APPENDIX A - NEW STRUCTURE CROSSING THE SHOAL LAKE AQUEDUCT AT MILE 93 PRELIMINARY DESIGN REPORT, JUNE 2013

City of Winnipeg
New Structure Crossing the Shoal Lake Aqueduct at Mile 93

Preliminary Design Report

June 2013

Submitted to:
City of Winnipeg

12-6029-1000

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Submitted by:
Dillon Consulting Limited

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APPENDICES

1 INTRODUCTION

The preliminary design for the new structure crossing the Shoal Lake Aqueduct (Aqueduct) at Mile 93 follows a preliminary design review of potential vehicle crossing locations by AECOM in 2010. In the design review Mile 93 was identified as the preferred location for a structure to be constructed crossing the Aqueduct. The preliminary design assignment included an initial site visit and topographic survey. A geotechnical investigation and analysis, preliminary design of the bridge structure and approach and preliminary design of the approach roadways were also completed. Regulatory approvals were not included in the preliminary design assignment.

The preliminary design of the new structure on crossing the Aqueduct proposes a 33.53 m (110 ft) single span ACROW panel bridge. The structure will need to span the Aqueduct without causing excessive stresses or deformations to the Aqueduct. The alignment of the structure is chosen to be perpendicular to the Aqueduct to minimize the length of the structure as well as to provide a clear line of sight to the Aqueduct and the Greater Winnipeg Water District (GWWD) rail line that is approximately 50 metres south of the Aqueduct.

This report presents the recommended structure alternative, including cost estimates, for the new structure crossing the Aqueduct at Mile 93.

2 GEOTECHNICAL INVESTIGATION

Dillon Consulting Limited (Dillon) retained TREK Geotechnical (TREK) to undertake a geotechnical site investigation, including groundwater conditions, to provide foundation recommendations, slope stability analysis, as well as stress and settlement analysis on the Aqueduct. An initial site investigation was carried out on February 17, 2012 with the subsurface investigation occurring on March 27, 2012.

The complete final geotechnical report is contained as an Appendix of this Preliminary Structural Design Report for ease of reference.

The following is a brief summary of the geotechnical investigation results.

2.1 Stratigraphy

Peat **–** A 0.5 m thick peat layer was encountered at surface in TH12-01. The peat is fibrous, fine, dark brown and wet. The peat is an H3 in degree of humification based on the Von Post peat classification system. The moisture content of one sample of the peat was 342%.

Alluvial Silts and Clays) **–** Interlaid alluvial silts and clays were encountered below the peat in TH12-01 to 7.6 m below ground surface (bgs). The alluvial soils contain trace sand, are light brown to grey and of low to intermediate plasticity.

Moisture contents range from 19 to 41% with an average of 26%. Bulk unit weights range from 17.3 to 22.0 kN/m^3 with an average of 20.4 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 11 to 53 kPa with an average of 36 kPa. The plastic limits from two samples of the clay are both 13% with liquid limits of 23% and 40%.

Lacustrine Clay **–** Lacustrine clay was found underlying the alluvial soils to a depth of 24.4 m bgs. The clay is silty, contains trace gravel, is grey, moist and of high plasticity.

Moisture contents range from 23 to 35% with an average of 32%. Bulk unit weights range from 18.6 to 19.2 kN/m^3 with an average of 19.0 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 28 to 51 kPa with an average of 42 kPa. The plastic and liquid limits from one sample of the clay are 15 and 51%, respectively.

Sand **–** A sand layer was found underlying the lacustrine clay to from 24.4 to 26.8 m bgs. The moisture content of one sample from the sand and gravel was 14%. Within the sand layer, a cobble, approximately 150 mm in diameter, was cored through at 26.4 m bgs.

Silt (Till) **–** Silt (till) was encountered below the sand from 26.8 to 28.3 m bgs. The silt (till) is dense and contains some sand, some gravel, and is grey. The moisture content of one sample was 10%.

Bedrock **–** Bedrock was encountered at 28.3 m bgs (Elev. 296.7). The drilling was advanced 3.8 m into the bedrock. The bedrock is amphibolite, greenish grey in color, strong to very strong (R4 to R5) and homogenous. The upper 1.0 m of the bedrock is strongly fractured and weathered with a rock quality designation (RQD) of 55%. The lower 2.8 m of bedrock is intact with an average RQD of 97%.

2.2 Groundwater Conditions

A groundwater level of 0.9 m bgs was measured immediately after drilling TH 12-01 on the south side of the crossing. Sloughing was observed at 3.1 m bgs during drilling. It is important to recognize that the measured groundwater levels should be considered short-term and may vary seasonally, after heavy precipitation events or as a result of construction activities. Seepage from the near surface organic soils can also be expected. Groundwater levels on the north side of the Aqueduct may be different than observed on the south side at TH 12-01 and should be confirmed prior to completing the detailed design.

2.3 Slope Stability Analysis

Slope stability analysis was completed for the proposed Mile 93 bridge geometry provided by Dillon. The preliminary assumptions included an earth fill approach embankment and concrete abutments (pile supported). The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Slip surfaces were specified with the grid and radius method, with factors of safety calculated using the Morgenstern-Price method of slices. Groundwater conditions were modelled using piezometric lines.

2.3.1 Model Geometry

The model geometry is based upon the topographic survey information collected by Dillon on October 12, 2012 supplemented with ditch inverts from ice auger soundings carried out during the initial site reconnaissance. The water level in the ditch of the SLA crossing is based on the top of ice level obtained in the Dillon October 12, 2012 survey. The cross section is taken just outside of the abutment where the fill is at a maximum height. The preferred layout has the middle of the bridge shifted to the south of the Aqueduct centerline and as a result, the north abutment is about 6 m closer to the Aqueduct than the south abutment.

2.3.2 Soil Properties and Groundwater Conditions

The soil parameters used in the slope stability analysis are based on the field and laboratory testing, the results of hand auger test holes in the backfill for the Aqueduct from previous studies (AECOM, 2010) and typical values for the nature of soils encountered. It was assumed that soil conditions are the same on the north side of the Aqueduct (in the vicinity of the north abutment) as determined on the south side during the sub surface investigation, in particular the near-surface soil unit (alluvial silt and clay).

