APPENDIX B – NEW STRUCTURE CROSSING THE FALCON RIVER DIVERSION PRELIMINARY DESIGN REPORT

City of Winnipeg
New Structure Crossing the Falcon River Diversion

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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TABLE OF CONTENTS

Page No.

APPENDICES

1 INTRODUCTION

Dillon Consulting Limited (Dillon) was retained by the City of Winnipeg to complete the preliminary design for the new structure crossing the Falcon River Diversion (Diversion). Prior to this assignment, a winter road clearing was previously constructed on either side of the Diversion for a proposed temporary bridge. The winter road geometry was assessed to determine if the existing winter road and crossing location could be used for the proposed crossing and approach roadways. A geotechnical investigation and analysis, preliminary design of the bridge structure, and preliminary design of the approach roadways were also completed. Hydraulic analysis and regulatory approvals were not included in the preliminary design assignment.

The preliminary design of the new structure on crossing the Diversion proposes a 33.53 m (110 ft) single span ACROW panel bridge. Four potential roadway alignment options were developed and presented to the City of Winnipeg. The preliminary design proceeded with the preferred option chosen by the City of Winnipeg. The alignment of the structure is chosen to be perpendicular to the Diversion to minimize the length of the structure. Approach embankments will be required to provide necessary navigation clearances for the proposed structure. Rip rap placement in the channel, restoring the channel to its original cross section, and timber retaining walls at each abutment are also proposed to address the slope stability of the Diversion at the crossing location.

This report presents the recommended structure alternative including cost estimates for the new structure crossing the Falcon River Diversion

2 GEOTECHNICAL INVESTIGATION

Dillon retained TREK Geotechnical (TREK) to undertake a geotechnical site investigation, including groundwater conditions, to provide foundation recommendations, and slope stability analysis. An initial site investigation was carried out on February 17, 2012 with the subsurface investigation occurring on October 24, 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

Alluvial Silts **–** Alluvial silt was encountered at surface in THI2-02 to 4.6 m bgs and to the end of hole in TH12-03. The silt is clayey, contains trace fine sand and trace gravel, is light brown, moist and of intermediate plasticity.

Based on measured undrained shear strengths the silt is firm to stiff with a trend of decreasing shear strength with depth. Moisture contents range from 17% to 42% with an average of 25%. Average bulk unit weights are 19.6 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 34 to 52 kPa with an average of 46 kPa.

Lacustrine Clay **–** Lacustrine clay was found underlying the silt to a depth of 12.8 m below surface. The clay is silty, contains trace fine sand and trace gravel, is grey, moist, soft to firm and of intermediate to high plasticity.

Moisture contents range from 27% to 69% with an average of 41%. Bulk unit weights range from 15.3 to 18.8 kN/m3 with an average of 17.5 kN/m3. Based on unconfined compression tests, undrained shear strengths range from 18 to 46 kPa with an average of 32 kPa with a trend of decreasing shear strength with depth. The plastic and liquid limits from one sample of the clay were 12% and 41%, respectively.

Silt (Till) **–** Silt (till) was encountered below the clay to 15.1 m below surface. The silt (till) is clayey, contains trace sand, trace gravel, is grey, moist, soft and of intermediate plasticity. The moisture content of one sample from the silt (till) was 27%. Sample recovery from the lower portion of the silt till was not possible due to the drilling method (NQ coring).

Bedrock **–** Bedrock was encountered at 15.1 m below surface (Elev. 309.0). The drilling was advanced, 2.6 m into the bedrock. The bedrock is amphibolite, greenish grey in color, strong to very strong (R4 to R5) and homogenous. The bedrock is intact with an average RQD of 95%.

2.2 Groundwater Conditions

The groundwater level was at 2.9 m after completion of drilling in THL2-O2, respectively. No seepage was observed in TH12-03. Minor sloughing was observed in the silt (till) in TH12-02 but the hole remained open to 14.5 m bgs after completion. 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. Groundwater levels on the north side of the channel may also vary and should be confirmed prior to completing the detailed design.

