Klohn Crippen Berger

City of Calgary

Assessment of Elbow River Upstream Bridge Structures Impact on Glenmore Dam

Hydrotechnical Assessment Report

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November 2015

Klohn Crippen Berger

November 24, 2015

The City of Calgary, Water Resources
625 25th Avenue S.E.
Calgary, Alberta
T2G 4K8

Mr. Frank Frigo, P.Eng.
Senior Planning Engineer – River Engineering Group

Dear Frank:

Assessments of Elbow River Upstream Bridge Structures Impact on Glenmore Dam Hydrotechnical Assessment Report

Please find enclosed our report entitled "Assessments of Elbow River Upstream Bridge Structures Impact on Glenmore Dam – Hydrotechnical Assessment Report".

We trust this fulfils your requirements at this time. Please contact us if you have any questions or require anything further.

Yours truly,

KLOHN CRIPPEN BERGER LTD.

Charles Slack

Chuck Slack, P.Eng.
Principal, Manager - Water Resources & Civil Projects

CS,WD,RC:kb

Cc: Ivan Kurbatfinski, P.Eng.
Acting Leader, Project Engineering - Glenmore

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City of Calgary

Assessment of Elbow River Upstream Bridge Structures Impact on Glenmore Dam

Hydrotechnical Assessment Report

EXECUTIVE SUMMARY

Alberta Transportation (AT) is advancing the design and execution of the West and Southwest Calgary Ring Road Project. The Southwest Calgary Ring Road (SWCRR) will cross the Elbow River approximately 6 km upstream of the City of Calgary's (City) Glenmore Dam. In 2006 and 2007, Alberta Transportation (AT) progressed the design of the proposed Elbow River crossing and Klohn Crippen Berger Ltd. (KCB) undertook a hydrotechnical review to assess its possible impacts on upstream and downstream flood risk.

In 2015, KCB was retained by the City, in cooperation with AT to review the latest SWCRR crossing of the Elbow River design concept, which included higher road elevations and larger bridge openings than the design proposed in 2007. The following describes the scope of work:

- Undertake a hydrotechnical analysis of proposed 2007 and 2015 bridge and road designs for a range of extreme event discharges to determine head loss through the structure, change in water levels and channel flow velocity, the discharge at which the proposed road / bridge is overtopped, and the erosion potential at the bridge and along the realigned channel.
- Undertake a geotechnical investigation to assess the likelihood of a piping or sliding failure of the road embankment and assess the risks of road embankment failure under rapid drawdown conditions.
- Undertake a dambreak assessment for potential overtopping or piping induced failure of the road embankment.
- Undertake a morphological assessment to assess the long term impact on river alignment, changed erosion and deposition potential within the channel due to the proposed crossing during large floods, and the potential for channel avulsion to affect the proposed crossing (bridge, road embankment, and stormwater ponds).
- Assess the impact of blockage due to debris or sediment on water levels.

KCB developed a hydraulic model of the Elbow River and Glenmore Reservoir that extended from Glenmore Dam to the Discovery Ridge subdivision, upstream of the proposed SWCRR. The system was simulated for a range of discharges up to 3,500 m^3/s to determine the hydraulic capacity of the SWCRR crossing. Separate breach models were developed to assess the potential consequences of a dambreach type of failure on the Glenmore Dam and associated reservoir structures. In addition, KCB undertook a seepage analysis to assess the likelihood of embankment failure during extreme events and a morphological assessment to assess the possible impacts of changes in river alignment on the proposed SWCRR. The following conclusions were drawn:

- The 2015 conceptual bridge design has a higher discharge capacity than the 2007 design as the top of road elevation is between 1.4 m and 1.8 m higher and the bridge opening is over 40% larger than the 2007 design (400 m^2 rather than the 570 m^2).
- The bridges (both 2007 and 2015) have the hydraulic capacity to convey the new (post-2013 flood) 1:100 year event with a minimum 1.0 m freeboard between predicted water level and

estimated bridge low chord for the majority of the bridge, though there is less than 1.0 m freeboard towards the southern end of the bridge in the 2007 design.

- The 2007 road design overtops at a discharge between 1,600 – 1,700 m^3/s, which corresponds to a return period of between the 1:400 and 1:500 year flood events. The 2015 road design overtops at a discharge between 3,400 – 3,500 m^3/s, which is greater than the current Probable Maximum Flood (PMF) estimate for Glenmore Dam of 2,550 m^3/s.
- Seepage analysis indicates that piping failure of the road embankment fill or foundation is highly unlikely. In addition, it is estimated that the embankment would take between 4 to 8 weeks for steady state seepage conditions to form through the embankment, significantly longer than the expected duration of a flood event (which could be up to one week).
- Embankment instability due to rapid draw down is considered unlikely due to the relatively short duration of a flood event.
- Overtopping failure of the 2015 road embankment results in peak breach outflow discharge of 600 m^3/s. The equivalent for the 2007 design is 250 m^3/s. The time to full breach formation is estimated to be between 2.5 to 4.0 hours, which is considered a conservatively fast estimate of breach formation time.
- The lower portion of the Southeast Dyke at Glenmore Reservoir could start to overtop at an approximate inflow discharge of 2,500 – 2,800 m^3/s, though this varies depending on the shape (and, therefore, the volume) of the inflow hydrograph. The City is currently assessing updated design inflow hydrographs for Glenmore Reservoir.
- The overtopping embankment failure of the 2015 design would occur at a discharge of 3,400 m^3/s, at which time the Southeast Dyke has already overtopped and potentially failed. The proposed 2015 bridge crossing does not, therefore, affect the timing of when the Southeast Dyke overtops.
- For the 2015 design, the increase in discharge over the Southeast Dyke as a result of road embankment failure is likely to be insignificant, when considering the impact of reservoir attenuation and timing of breaches. In this scenario it is likely there would be a first peak from the inflow flood and a second peak from the road embankment failure. The second peak is lower than the first but this is dependent on the shape of the inflow hydrograph.
- The 2007 design could fail when the water levels in the Glenmore Reservoir are at the crest elevation of the Southeast Dyke. In this scenario, however unlikely, the additional flood wave from the breach could overtop the Southeast Dyke. If the proposed SWCRR is designed to overtop at a discharge greater than 3,000 m^3/s, then its failure would not affect the timing of when the Southeast Dyke overtops or fails.
- The stormwater ponds upstream of the dam would be inundated during Elbow River discharges of 1,500 m^3/s (equivalent to a 1:350 year flood event) and overtopping failure of the dykes could occur once the peak flood discharge has passed and water levels in the Elbow River are receding. Dyke failure would result in the release of a flood wave with a significantly

smaller peak discharge than the minimum 1,500 m^3/s Elbow River discharge required to overtop the stormwater pond dykes. In addition, failure of the dykes would occur during the receding limb of the flood hydrograph and, therefore, after peak water levels in Glenmore Reservoir. It is for these reasons that the failure of the storm ponds would not affect maximum flood levels in Glenmore Reservoir during floods exceeding the 1:350 year event.

- Provided the stormwater pond dykes are constructed of suitable water retaining fill material and the foundation has been suitably prepared (i.e. no sand or fine silt present), then there is a very low risk of piping failure of these dykes.
- Provision to incorporate a conservative debris blockage equivalent to 10% of the bridge opening is suitable for detailed design purposes. This blockage area would be 80 m^2 for the 2015 design and 40 m^2 in the 2007 design and should be applied to the flow area below the estimated water level upstream of the bridge crossing.
- Sensitivity tests indicate that with very conservative bridge expansion / contraction loss coefficients and 10% blockage, the 1:100 year water levels upstream of the bridges could increase by 0.75 m in the 2015 design and 0.52 m in the 2007 design. The 2015 design results in a larger increase in water levels due to the larger absolute blockage area of 80 m^2. There would still be a minimum freeboard of 1.0 m between the predicted 1:100 year flood levels and the bottom of the Weaselhead Road and SWCRR southbound lanes bridge girders for the 2015 design. In the 2007 road design, however, there would be no freeboard at the south end of the road bridges, which would increase the risk of partial blockage of the structure.
- The morphological assessment indicates that the river upstream and downstream of the bridge crossings is highly mobile, and could occupy any point across the floodplain, which is 1.0 to 1.2 km wide in the vicinity of the bridge. There is, therefore, the possibility that movement of the river will result in erosive attack on the road embankment and the dykes around the stormwater ponds which may result in additional maintenance costs.
- The proposed channel realignment will flatten an already flat local gradient, promoting channel avulsion upstream. Even without that effect, channel avulsion from river km 21 (approximately 150 m upstream of the proposed river realignment) into the proposed channel realignment would be expected sometime between 50 and 100 years if historical rates of movement continued.
- Channel incision is likely in the downstream portion of the proposed channel realignment, through the bridge opening. Sediment accumulation is unlikely to be a significant problem for the bridge opening over the short or medium term, though could be an issue in the long term. Channel movement downstream of the crossing is likely to increase.
- The hydraulic model of the 2007 design predicts a high exit velocity from the bridges and realigned Elbow River. Velocities through the bridge are of the order of 3.5 m/s, but the exit velocity is predicted to be 5.3 m/s during the 1:100 year flood event. The velocities through the 2015 bridge design are typically 3.5 m/s. Morphological changes could result in increased design velocities through the bridge. It is noted that the hydraulic assessments were based on

conceptual designs of the 2007 and 2015 bridges and may not reflect the eventual transition from the realigned river channel to the existing Elbow River. This transition and required erosion protection should be evaluated in further phases of design.