In the vicinity of the proposed abutments, groundwater levels were assumed to be approximately at the base of the embankment fill, sloping towards the surveyed ice level in the ditches. Although this ground water level is higher than measured during drilling, it is considered representative of potential ground saturation due to seasonal changes and environmental effects.

2.3.3 Modeling Results

The factors of safety (FS) for potential slip surfaces (PSS) through the approach fill immediately adjacent to the abutment on both sides of the Aqueduct were determined for the original proposed bridge geometry. Any structural support provided by the piles and/or abutment was neglected in the analysis. Three key slip surfaces were examined, the slip surface with the minimum FS at the crossing (critical) which could negatively impact the bridge abutment, a slip surface that extends to the top of the aqueduct, and a slip surface that extends below the aqueduct. The latter two are considered potential slip surfaces that could impact the integrity of the Aqueduct.

Modelling of the originally proposed bridge geometry resulted in an estimated FS for the critical slip surfaces on the north and south sides of the crossing of 1.41 and 1.45, respectively. The following modifications were then incorporated into the model to achieve the target FS:

- Increase the depth of granular fill around the abutments to improve soil strength and reduce groundwater levels in the vicinity of the abutment; and
- Construct wing walls behind the abutments to offset fill loading away from the top of riverbank. This was accomplished by analyzing wing wall lengths which are considered practical of 3, 4 and 5 m.

The modelling with the proposed modifications and with a 4 m long wing wall on the north side of the crossing resulted in an estimated FS for the critical slip surface of 1.50.

To account for potential variability in soil conditions on the north side of the Aqueduct, further analysis was carried out assuming clay (rather than silt) in the upper soil horizon (top 7.5 m). This assumption lowers the FS below the target of 1.5 by about 10%. This result reinforces the need to confirm near surface soil and groundwater conditions at the north abutment prior to completing the detailed design.

2.4 Stress and Settlement Analysis

A stress-deformation analysis was completed to evaluate the stresses that may be imposed on the Aqueduct structure and associated settlements as a result of bridge construction. The cross section geometry used in the analysis was taken through the centre of the approach fill on both the north and south sides. The stress analysis was completed using a stress-deformation finite element model (Sigma/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Deformations were modelled using linear elastic constitutive soil models. Soil properties used in the analysis were based off measured values or were assumed based on typical values used for similar soil types.

2.4.1 Stress Analysis

The Aqueduct structure was modelled both as a rigid member (no displacement allowed) and as a free moving member. The model assumes 1.2 m of clay backfill at the Aqueduct base with peat backfill to surface based on previous investigations at Mile 92.99 (UMA, 1994). The estimated increase in stress in both the horizontal (x-direction) and vertical (y-direction) direction were then determined at various locations (nodes) along the outside surface of the structure.

The maximum stresses in the horizontal and vertical directions are 9 and 3 kPa respectively. For comparison, maximum stress changes in the order of 23 (horizontal) and 14 kPa (vertical) were estimated by TREK in 2010 as part of a conceptual evaluation of a bridge crossing carried out for AECOM in May 2010. The 2010 modelling, however; assumed a shorter bridge span with approach fills in closer proximity to the Aqueduct. In both the 2010 and 2013 modelling, the maximum horizontal and vertical stress increases occur at the outside edges of the base of the structure (invert). If the estimated stresses are greater than what can be tolerated by the structure, a more rigorous analysis should be carried out during detailed design. Additionally, options to reduce the loading from proposed fills, such as lightweight fill or increasing the setback distance of the abutments could be investigated.

2.4.2 Settlement Analysis

Consolidation settlement of the soils beneath the approach fills can be expected although it will take a number of years for the settlement to occur due the fine grained nature of the soils on site. The largest settlement magnitudes will be immediately beneath the maximum fill heights and will dissipate with increasing distance away from the fill. Settlement of the approach fills can likely be accommodated in the bridge design, however; any associated settlement of the soil beneath the Aqueduct must be within an acceptable range for the structure. In this regard, a l-dimensional (l-D) analysis was carried out to predict consolidation settlements under the north and south abutments and under the centre of the Aqueduct using the maximum vertical stresses estimated from the finite element model. From this analysis, the estimated settlement under the aqueduct is 10 mm.

In the event that this magnitude of settlement at the abutment locations cannot be accommodated by regular maintenance (e.g. asphalt overlays at the bridge approaches) techniques to accelerate consolidation settlement such as preloading or the installation of vertical drains may be considered. If the estimated settlements of the Aqueduct are greater than what can be tolerated by the structure, options to reduce the loading from proposed fills, such as lightweight fill or increasing the setback distance of the abutments should be investigated.

2.5 Foundation Considerations

The soil conditions encountered at the Aqueduct crossing location make cast-in-place concrete friction piles and driven steel piles end bearing on the bedrock viable foundation options. If cast-in-place concrete friction piles do not provide sufficient resistance for the anticipated loads, driven steel end bearing piles should be used. Due to the sloughing and groundwater conditions encountered during drilling, it is likely that cast-in-place concrete piles end bearing in the till or bedrock is not a viable option as full length sleeving would be required to maintain an open hole.

2.5.1 Limit States Design

Limit states design requires consideration of distinct loading scenarios and prescribes resistance factors (reduction factors) that are based upon the method used to evaluate pile capacity. The ultimate bearing capacity values for the soils at the site need to be factored using resistance factors as defined in the 2010 Canadian Highway Bridge Design Code (CHBDC). The ultimate pile capacities are to be multiplied by the appropriate resistance factors to establish the Ultimate Limit State (ULS) pile capacity, which can be compared against the ULS (factored) load combinations defined for the structure. The Service Limit State (SLS) is concerned with limiting the deformation or settlement of the foundation under static loading conditions such that the integrity of the structure will not be impacted by comparing SLS (unfactored) structural loads to the SLS pile capacity.