2.3 Slope Stability Analysis

Slope stability analysis was completed for the proposed 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 and Geometry

The model geometry is based upon the topographic survey information collected by Dillon on October 12, 2012. The cross section is taken just outside of the abutment where the fill is at a maximum height. The water level in the channel is based on the top of ice level obtained in the Dillon October 12, 2012 survey. The preferred layout has the 33.4 m long bridge centered on the channel with east and west abutments set back about 3 m from the existing top of bank.

2.3.2 Soil Properties and Groundwater Conditions

The soil parameters used in the analysis are based on the field and laboratory testing and the nature of the soils encountered. It was assumed that soil conditions are the same on the west side of the channel as determined on the east side during the sub surface investigation, in particular the near-surface soil unit (alluvial silt and clay). A friction angle of 22^{o} was assumed based on the appreciable silt and sand content.

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 water level in the channel. These levels are higher than those observed during drilling, however; they reflect the potential for the ground to be saturated. Groundwater levels on both sides of the channel at the new crossing location should be verified prior to completing the detailed design.

2.3.3 Modeling Results

The Factors of Safety (FS) for potential slip surfaces through the approach fill immediately adjacent to the abutment on both sides of the channel were determined. The critical slip surface is representative of one that potentially could affect the bridge abutments which is also the slip surface with the minimum FS for the cross-section analyzed. A minimum FS of 1.5 was targeted for the critical slip surface. Modelling of the originally proposed bridge geometry resulted in calculated FS for the critical slip surfaces on the west and east sides of the channel of 1.22 and 1.24 respectively. The following modifications were then incorporated into the model to achieve the target FS of 1.5:

- Increase the depth of granular fill around the abutments to improve soil strength and lower groundwater levels in the vicinity of the abutments;

- Construct wing walls behind the abutments to offset fill loading away from the top of riverbank. In this regard, wing walls wall lengths of 3, 4 and 5 m were considered practical;
- Extend the proposed rip rap and adjust the thickness of the blanket for additional toe support and scour protection.

The modelling with the proposed modifications and with a 4 m long wing wall on both sides of the channel resulted in an estimated FS for the critical slip surface of 1.5. Once a final crossing location and elevation has been determined, the design should be optimized based on site specific geometry, and soil and groundwater conditions. For example, it may be possible to reduce the rip rap blanket thickness by incorporating a 5 m long (rather than 4 m) wing wall.

2.4 Foundation Considerations

The soil conditions encountered at the Diversion 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 are not a viable option as full length sleeving would be required to maintain an open hole.

2.4.1 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. 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.

2.4.2 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 Diversion. 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.

2.4.3 Driven Steel Piles

Piles driven to refusal on the bedrock are considered a viable option for support of bridge abutments at the proposed Diversion crossing. It is anticipated that piles can be driven through the clays and tills to the underlying bedrock at each crossing location. At the Diversion crossing location, steel piles driven to refusal on bedrock may be designed with an ULS capacity of 50% of the yield stress of the steel, multiplied by the cross sectional area of the steel. Steel piles driven to refusal on bedrock may be designed with a SLS capacity of 30% of the yield stress of the steel, multiplied by the cross sectional area of the steel.
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.

2.4.4 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.

2.5 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.

2.6 Recommendations

- Once the crossing location is established for the Diversion crossing, an additional topographic and bathymetric survey should be completed to confirm the crossing geometry used in the stability analysis.
- Two hand auger test holes should be completed at the new Diversion crossing to confirm the presence of alluvial silts and clays and to establish the alluvial soils/lacustrine clay contact elevation. Piezometers should also be installed in the hand augured test holes to confirm the groundwater levels used in the stability analysis.
- A deep (drill rig) test hole should be completed on one side of the new Diversion crossing location to establish the till and bedrock contact elevations.
- The hydraulic and environmental impacts of the proposed rip rap at the Diversion crossing should be considered in the detailed design.
- 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 as 4:1 on the drawings for the approach roadway embankments. Roadway embankment side slopes to be confirmed during detailed design.

