migrating towards the river channel since the bearing measurement program was started by the City of Winnipeg after the 1988 rehabilitation. The following key conclusions were found:

- The overall bearing stiffener eccentricity indicated pier migration of 100 mm ± towards the river;
- The pier top was rotated away from the river with an average top rotation of 0.3 degrees towards the north;
- Pier migration was noted to accelerate at a rate faster than the historical average after 2019, increasing to between 4 mm to 5 mm movement per year, compared to the previous average of 1 mm to 2 mm per year.

3. STRUCTURE MODELING

Following the initial assessment of Pier 3 it was determined that the pier required stabilization. The design team reviewed the loading demand versus calculated capacity to determine appropriate stabilization measures. A plate and frame model of the existing Pier 3 properties was created to model the movement of the pier and load distribution to each of the piles. The model, shown in Figure 7, considers the effects of slope movement on the pier and the nonlinear lateral soil response on the piles, while attempting to reduce the approximation and conservatism of traditional design methodologies (method of squares). The following sections outline the soil lateral response, modeling methodology, model calibration, and results of the analysis.

Figure 7: Structural 3D Model



3.1 Model Setup

The model was developed based on the geometry of the pier from the 1964 record drawings, as shown in Figure 4. The pier shaft and pile cap were both modelled as plate elements, while the piles were modelled as frame elements. The boundary conditions of the model were modelled as follows:

- Soil springs were modelled using multi-linear springs at one (1) metre intervals, based on P-Y curves defined from using L-pile analysis, that were developed by the geotechnical engineer;
- The bases of the piles were modelled with pin supports (fixed for displacement, free to rotate), having been based on being driven into till. The exact pile length is unknown as it was not provided on the original record drawings and driving records were not available, so it was recommended by the geotechnical engineer to assume all piles were driven approximately two (2) metres into dense till, leading to an assumed nine (9) metre pile length; and
- Piles were modelled with the 1:20 batter based on the original record drawings.

3.2 Lateral Soil Response

The model used non-linear soil springs formulated by the geotechnical engineer in L-Pile. These springs allowed for force deflection soil response curves to be formulated that represent the changing behaviour of the soil as it compressed under lateral load. Each curve represents a different depth below the pile head and recognizes the changing behaviour of soil with depth. The curves were based on multiple recent and historical boreholes at the site. Figure 8 shows the original boreholes from the 1960's while Figure 9 shows the resultant P-Y curves used in the analysis.

