Waverley West Arterial Roads Project
Kenaston Overpass

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ABSTRACT

Kenaston Boulevard was designated a primary economic route within the City of Winnipeg, Capital Region and the Province of Manitoba due to the linkage it provides between major industrial/commercial sites and national/international trade routes. The Waverley West Arterial Roads Project (WWARP) extends Kenaston Boulevard to the Perimeter Highway and handles a large volume of truck traffic. The extension has produced significant cost savings for the transport industry as well as relieving congestion on adjacent arterial routes and routes servicing new neighbourhoods in southwest Winnipeg. This session will focus on the unique grade-separated structure constructed as part of the WWARP project, known as the Kenaston Overpass. The first flyover (grade-separation) structure of its kind in Winnipeg, the Kenaston Overpass consists of a 105 m long two-span structure that spans across Kenaston Boulevard. The new flyover structure facilitates uninterrupted travel along the extended Kenaston Boulevard while facilitating efficient movement of traffic to Bishop Grandin Boulevard, another major arterial route.

This complex project had unique design and construction features that were considered during its delivery. One of the fundamental challenges for the Kenaston Overpass was accommodating for the future extension of Bishop Grandin Boulevard in addition to satisfying the roadway alignment of the realigned Bishop Grandin Boulevard and extended Kenaston Boulevard. This greatly restricted a number of the structural configurations that could have been used to simplify the design.

Unique design features include the horizontally curved, 5.2% superelevated deck constructed on steel trapezoidal box girders of varying span lengths, with semi-integral abutments. To keep the approach embankments within the available property and maintain stability of the surrounding (poor quality) soil, mechanically stabilized earth wall embankments with light-weight concrete backfill were utilized.

Unique construction challenges include dealing with poor subsurface soil conditions, the presence of fibre optic communication cables and a high pressure gas line on site, working along the boundary of high-power transmission lines, construction during the coldest winter in over 100 years, maintaining four lanes of traffic through the site during construction, and fabrication and erection the geometrically complex steel box girders.

The Kenaston Overpass project highlights the transportation field's promotion of efficient transportation by using innovative design methodologies and construction techniques to meet the increasing demands on infrastructure today.

1.0 PROJECT BACKGROUND

Manitoba's capital and largest city, Winnipeg (herein referred to as "the City") is expanding on Canada's prairie plains. Planning for development of the City's southwest quadrant began in the early 2000's. The area is typically referred to as the "Waverley West" neighbourhoods. At full build out, it is anticipated that the area will be home to over 40,000 people. To service an area of this size, and provide improved arterial connections with the greater region, the City initiated the Waverley West Arterial Roads Project (WWARP). The WWARP project as a whole is comprised of over 40 lane-kilometres of high speed roadway connecting Kenaston Boulevard (Kenaston) to the City's Perimeter Highway (P.T.H. 100), including a 105 m overpass structure. In addition to vehicular traffic, the project included multi-use pathways along its length to connect pedestrians, cyclists, and other active transportation (AT) users to the existing AT network. There were multiple phases to the WWARP project and each phase was further divided into multiple contracts for the purposes of construction tendering. This paper focuses on the Kenaston Overpass which is located at the north end of the WWARP project and is shown in green in **Figure 1**.