3 DESIGN CRITERIA

3.1 Geometrics

The existing granular roadway that was previously constructed to allow for a temporary crossing of the Falcon Creek Diversion was analyzed to determine if it was suitable for the approach roadways. It was found that the alignment of the existing granular road did now allow for a perpendicular crossing of the diversion which would create a longer more costly structure. In addition, the curves required to use the existing crossing location would not meet a 40 km/hr design speed requirements due to the close proximity of Shoal Lake to the northeast of the crossing. It was determined that a large segment of the existing roadway would require re-alignment in order to cross perpendicular to the diversion at the existing road. Therefore, several alternative alignments were developed and presented to the City of Winnipeg. Following consultation with Shoal Lake No. 40 First Nation, the City of Winnipeg proceeded with the proposed approach roadways shown in Appendix A.

The proposed alternative was selected because it allows for a perpendicular crossing to the diversion on a tangent portion of the diversion, reducing the length of the structure. This alternative also increases the safety of the approach roadways by including longer roadway tangents approaching the structure.

As the width of the structure will only permit one vehicle crossing the diversion 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 meters 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%

Further clearing and subsequent ground proofing along the proposed approach roadways east and west of the Diversion 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 Falcon River Diversion crossing is shown on Drawing No. 2 in the Appendix A.

4 UTILITIES

4.1 Existing

There are no known utilities at near crossing location that would be impacted by the proposed structure that will cross the Falcon River Diversion. Concrete barriers should be installed along the east and west approach roadways to protect the Diversion 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 Falcon River Diversion 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 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 savings incurred by placing 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. 309.0 m±; therefore; the pile lengths required will be 15.1 m±.

6 SUPERSTRUCTURE ALTERNATIVES

6.1 General

The following superstructure alternatives were evaluated for the new structure crossing the Falcon River Diversion.

- 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 Diversion. 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 require extensive falsework constructed in the Diversion to facilitate the deck slab concrete placement. A cast-in-place post-tensioned concrete deck slab superstructure was not considered appropriate for the structure crossing the Diversion.

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 at the site. Further, cast-in-place concrete curbs and steel guardrails would be required to be constructed to this remote 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 Diversion.

The final option considered for the Diversion 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 easily connected to the trusses. The assembly and installation of the bridge and timber 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 Diversion.

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 entering into the Diversion. 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 Diversion 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/m^2.

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 Falcon River Diversion. We estimate that the additional cost to construct the deep foundations will be approximately $350,000.

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

7.2 Cost Estimate

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

(33.4 m x 6.9 m x $4,750) + $350,000 = <u>$ 1,440,000</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. The proposed tender date is July 29th, 2013. The tendering period would be during the month of August with an anticipated contract award date of August 26th, 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.