2.5.2 Cast-in-Place Concrete Friction Piles

ULS and SLS geotechnical resistances are provided in the geotechnical report for cast-in-place friction piles for the structure crossing the Aqueduct. Adhesion within the upper 2.5 m of the pile should be ignored to take into consideration potential shrinkage and environmental effects such as frost action over that depth. Shaft support within any fill materials should also be ignored. A minimum pile length of 8 m below ground surface is recommended for straight shaft piles to protect against frost jacking.

Additional Design and Construction Recommendations

Additional design and construction recommendations for cast-in-place concrete piles are provided below:

1. The weight of the embedded portion of the pile may be neglected.
2. The contribution from end bearing should be ignored.
3. Based on observed conditions sleeving of pile holes may be necessary. If seepage and sloughing conditions are observed during shaft drilling the holes should be sleeved.
4. Drilling and concrete placement for the piles should be inspected by geotechnical personnel to verify the soil conditions and proper installation of the piles.
5. Prior to casting the pile, any groundwater within the shaft should be removed or controlled.
6. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre.
7. Once the pile spacing, length and layout of pile groups are known, the foundation system should be evaluated to determine if pile group effects are applicable.
8. All cast-in-place piles require reinforcement design by a qualified structural engineer for the anticipated axial, lateral and bending loads from the structure.

2.5.3 Driven Steel Piles

Piles driven to refusal on the bedrock are considered a viable option for support of bridge abutments at the proposed Aqueduct crossing. It is anticipated that piles can be driven through the clays and tills to the underlying bedrock at each crossing location. At the Aqueduct crossing, the presence of cobbles within the sand layer above the bedrock may create some installation difficulties; there is a risk of reduced capacity resulting from shallow refusal or the need for a replacement pile(s). The ULS design criteria outlined in the CHBDC (Clause C10.22.2) present three resistance factors that should be considered when driving steel piles. Due to the nature of driving steel piles to refusal on bedrock, all three resistance factors should be used for the ULS design case. The product of all three results in a resistance factor of 0.5 (rounded).

Refusal criteria and load capacity for specific piles should be established once the pile sizes and driving method are known in order to verify that the geotechnical and structural capacity has been adequately addressed to minimize the potential for pile damage during driving. Driving should proceed under careful observation near bedrock to avoid overdriving the pile, which could lead to pile damage or misalignment. It is common for bedrock in these areas to slope significantly. In the event that it appears that piles are sliding on bedrock during construction, misalignment and pile damage could occur. Where this occurs, driving should be discontinued to avoid further misalignment of the pile, and an assessment made of the pile capacity and anticipated performance. Where the pile capacity is found to be insufficient to support the design loads, additional piles may be required.

The following additional recommendations regarding steel piles are provided.

1. The allowable capacities noted pertain to geotechnical resistance only. The pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.
2. The weight of the embedded portion of the pile may be neglected in design.
3. If drop hammers are used, the drop hammer should have a minimum mass equivalent to three times the mass of the pile.
4. The driving of all piles should be documented and approved by qualified geotechnical personnel.
5. Pile spacing should be a minimum of 2.5 pile diameters measured centre to centre.
6. All piles driven within 5 pile diameters of one another should be monitored for heave and where heave is observed the piles should be re-driven to the specified refusal criteria.
7. All piles should be fitted with rock points (driving shoes) to reduce potential damage to the toe of the pile when driving through cobbles or boulders onto bedrock.
8. Driven steel piles should extend a minimum of 8 m below grade to resist adfreezing forces.
9. During the final set, piles should be driven continuously once driving is initiated to the required refusal criteria.
10. A steel follower should not be used for driving of steel piles.

2.5.4 Lateral Pile Capacity

The lateral loads for the bridges will be accommodated by using battered piles. Additional recommendations or detailed lateral pile analysis should be determined if lateral pile capacity needs to be assessed.

2.6 Excavations and Shoring

All excavations must be carried out in compliance with the appropriate regulation(s) under the Manitoba Workplace Safety and Health Act. Flattening of open excavation side slopes may be required, in particular if saturated soils are encountered. Gravel buttresses could be used to prevent wet silts from flowing into excavations, in conjunction with sump pits used to dewater the excavation.

2.7 Recommendations

- A hand auger test hole should be completed on the north side of the Aqueduct crossing to confirm the presence of alluvial silts and clays and to establish the alluvial soils/lacustrine clay contact elevation. A piezometer should also be installed in the hand augured test hole to confirm the groundwater levels used in the stability analysis. Should it be considered necessary to confirm the depth to bedrock at the north abutment, it may be preferable to mobilize a drill rig once the road on the north side of the Aqueduct ROW has been cleared.
- For any pile driving, it is recommended that Pile Dynamic Analyzer (PDA) be used during driving to verify that calculated pile capacities for each pile are developed.
- Side slopes are shown on the drawings as 4:1 for the approach roadway embankments. Roadway embankment side slopes to be confirmed during detailed design.

3 DESIGN CRITERIA

3.1 Geometrics

The location of the structure crossing the aqueduct was determined in the preliminary design report completed by AECOM due to minimal rock outcrops in this location. The proposed approach roadways are shown on Drawing No. 1 in Appendix A. The elevation of the structure was dictated by the elevation of the GWWD Rail Line located approximately 50 meters south of the Aqueduct. In order to reduce the risk of a vehicle running off the road and potentially causing damage to the aqueduct, a minimal grade was utilized to maintain vehicle stability at the structure.

As the width of the structure will only permit one vehicle crossing at a time, it is recommended that a stop sign be utilized to avoid potential conflicts on the structure. The stop sign should be placed a minimum of 25 m from the structure to allow vehicles to pass on the opposing side of the structure. In addition it is recommended that a "Narrow Structure" sign with a supplementary "1 Lane" sign (WA-24 and WA-24S respectively, as per the Manual for Uniform Traffic Control Devices) be installed in close proximity to the stop sign.

With these conditions in place, the following design criteria were utilized for the alignment of the bridge approaches.

- Design Speed = 40 km/hr
- Maximum Superelevation = 0.06 m/m
- Lane Width = 4 m
- Typical Cross Slope = 3%

It is recommended that the crossing of the GWWD Rail Line meet the requirements of RTD 10 – Road/Railway Grade Crossing Technical Standards and Inspection. Additional clearing may be required south of the crossing to allow for the appropriate sight distance between the vehicles and trains.