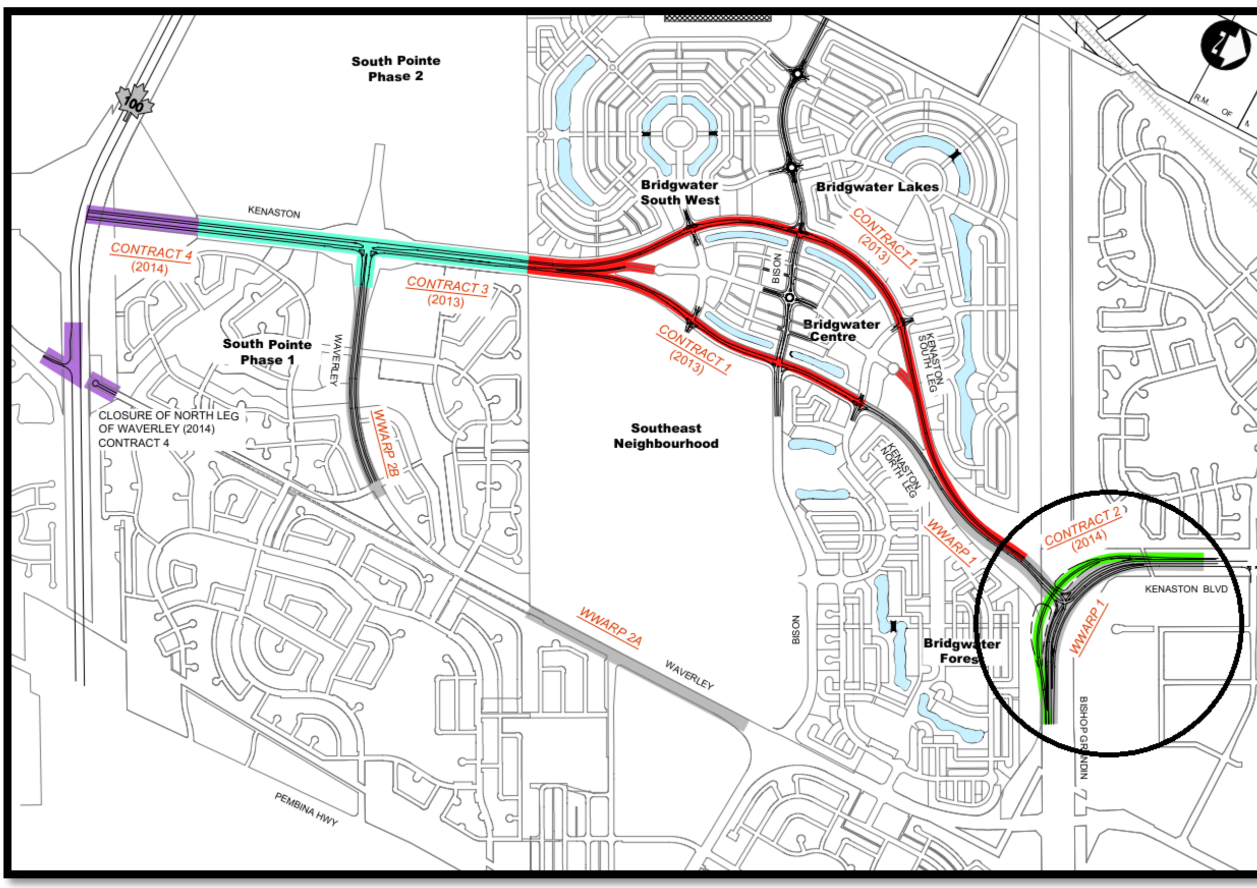

FIGURE 1: PROJECT LIMITS – KENASTON BOULEVARD TO P.T.H. 100

Kenaston was designated an economic route within the City, Capital Region and Province of Manitoba. Due to the linkage it provides between major industrial/commercial sites and national/ international trade routes, it handles a large volume of truck traffic that has resulted in significant cost savings for the transport industry. Kenaston also provides access to new residential and commercial zones within the Waverley West neighbourhoods and provides service to high volumes of commuter traffic. Lastly, Kenaston functions to provide a corridor for inter-neighbourhood travel, commuter active transportation, and transit networks.

FIGURE 2: KENASTON OVERPASS

The Kenaston Overpass is the first flyover (grade-separated) structure of its kind constructed in the City and consists of a two-lane bridge structure and ramp lanes that connect southbound Kenaston to eastbound Bishop Grandin Boulevard (Bishop Grandin), two major arterial routes in the City. With the realignment of the through-lane over the bridge, modifications and improvements at the Kenaston and Bishop Grandin at-grade intersection were also completed.

2.0 Design Challenges

There were a number of design considerations and project constraints that were addressed during the design development of the overpass structure and the ramp lanes. The primary design codes used for the design were the Canadian Highway Bridge Design Code (CHBDC, CSA S6) and the TAC Geometric Design Guide for Canadian Roads.

The following sections will highlight several of the key design challenges and project constraints and the associated measures that were adopted to address them.

2.1 Overpass Arrangement and Configurations

One of the main design challenges for the Kenaston Overpass was determining the location of the overpass. There were several constraints for the location of overpass which included:

- The existing lanes of Bishop Grandin and Kenaston to the north and the recently constructed at-grade intersection that connects the extended south leg of Kenaston. While these roadways were shifted to the north and east to maximize the available area for the overpass, they were restricted by a major drainage channel that could not be impacted.
- The future extension of Bishop Grandin to P.T.H. 100 to the west of the Kenaston Overpass; in concept this is similar to the WWARP project with Bishop Grandin extending west instead of Kenaston extending south. What made accommodation of this potential future connection complex was that the geometric design had not been finalized and conservative assumptions had to be made regarding the long-term arrangements of this interchange.
- High voltage transmission lines to the south. There was a buffer zone extending 6 m on either side of the transmission line where the grade could not be altered. Arcing from the lines also had to be considered when constructing any roadway or structural elements within a given radius from the power lines.
- Along southbound Kenaston north of the overpass, the roadway passes over a concrete box culvert which allows the major drainage channel to cross under the roadway. This box culvert was extended to accommodate the existing lanes of southbound Kenaston, the two new ramp lanes and an active transportation pathway. There were challenges with the vertical profile as the box culvert was fixed at a lower elevation followed by an immediate upgrade to gain height for the overpass.
- There were a number of critical utilities in the area such as gas, fibre optic telecommunications, and land drainage sewer. Due to scope and schedule concerns, there was a desire to minimize the relocation of these utilities and so the Kenaston Overpass was designed to avoid or accommodate all utilities within the footprint of the overpass and ramp lanes.