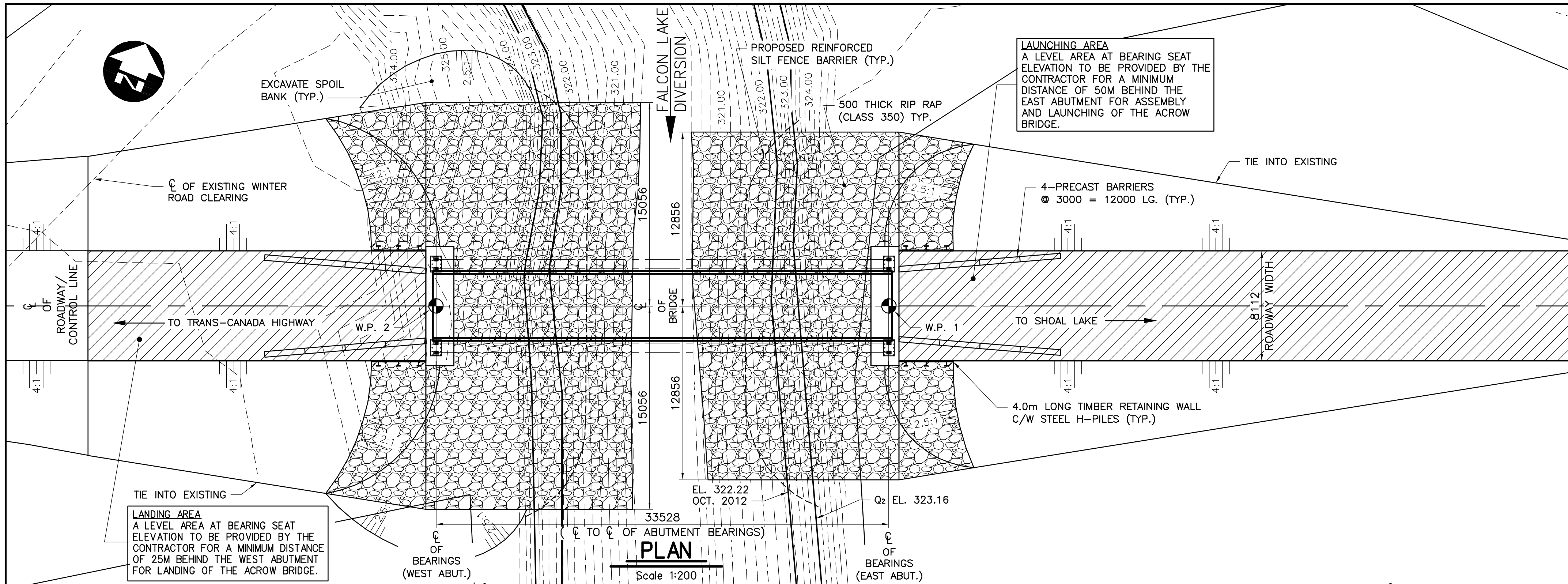
9 CONCLUSIONS

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 Diversion.
- 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 $1,440,000.00, not including; a contingency, engineering fees for detailed design or contract administration, or city administration costs.

APPENDIX A

Drawings



LOCATION PLAN

NOTES:

NAVIGABLE WATER INFORMATION

- HYDRAULIC ANALYSIS TO BE COMPLETED BY OTHERS.
- Q2 ELEVATION TO BE CONFIRMED BY OTHERS.
- DEPARTMENT OF FISHERIES AND OCEANS AND NAVIGATION CANADA APPROVALS REQUIRED.

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 H-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. RIP RAP WILL BE PLACED ON THE HEAD SLOPES TO PROTECT AGAINST POSSIBLE EROSION.
3. SUPERSTRUCTURE: SUPERSTRUCTURE WILL BE A CLEAR SPAN ACROW PANEL BRIDGE AND WILL BE CONSTRUCTED WITHOUT ENCROACHING ON THE STREAM BED.

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 = 5501744.24m, EASTING = 774691.29m
- LATITUDE = 49°36'20.66"N, LONGITUDE = 95°11'51.64W"

BRIDGE INFORMATION:

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

DESIGN DATA

SPECIFICATIONS:

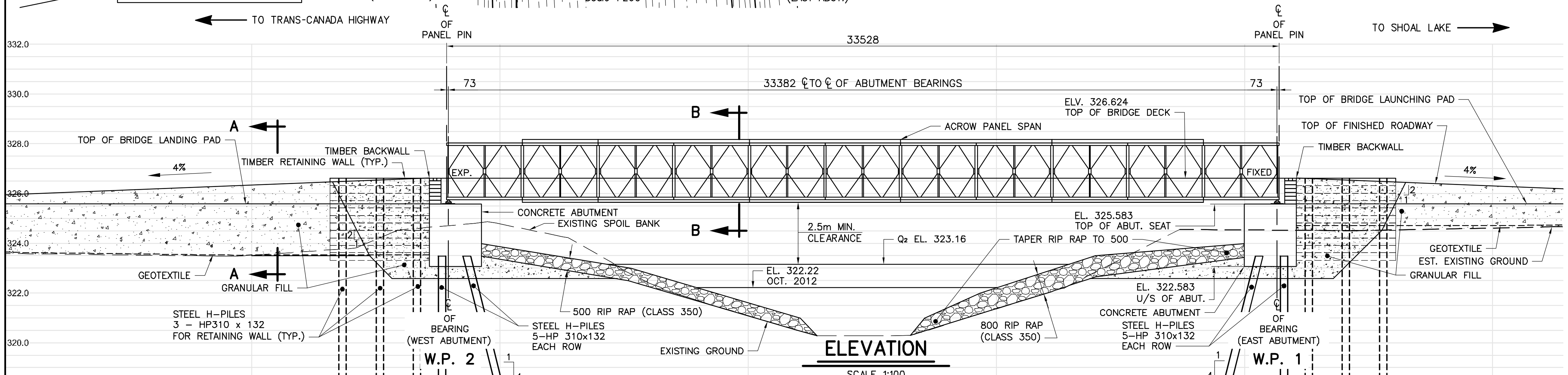
- AASHTO LRFD BRIDGE DESIGN SPECIFICATION 5th EDITION PLUS INTERIMS AASHTO/AWS BRIDGE WELDING CODE D1.5M 2002.

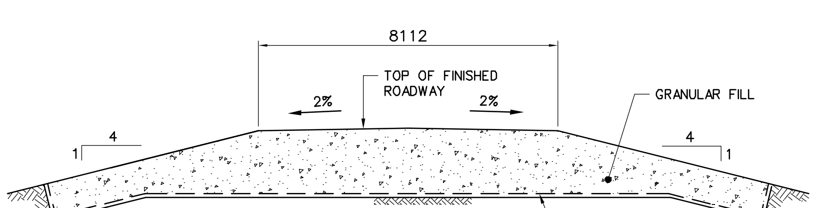
VEHICULAR LIVE LOADING (WHICHEVER GOVERNS):

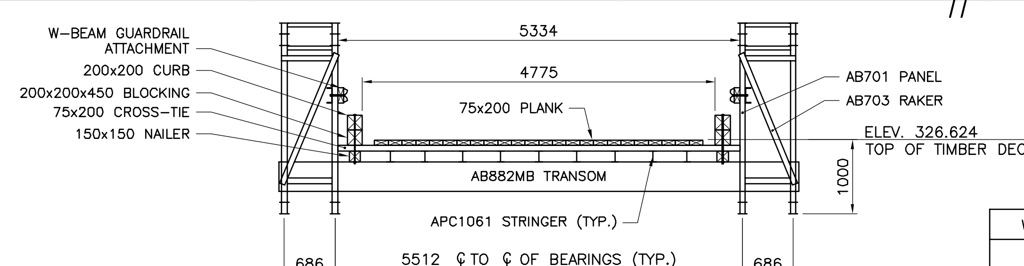
- MODIFIED AASHTO HSS 30 TRUCK DESIGN LOAD.
- AASHTO LRFD "HL-93" TRUCK AND LANE LOAD.

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	774706.810	5501750.575	325.583
W.P. 2	774675.764	5501737.914	325.583

TEST HOLE CO-ORDINATES (UTM ZONE 14 U)

TEST HOLE MK. NO.	"X"	"Y"
TH12-02	774687.39m	5501778.54m
TH12-03	774667.78m	5501758.61m

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

B.M. ELEV.