Further clearing and subsequent ground proofing along the approach roadways north and south of the aqueduct crossing are required to determine the profile of the existing ground and verify the alignment and profile of the approach roadways at the crossing location.

3.2 Loading

The new structure will be designed in accordance with the following:

- AASHTO LRFD Bridge Design Specifications (latest edition);
- 25 year design life; and
- Loading to HSS30 and AASHTO HL-93 Design Vehicles.

3.3 General Arrangement Drawing

The General Arrangement of the proposed Aqueduct crossing at Mile 93 is shown on Drawing No. 2 in the Appendix A.

4 UTILITIES

4.1 Existing

The new structure will cross the Aqueduct at Mile 93. The Aqueduct will require to be protected from stresses imposed during construction. Also, concrete barriers should be installed along the North and South approach roadways to protect the Aqueduct from errant vehicles.

4.2 Proposed

At this time, there are no proposed utilities planned to be installed near the proposed crossing location by the City of Winnipeg.

5 SUBSTRUCTURE ALTERNATIVES

5.1 General

As the proposed structure is a clear span over the Aqueduct only abutment substructures were considered. The choice of substructure units depends, at least partly; on the choice of superstructure. Several basic abutments were considered for the new structure.

5.2 Abutments

Shelf, semi-integral, and integral abutments are all potential abutment types that could be used with the proposed structure span of 33.38 m. Shelf type abutment is recommended due to the remoteness of the site as well as the recommended ACROW panel superstructure. A shelf type abutment is the least complex and will require the least amount of time to construct.

A reinforced concrete shelf-type abutment would consist of a concrete pile cap extending up to the bearing seat. The abutment would include a timber backwalls and wingwalls to contain approach fill, with steel H-piles supporting the timber wingwall.

The principal advantages of this type of abutment is the ease of construction and the stability it provides against lateral loads. The large concrete pile cap along with the battered toe piles provides excellent resistance to backwall pressures. The main disadvantage of this alternative is the increased cost since more concrete is required. This cost would be offset by all of the foundation concrete can be placed at one time.

5.3 Abutment Foundation

The recommended foundation support for the shelf-type abutment is two rows of HP 310 x 132 steel H-piles driven to refusal. The front row of the piles will be battered to resist lateral force. Refusal is anticipated at elev. 296.7 m±; therefore the pile lengths required will be 28.3 m±.

6 SUPERSTRUCTURE ALTERNATIVES

6.1 General

The following superstructure alternatives were evaluated for the new structure crossing the Aqueduct.

- Structure Steel Plate Girders;
- Precast Prestressed Concrete I – Girders;
- Cast-in-place Concrete Deck Slab;
- Precast Prestressed Concrete Box Girders; and
- ACROW 700XS Steel Truss.

Steel and concrete I-girder designs require more time and labour in order to construct a composite concrete deck on top of them. Also, both structural steel plate girders and precast concrete I-girders have a relatively deep superstructure when compared to a cast-in-place concrete deck slab, precast concrete box girder, or an ACROW 700XS steel truss bridge and would not allow as much access to the top of the Aqueduct. For these reasons, structural steel plate girders and/or precast concrete I-girder designs are not considered appropriate for the structures at this interchange.

A third superstructure alternative considered was a cast-in-place post-tensioned concrete deck slab. This alternative requires the least superstructure depth, but would need extensive falsework for construction which could result in significant stress on the Aqueduct. A cast-in-place post-tensioned concrete deck slab superstructure was not considered appropriate for the structure crossing the Aqueduct.

The fourth superstructure alternative considered was precast concrete box girders with a 150 mm composite reinforced concrete deck. This alternative has the advantages of a relatively shallow superstructure depth and the precast units are fabricated off-site, thereby reducing on-site construction and shortening the overall construction schedule. The main disadvantage of this option would be the transportation and erection of the concrete box girders to this remote site. Further, cast-in-place concrete curbs and steel guardrails would be required to be constructed at the site increasing the cost of the structure. For these reasons, the precast prestressed concrete box girder superstructure was not considered appropriate for a structure crossing the Aqueduct.

The final option considered for the Aqueduct crossing is the ACROW 700XS steel truss. Although the ACROW 700XS steel trusses are the deepest of all the proposed options, this superstructure has a relatively low structure depth below the top of the bridge deck of approximately 900 mm. The main advantage to the ACROW trusses is the fact that the trusses are constructed of steel components that are shipped by truck to the site. The trusses are then assembled on the approach embankment by bolting the components together and the bridge is then launched into place. The ACROW bridge also includes a timber deck curb and steel W-beam guardrail that are all connected to the trusses. The assembly and installation of the bridge and deck also provide opportunities for training local labourers and community development. Due to the reasons listed above, the ACROW steel truss is the recommended option for the new structure crossing the Aqueduct.

6.2 Other Elements

6.2.1 Traffic Barriers

Concrete barriers are recommended at each of the approaches to the bridge to prevent any errant vehicles from contacting the bridge or the Aqueduct. Steel W-beam guardrails and timber curbs are also recommended to be installed on the ACROW steel trusses to prevent vehicles from damaging the bridge structure.

6.2.2 Bearings

Both expansion and fixed bearings are provided by ACROW with the superstructure components.

6.2.3 Drainage

Drainage is provided through the joints in the timber deck and through the timber curb.

7 COST ESTIMATES

7.1 Basis of Cost Estimate

The basis of cost estimate for the recommended structure crossing the Aqueduct was based on a data from tendered ACROW bridge structures in remote locations for Manitoba Infrastructure and Transportation (MIT). The following sites were reviewed in the development of the estimate:

- God's Lake Narrows Bridge (MIT);
- Panko Narrows Bridge (MIT);

a) God's Lake Narrows Bridge (MIT)

This bridge was constructed in 2008 and is approximately 150 m (center line north abutment bearing to center line south abutment bearing) long and has a deck width of 6.325 m (out to out of chords). The substructure consisted of two cast-in-place concrete abutments and two cast-in-place concrete piers anchored into the existing bedrock. The superstructure consisted of a combination of ACROW Panels (DSR2, TSR2 and TDR3H types). The tendered price for the bridge was $4,776,388.00.