- Neither the proposed 2007 or 2015 designs affect water levels in the Discovery Ridge subdivision for any inflow up to the simulated 3,500 m^3/s discharge.

The following recommendations are made:

- Channel mobility in this reach of the Elbow River is naturally high. Attempting to prevent or control that mobility would be difficult and would have morphological consequences upstream and downstream. Therefore further designs should accept and expect river mobility as much as possible. The design philosophy should be to protect the infrastructure (i.e. road embankment, bridge piers and abutments, and stormwater ponds) to an appropriate level, rather than to attempt to control the river.
- The design of the stormwater ponds, bridge abutments, and bridge piers should consider: potential changes in river position and profile, including the likely future avulsion from river km 21 (150 m upstream) to the realigned channel; likely erosion through the bridge opening; and, possible future migration of the upstream channel to any point across the width of the valley.
- AT should monitor the migration of meanders within approximately a kilometer upstream and downstream of the proposed bridge crossing annually after each flood season. In addition, the City should monitor (i.e. georeferenced air photos in the City GIS system) the migration of meanders within the City limits at the same monitoring frequency.
- The hydrotechnical assessments of the SWCRR bridges were based on the 2007 and 2015 conceptual-level bridge and road designs provided by Alberta Transportation. Upon completion of the final design, the conclusions of this report should be confirmed by the City to verify that the final crossing configuration (i.e. bridge span, river realignment, and minimum road elevation) does not have adverse impacts to the Glenmore Dam, Glenmore Trail SW Causeway, and the Southeast Dyke.

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1 INTRODUCTION

Alberta Transportation (AT) is advancing the design and execution of the West and Southwest Calgary Ring Road Project. The Southwest Calgary Ring Road (SWCRR) will cross the Elbow River approximately 1.5 km upstream of the west end of Glenmore Reservoir, 5.6 km upstream of the Glenmore Causeway, approximately 6 km upstream of the City of Calgary's (City) Glenmore Dam, and approximately 2.8 km downstream of the Discovery Ridge subdivision, as shown on the Location Plan in Figure 1.

In 2006, Klohn Crippen Berger Ltd. (KCB) was retained by the City to provide preliminary hydraulic findings regarding the proposed SWCRR bridge over the Elbow River and its potential impact on the Glenmore Reservoir. KCB (2006) presented the hydrotechnical analysis for a proposed conceptual bridge design that included preliminary assessments of the bridge hydraulic performance during extreme flood events (1:100 year to the Probable Maximum Flood). KCB was then retained by TransTech Engineering to review an updated concept design, with results presented in KCB (2007).

In 2015, KCB was retained by the City, in cooperation with AT to review the latest concept design that includes: three bridges over the Elbow River (Weaselhead Road, Southbound Calgary Ring Road, and Northbound Calgary Ring Road); a road embankment across the Elbow River floodplain; two stormwater ponds upstream of the Weaselhead Road crossing; and a realigned reach of the Elbow River. The bridges consist of three spans that are significantly larger than the spans proposed in the 2007 design. The larger spans were required to accommodate wildlife corridors on either side of the realigned Elbow River. Construction of the proposed SWCRR has the potential to affect:

- Upstream communities through increased water levels due to the constriction of the Elbow River floodplain;
- The morphology of the Elbow River, as the location of the river will be fixed at the proposed bridge; and
- The downstream Glenmore Dam and associated infrastructure through potential breaching of the road embankment during extreme discharges.

Of particular concern is the Southeast Dyke at Glenmore Reservoir, which is an earthfill embankment that has the potential to be overtopped in extreme events. Given the urbanization that has taken place downstream of Glenmore Reservoir, the Glenmore Dam is an Extreme Consequence Dam as defined by Canadian Dam Association (2007) Guidelines.

Given the potential impact on the Glenmore Dam and associated reservoir structures, and the Discovery Ridge subdivision, the City developed the following scope of work to assess the hydrotechnial, geotechnical and geomorphologic impacts of the proposed 2007 and 2015 bridge designs.

- Undertake a hydrotechnical analysis of proposed 2007 and 2015 bridge and road crossing designs for a range of extreme event discharges to determine headlosses over the structure, change in water levels and channel flow velocity, the discharge at which the proposed structure is overtopped, and the erosion potential at the bridge and along the realigned channel.

- Undertake a geotechnical investigation to assess the likelihood of a piping failure or sliding of the road embankment and assess the risks of road embankment failure under rapid drawdown conditions.
- Undertake a dambreak assessment for potential overtopping or piping induced failure of the road embankment.
- Undertake a morphological assessment to assess the long term impact on river alignment, changed erosion and deposition potential due to the proposed crossing during large floods, and the potential for channel avulsion to affect the proposed crossing (bridge, road embankment, and stormwater ponds).
- Assess the impact of blockage due to debris or sediment on water levels.

The assessment is to use updated Elbow River discharge predictions as described in Golder (2014) as well as the LiDAR data collected by the City after the June 2013 flood. This report presents the methodology and outcomes of the hydrotechnical, geotechnical and morphological investigations undertaken to address the project objectives described in the bullet points above. It is understood that this report will be provided to AT for incorporation in further phases of the bridge crossing design.

2 PROJECT INFORMATION

2.1 Project and Site Description

Figure 1 shows the location of the proposed SWCRR crossing of the Elbow River in relation to Glenmore Reservoir and the Discovery Ridge subdivision. The crossing is located to the south of the intersection of Sarcee Trail and Glenmore Trail and approximately 1.5 km upstream of Glenmore Reservoir. The crossing is within AT's Transportation/Utility Corridor (TUC) and is located just upstream of the City limits. In the vicinity of the proposed bridge crossing, the Elbow River upstream of Glenmore Reservoir consists of a meandering channel and wide floodplain that is flanked on either side by steep escarpments to old river terraces. The floodplains in the vicinity of the proposed crossing are densely forested.

The SWCRR crossing of the Elbow River consists of southbound and northbound lanes on a road embankment, two bridges across a realigned Elbow River, a realigned Weaselhead road and two stormwater ponds. The Weaselhead Road realignment consists of a new river crossing upstream of the SWCRR and an underpass under the northern SWCRR embankment, to facilitate a tie-in to the existing road. The two stormwater ponds are located upstream of the proposed Weaselhead Road, located either side of the realigned Elbow River.

The proposed SWCRR will replace the existing Weaselhead Bridge (formerly known as the Sarcee Bridge), which consists of two piers and a total span of 42 m. The approach roads to the bridge are not elevated from the floodplain and become inundated during flood events.

A significant amount of work has been carried out in the area over the last 20 years that is relevant for the current hydrotechnical, geotechnical and morphological analysis required to meet the objectives of the current project. Specific areas of work are described in the bullet points below.

- The 1996 Elbow River Flood Mapping project described by AGRA (1996), with flood maps included in Appendix I;
- The 2006/2007 SWCRR design drawings included in Appendix II, and associated hydrotechnical analysis;
- The updated hydrological analysis undertaken for the Elbow River catchment;
- The 2015 SWCRR design drawings included in Appendix III;
- Information related to the June 2013 flood and updated Elbow River hydrology;
- LiDAR, Glenmore Reservoir bathymetry, and Aerial Imagery captured post June 2013 flood; and
- Published information related to blockage.

The following sections provide a summary of the background relevant for this current project.

2.2 1996 Floodplain Mapping AGRA (1996)

The Elbow River M.D of Rocky View Floodplain Mapping Project (AGRA 1996) involved the construction and calibration of a steady state HEC-2 model of approximately 39 km of the Elbow River upstream of Glenmore Reservoir. AGRA (1996) stated that the lower reach of the river has an average bed slope of 0.0058 with the river channel bed consisting of shallow gravel over moderately erodible bedrock.