Figure 8: Original Bridge Construction Boreholes on North Riverbank

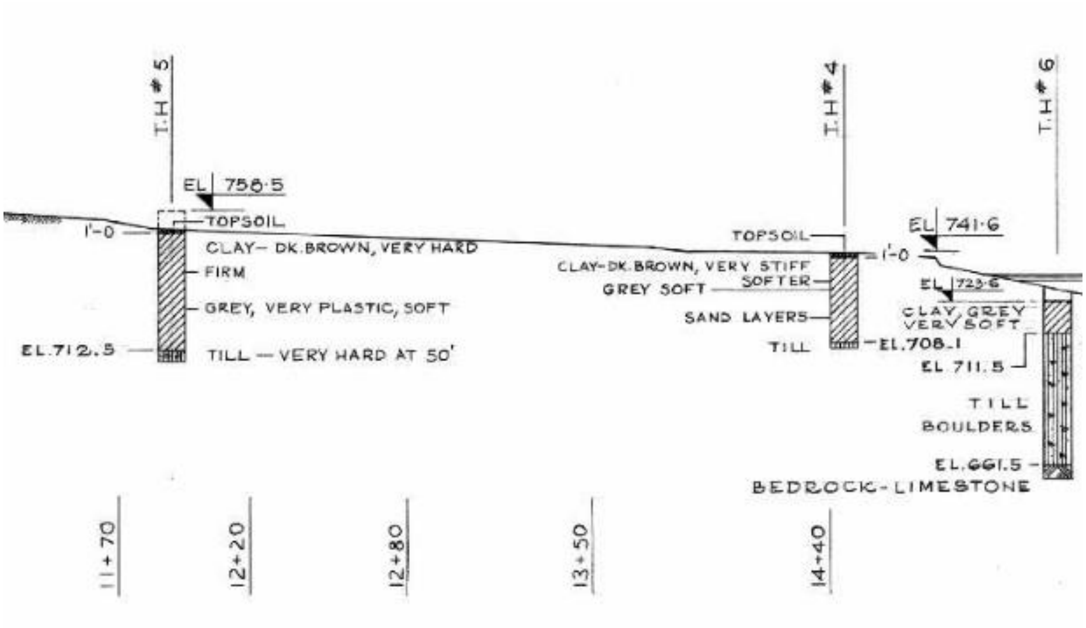
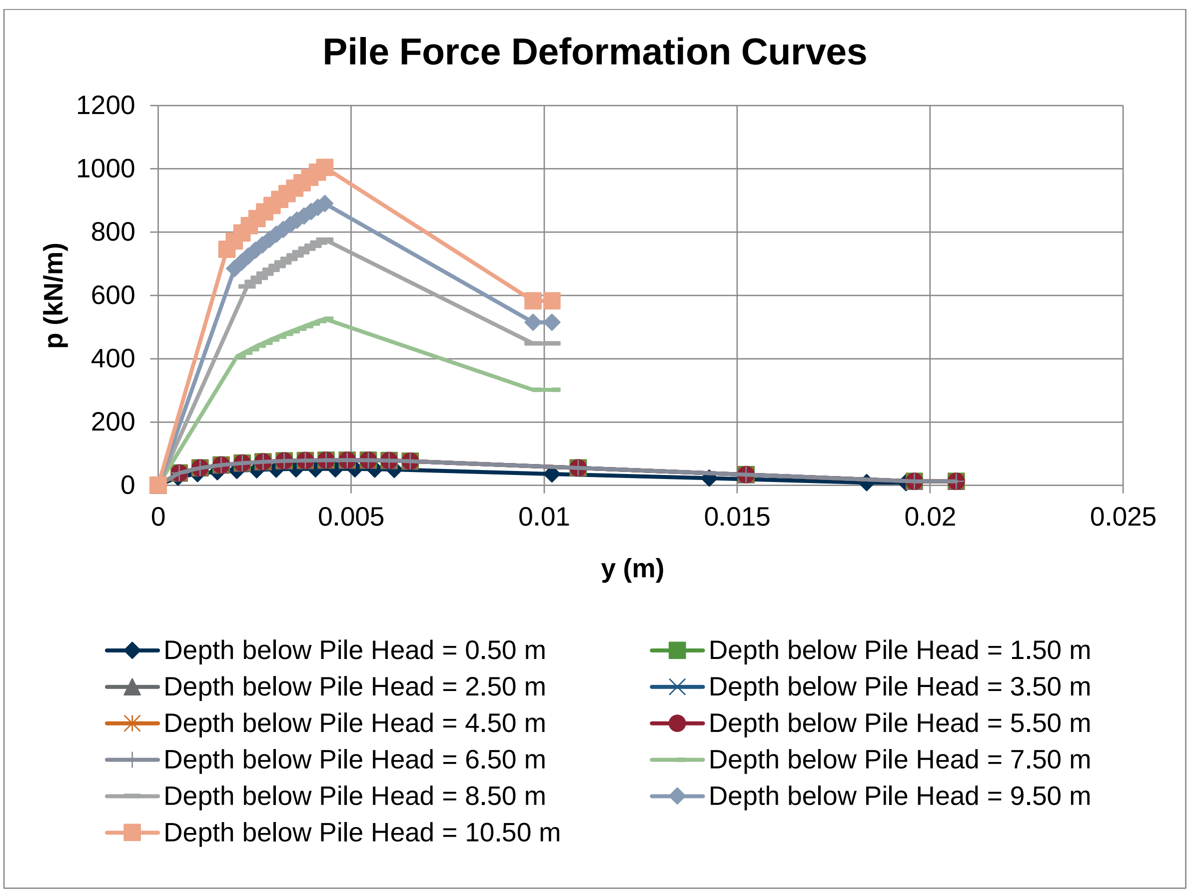