This equates to a structure cost of $5,035/m²$.

b) Panko Narrows Bridge (MIT)

This bridge was tendered in January 2013 and is scheduled to be completed in March 2014. The Panko Narrows Bridge is 61 m (center line north abutment bearing to center line south abutment bearing) long and has a deck width of 6.9 m (out to out of chords). The substructure consists of a granular embankment. The superstructure consisted of Acrow Panels (type DDR2H). The tendered price for the bridge was $1,969,699.00

This equates to a structure cost of $4,680/m^2

c) Summary costs/m^2

God's Lake Narrows	$5,035/m^2
Panko Narrows	$4,680/m^2

It should be noted that both the God's Lake Narrows and Panko Narrows bridges had shallow foundations which are less costly than the deep foundations which are required for the crossing over the Aqueduct. We estimate that the additional cost to construct the deep foundations will be approximately $525,000.

Based on the analysis of the above data, the recommended unit price cost estimate for the structure crossing the Aqueduct is $4,750/m^2.

7.2 Cost Estimate

The cost estimate for the new structure crossing the Aqueduct at a preliminary level are based on square meterage areas as follows:

(33.4 m x 6.9 m x $4,750) + $525,000 = <u>$ 1,620,000</u>

8 PROJECT SCHEDULES

8.1 Overall Project Schedule

The proposed project schedule, included in Appendix B, is based on our understanding that the City of Winnipeg is intending to proceed with this project and complete the construction of the crossing by September 30, 2014. This will allow the crossing to be in operation for the 2015 winter road season. The detailed design, including tender preparation, is scheduled to occur during June and July, 2013, with a proposed tender date of July 29, 2013. The tendering period would be during the month of August with an anticipated contract award date of August 26, 2013. Construction could start following the award however access to the site will be limited and will likely start following the completion of the winter road in January 2014. It is anticipated that the construction of the Aqueduct crossing will be completed by October 14, 2014.

8.2 Construction Schedule

The proposed construction schedule, included in Appendix B, is based on the assumption that the successful contractor will commence construction following the opening of the winter road in January 2014. It is estimated that the steel H pile installation, excavation of the frozen ground around at each substructure will occur during the month of February. The concrete works would then follow and would be completed by the end of March. The abutments would then be backfilled and the launch pad would be constructed to facilitate the assembly of the ACROW superstructure, including the timber deck and backwalls. The superstructure assembly and installation is anticipated to be complete by the end of April. The roadworks would likely commence in June, following the spring thaw, and would likely be completed by the end of June. Site clean-up is anticipated to be complete by the middle of July.

Although the proposed schedule shows the construction occurring from January to July 2014, the construction schedule may be shortened if the contractor chose to work multiple shifts each day or have numerous construction activities occurring simultaneously. This could potentially allow the construction to be completed prior to the winter road closing in spring 2014.

Alternatively, the construction could occur during two winter road seasons with a completion date of March 30, 2015. This would provide the contractor with almost twice the amount of time with vehicular access the site and the option to complete the work without having to keep the equipment at the site until the start of the winter road season in 2015. Providing the contractor this option may lead to a reduced construction price.

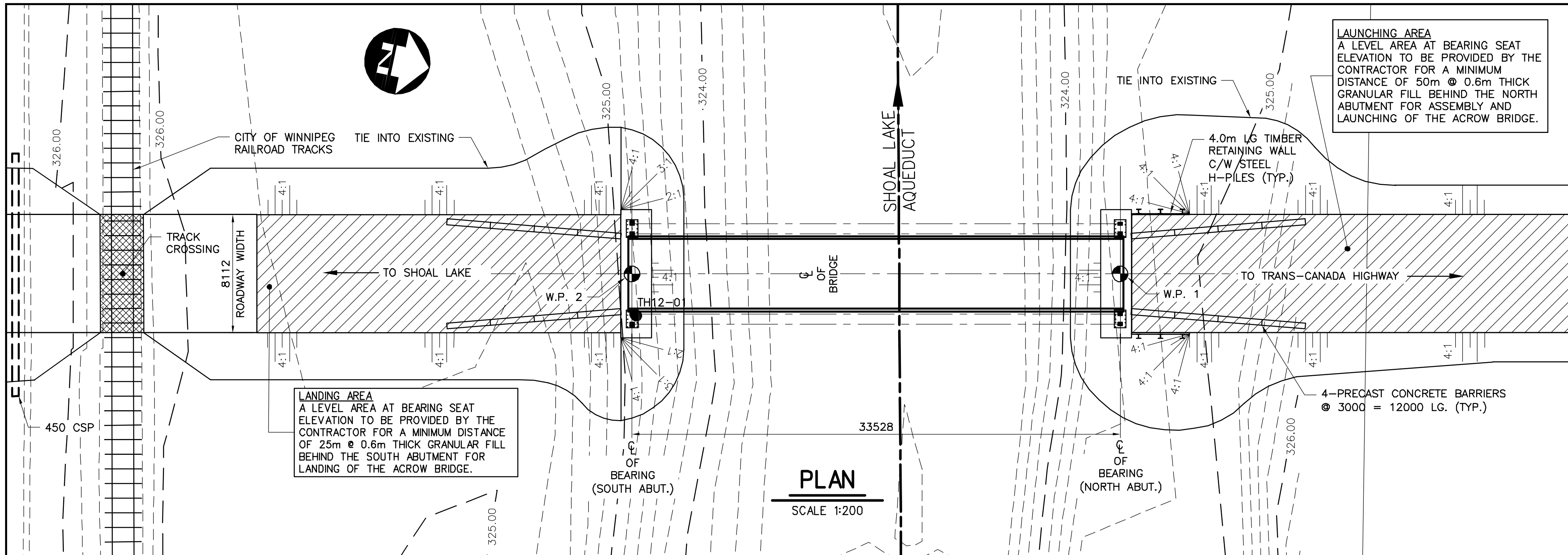
9 CONCLUSIONS

Based on our preliminary design study we have reached the following conclusions:

- Steel H piles with a cast-in-place concrete cap are the most suitable foundation alternative for a structure crossing the Aqueduct.
- An ACROW panel steel truss bridge with timber deck and backwalls is the most suitable superstructure alternative.
- The cost estimate for the construction of the new structure is $1,620,000.00 not including contingency, engineering fees for detailed design or contract administration, or city administration costs.