Manning's *n* for the channel was determined through model calibration against high water marks measured in the 1990 and 1995 flood events. For the lower reaches of the model, calibration was achieved with a Manning's *n* of 0.027 for the 1990 event (estimated discharge of 172 m^3/s) and a value of 0.033 for the 1995 event (estimated discharge of 352 m^3/s). There was some uncertainty regarding the estimated discharge for each event and so to be conservative, the higher Manning's *n* values determined for the 1995 event were selected for flood hazard mapping purposes.

It was not possible to calibrate Manning's *n* values for the floodplain due to the lack of observed water level data during the flood events. Manning's *n* was, therefore, estimated for the floodplain areas, with values in the range of 0.1 to 0.2, reflecting the medium to densely vegetated and highly irregular floodplain characteristics.

Bridge contraction and expansion coefficients were assumed to be 0.3 and 0.5 respectively for modelled structures.

AGRA (1996) describes the flood frequency analysis undertaken by the Hydrology Branch of the Alberta Environmental Protection in 1990 (now Alberta Environment and Parks). Table 2.1 below presents the predicted discharges for a range of return period events used in the 1996 floodplain mapping project.

Table 2.1 Summary of Hydrological Inflows for the 1996 Floodplain Project (AGRA 1996)

Return Period (years)	Discharge (m^3/s)
100	842
50	676
25	516
20	466
10	318
5	185
2	52

Appendix I contains the AGRA (1996) flood frequency maps for the lower modelled reaches of the Elbow River. The photocopy is of poor quality, but it is possible to see on Sheet 8/8 that the predicted 1:100 year water level immediately upstream of the Sarcee Trail Bridge crossing was 1083.00 m.

2.3 The 2007 Bridge Design

KCB (2006) describes the proposed SWCRR crossing of the Elbow River and associated hydrotechnical analysis undertaken on behalf of the City. At the time, the SWCRR design was at a very preliminary stage. Even though the road alignment and bridge concept (including deck elevations) were almost fixed, detailed dimensioning and drawings were not be available for over 6 months. Given that design drawings were not available at the time of the hydrotechnical analysis, preliminary design dimensions details were obtained from TransTech Engineering and are summarized below.

- The low point of roadway was set at elevation 1085.8 m, approximately 5.5 m above the floodplain.
- The road sloped up at 1% in each direction from the low point, until it changed to cut into the sides of the river valley.
- The river valley at this location is approximately 900 m wide between escarpments.
- The bridge deck spans were approximately 100 m long, with a road embankment across the remaining floodplain; and
- Clear height from floodplain (riverbank) to lowest underside of bridge deck (north end) was approximately 4.5 m.

KCB (2006) states that Alberta Infrastructure and Transportation (now AT) had set the 1:100 year flow as the design discharge for the bridge opening. Including freeboard allowance, this resulted in approximately 2.5 m vertical clearance between the low chord of the bridge and the riverbank. Wildlife corridor considerations took precedence, however, so the minimum vertical clearance was increased to approximately 4.5 m at the lower (north) end of the bridge span.

Terrace (2006) presents a hydrotechnical analysis up to the 1:100 year return period and computed a Design High Water Level of 1082.1 m. This was based on a design discharge of 700 m^3/s, which was the estimated discharge in the 1932 flood. The following notes on the 1932 event were extracted from Terrace (2006):

> *The 1932 event is the largest known event on the Elbow River since 1908 or earlier. The 1932 flood occurred as the Glenmore reservoir was nearing completion and the reservoir was empty. The flood filled the reservoir from empty to within 0.5 m of the crest over a period of 1 to 2 days and significantly reduced the flood flows downstream of the dam. The flood was calculated based on stage elevations at the Glenmore reservoir and these calculations resulted in a very sharp peak in the hydrograph, with a top hourly flow of 725 m^3/s, and which exceeded 600 m^3/s for only 3 hours. The length of time that the flows exceeded 250 m^3/s in 1932 was only one and a half days.*

Although KCB (2006) states that the bridge was to be designed to the 1:100 year water level, it appears that Terrace (2006) developed the Design High Water Level based on the hydraulic design criteria for bridge design as described in AT (2014). This approach describes two methods for sizing bridge openings: Channel Capacity or Historic High Water Observations. It appears the latter approach was used to determine the design water level.

In October 2006, TransTech provided an updated bridge design for assessment. KCB (2007) describes the outcomes of the updated analysis, which found that: the net flow (orifice) area of the updated bridge design was similar to that of the previous design; the bottom chord elevations of the new bridge design were approximately 0.2 m higher than the previous design on average; and the road elevation was approximately 1 m higher than the previous design. KCB (2007) concluded that surcharging of the Ring Road bridge and roadway overtopping would occur slightly "rarer" than was estimated in February 2006 (say 1:400 years instead of 1:300 years).

Following issue of KCB (2006) it appears TransTech formalized the design into a set of bridge design drawings as shown in Appendix II. Key design information pertinent to the current hydraulic analysis of the structure is given in Table 2.2.

Table 2.2 Key Bridge Design Information (2007 Design)

Category	Dimension
Length of Bridge (m)	93.5
Area under the Bridge (m^2)	400
Design Streambed Elevation (m)	1078.1
Design High Water Level (m)	1082.6
Minimum Bridge Deck Elevation (m)	1087.1
Maximum Bridge Deck Elevation (m)	1088.8
Minimum Low Chord Elevation	1084.3
Maximum Low Chord Elevation	1085.8
Minimum Road Embankment Elevation (m)	1085.37

It appears that the design High Water Level given in Terrace (2006) was revised upwards during development of the 2007 bridge design, with a level of 1082.6 m given in Table 2.2 rather than 1082.1 m given in Terrace (2006). This could be in-line with the statement made in KCB (2006) that the 1:100 year water level would be used in the design.

Figures 2 and 3 show longitudinal profiles through the 2007 bridge design. Figure 3 assumes the bridges are discrete with their own individual abutments. Figure 4 shows one continuous SWCRR bridge abutment, i.e. there is a continuous abutment between southbound and northbound bridges. Hydraulic analysis was carried out for both bridge scenarios to determine which set-up would result in higher upstream water levels.

2.4 The 2015 Bridge Design

Appendix III presents the preliminary design drawings for the 2015 bridge and road as developed by CH2MHill and ISL. The plan shown in Drawing *10S-RD-SK03a* shows that the principal components of the 2015 design, which includes:

- Two SWCRR bridges across the Elbow River and road embankments across the floodplain, with the southbound lanes upstream of the northbound lanes.
- A realigned Weaselhead Road with new bridge upstream of the proposed SWCRR and an underpasss through the northern SWCRR road embankment, to facilitate tie-in to the existing Weaselhead Road.
- Realigned Elbow River through the three bridges, with wildlife corridors on the left and right overbank areas (both large and small wildlife corridors are included in the design).
- Two stormwater ponds located upstream of the Weaselhead Road bridge and either side of the realigned Elbow River.

Drawing *10S-RD-SK03a* also shows the location of the toe and top of the steep escarpment flanking the Elbow River floodplain at this location. The floodplain is approximately 750 m wide at the SWCRR bridge location.

It is understood that although only two lanes will be constructed in each direction to begin with, ultimately there will be 4 lanes in each direction, with the easternmost and westernmost two lanes constructed first. Based on the information presented on Drawing *10S-RD-SK03a,* it is assumed that the bridge earthworks will be constructed for the entire future 8 lanes of traffic, resulting in continuous northern and southern bridge abutments between the two bridges. The hydraulic analysis was, however, undertaken on both discrete and continuous abutments between bridges for comparison purposes.

Drawing *10S-RD-SK03b* shows sections through the three bridges. Drawing *10S-RD-SK03c* shows two sections across the Elbow River. The first is a section taken 50 m upstream of the Weaselhead Road and shows the relative locations of the small and large wildlife corridors on either side of the river and the proposed armoured erosion protection keyed into the ground.

The second section is located approximately 200 m upstream of the Weaselhead Road and shows the wildlife corridors as well as the embankment separating the realigned river from the north and south ponds and the profile into the ponds. The base elevation of the ponds is approximately 1079 m, with the permanent water level set at 1081 m and the design high water level set at 1083 m. The top of the embankment separating the pond from the river is 5 m wide. Drawing *10S-RD-SK01a* shows additional sections through the realigned Elbow River and the two storm ponds.