FIGURE 3: SITE CONSTRAINTS (LOOKING EAST)

Using a roadway design speed of 90 km/hr and meeting the various geometric design requirements, a horizontal and vertical profile was developed for the roadway/overpass. Several iterations were required to address conflicting design requirements. For example, the inside shy distance for the overpass had to be increased from 1.5 m to 3.5 m in order to satisfy the inadequate stopping site distance associated with the visual interference from the concrete barriers on the overpass. The barriers were selected as 1070 mm tall solid concrete barriers to minimize the visual impact of traffic to the nearby residential subdivision but in doing so generated the sightline interference on the north side of the bridge. The horizontal curvature (radius) of the ramp lanes on either side of the overpass were reduced to permit the increase in radius for the overpass and together with the increase in shy distance assisted in "seeing across the curve" and meeting the required stopping sight distances. These changes in radius for the roadway ultimately led to an increase in superelevation, although it was still deemed safe in terms of deck icing and driver comfort.

The final roadway configuration of the overpass was selected as follows:

- Horizontal radius of 340 m for the ramp lanes and 510 m for the overpass structure.
- A maximum superelevation of 5.2% for the length of the overpass structure tapering to a crowned roadway along the ramp lanes.
- A vertical curve of K =32 for the ramp lanes and overpass structure.

The site constraints and final roadway configuration heavily restricted the available footprint for the bridge structure. A number of configurations for the bridge structure (spans, foundation locations, skew) were investigated and ultimately a configuration with two longer spans was selected; 44.5 m – 59.2 m for a total span length of 103.7 m. This configuration fits within the available footprint, allows for the future extension of Bishop Grandin to the west, and accommodates the required clearance to the transmission lines and other utility lines in the area.
In order to achieve the necessary horizontal and vertical curvature, superelevation, and long span lengths required by the roadway geometry and site constraints, steel trapezoidal box girders were selected for the bridge superstructure. Box girders typically perform well for curved bridges due to their high torsional stability (eccentric loading on a horizontally curved roadway causing torsion), ability to have longer span lengths, and can be broken into multiple segments to make fabrication and erection simpler. Other girder alternatives were reviewed as well but the box girders were selected as they were more efficient than a plate girder system and the span lengths and horizontal curvature were excessive for the use of (precast prestressed) concrete girders.

Two trapezoidal box girders were selected and had a width of 3.5 m and a depth of 2.2 m and 2.0 m for exterior and interior girder, respectively. The difference in height between the girders was due to the different span lengths for the exterior and interior girder and superelevation of the roadway. A 225 mm thick cast-in-place concrete deck with 90 mm thick asphalt overlay was selected as the riding surface. A single pot bearing was provided at each of the substructure locations (two abutments, one pier) due to their ability to accommodate high loads from the superstructure and larger thermal movements. The foundation for the substructure units consisted of steel H-piles, with battered piles at the fixed foundation (pier) for additional lateral capacity. A cross-section of the bridge is shown in **Figure 4** below.

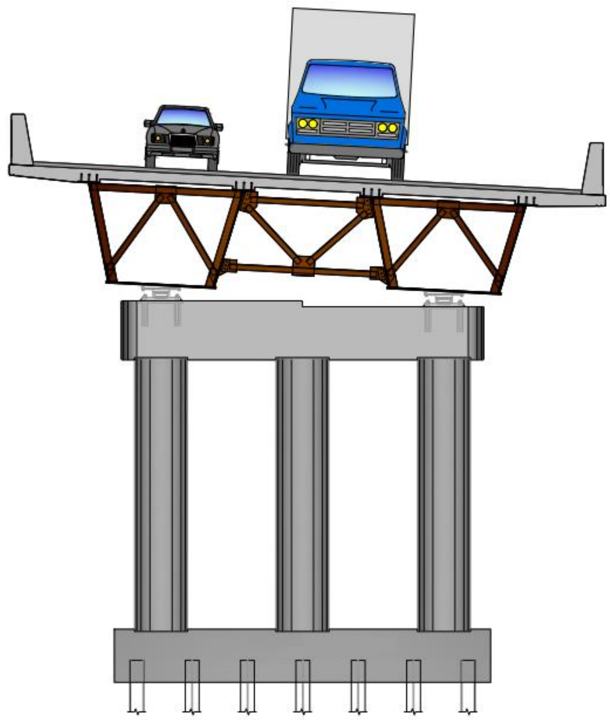

FIGURE 4: OVERPASS CROSS-SECTION (AT PIER)

A center pier and two semi-integral abutments, each consisting of a grade beam and abutment bearing seat, make up the five substructure units. The foundations for the substructure units were selected as steel H-piles. For the (fixed) pier, three round columns were provided due to the increased rigidity in the longitudinal and transverse direction. A reinforced structural stab extends from the end of the deck to the grade beam and expansion joint as part of the semi-integral abutment configuration. The layout of one span of the bridge is shown in **Figure 5** below.