APPENDIX A

Drawings



PLAN

SCALE 1:200

LOCATION PLAN

NOTES:

CONSTRUCTION INFORMATION

- IN GENERAL ALL EFFORTS SHALL BE MADE TO LIMIT THE DISRUPTION OF NATURAL VEGETATION AND GROUND COVER.

1. BRIDGE ABUTMENTS:
   BOTH ABUTMENTS WILL BE ON STEEL PILES DRIVEN DOWN TO BEDROCK WITH A REINFORCED CONCRETE PILE CAP.
2. APPROACHES:
   THE APPROACHES SHALL BE CONSTRUCTED WITH CLEAN GRANULAR FILL, HAVE A TOP WIDTH OF 8.112m AND A TAPERING LENGTH.
3. SUPERSTRUCTURE:
   SUPERSTRUCTURE WILL BE A CLEAR SPAN ACROW PANEL BRIDGE AND WILL BE CONSTRUCTED WITHOUT DISRUPTING THE SHOAL LAKE AQUEDUCT.

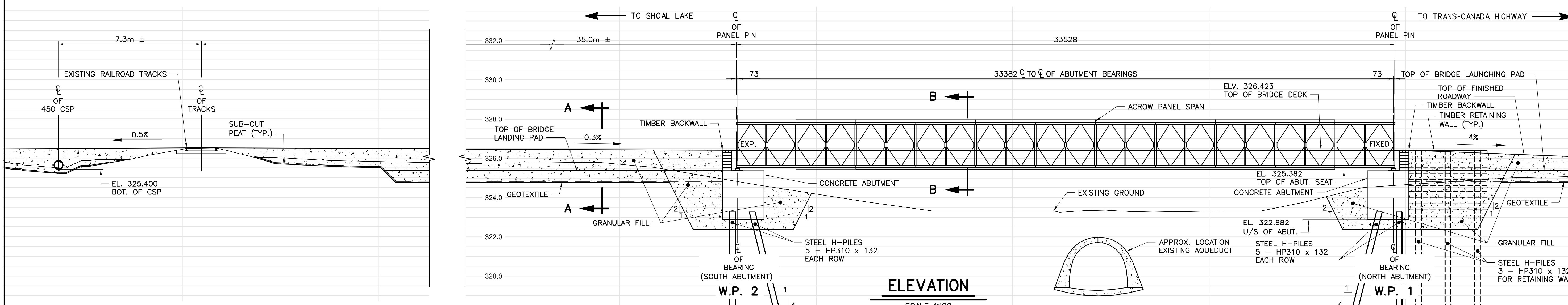
GPS INFORMATION:

COORDINATES PROVIDED TAKEN FROM SURVEY DATA TYING INTO THE SHOAL LAKE AQUEDUCT CONTROL SURVEY WHICH IS REFERENCED TO ZONE 14 U, ACTUAL LOCATION OF BRIDGE IS LOCATED IN ZONE 15 U.

- UTM CO-ORDINATES: 14U NORTHING = 5503438.74m, EASTING = 767632.66m
- LATITUDE = 49°37'26.83"N, LONGITUDE = 95°17'38.46W"

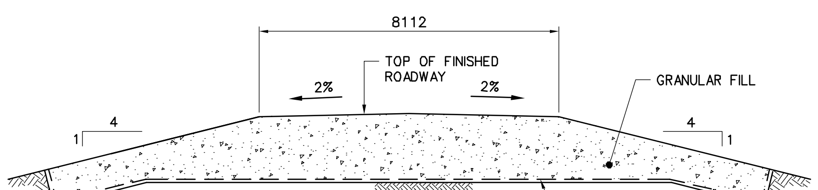
BRIDGE INFORMATION:

- BRIDGE DIMENSIONS: 33.528m LONG (110'), 5.334m WIDE (17'-6") INSIDE OF TRUSS TO INSIDE OF TRUSS.



ELEVATION

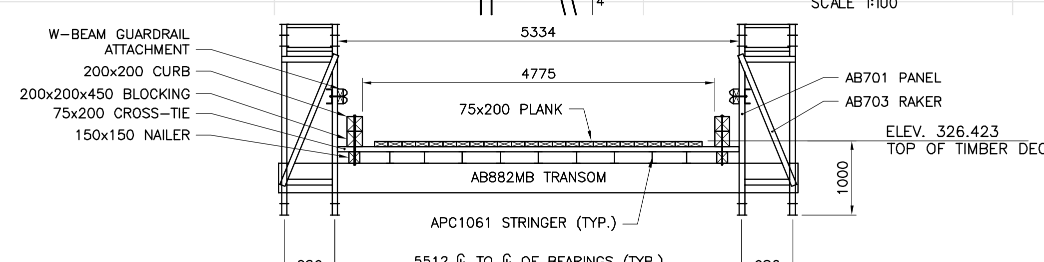
SCALE 1:100



SECTION A-A

SCALE 1:100

TYPICAL APPROACH CROSS-SECTION



SECTION B-B DSR2 SPAN

SCALE 1:50

DESIGN DATA

SPECIFICATIONS:

- AASHTO LRFD BRIDGE DESIGN SPECIFICATION 5th EDITION PLUS INTERIMS.
- AASHTO/AWS BRIDGE WELDING CODE D1.5M 2002.

VEHICULAR LIVE LOADING (WHICHEVER GOVERNS):

- MODIFIED AASHTO HSS 30.
- AASHTO LRFD "HL-93" TRUCK AND LANE LOAD.

MISCELLANEOUS METAL:

- ALL STRUCTURAL STEEL SHALL CONFORM TO CSA-G40.21 GRADE 300W UNLESS NOTED OTHERWISE.
- ALL REINFORCING STEEL SHALL CONFORM TO CSA-G30.18 GRADE 400W UNLESS NOTED OTHERWISE.