Drawing *10S-RD-SK06* shows longitudinal profiles along the proposed road. It can be seen that the minimum road elevation occurs to the south of the proposed bridges and it is this part of the Elbow River crossing that would overtop first during an extreme event.

Table 2.3 presents pertinent bridge and associated road design information that will be used in the analysis presented in the remainder of this report.

Table 2.3 Key Bridge Design Information Interpreted from the 2015 Design Drawings

Category	Dimension
Northbound	
Area under the Northbound Bridge (m^2)	570
Length of Bridge (m)	157
Minimum Bridge Deck Elevation (m)	1087.5
Maximum Bridge Deck Elevation (m)	1090.0
Minimum Low Chord Elevation	1083.3
Maximum Low Chord Elevation	1085.2
Minimum Road Embankment Elevation (m) (1)	1086.0
Design Streambed Elevation (m)	1078.1
Design High Water Level (m) (2)	1082.6
Southbound	
Area under the Southbound Bridge (m^2)	763
Length of Bridge (m)	157
Minimum Bridge Deck Elevation (m)	1089.5
Maximum Bridge Deck Elevation (m)	1092.0
Minimum Low Chord Elevation	1085.0
Maximum Low Chord Elevation	1087.1
Minimum Road Embankment Elevation (m) (1)	1087.2
Design Streambed Elevation (m)	1078.4
Design High Water Level (m) (2)	1082.9
Weaselhead Road	
Area under bridge (m^2)	789
Length of Bridge (m)	157
Minimum Bridge Deck Elevation (m)	1089.5
Maximum Bridge Deck Elevation (m)	1092.0
Minimum Low Chord Elevation	1085.0
Maximum Low Chord Elevation	1087.6
Minimum Road Embankment Elevation (m) (1)	1087
Design Streambed Elevation (m)	1078.5
Design High Water Level (m) (2)	1083.0

Note (1) Minimum road elevation occurs to the south of the bridge, although this excludes the fact that low point of the Weaselhead Road is actually in the underpass.

Note (2) Design High Water Level at the northbound bridge is the same as the design high water level given in Table 2.2. The methodology for deriving the design High Water Level appears similar to the methodology described in AT (2014).

Figures 4 and 5 show longitudinal profiles showing pertinent dimensions and elevations for the 2015 bridge design. Figure 4 assumes the bridges are discrete with their own individual abutments. Figure 5 shows one continuous SWCRR bridge, i.e. there is a continuous abutment between southbound and northbound bridges. Hydraulic analysis was carried out for both bridge scenarios to determine which set-up would result in higher upstream water levels.

It can be seen from Table 2.3 that the northbound bridge is lower than the southbound bridge with a correspondingly smaller flow area under the bridge (570 m^2 rather than 763 m^2). Comparing the information in Table 2.2 and Table 2.3, it can be seen that the 2015 design consists of a significantly longer bridge than the 2007 design (157 m as opposed to 93.5 m) with the flow area under the 2015 bridge approximately 43% greater than the 2007 design (570 m^2 compared to 400 m^2). The comparison of the 2007 and the 2015 bridge designs is shown on Figure 6.

2.5 Elbow River Discharges

The June 2013 event caused widespread flooding in Calgary. During the event, City crews were out surveying water levels and Appendix IV contains the results around the Discovery Ridge subdivision and Table 2.4 presents a selection of surveyed water levels in this area.

Table 2.4 Summary of Water Levels Surveyed near Discovery Ridge during June 2013 Flood

KCB ID	City ID	Northing (m)	Easting (m)	Elevation (m)	Date of survey	Time of Survey	Comment
1	68	5653211	-15413.3	1097.917	6/20/2013	10:20 PM	
2	69	5653151	-15327	1097.38	6/20/2013	10:23 PM	
3	70	5653148	-15308	1097.35	6/20/2013	10:25 PM	
4	1512	5653011	-15145	1096.98	6/20/2013	8:55 PM	
4 (HWM)		5653013	-15144	1097.10	6/20/2013	9:04 PM	High Water Mark
5	1510	5652851	-14866	1095.16	6/20/2013	8:30 PM	
6	1495	5652718	-14560	1093.99	6/20/2013	7:42 PM	
6 (HWM)		5652718	-14559	1093.93	6/20/2013	9:33 PM	High Water Mark
7	52	5652705	-14495	1093.49	6/20/2013	8:44 PM	
8	57	5652561	-14218	1092.25	6/20/2013	8:57 PM	
9	62	5652504	-13898	1091.49	6/20/2013	9:10 PM	

Figure 7 presents the reservoir inflow hydrograph developed by the City based on water levels recorded at Glenmore Dam. It can be seen that the peak inflow discharge was approximately 1,240 m^3/s and it occurred at 10:00 pm on June 20, 2013. The hydrograph has a very steep rising limb with discharges increasing from 200 m^3/s to 1,240 m^3/s in approximately 4 hours. It is interesting to note the Terrace (2006) describes the 1932 event as having a very peaky hydrograph shape around the time of the peak discharge.

It can be seen from Table 2.4 that the water levels were surveyed between 7:42 pm and 10:25 pm on June 20, 2013. The peak discharge in the Elbow River (estimated at 1240 m^3/s) occurred at approximately 10:00pm at Glenmore Dam. The High Water Marks given in Table 2.4 indicate that the peak discharge in the Elbow River occurs sometime around 9:04 pm. This would suggest the time of travel between Discovery Ridge and Glenmore Dam is approximately 1 hour.

Figure 8 shows an aerial image of flooding in the Elbow River in the vicinity of the proposed SWCRR taken sometime between 08:00 and 09:30 on June 22, 2013 (approximately 36 hours after the peak discharge in the river). This photograph shows a meandering Elbow River upstream of the Glenmore

Reservoir, with inactive river channels becoming active during the flood. The flooding is extensive, with the whole valley floor under water during the event.

After the flood, Alberta Environment and Sustainable Development (now Alberta Environment and Parks (AEP)) and the City retained Golder to update the flood frequency analysis for the Elbow and Bow Rivers. Golder (2014) presents updated discharge predictions for a range of events upstream of Glenmore Dam, and these discharges are reproduced in Table 2.5. The predicted discharge for the 1:100 year event is 954 m^3/s, which is approximately 13% larger than the corresponding discharge of 842 m^3/s reported in AGRA (1996), see Table 2.1.

Table 2.5 Predicted Discharges in the Elbow River upstream of Glenmore Dam (Golder 2014)

Return Period (years)	Elbow River Discharge above Glenmore Dam (m^3/s)
2	84.6
5	194
10	307
20	454
50	708
100	954
200	1,250
500	1,770
1000	2,220

The June 2013 flood had an estimated peak discharge of 1,240 m^3/s above Glenmore Dam and this is equivalent to a 1:200 year event, based on the revised hydrology presented in Golder (2014).

In addition to the hydrological analysis update for the Elbow River, the Province is reassessing the predicted probable maximum flood (PMF) discharge. The current estimated PMF discharge upstream of Glenmore Reservoir is 2,550 m^3/s, but it is expected that this will be revised upwards. The hydrograph shape for the current PMF is unknown.

2.6 Mapping and Air Photos

The City provided post June 2013 flood LiDAR and reservoir bathymetry data, and aerial imagery for use in this project. The LiDAR data and associated imagery was captured between September 9 and October 7, 2013. The information was provided in a 0.2 x 0.2 m grid of x, y, z coordinate data, which was rationalized down to a more manageable 2.0 x 2.0 m grid of data points. KCB then developed 0.5 m contours from the 2.0 m grid of data points.

2.7 Literature Review of Blockage (or Drift) Scenarios

AT (2014) provides recommendations on how to reduce impact of blockage, which include:

- Locating piers outside of main flow;
- Increasing span length over main flow;

- Providing additional freeboard to minimize risk to superstructure;
- Consider potential flow deflection against banks in configuring protection works; and
- Remove drift from piers at existing bridges with pier scour vulnerability.

The recommended maximum freeboard given in AT (2014) is 1.0 m., which is deemed sufficient to help allow the passage of drift/debris through the structure during the design flood.

AT (2014) does not, however, advise on blockage levels (i.e. what proportion of the bridge opening should be considered blocked) for a hydrotechnical analysis of bridge crossings. In view of this, KCB undertook a literature review to assess what the blockage potential would be for the proposed bridges with Wallerstein (1997) and Engineers Australia (EA 2014) providing the most useful information related to blockage. The following sections summarize our findings.