Figure 9: Pier 3 Non-Linear P-Y Curves



3.3 Model Scenarios and Loading

The general slope stability conditions at Pier 3, modeled by the geotechnical engineer, demonstrated a marginal factor of safety with the critical slope slip surface approximately 4.5 m below existing ground surface. During detailed design of the overall bridge works, riverbank offloading of the slope was proposed to increase the factor of safety of the slope to stabilize it.

As a result of the current and planned increase to the slope factor of safety, three modeling scenarios were developed to provide an indication of a range of pile loading and movement conditions. These scenarios considered the original bridge if the slope had not experienced movement as well as the potential impact of historic and current movement of the slope. These three scenarios included:

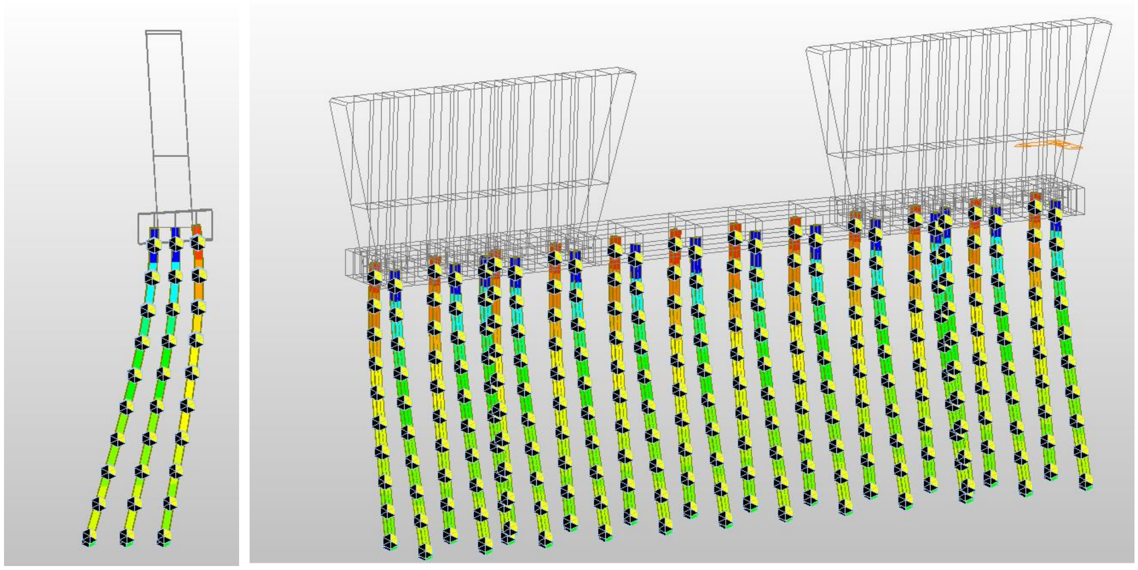
- **Case 1:** No historic slope movement considering the original condition (Slope FS >>1). Modelled with soil springs along the compete pile depth and no lateral force on the piles or pile cap.
- **Case 2:** Stable slope following movement along the slip plane (Slope FS >1). The upper 4.5 m of the slope was assumed to have no lateral soil support but an equivalent lateral force on the pile in this upper surface equal to maximum loading on the P-Y curve. After this loading is applied, the slope theoretically stabilizes, and the upper soil springs then engage.
- **Case 3:** Unstable slope following movement along the slip plane (Slope FS<1). The upper 4.5 m of the slope is assumed to have no lateral soil support but an equivalent lateral force on the pile in this upper surface equal to maximum loading on the P-Y curve. After this loading is applied, the slope does not stabilize, and the upper soil springs will not engage and were left out of the analysis.