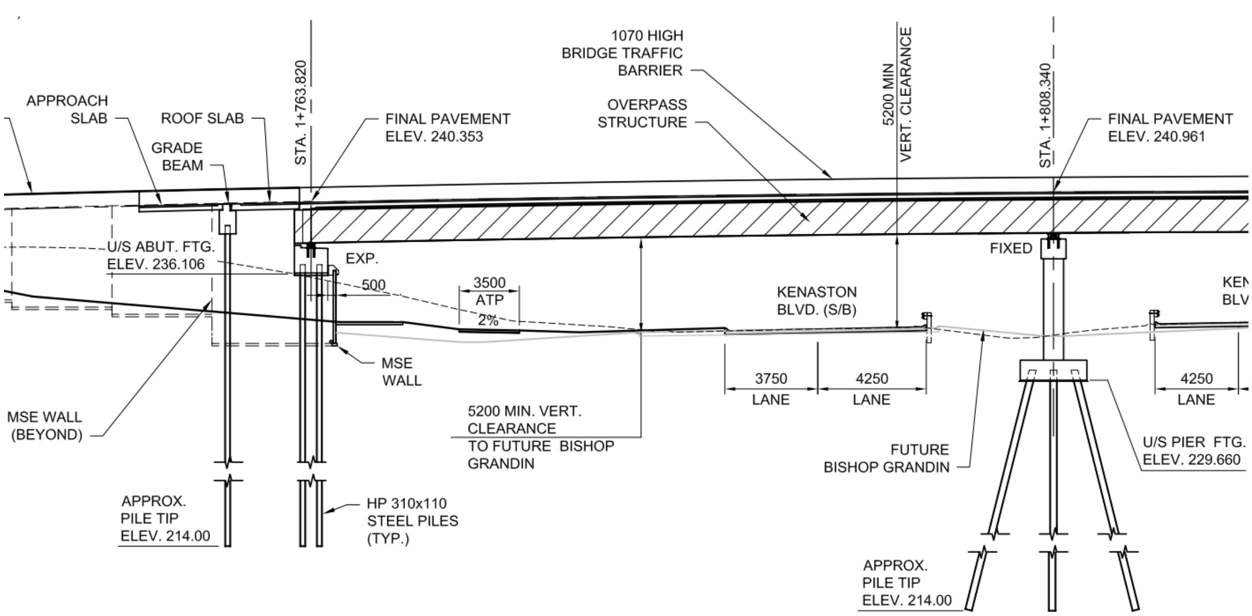

FIGURE 5: STRUCTURAL ARRANGMENT (WEST SPAN)

2.2 Approach Roadways and Retaining Walls

The elevation difference between the overpass riding surface and the roadways underneath was approximately 8 m. Due to the poor clay soils in the City, a minimum slope of 4V:1H is typically required for stable embankments and for the Kenaston Overpass the selected slopes were closer to 5V:1H. Based on the 5:1 slope requirements and the elevation difference at the overpass, the approach embankments required a minimum footprint of 40 m in each direction. However, this embankment footprint would have conflicted with the existing roadways and the clearance zone of the high voltage transmission towers. Therefore a retaining wall system was required to reduce the footprint of the approach embankments to eliminate the conflict with the roadway and transmission towers. Different systems were considered including steel sheet piling and a mechanically stabilized earth wall (MSE wall). Sheet piling walls were not selected due to the presence of critical utility infrastructure under the ramp lane embankments. A MSE wall was chosen as the retaining wall system due to its simplicity of construction and shallow excavation requirements which eliminated the utility conflicts.

Initial design of the MSE wall using standard granular backfill yielded excessive long-term settlements of the approach embankments and it was determined that the final design would need to mitigate these settlements. Short-term settlements were expedited through partial pre-loading of the approach embankment areas one year prior to construction of the Kenaston Overpass. As the final location and configuration of the bridge had not been finalized during preloading efforts, it was decided that additional dewatering/settlement measures or other ground improvements would not be included as part of the preloading.
Long-term settlements were addressed by reducing the loading on the subgrade material. This was achieved through the use of lightweight fill, in this case lightweight concrete. Lightweight concrete is comprised of cement, flyash, water, and a foaming agent. By using a foaming agent in the cement/flyash slurry, the slurry density can be reduced from 1600 kg/m^3 to 450 kg/m^3. While the compressive strength of the lightweight backfill is drastically reduced, to around 0.5 MPa, the material performs similarly to clay with the added benefit of having a density of less than 1/3 of clay.