Debris Control at Hydraulic Structures in Selected Areas of the United States and Europe

Wallerstein et al (1997) describes theoretical research undertaken by Godtland & Tesaker (1994), examining the potential for debris clogging at spillways on two dams in Norway. In both cases, the spillway was of the fixed overflow type with a bridge supported by piers along the crest. Model tests were carried out to determine under which conditions debris tangles may cause clogging of spillways and the following approximate guidelines were derived from the tests:

- Spacing of bridge piers should be at least 80% of the length of the arriving trees.
- Vertical free opening between the crest and the superstructure of the bridge should be at least 15% of the tree length.
- Where a superstructure (such as a bridge) is present on a spillway crest, and with spacing between bridge piers of at least 110% of the tree length, most debris tangles will pass when overflow height reaches 16-20% of the tree length.

Australian Rainfall & Runoff

The aim of EA (2014) is to provide design guidance on the blockage of hydraulic structures during flood events. It is noted that the term "hydraulic structures" refers to culverts and small bridges over drainage channels (rather than major bridge structures). EA (2014) describes the mechanisms of blockage, which include Debris Availability, Debris Mobility, Debris Transportability, and Structure Interaction. The first three mechanisms are deemed to have a high probability of occurrence in the Elbow River, based on the observed accumulation of debris along the river and within Glenmore Reservoir during the June 2013 flood. This results in a *high* at-site Base Debris Potential.

The Structure Interaction mechanism describes the likelihood of blockage of the structure and is dependent on whether the debris is able to bridge across the structure or become trapped within the structure. Notwithstanding this point, Table 2.6 (which is a reproduction of Table 6 in Engineers Australia 2014) summarizes the blockage level that should be considered in the analysis.

Table 2.6 Most Likely Percentage of Blockage to be used in the Design

Control Dimension	At-Site Debris Potential		
	High	Medium	Low
$W < L_{10}$	100%	50%	25%
$W >= L_{10} =< 3^* L_{10}$	20%	10%	0%
$W > 3^*L_{10}$	10%	0%	0%

Note *W* is the spacing between bridge piers; L_{10} is the length of the longest 10% of debris.

Given that the focus of EA (2014) is blockage of culverts and small bridges, it is assumed that the blockage is as a percentage of the entire opening.

Recommended Blockage Ratio

Both methods described above refer to the length of trees and given the heavily wooded nature of the upstream river valley, the Elbow River will contribute floating logs to the river, particularly during flood events. A significant number of logs arrive at Glenmore Reservoir every year, and are collected, stored temporarily and removed (J. Slaney, City of Calgary, pers.comm.). The debris arriving at the bridge can be expected to be composed of logs similar to those in Photographs 2.1 and 2.2.

Photograph 2.1 Aerial view of log accumulation in Glenmore Reservoir near Heritage Park (2014).

Based on Photograph 2.1, the arriving logs are typically 10 to 15 m long, although there are logs as long as 20 m in the photo.

Photograph 2.2 Floating logs in Glenmore Reservoir (Photo credit: City of Calgary)

The central bridge opening is 61 m in the 2015 design (the minimum central span in the 2007 design is 30 m) and the maximum observed log length is approximately 20 m, so the spacing between bridge piers (W) is greater than 3 times the longest log length of 20 m in Table 2.6. Based on a High level of At-Site Debris Potential, Engineers Australia (2014) recommends that a 10% of the bridge opening should be considered during detailed design. In reality, although it is possible that some debris could get trapped on the bridge piers, significant blockage of the structure is not likely.

For the purposes of this assessment, a 10% blockage scenario was investigated based on the EA (2014) recommendations, with results presented in Section 3.6. This is considered conservative, especially if the upstream face of the piers are rounded, the 1:100 year flood depths at the bridge piers would be greater than 4 m (i.e. greater than 20% of the tree length), and the velocity through the main structure would be relatively high. It is, therefore, unlikely that debris could be trapped to cause 10 % blockage of the opening, particularly because there is significant freeboard between the predicted flood levels and the bottom of the bridge girders.

3 BRIDGE AND ROAD HYDRAULIC CAPACITY ASSESSMENTS

The objective of the hydrotechnical assessment is to determine the impact of the proposed SWCRR on water levels and velocities in the Elbow River. A hydraulic model was developed for the assessment and was validated against information collected during the June 2013 flood event. This model was then updated to assess a range of bridge design scenarios to assess their impact on the Elbow River and to determine the discharge at which the road embankment overtops. The following sections describe the work undertaken and results of the hydraulic assessments for the SWCRR crossings (2007 ad 2015 designs) of the Elbow River and floodplain.

3.1 Summary of Discharges Simulated for this Project

Table 3.1 presents the range of modelled discharges simulated as part of this project. The minimum discharge simulated was 954 m^3/s (i.e. the updated 1:100 year event). KCB validated the hydraulic model predictions against observations recorded during the June 2013 event and then simulated a range of discharges to determine when the proposed SWCRR road embankment would overtop. The peak PMF discharge is not known at the time of writing, so a range of extreme discharges was assessed to determine hydraulic capacity of the proposed bridges and road crossings of the Elbow River floodplain.

Table 3.1 Modelled Discharges in the Elbow River upstream of Glenmore Reservoir

Modelled Discharge (m^3/s)	Comment
954	1:100 year event
1,240	The approximate discharge in the June 2013 event and approximately the 1:200 year event.
2,000	-
2,500	Current estimated PMF discharge
3,000	-
3,250	-
3,500	Upper limit of potential PMF discharge upstream of Glenmore Dam.

One of the objectives of the project was to determine the return period at which the proposed ring road will overtop. Given that the peak PMF discharge is not known at the time of writing, it will be necessary to assess the frequency of overtopping once the PMF update report has been released.

3.2 Hydraulic Model Development

KCB developed a HEC-RAS model of the Elbow River upstream of Glenmore Dam. The modelled reach was 11.1 km in length and included Glenmore Reservoir, Glenmore Causeway, and approximately 6.3 km of the Elbow River upstream of the reservoir. The upstream model limit was at the Discovery Ridge subdivision. Figure 9 shows the hydraulic model extents.

The model was developed using the post June 2013 flood LiDAR data described in Section 2.6, which did not capture elevations below the water surface. Elbow River discharges recorded at the Sarcee

Bridge (Water Survey of Canada Gauge Number 05B5010) ranged from 14.4 m^3/s down to 8.7 m^3/s over the period that the LiDAR data was collected, indicating that river levels would have been relatively low at the time of the survey. Even so, part of the channel was under water at the time LiDAR data was collected and was not, therefore included in the model cross sections. This means the hydraulic model is probably slightly under representing the actual hydraulic radius of channel given that the entire flow area has not been captured.

A total of 25 cross sections were used to model the Elbow River upstream of Glenmore Dam (excluding those required to model the proposed crossing configurations) with an average cross section spacing of 466 m. The inflows at the upstream model boundary are as given in Table 3.1. The downstream model boundary is the rating curve at Glenmore Dam. Section 3.6 describes the sensitivity tests undertaken to assess the impact of a changed rating curve on water levels at the SWCRR bridge.

Manning's *n* was used to model the channel and floodplain roughness, with initial values selected based on engineering judgement and the calibrated values described in AGRA (1996). The outcome was that Manning's *n* for the channel was set at 0.03 and the value for the floodplain was 0.15. This resulted in a composite value for the modelled cross sections of 0.132. A value of 0.03 for the channel represents the average values determined through calibration of the 1990 and 1995 flood events, as described in AGRA (1996).

Bridge contraction and expansion losses were included in the model to determine the losses over the structure and Table 3.2 provides a summary of typical loss coefficients encountered for a range of constriction types.

Table 3.2 Typical Expansion and Contraction Loss Coefficients

Description of type of constriction	Contraction Loss Coefficient	Expansion Loss Coefficient
No Transition	0.0	0.0
Gradual Transitions	0.1	0.3
Typical Bridge Transition	0.3	0.5
Abrupt Transition	0.6	0.8

Note: Information taken from the HEC-RAS Hydraulic Reference Manual (USACE 2010)

Given the proposed bridges are long at approximately 90 to 150 m (compared to the floodplain width of 750 m at the location of the SWCRR bridges) and given that the entrance to the bridges has been designed to give a gradual transition from wide floodplain flow to the constricted flow through the bridge, it is assumed that the proposed configuration is a typical bridge transition, with contraction and expansion loss coefficients of 0.1 and 0.3 respectively. Section 3.6 describes the sensitivity analysis on bridge expansion and contraction losses.