For each case, passive pressure was placed on the upslope (north) slide side of the pile cap and at-rest pressure on the downslope side to coincide with the soil gap at the pier base. The loads transmitted from the bearings were placed on the top of the pier plate elements. It was assumed that the maximum longitudinal force from bearing friction was 5% of the vertical loading.

3.4 Pier Assessment

The pier top rotations and movements were modeled for each case and compared to field measurements. The field measured pier top rotation of 0.005 rads and peak eccentricity of 104 mm compared well with Case 2, stable slope following initial movement, at ULS. The model calculated similar results of 113 mm of movement towards the river and a rotation of 0.0059 rads when considering a range of probable loading scenarios. Deformation of the model is shown in Figure *10*. Notably, the movement mode was very similar to field measurements, with the pile cap shifted closer to the river and the pier top rotated away from the river.

Figure 10: Calibration Model Deformation



Following model calibration, load combinations outlined in the CHBDC S6-19 bridge code were reviewed for the stable and unstable slope cases to produce a range of likely loading conditions on the piles supporting the pier.

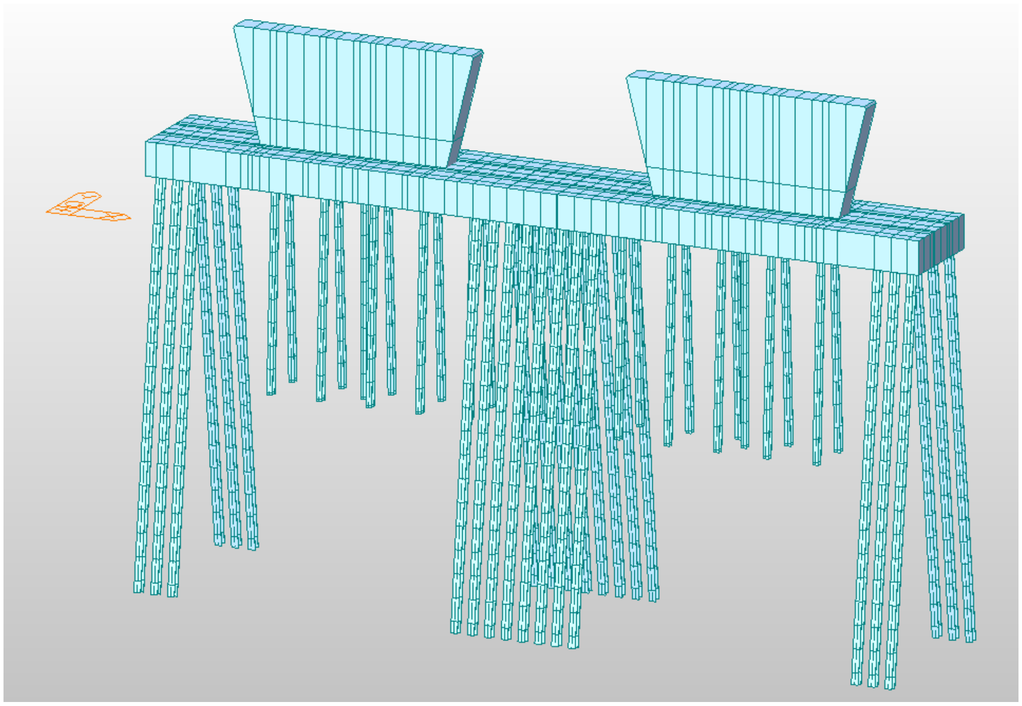
All cases included the dead loads and proposed live loads with lane positions for the rehabilitated bridge cross-section following planned construction work. A range of possible loading scenarios were reviewed to develop a better understanding of the influence of slope movement on pile loading. These cases demonstrated that the piles were overstressed on the existing bridge by meaningful amounts and required underpinning to offload the foundation. The primary cause of both the pier movement and overstress was determined to be slope movements.