One of the primary design requirements associated with the design of MSE walls is limiting the differential movement of the MSE wall along its length; typically differential movement greater than 1% is considered excessive. While an MSE wall system can withstand the entire wall settling equally, excessive differential movement will cause panels to move relative to each other and can cause cracking, moving out of plane, or overstressing/damage to the connections with the reinforcing elements (steel strips, geo-grid, etc.), all of which can reduce the service life or cause premature failure of the MSE wall system. For the Kenaston Overpass, differential settlement was addressed through the use of variable thicknesses of standard and lightweight backfill along the length of the wall. This allowed the effective pressure of approach embankments to be uniform across the length of the MSE wall. The varying depths of backfill material are shown in **Figure 6** below. The lightweight backfill extends from the limit of excavation of the embankment preloading (shown in red) to the blue line. Compacted granular backfill was used for the remainder of the backfill (shown in green).

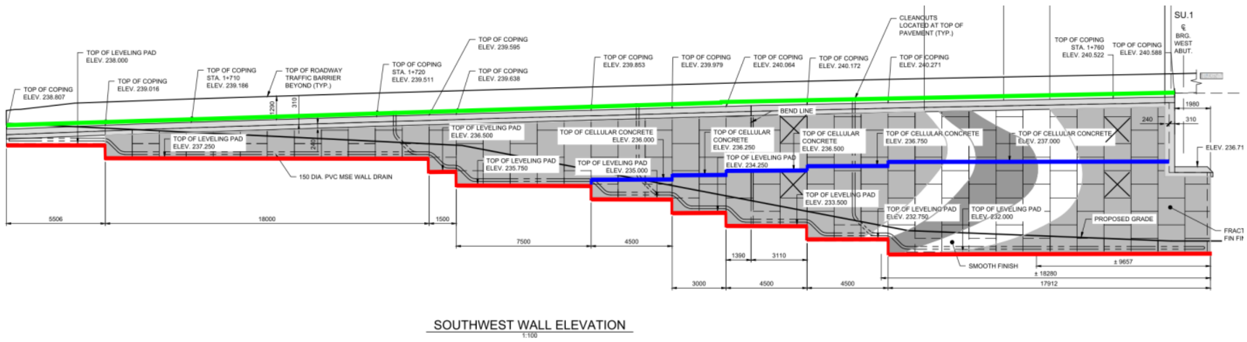

FIGURE 6: LAYOUT OF MSE WALL

2.3 Durability and Maintenance

With the growing infrastructure deficit in Canada, it is a constant effort to ensure that the service life of new infrastructure is maximized. The Kenaston Overpass was no different and many different design decisions were made to ensure that the structures achieved or surpassed their design service life and maintenance and rehabilitation was minimized for these major thoroughfares. Of note, the required design service life for the overpass structure is 75 years in accordance with the CHBDC and City requirements. The design service life for the MSE wall system was increased to 100 years due to the complexity associated with rehabilitation/reconstruction of MSE wall systems.

The design team made both configurational and material selection decisions as part of achieving the design service life for the bridge and MSE wall structures. A number of the configurational design selections are as follows:

- Semi-integral abutments were chosen for the bridge structure. Through selecting semi-integral abutments, the expansion joint system was shifted back onto a grade beam that was 5 m into the approach embankments. Inevitably all expansion joint systems leak at some point in their service life and can lead to deterioration of the deck, girders, bearings, and abutment. By moving the expansion joint away from the abutment area onto a sacrificial grade beam, the maintenance and rehabilitation efforts, complexity, and associated costs are significantly reduced.
- All structural concrete was provided with a slope to ensure that water will shed from the structure. This included the substructure, superstructure, and riding surface of the overpass to avoid build-up of material or ponding that can cause saturation of the concrete. Long-term saturation can lead to ingress of moisture and hazardous materials (chlorides, sulphates) which can lead to premature freeze-thaw deterioration, chemical degradation of the concrete paste, or expediting the corrosion of the reinforcing.
- An asphaltic wearing surface with a hot poured rubberized asphalt waterproofing system was provided on top of the cast-in-place concrete deck to protect the deck from salt laden moisture and weathering effects.
- An asphalt roadway structure was selected for the ramp lanes (on the approach embankments). As noted above in **Section 2.2**, large long-term settlements of the MSE wall were expected; approximately 600 mm during the service life of the MSE wall. As the bridge structure is supported on pile foundations to bedrock there will be different settlements between the bridge and the MSE walls. By using an asphalt roadway structure, the ramp lanes can be "topped up" with asphalt whereas a concrete roadway would require more extensive rehabilitation efforts.