3.3 Validation of the Baseline Hydraulic Model

The baseline model was simulated for the 1,240 m^3/s discharge (i.e. the estimated peak discharge in the June 2013 flood event) and Table 3.3 compares model predictions with a selection of water levels around Discovery Ridge, which were surveyed during the June 2013 event (see Table 2.4). Figure 10 shows the location of the surveyed water levels in relation to the modelled cross sections, with Table 3.3 reproduced on this figure for ease of reference. The aerial image in the figure was taken approximately 36 hours after the peak discharge in the Elbow River and gives an indication of the flood extent around Discovery Ridge.

Table 3.3 Results of Model Validation

KCB ID	Surveyed Water Level (m)	Time of Survey	Predicted Water Level with Q=1240 (m)	Difference from observations (m)
1	1097.92	10:20 PM	1098.07(1)	n/a
2	1097.38	10:23 PM	1097.83	+0.45
3	1097.35	10:25 PM	1097.75	+0.40
4	1096.98	8:55 PM	1096.93	-0.05
4 (HWM)	1097.10	9:04 PM	1096.93	-0.17
5	1095.16	8:30 PM	1095.68	+0.52
6	1093.99	7:42 PM	1094.25	+0.26
6 (HWM)	1093.93	9:33 PM	1094.25	0.26
7	1093.49	8:44 PM	1093.98	+0.49
8	1092.25	8:57 PM	1092.81	+0.56
9	1091.49	9:10 PM	1091.61	0.12

Note (1) Based on model prediction at upstream cross section.

The above table indicates that on the whole, water level predictions are higher than the observations, suggesting that the baseline model is conservative. Based on the information available, the results of the validation exercise indicate that the selected baseline hydraulic parameters described in Section 3.2 are appropriate for this level of study.

The validated model was compared against the AGRA (1996) model predictions. As discussed in Section 2.2, the 1:100 year flood level at the Weaselhead Road Bridge was 1083.0 m for a 1:100 year discharge of 842 m^3/s. The KCB model predicts a 1:100 year water level of 1082.5 m, assuming a discharge of 954 m^3/s. It is difficult to explain why the AGRA (1996) model predictions are so much greater than the KCB model predictions without undertaking a detailed review of the AGRA 1996 model, which is outside the scope of this study.

3.4 Bridge Crossing Hydraulic Assessment during the 1:100 Year Event

Based on the model validation, the baseline model was used for model simulations to assess the impact of the proposed bridges. Two scenarios were modelled for each of the 2007 and 2015 bridge designs, as follows:

- two individual SWCRR bridges and the upstream Weaselhead Road Bridge;
- a single SWCRR bridge with continuous abutment and the upstream Weaselhead Road Bridge.

The reason for this approach was to determine the worst case scenario in terms of water levels upstream of the proposed 2007 and 2015 bridge designs. Although it is assumed that ultimately the SWCRR will have continuous abutments between bridges, it is possible that the initial 2 lane bridges could have independent abutments.

Figures 11 and 12 show the hydraulic grade line through the 2015 and 2007 bridges assuming the ring road bridges are separate entities, acting independently of each other (i.e. there is an expansion and contraction loss upstream and downstream of each bridge). Figures 13 and 14 show the hydraulic grade line through the bridges assuming there is a single, continuous bridge abutment between bridges in the direction of flow. These profiles show that the single bridge scenario with continuous abutments gives slightly higher water level predictions upstream of the bridge than the discrete bridges, and so the results of the single bridge scenario are reported in Table 3.4.

Table 3.4 Predicted 1:100 year Water Levels for the Single Bridge Configuration

Comment	Water levels (m)		
	2007 Bridge Design	2015 Bridge Design	Difference between 2015 and 2007 Designs
Downstream of Northbound Lanes	1081.95	1082.15	0.2
Upstream of Southbound Lanes	1083.95	1082.85	-1.1
Upstream of Weaselhead Road	1084.70	1083.10	-1.4

The water levels downstream of the 2015 Northbound Lanes Bridge are slightly higher than those downstream of the 2007 bridge. This is due to the differing channel configurations downstream of the 2007 and 2015 bridges. Downstream of the 2015 bridge design, there is a significant channel bend required to tie into the existing Elbow River channel. There is a more gradual bend downstream of the 2007 design.

Table 3.5 shows the difference in elevation between the bottom of the proposed girders and the 1:100 year flood levels for the 2007 design. It can be seen that there is approximately 0.5 m freeboard between the predicted water levels and the minimum bottom elevation of the Weaselhead Bridge and Southbound Ring Road Bridge girders. The minimum low chord is at the southern end of the bridge, and given the low chord at the northern end of the bridge is approximately 1.5 m higher, then the minimum 1.0 m freeboard should be achieved for the majority of the span (the minimum average freeboard for the southbound lanes bridge is 1.2 m). For the bridge carrying the northbound lanes, the minimum freeboard is predicted to be over 1.0 m.

Table 3.5 Comparison of 1:100 Year Water Levels and the Low Chord of the 2007 Bridge

Comment	Levels (m)		
	Upstream of Northbound Lanes	Upstream of Southbound Lanes	Upstream of Weaselhead Road
1:100 year Water Level[1]	1082.9	1084.0	1084.0
Minimum Elevation of Low Chord of Bridge	1084.3	1084.3	1084.6
Freeboard to Minimum Low Chord Elevation	1.3	0.3	0.5
Maximum Elevation of Low Chord of Bridge	1085.8	1086.0	1086.1
Freeboard to Maximum Low Chord Elevation	2.9	2.0	2.6
Average 1:100 Year Freeboard	2.1	1.2	1.6

Note (1) These elevations can also be considered the Design High Water Levels for the bridges and are comparable to the equivalent elevations given in Table 2.2.

Table 3.6 shows the difference in elevation between the bottom of the proposed girders and the 1:100 year flood levels for the 2015 design. It can be seen that there is at least 1.9 m freeboard between the predicted water levels and the minimum bottom elevation of the Weaselhead Bridge and Southbound Ring Road Bridge girders. For the bridge carrying the northbound lanes, the minimum freeboard is predicted to be 1.0 m at the southern end of the bridge, though the average freeboard is 2.0 m.

Table 3.6 Comparison of 1:100 Year Water Levels and the Low Chord of the 2015 Bridge

Location	Levels (m)		
	Upstream of Northbound Lanes	Upstream of Southbound Lanes	Upstream of Weaselhead Road
1:100 year Water Level[1]	1082.3	1082.9	1083.1
Minimum Elevation of Low Chord of Bridge	1083.3	1085.0	1085.0
Freeboard to Minimum Low Chord Elevation	1.0	2.1	1.9
Maximum Elevation of Low Chord of Bridge	1085.2	1087.1	1087.6
Freeboard to Maximum Low Chord Elevation	2.9	4.2	4.5
Average 1:100 Year Freeboard	2.0	3.2	3.2

Note (1) These elevations can also be considered the Design High Water Levels for the bridges and are comparable to the equivalent elevations given in Table 2.3.

It can be seen from the results that both the 2007 and the 2015 bridge designs have the capacity to handle the predicted 1:100 year event whilst predominantly maintaining a minimum 1.0 m freeboard between predicted 1:100 year water levels and the bottom elevation of the bridge girders.

Overtopping of the SWCRR southbound lanes will take place when flood levels exceed an elevation of 1087.2 m (see Table 2.3). As can be seen in Table 3.5, the predicted 1:100 year water level upstream of the southbound lanes bridge is 1082.85 m, which is approximately 4.3 m lower than the minimum road elevation.

Table 3.7 summarizes river velocities through the bridge, which tend to be of the order of 3.5 m/s in the 1:100 year event for both bridge designs. The model is, however, predicting a high, supercritical exit velocity in the 2007 bridge design. Given the findings of the morphological assessment described in Section 5 and the potential need for robust erosion protection at the outlet, it would be preferable to keep to a low, sub-critical exit velocity to minimize the erosive potential of the river downstream of the bridge. In view of this, the 2015 bridge design is more appropriate. Refer to Section 5.2.2 for a more detailed discussion on potential geomorphic impacts resulting from the proposed Elbow River crossing.

Table 3.7 River Velocities through the Bridge during the 1:100 year Event

Parameter	2007 Bridge Design	2015 Bridge Design	Difference
Velocity upstream of the Weaselhead Road Crossing (m/s)	3.5	3.2	-0.3
Velocity upstream of the Ring Road Bridge (m/s)	3.5	3.3	-0.2
Velocity downstream of the Ring Road Bridge (m/s)	5.3[1]	3.5	-1.8

Note (1) The model predicts a high exit velocity in the 2007 bridge design with Froude number exceeding 1.0, i.e. exit discharge is supercritical.