3.5 Pier Stabilization Design

It was determined that pier underpinning was required to stabilize the pier against future movements and provide supplementary foundation / substructure capacity due to observed field conditions and measurements as well as structural analysis results. Building on the previous model of the pier, a new model was built that included an encasement pile cap and additional H-piles placed at locations where the existing pier and bridge geometry would permit driving. The model is shown below in Figure 11. The new pile cap was designed to carry the full weight of the superstructure in the event that the existing foundation was overstressed due to pier / slope movements. This was accomplished by modelling the pier in six separate stages discussed below:

- **Stage 1:** Original conditions with movement applied;
- **Stage 2:** Passive pressure removed to model offloading works;
- **Stage 3:** New underpinning design added;
- **Stage 4:** New bridge deck placed on southbound structure;
- **Stage 5:** Deck demolished on northbound structure;
- **Stage 6 (Final Condition):** New deck on northbound structure.

Figure 11: Foundation Underpinning Model



The final design, shown in Figure *12* and Figure 13, consisted of 28 piles placed at offsets around the existing pier. Spatial constraints of the existing bridge substructure and superstructure and the bridge rehabilitation construction being in-progress only permitted piles to be installed at the transverse edges of the pier and in between the carriageways. The piles are embedded into a new encasement pile cap that is 4.5 m wide by 1.5 m tall.

This cap is specifically engineered to withstand the entirety of the loading from the pier above and effectively transmit it to the new piles. To facilitate this load transfer, tension ties placed in bridge direction were made continuous, by coring through the existing pile cap to allow installation. Shear reinforcement was drilled and epoxy doweled into the pier shaft around its perimeter.