WORKING POINT CO-ORDINATES (UTM ZONE 14 U)

	EASTING	NORTHING	ELEVATION
W.P. 1	767629.189	5503455.142	325.382
W.P. 2	767636.130	5503422.341	325.382

TEST HOLE CO-ORDINATES (UTM ZONE 14 U)

TEST HOLE MK. NO.	"X"	"Y"
TH12-01	767638.87m	5503423.20m

NOTE:
TEST HOLE LOCATIONS HAVE BEEN SUPPLIED BY OTHERS AND DILLON CONSULTING DOES NOT GUARANTEE THEIR ACCURACY. REFER TO TREK GEOTECHNICAL REPORT FOR DESCRIPTION OF EACH TEST HOLE.

PRELIMINARY ONLY
NOT FOR CONSTRUCTION

NO.	REVISIONS	DATE	BY
1	ISSUED FOR PRELIM. DESIGN	03/27/13	GL

B.M. ELEV.

DESIGNED BY GL
DRAWN BY KB
CHECKED BY RE
APPROVED BY ML
HOR. SCALE
VERTICAL
DATE 03/27/13

DILLON CONSULTING

RELEASED FOR CONSTRUCTION
DATE

ENGINEER'S SEAL

CONSULTANT PROJECT NUMBER 12-6029

Winnipeg
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT

BRIDGE AT SHOAL LAKE AQUEDUCT
& ASSOCIATED ROADWORKS

BRIDGE GENERAL ARRANGEMENT

CITY DRAWING NUMBER

SHEET 2 OF 2

CONSULTANT DRAWING NUMBER 126029-SL-01

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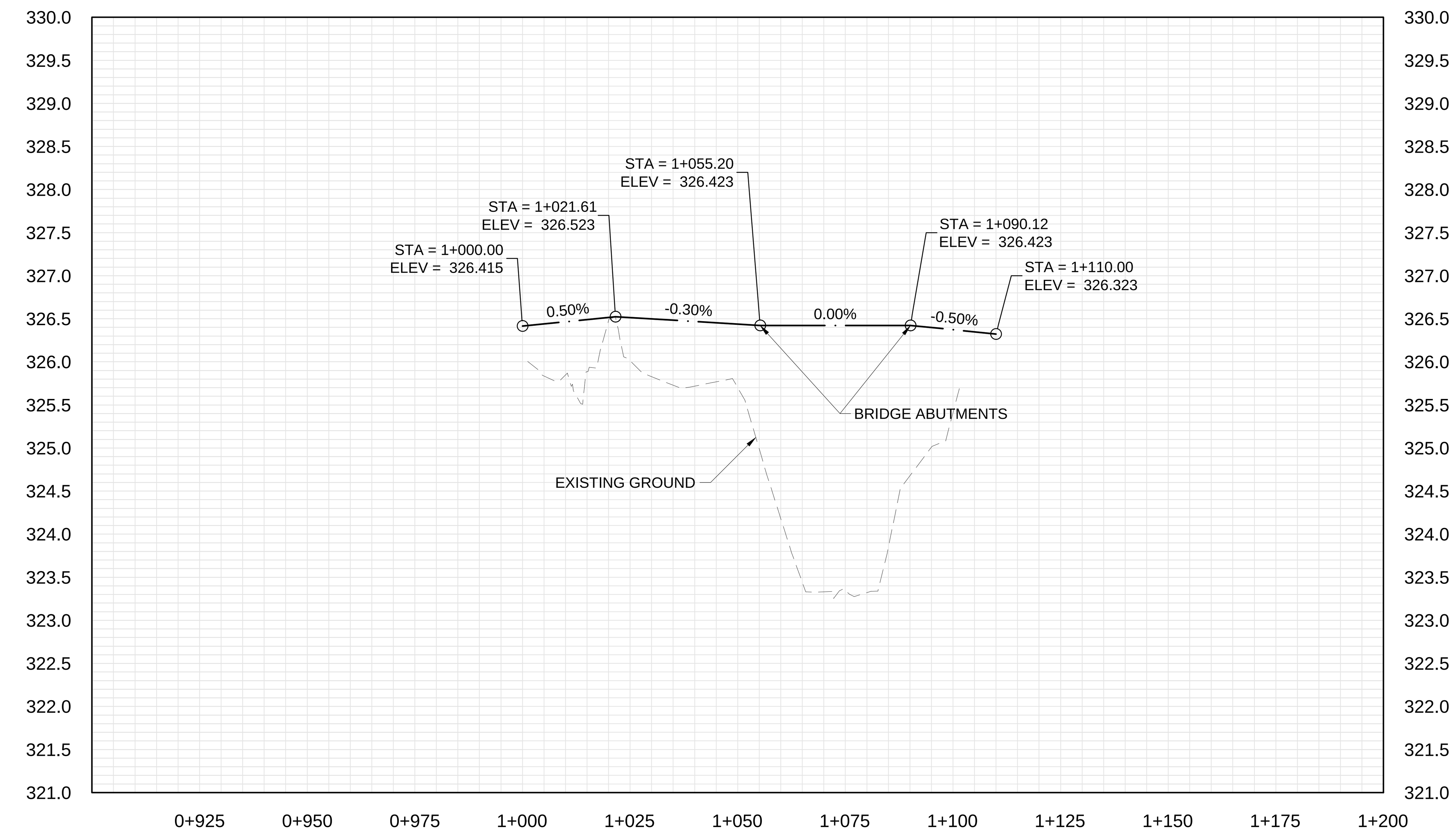

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PRELIMINARY ONLY
NOT FOR CONSTRUCTION

B.M.
ELEV.