3.5 Bridge/Road Crossing Maximum Hydraulic Capacity Assessment

KCB simulated the range of discharges given in Table 3.1 using the HEC-RAS model to determine the discharge that overtops the SWCRR road and bridge (2007 and 2015 bridge designs). Longitudinal profiles showing the results are shown on Figures 15 to 18 for the four assessed bridge scenarios. General points of note in relation to Figures 17 and 18 (i.e. the continuous bridge scenarios for the 2007 and 2015 designs) are summarized in Table 3.8.

Table 3.8 Comparison of Hydraulic Performance of 2007 and 2015 Bridge Design

Comment	Discharge (m^3/s)		
	2007 Bridge Design	2015 Bridge Design	Difference
Discharge at which Weaselhead Road Bridge Surcharges	1,250	2,000	750
Discharge at which Weaselhead Road Bridge Overtops	1,600	3,400	1,800
Discharge at which Southbound Lanes Bridge Surcharges	1,250	2,000	750
Discharge at which the Southbound Road Embankment Overtops [(1)]	1,650	3,400	1,800
Discharge at which Northbound Lanes Bridge Surcharges	1,900	1,950	50

Note (1) Overtopping of the SWCRR occurs when the southbound lanes overtop. From Table 2.2, the minimum road elevation of the 2007 design is 1085.2 m. From Table 2.3, the minimum elevation of the southbound lanes embankment is 1087 m in the 2015 design. The southbound lane embankment elevation for both the 2007 and 2015 bridge designs is critical for overtopping of the ring road.

It can be seen that the road embankment in the 2007 design overtops at a discharge between 1,600 - 1700 m^3/s. This corresponds to a return period of between the 1:400 and 1:500 year flood events, based on the updated Elbow River discharge predictions given in Table 2.5. The 2015 road design overtops at a discharge in excess of 3,400 m^3/s, which is greater than the current estimated PMF at Glenmore Dam of 2,550 m^3/s. The results of these simulations demonstrate the impact of the larger bridge and higher road embankment on conveying higher Elbow River discharges. The upside is that the new design could be open for traffic during much larger events than the 2007 design.

The higher road embankment in the 2015 design has the potential to create a larger flood wave than the 2007 design should a breach of the embankment occur. This flood wave is discussed further in Section 4.2.

3.6 Sensitivity Analysis

A number of sensitivity tests were carried out to assess the impact of assuming increased contraction and expansion losses, and potential 10% debris blockage of the bridge, as recommended in Section 2.7.

KCB investigated an extreme loss coefficient scenario with the contraction losses set at 0.6 and the expansion losses set at 0.8. The values used in this scenario are appropriate if there are abrupt changes between the floodplain flow and bridge flow conditions (see Table 3.2) and are considered conservative for the proposed SWCRR bridges. The results of these sensitivity tests are summarized in Table 3.9.

Table 3.9 Results of Sensitivity Tests Carried out on the 1:100 year Event for the Continuous Abutment Scenario

Description	Water Levels Upstream of the Weaselhead Road Bridge (m)			
	2007 Bridge Design	Difference from Baseline Scenario	2015 Bridge Design	Difference from Baseline Scenario
Baseline Scenario	1084.67	-	1083.17	-
Contraction/ Expansion Loss Coefficients increased to 0.6/0.8	1084.87	0.20	1083.24	0.07
10% Blockage of the Bridge Opening due to Debris	1085.01	0.34	1083.71	0.54
Increased Loss coefficients and 10% blockage	1085.19	0.52	1083.92	0.75

To model blockage, it is assumed that 10% of the entire Weaselhead Road and SWCRR southbound bridge openings was removed from the flow area during the 1:100 year flood event. This amounts to the blocking of approximately 80 m^2 of flow area below the predicted 1:100 year water level in the 2015 bridge design and 40 m^2 in the 2007 design. The results of this blockage scenario indicate that water levels increase by 0.54 m in the 2015 bridge design and 0.34 m in the 2007 bridge design, with the 2015 design showing a greater increase in flood levels due to the larger absolute area of blockage. It is recognized that this blockage scenario is considered very conservative based on the fact that methodology described in EA (2014) was developed for culverts and small bridges.

The cumulative effect of higher contraction and expansion losses and debris blockage is a 0.75 m increase in water levels upstream of the proposed 2015 bridge design. The freeboard between the bottom of the Weaselhead Road (or southbound lanes bridge) girder and the predicted 1:100 year water levels is 1.90 m for the 2015 designs (see Table 3.6), and so even with blockage and very conservative expansion and contraction loss coefficients, there will still be at least 1.0 m freeboard during the 1:100 year flood.

The minimum freeboard in the 2007 design is approximately 0.3 m (southbound lanes bridge), and so blockage and higher loss coefficients could reduce the minimum freeboard to zero at the southern end of the bridge. The larger freeboards in the 2015 bridge structure means that the 1.0 m freeboard is likely to be maintained for the southbound lanes of the SWCRR and the Weaselhead Road Bridge, even with higher assumed losses and blockage taken into consideration.

The City is currently progressing designs to replace the stoplogs used on the Glenmore Dam spillway crest to increase water levels by 1.5 m to the full supply level with a new gates system. The modifications required to the spillway crest to accommodate the new gates will result in higher Glenmore Reservoir water levels. KCB undertook a sensitivity test on water levels at the SWCRR bridges to assess the impact of an updated rating curve for the new spillway configuration (assumed four gates per spillway bay) and Table 3.10 compares the results of the existing and proposed rating curve scenarios.

Table 3.10 Effect of Revised Glenmore Dam Spillway Rating Curve on SWCRR Bridge Water Levels

Glenmore Dam Rating Curve	Discharge (m^3/s)	Water Levels in the 2007 River Crossing Design		2015 River Crossing Design	
		Downstream of Bridges	Upstream of Bridges	Downstream of Bridges	Upstream of Bridges
Existing Spillway	3,500	1083.7	1088.6	1083.9	1089.2
New Gates on Spillway	3,500	1084.0	1088.6	1084.2	1089.2
Difference	-	0.3	0.0	0.3	0.0
Existing Spillway	1,000	1081.8	1084.8	1081.7	1083.3
New Gates on Spillway	1,000	1081.8	1084.8	1081.7	1083.3
Difference	-	0.0	0.0	0.0	0.0

The results of this sensitivity test indicate that a changed spillway configuration will have no impact on water levels upstream of the SWCRR up to a simulated reservoir inflow of 3,500 m^3/s. In addition, there will be no impact on water levels downstream of the SWCRR for the 1:100 year flood event. There is a relatively small increase in water level predictions (approximately 0.3 m) downstream of the SWCRR bridges at a reservoir inflow discharge of 3,500 m^3/s.

4 IMPACT OF POTENTIAL IMPOUNDMENT UPSTREAM OF BRIDGE CROSSING

As discussed in Section 3.5, the proposed 2015 bridge and approach roads are higher in elevation than the previous 2007 design. Although the bridge opening is larger in the 2015 design, there is still the potential for the new structure to impound a large volume of water during an extreme event. The following sections describe the impact of the bridge on upstream water levels and the impact of potential failure of the road embankment during extreme events.

4.1 Flood Levels within Discovery Ridge

The baseline hydraulic model was simulated for a range of discharges. Water level predictions in Discovery Ridge for the 2015 bridge design are presented in Table 4.1.

Table 4.1 Impact of the Proposed Bridges on Water Level in Discovery Ridge

Discharge (m^3/s)	Water levels at Discovery Ridge (m)[1]				
	Existing Conditions	2007 Bridge Design	Difference from Existing	2015 Bridge Design	Difference from Existing
954	1093.78	1093.78	0.00	1093.78	0.00
1,240	1094.02	1094.02	0.00	1094.02	0.00
1,500	1094.23	1094.23	0.00	1094.23	0.00
2,000	1094.63	1094.63	0.00	1094.63	0.00
2,500	1094.96	1094.96	0.00	1094.96	0.00
3,000	1095.26	1095.26	0.00	1095.26	0.00
3,500	1095.54	1095.54	0.00	1095.54	0.00

Note 1 – Water Levels were extracted from cross section 10150.

It can be seen that both the 2015 and 2007 bridge designs result in the same water level predictions at Discovery Ridge. The predicted water level immediately upstream of the 2007 bridge design is approximately 1.2 m higher than the equivalent water level upstream of the 2015 design for a 3000 m^3/s discharge. The impact of this backwater effect diminishes to zero approximately 2.5 km upstream of the new bridges (at cross section 9450). Cross section 9450 is just downstream of the residential properties adjacent to the river. Based on this result, the proposed bridge designs will have no impact on water levels at Discovery Ridge.