Figure 12: Foundation Underpinning Reinforcing

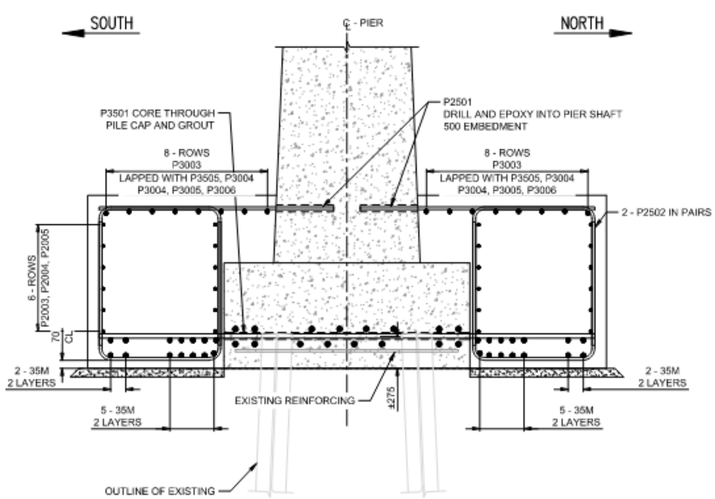
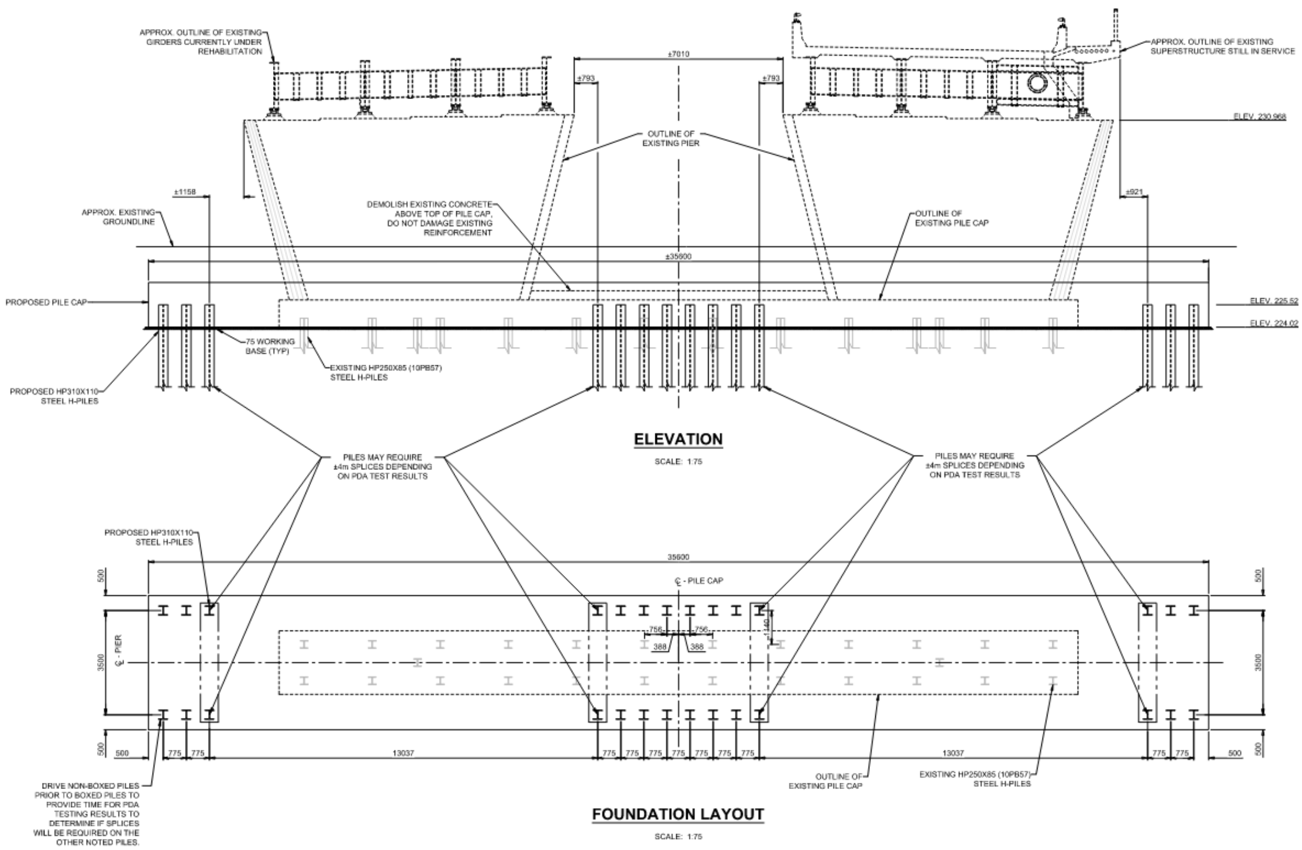


Figure 13: Foundation Underpinning Pile Layout



4. PIER STABILIZATION CONSTRUCTION

Throughout the design of the pier stabilization, the project team regularly reviewed constructability with the contractor. Partnership and collaboration between the contractor and the engineering team significantly supported the pier stabilization process. The overall construction staging of the pier stabilization was undertaken as follows:

1. The soil surrounding the pier was offloaded to reduce the slope driving pressures and a rock berm was placed on the toe of the slope in the river channel to increase the short-term slope stability Factor of Safety, as shown in Figure 14. The slope offloading removed between two and three metres of fill over the existing pile cap. A silt curtain was used to control sedimentation from rock placement in the river. No cofferdam was required to complete the pier stabilization works as ground seepage from the river was able to be controlled by pumping.

Figure 14: Riverbank Offloading and Stabilization Berm Rock Placement

2. The existing reinforcement in the pile cap and the steel H-piles were located using a GPR reinforcement survey. Holes were cored through the existing pile cap for grouted tension ties, which were installed following pile driving. Coring was completed using a tractor mounted rock drill, as shown in Figure 15. Layout of core holes considered the locations of existing piles, future piles, and pile cap reinforcement. The design of tension ties was adapted to accommodate a range of distances from the underside of the new cap to facilitate rock hammer drilling. The track mounted rig provided a cost-effective alternative to coring the holes through the existing pile cap. Careful monitoring was required to confirm the holes were cored as level as possible through the pile cap.