- MSE wall systems are not designed to include the effect of hydrostatic pressures within the backfill material and even when retaining wall systems are under-designed, water is most commonly the initiator of failure. Therefore it was critical that extensive drainage piping was provided for the MSE walls to ensure that any water that entered the backfill would be immediately removed. Piping was provided along the base of all wall footings and drained into the City's land drainage sewer system. An impermeable geomembrane was also provided immediately under the roadway structure system to collect any salt laden water that may have leaked through the pavement. Drainage piping was provided in a depression formed in the geomembrane and similarly drained into the City's land drainage sewer system.

A number of the material design selections are as follows:

- Concrete types. Type S1 sulphate resistant concrete was selected for all concrete in contact with in-situ soil as Winnipeg's clays are associated with especially high sulphate contents. Type C1 concrete was selected for all deck and barrier concrete due to their continuous exposure to chlorides. Fibre reinforced concrete (with non-metallic fibres) was required for all structural concrete, with the exception of the footings, to assist with the control of early shrinkage cracking.
- Reinforcing types. Galvanized reinforcing was specified for all substructure concrete and stainless steel reinforcing was specified for all superstructure concrete. Together with the use of an asphaltic wearing surface and waterproofing, the intent was to maximize the service life of concrete deck. Reinforcing depth testing as part of quality control for the deck and barriers was specified to ensure the required concrete cover was achieved.
- Weathering steel for the girders with a structural steel coating system at the girder ends. Drip strips, both on the concrete deck (inset) and underside of girders (attachments), were also detailed to remove any water running along the length of the deck and girders.
- The MSE wall system had stringent material requirements to ensure the 100-year service life of the MSE wall would be achieved. The backfill material had to be chemically controlled for pH, chlorides, sulphates, and resistivity. Additionally, it had to be controlled for physical parameters, including gradation, friction angle, deleterious substances, and soundness so the mechanical performance of the system could be maintained. The reinforcing strips were galvanized and extra removable test strips were installed to provide a method of confirming the performance and section loss of the reinforcing, as performance of the reinforcing system is directly associated with performance of the overall MSE wall system.

3.0 CONSTRUCTION CONSIDERATIONS

Converting drawings to structure and pavement is equally complex as the design process and effective specifications and oversight were provided to ensure the final constructed elements achieved the requirements of the design. As with any construction project, there were a number of challenges encountered during the construction process that required effective communication and coordination between the contracting groups and the design engineers. The following sections will outline some of the key considerations and challenges that were encountered.

3.1 Substructure

Bridge construction commenced with the installation of the steel H-piles for the substructure units. While there was significant laydown area for the installation of the abutment piles, the pier foundation was located in the narrow median between four active lanes of Kenaston. Temporary shoring was installed to facilitate the installation of piles and construction of the pier. Based on the size of the pier, the shoring was installed within 1 m of the roadway. The contractor had originally intended to temporarily place equipment on the roadway for pile driving operations; however, after review of the working radius of the piling equipment, it was determined that a full roadway closure would be required but could not be permitted due to the high volume of traffic along the arterial route. Ultimately staged excavation of the pier was completed to satisfy the access requirements of the piling equipment. The median lanes of traffic were closed as a safety precaution for passing motorists and to allow workers and small equipment to access the area surrounding the shoring.

FIGURE 7: H-PILE FOUNDATION – PILE DRIVING

During piling operations, pile dynamic analysis testing was completed to monitor and confirm hammer and driving system performance, assess pile installation stresses and integrity, as well as evaluate pile capacity. The testing yielded that the pile driving system was performing as required and the mobilized pile capacities were higher than the desired ultimate pile capacities.

The substructure was constructed during Winnipeg's coldest winter in over 100 years and a hoarding and heating system was implemented for the protection of the concrete against the ambient conditions. Based on the size of the substructure units, they were considered mass concrete. Due to the cold-weather concrete and mass concrete, the concrete was closely monitored including thermal differentials between the core and surface, surface and ambient (within the hoarding), and overall cooling rate. Based on the temperature management plan developed by the contractor, the temperatures were monitoring multiple times per day and the heating was adjusted to ensure and the required monitoring parameters were not exceeded and the concrete cured properly (i.e., thermal cracking controlled).

FIGURE 8: COMPLETED SUBSTRUCTURE UNITS

After installation of the abutment piles and construction of the pier and abutments, excavation was required to prepare the area under the abutments for installation of the MSE wall system. During the excavation, nine of the 20 piles at the east abutment piles were damaged by the excavation equipment. The excavation contractor was required to retain an independent structural engineer to develop a repair procedure for the piles. The repairs included cold straightening of any flange deformations less than 12 mm in depth and flange deformations greater than 12 mm were flame heated before jacking into place. After the flange shapes were restored, a 12 mm steel plate was then welded across the flanges to reinforce the impacted areas of the piles.