NO.	REVISIONS	DATE	BY

DESIGNED BY	AMC
DRAWN BY	TJH
CHECKED BY	GCL
APPROVED BY	MBL
HOR. SCALE	1:1000
VERTICAL	1:20
DATE	

DILLON CONSULTING

RELEASED FOR CONSTRUCTION

DATE

ENGINEER'S SEAL

CONSULTANT PROJECT NUMBER
12-6029

Winnipeg

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT

SHOAL LAKE AQUEDUCT CROSSING

CITY DRAWING NUMBER

SHEET 1 OF 2

CONSULTANT DRAWING NUMBER

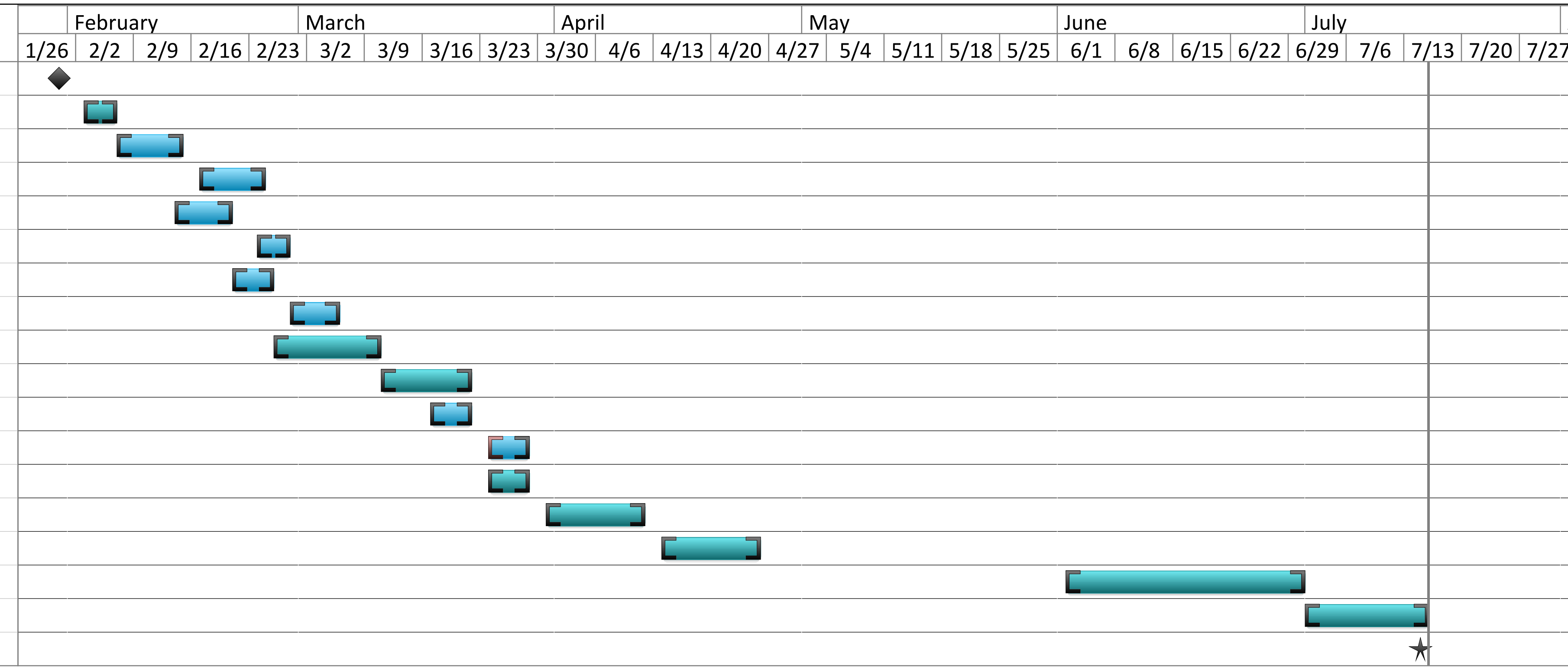
APPENDIX B

Schedules

DILLON
CONSULTING

Shoal Lake Aqueduct Bridge
Dillon Project No. 12-6029 Preliminary Construction Schedule

ID	Task Name	Start	Finish
1	Winter Roads Open	Fri 1/31/14	Fri 1/31/14
2	Mobilize to Site/Camp Start-up	Mon 2/3/14	Thu 2/6/14
3	Place H-Piles North Side	Fri 2/7/14	Fri 2/14/14
4	Place H-Piles South Side	Mon 2/17/14	Mon 2/24/14
5	Excavation North Side	Fri 2/14/14	Thu 2/20/14
6	Excavation South Side	Mon 2/24/14	Thu 2/27/14
7	Working Slab North Side	Fri 2/21/14	Tue 2/25/14
8	Working Slab South Side	Fri 2/28/14	Wed 3/5/14
9	Install Reinforcing & Place Concrete at North Abutment	Wed 2/26/14	Mon 3/10/14
10	Install Reinforcing & Place Concrete at South Abutment	Tue 3/11/14	Fri 3/21/14
11	North Abutment Dampproof & Backfill	Mon 3/17/14	Fri 3/21/14
12	South Abutment Dampproof & Backfill	Mon 3/24/14	Fri 3/28/14
13	Launchpad Construction	Mon 3/24/14	Fri 3/28/14
14	Acrow Bridge Construction	Mon 3/31/14	Fri 4/11/14
15	Timber Deck & Back Walls	Mon 4/14/14	Fri 4/25/14
16	Roadworks	Mon 6/2/14	Mon 6/30/14
17	De-Mobilization & Site Cleanup	Tue 7/1/14	Tue 7/15/14
18	Completion	Tue 7/15/14	Tue 7/15/14



Shoal Lake Aqueduct and Falcon River Diversion Bridges
Dillon Project No. 12-6029 Proposed Project Schedule

ID	Task Name	Start	Finish
1	Detailed Design	Wed 5/29/13	Mon 7/29/13
2	Tender Preparation	Mon 7/1/13	Mon 7/29/13
3	Issue Tender	Mon 7/29/13	Mon 7/29/13
4	Pre-Tender Meeting	Mon 8/5/13	Mon 8/5/13
5	Tender Close	Mon 8/12/13	Mon 8/12/13
6	Pre-Award Meeting	Mon 8/19/13	Mon 8/19/13
7	Contract Award	Mon 8/26/13	Mon 8/26/13
8	Pre-Construction Meeting	Mon 9/9/13	Mon 9/9/13
9	Material Procurement	Mon 9/9/13	Thu 10/10/13
10	Construction	Mon 9/9/13	Tue 9/16/14
11	Substantial Performance	Tue 9/30/14	Tue 9/30/14
12	Total Performance	Tue 10/14/14	Tue 10/14/14