Figure 15: Fully Excavated Pile Cap and Rock Drilling of Tension Ties Through Pile Cap

3. Six pile driving rig setups were required to gain access to appropriate zones and a total of 28 steel H-piles (HP310 X 110) were driven to refusal surrounding the pier at locations and offsets that accommodated the existing bridge, ongoing construction activities, and traffic staging (e.g. deck was removed from southbound carriageway at this point in the rehabilitation and traffic was bi-directional on the northbound carriageway). Figure *16* provides images of the pile driving. Pile driving works had to avoid the deck overhangs and girders of the existing bridge as well as accommodate overhead clearances for extending the pile driving lead. Pile capacities were validated using pile driving analysis (PDA) test results.

Figure 16: Driving H-Piles to Refusal

4. The piles were cutoff to the desired embedment into the new encasement pile cap. A working base slab was placed surrounding the existing pile cap to allow for assembly of the reinforcing steel cage around the piles, as shown in Figure 17. The working base slab further allowed the contractor to control seepage from the river, which was near the same elevation as the excavation base.

Figure 17: Pile Cutoff and Working Base Concrete

5. Following pile installation, the existing pile cap was blasted to roughen the surface and tension ties were placed and grouted across the existing pile cap as shown in Figure *18*. Bulkheads were placed on one side of the bar and flowable grout injected from the other side. Dowels were also drilled and epoxied around the perimeter of the pier shaft and the remaining two faces of the existing pile cap that would be embedded within the new pile cap.

Figure 18: Tension Tie Arrangement and Grouting

6. New pile cap reinforcement was placed surrounding the existing pile cap and perimeter formwork was erected, as shown in Figure 19. Temperature sensors were embedded within the concrete to monitor the heat evolution and maintain appropriate thermal gradients for a mass concrete pour. Approximately 180 cubic meters of concrete was placed within the form over eight (8) hours by means of concrete pumping. The formwork was left in place for seven (7) days while the concrete cured with a wet cure maintained on the surface until form stripping. Following the formwork removal the pile cap was coated in damp proofing and backfilled with granular material.

Figure 19: Final Reinforcement and Concrete Placement

7. Lastly, the overall riverbank surrounding the pier was covered in a rip rap apron to provide further slope stabilization and erosion control, as shown in Figure 20.

Figure 20: Backfill around Pile Cap and Final Rip Rap

5. CONCLUSIONS

The Pier 3 underpinning assessment, design, and construction was completed successfully on an accelerated timeline while construction on the broader project continued. Rigorous and innovative engineering practice was implemented to recreate the behaviour of the existing structure and propose appropriate design solutions. The design team worked closely with the contractor completing the work to confirm details were constructable and was an excellent example of engineering innovation and teamwork.

A number of key lessons learned may be taken away from this paper:

- Bearing monitoring provides useful data both for measuring bearing performance and as an indication of potential substructure movements which can lead to pile overstress;
- Piling design of previous codes are less conservative than current codes and this impacted the overall design of the underpinning;
- Substructure migration movements can accelerate over time, which requires appropriate monitoring programs to observe and document changes;
- It is critical to understand the impact that the pier movement has on the stresses being imparted on the existing piles; and
- Three-dimensional analysis utilizing P-Y curves can be implemented to successfully recreate field measured pier movements and assess any overstress.

6. ACKNOWLEDGEMENTS

The overall St Vital bridge rehabilitation project is funded by the City of Winnipeg. The authors would like to thank the City of Winnipeg for their strong support throughout the project. In addition, construction was completed by M.D. Steele construction, who provided excellent collaboration and constructability feedback throughout the design of the stabilization. Finally, we would like to thank our geotechnical engineer Trek Geotechnical for there ongoing collaboration through the design and construction process.