FIGURE 9: DAMAGED EAST ABUTMENT PILES

3.2 Superstructure

The superstructure is comprised of two trapezoid box girders lines, each girder line made of three segments. The segments are approximately 35 m in length and 2.2 m and 2.0 m in height; heights varying to account for the superelevation. Trapezoidal box girders are complex structural elements primarily due to the extensive number of stiffeners, bracing, and diaphragms and the interactions between each of these elements. An added complexity for this project is the horizontal and vertical curvature and superelevation set by the roadway geometry. Translating stock plate into highly controlled shapes required significant attention paid to the layout and assembly of the various elements of each girder. Layout and checks were completed with total station and the tolerance for each individual element of the girders was less than 1 mm.

FIGURE 10: GIRDER FABRICATION

Once a girder section was assembled it was transferred to progressive assembly, i.e., the adjacent girder was butted end to end and unique splice plates were drilled for that connection. Due to the length and size of the girder segments, the fabrication plant was only long enough for a single progressive assembly setup. The schedule was monitored rigorously to ensure there was no lost time waiting for progressive assembly, as there was limited room in the assembly space.

The intricate girder system added further complexity during girder erection. The horizontal and vertical curvature and superelevation meant that the girders could not be erected "flat" and each corner of the girder was at a different elevation. A hydraulic jack system was connected to the crane rigging and allowed for any of the four lifting points of the girder to be adjusted. The primary adjustment was for the 5.2% superelevation which allowed for the fit-up of the girder segments to occur with fewer adjustments. Progressive assembly mitigated the difficulty installing the field splices and the segments fit together without reaming for the bolts of the slice plates.

FIGURE 11: GIRDER ERECTION

The pot bearings were installed on galvanized steel shims and horizontally braced until the girders could be installed. This was to ensure the bearings could be adjusted to account for variations in the girder shape. The permitted tolerances for the bearing/girder connection were set at no more than 5 mm in each direction so that the dead and superimposed loads were transferred properly through the girder diaphragms and into the bearings. After the girders were installed and final adjustments were made to the bearing locations, the bearings were grouted and fixed to the substructure units.

FIGURE 12: POT BEARINGS PRIOR TO GIRDER ERECTION AND FINAL GROUTING

The girder bracing system consisted of vertical and lateral angle braces and diaphragms. The lateral bracing spanned from flange to flange of the individual girders as well as spanning between the two girder lines. This intricate bracing design added challenges to deck forming. Four layers of joists were used for deck forming because the top layer disconnected at each of the girder's 56 lateral braces. Overhangs, end concrete diaphragms, fascia, drip strips, girder flange haunches, screed machine rails also contributed to the complexity of the forming process. Foreseeing the challenges with forming the deck, formwork was prefabricated during a short window between substructure construction, and girder erection. The result of the prefabrication was a deck that was completely formed in four weeks, a feat remarked by all members of the project team considering the complexity of forming.

During erection and formwork installation, the girders were surveyed along the length of the girders to verify the girders had been fabricated with the required camber and were behaving as expected. Following formwork construction for the bridge deck, the stainless steel reinforcing was installed and the survey information was used to set the screed elevations for deck concrete placement. The barriers were installed and the deck was surveyed again to confirm the girder-deck behaviour and determine the final wearing surface thicknesses. Immediately prior to installation of the asphaltic wearing surface a hot poured rubberized asphalt waterproofing system was installed. Wick drains were provided along the perimeter of the deck to capture any water that may infiltrate through the asphalt overlay.

FIGURE 13: DECK CONCRETE PLACEMENT

3.3 Mechanically Stabilized Earth Walls

MSE walls are relatively new to Manitoba and the Kenaston Overpass is one of the largest applications in Manitoba to date. The MSE walls selected for the Kenaston Overpass were single-stage retaining walls made of precast concrete panels forming the face of the walls and galvanized steel reinforcing strips anchoring the panels into and reinforcing the backfill. An aesthetic treatment, inspired by the Aurora Borealis, was imprinted on the faces of the embankment walls and is shown in **Figure 14** below.

Modular MSE wall systems are highly susceptible to differential movement caused by expansion and settlement. At its highest point, the MSE wall is almost 9 m tall and long-term settlement on the underlying clay substrata was a major concern. As noted in **Section 2.2**, the loading of the MSE wall was made uniform along the length through the use of variable thicknesses of standard and lightweight backfill. However, during excavation of the lowest MSE wall section around the abutments, a layer of silt material was discovered. The silt material was deemed not appropriate for a base under the MSE wall and was excavated and replaced with compacted limestone. While the silt seam extended throughout the site, only the exposed portions near the abutments were of concern as the other sections of the MSE walls had a thick layer of consolidated clay protecting against water migration.