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

DESIGNED BY GC
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 FALCON LAKE DIVERSION
& ASSOCIATED ROADWORKS

BRIDGE GENERAL ARRANGEMENT

CITY DRAWING NUMBER

SHEET 2 OF 2

CONSULTANT DRAWING NUMBER 126029-FLD-01

NOTE:
FURTHER CLEARING AND SUBSEQUENT GROUND PROOFING REQUIRED TO VERIFY PROPOSED ALIGNMENT AND PROFILE

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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

FALCON LAKE DIVERSION CROSSING

CITY DRAWING NUMBER

SHEET 1 OF 2

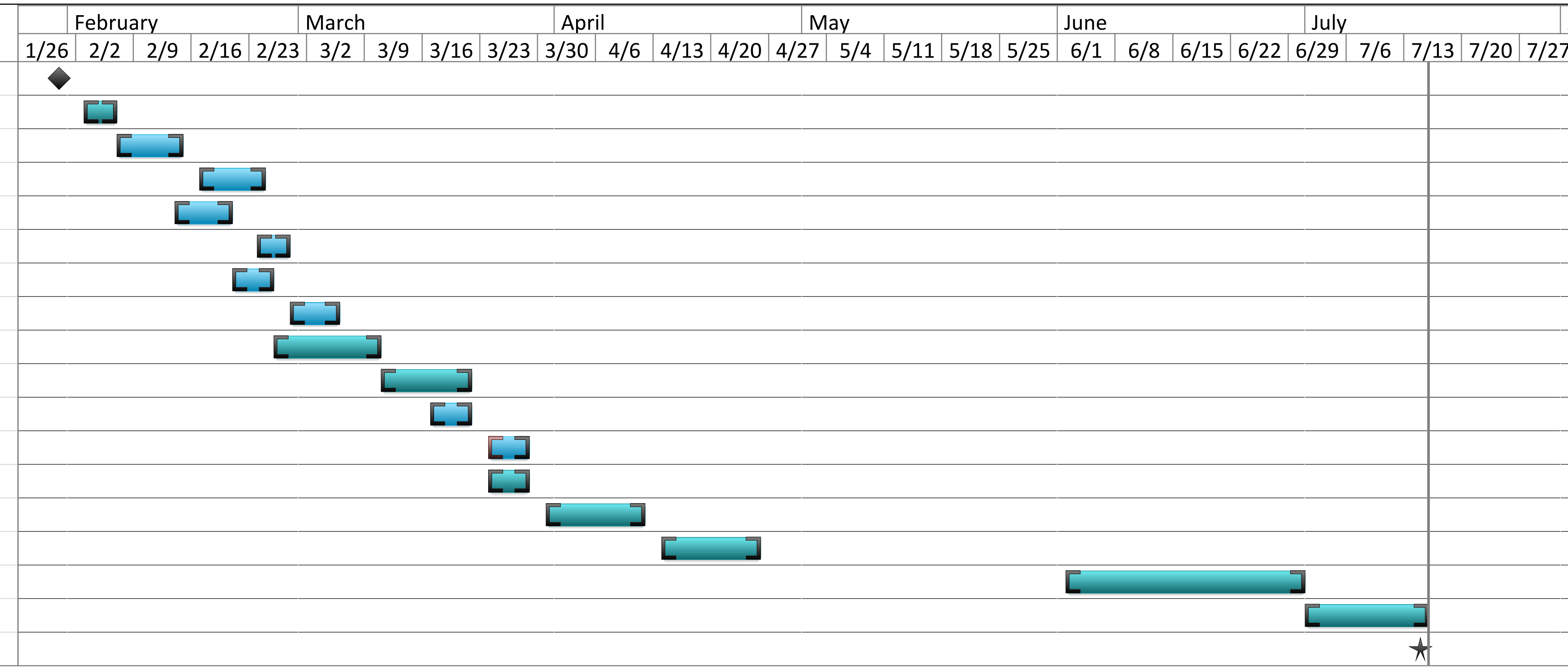
CONSULTANT DRAWING NUMBER

APPENDIX B

Schedules

Falcon River Diversion 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



DILLON CONSULTING

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