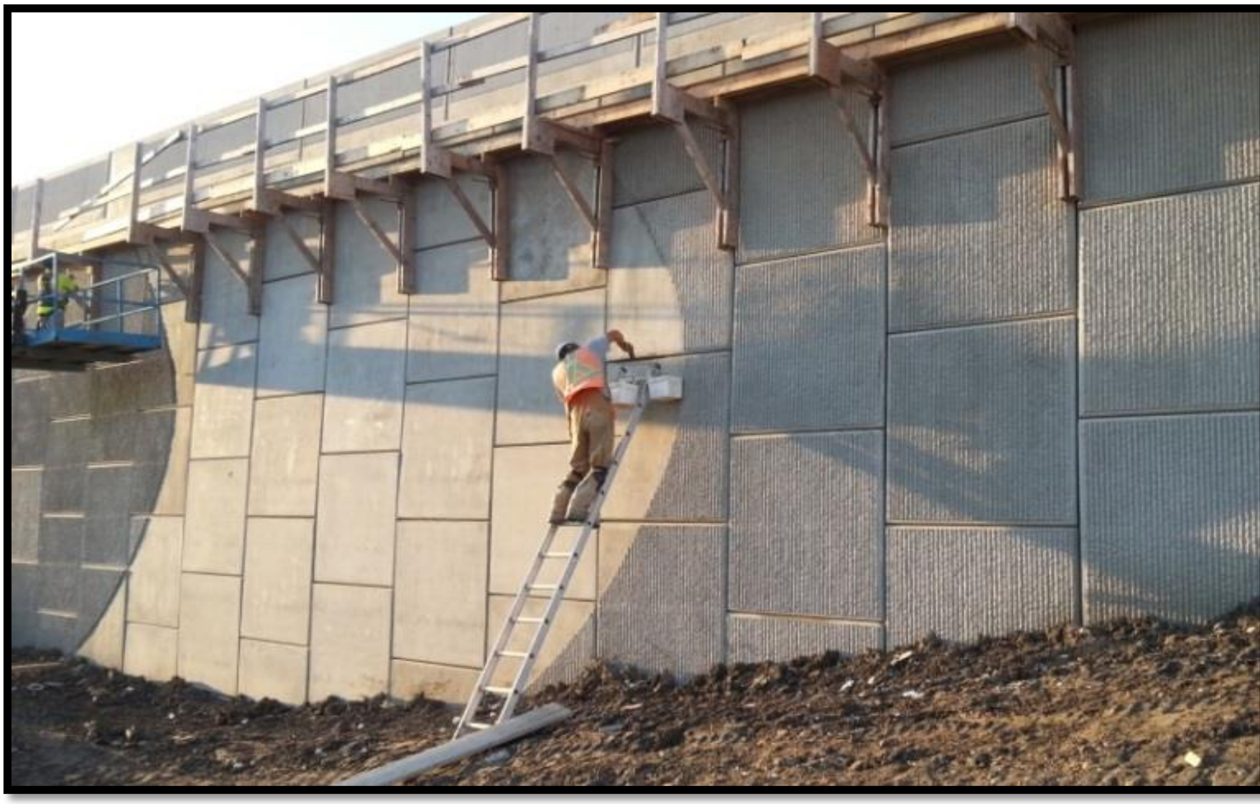

FIGURE 14: MSE WALL SHOWING AESTHETIC TREATMENT

Several fibre optic telecommunication cables and a high pressure gas line crossed the area where the MSE walls were to be constructed. During the design process the utilities were exposed to determine their exact locations and the MSE wall panel elevations were adjusted to avoid conflicts. However, extreme care was exercised to confirm the location of each of the utilities prior to excavation and MSE wall footing construction to ensure no conflicts would occur.

Over 900 panels were transported to site from the fabrication facility in British Columbia. Fabrication and delivery were staged to accommodate the progression of wall construction. Bracing panels for stability during construction was a challenge and required constant monitoring during panel placement and backfill operations. Further complicating the bracing of the panels was the vibrations from compaction equipment that caused outward deflection of the panels. For the panels that were backfilled with lightweight concrete, the hydrostatic pressure of the foam injected slurry also caused outward deflection of the panels. For both backfill types, the magnitude of deflection had to be predicted and panels were set out-of-plumb inwards to counteract deflections. **Figure 15** below shows the placement of the lightweight concrete fill and the bracing of the panels.

FIGURE 15: POURING LIGHTWEIGHT CONCRETE BACKFILL

4.0 PROJECT SUMMARY

Designed with sustainability and economic viability top of mind, Kenaston Overpass demonstrates the City's efforts to reduce infrastructure life-cycle costs and meet the increasing demands on infrastructure in a limited space. This project highlights industry technologies and techniques available to efficiently use resources, including land and funds, to meet Winnipeg's growing transportation needs.

FIGURE 16: KENASTON OVERPASS