4.2 Assessment of Road Embankment Stability during Flood Events

There are four possible mechanisms that could trigger a failure of the road embankment during flood conditions:

- Overtopping induced failure;
- Piping failure of the road foundation;
- Piping failure of the road embankment; and
- Instability due to rapid drawdown.

The following sections describe the likelihood of these failure mechanisms being realised.

4.2.1 Overtopping Failure

Table 3.8 indicates that both the 2015 and 2007 road embankments could overtop during extreme events. In view of this, the overtopping failure mechanism is considered possible and will be assessed in Section 4.2.

4.2.2 Piping Failure

This section presents the outcomes of the piping failure assessment, with the assumed parameters summarized in Table 4.2.

Table 4.2 Parameters for Piping Failure Analysis

Parameter	Value for the 2007 Bridge Design	Value for the 2015 Bridge Design
Crest width (m)	145	145
Embankment height (m)	3.5	5.0
Side slopes	3H:1V	3H:1V
Horizontal length of embankment (m)	166	175
Upstream water depth (m)	3.5	5.0
Downstream water depth (m)	0	1.5
Differential Head (m)	3.5	3.5
Horizontal Length/Differential Head(1)	47	50
1/3 Horizontal Length/Differential Head(2)	16	17
Embankment material composition	Likely silty clay till, but considered as a silty sand	Likely silty clay till, but considered as a silty sand
Foundation material	Sand and silt	Sand and silt

Notes (1) This ratio is required for the empirical relationship described in Bligh (1910)
(2) This ratio is required for the empirical relationship described in Lane (1935)

Foundation Piping

Piping potential is directly proportional to hydraulic head and inversely proportional to the length of seepage path. The empirical relationships established by Bligh (1910) and Lane (1935) are still considered valid today for non-dispersive soils.

- Bligh (worst case): riverbeds of light silty sand - L/H > 18 for no piping potential.
- Lane (worst case): very fine sand or silt – 0.3L/H > 8.5 for no piping potential.

For the ring road embankment, the calculated values of L/H and 0.3L/H given in Table 4.2 are typically double these minimum requirements. In view of this, foundation piping is considered very unlikely and is not considered further.

Embankment Fill Piping

A steady state seepage model of the two embankment options was created in SEEP/w. Fill and foundation hydraulic conductivities of 10^{-4} and 10^{-5} m/s were adopted to model silty sand materials. Under steady state conditions, a maximum exit gradient (vertical and horizontal) at the downstream toe was calculated to be about 0.10. Simple transient analyses indicate that it would take about 4 to 8 weeks to achieve steady state phreatic conditions for the range of conductivity values.

The piping mechanism by vertical seepage gradients is defined as the critical balance between the soils effective weight and the upward seepage force. Considering a specific gravity of 2.65 and a void ratio of 0.65, the equation for allowable vertical seepage gradient is 1.0/Factor of Safety. The commonly accepted factor of safety against piping is about 5. Therefore, the allowable vertical gradient is 0.20, which is double the calculated exit gradient.

For horizontal seepage, an expression for moment equilibrium at a particulate level (assumed as spheres) was developed. Considering a specific gravity of 2.65 and a void ratio of 0.65, the equation for allowable horizontal seepage gradient is 0.58/Factor of Safety. Assuming a factor of safety of 5, the allowable horizontal gradient is 0.12, which is 20% higher than the calculated gradient of 0.10.

The calculated exit gradient is less than the allowable seepage gradient and so the formation of piping within the embankment is considered unlikely. In addition, it is likely that the PMF on the Elbow River will last days rather than the required 4 to 8 weeks to achieve steady state conditions, though the PMF hydrograph is unknown. This failure mechanism was not considered further.

Instability due to rapid drawdown

Instability due to rapid drawdown is considered very unlikely due to the relatively short impoundment period during an extreme flood event.

4.3 Overtopping Failure of the Road Embankment

KCB developed a model to assess the impact of a dambreach failure on peak discharges entering the Glenmore Reservoir. A dambreach model was set up in HEC RAS using the road profiles shown in Appendix II and Appendix III and a stage-storage relationship developed for the impoundment area upstream of the road computed using the LiDAR data.

The stage-storage relationship for the 2015 design is shown on Figure 19. The stage-storage volume was developed to an elevation of 1090 m, which is higher than the maximum predicted water level over the 2015 road design shown in Figure 20. The 1090 m contour is highlighted on Figure 20 for information and the impoundment volume at a maximum water level of 1090 m is approximately 12,000 dam^3. The minimum road elevation in the 2015 design is 1087.2 m and the corresponding impoundment volume is 5,500 dam^3. The 1087.2 m contour is also shown on Figure 20.

A number of dambreach simulations were carried out for the 2015 design with breach parameters (maximum breach width, breach side slopes, and time to full breach formation) determined using standard equations given in Froehlich (2008). The breach simulations for the 2015 bridge design are shown on Figure 21 and are summarized below.

- Scenario 1A – Breach initiates when road just overtops, there is a medium head differential between upstream and downstream water levels, and a fast breach formation time.
- Scenario 1B - Breach initiates when road just overtops, there is a large head differential between upstream and downstream water levels, and a very fast breach formation time.
- Scenario 2A –Breach initiates at a high Elbow River discharge, there is a discharge of 650 m^3/s overtopping the road embankment, flood flows are starting to recede in the river, there is a large head differential between upstream and downstream water levels, and there is a fast breach formation time.
- Scenario 2B –Breach initiates at a high Elbow River discharge, there is a discharge of 650 m^3/s overtopping the road embankment, flood flows are starting to recede in the river, there is a very large head differential between upstream and downstream water levels, and there is a slow breach formation time.

Table 4.3 presents the breach parameter values and model results. Results are also presented graphically on Figure 22. Two scenarios for the breach bottom elevation assessed. The first is assuming the breach stops at the tailwater elevation of 1083.8. The second assumes there is no tailwater and the breach cuts down to the floodplain (elevation 1082 m).

Table 4.3 Summary of Dambreach Parameters and Results for the 2015 Bridge Design

Parameter	Scenario ID			
	Scenario 1A	Scenario 1B	Scenario 2A	Scenario 2B
Upstream River Level at time of Breach Initiation (m)	1087.2	1087.2	1089.2	1089.2
Impoundment volume (dam^3)	5,500	5,500	10,000	10,000
Elevation of bottom of breach (m)	1083.8	1082.0	1083.8	1082.0
Maximum head above bottom of the breach (m)	3.4	5.2	5.4	7.2
Breach bottom width (m)	46	49	58	60
Breach side slopes	1H:1V	1H:1V	1H:1V	1H:1V
Time to full breach formation (hrs)	3.5	2.5	3.4	5.0
Peak dambreak discharge (m^3/s)	300	600	575(1)	200(1)
River Discharge at which road overtopping occurs (m^3/s)	3400	3400	3400	3400

Note (1) This value is the difference between the peak discharge and the discharge on the receding limb of the hydrograph shown in Figure 21.

The minimum road elevation in the 2007 bridge design is approximately 1085.37 m and the corresponding impoundment volume is approximately 2,750 dam^3, see Figure 23. The 1085.4 m contour is shown on Figure 20. The breach simulations for the 2007 bridge design are shown on Figure 24 and summarized in the bullet points below.

- Scenario A – Breach initiates when road just overtops, there is a small head differential between upstream and downstream water levels, and a fast breach formation time.
- Scenario 1B - Breach initiates at a high Elbow River discharge, there is a discharge of 450 m^3/s overtopping the road embankment, flood flows are starting to recede in the river, there is a large head differential between upstream and downstream water levels, and a fast breach formation time.

Table 4.4 presents the breach parameter values and associated results. The results are also presented graphically on Figure 25.

Table 4.4 Summary of Breach Parameters and Results for the 2007 Bridge Design

Parameter	Scenario ID	
	Scenario A	Scenario B
River Level at time of Road Overtopping (m)	1085.4	1087.0
Impoundment volume (dam^3)	2,800	5,100
Elevation of bottom of breach (m)	1082.0	1082.0
Maximum differential head	3.4	5.0
Breach bottom width (m)	39	49
Breach side slopes	1H:1V	1H:1V
Time to full breach formation (hrs)	2.8	3.8
Peak dam breach discharge (m^3/s)	250	220
Discharge at which road overtopping occurs (m^3/s)	1600	1600

The results of the analysis indicate that the peak discharge resulting from a breach of the 2015 design road embankment is approximately 600 m^3/s, with the peak discharge resulting from a breach of the 2007 road design is approximately 250 m^3/s. The breach is most likely to occur once the road embankment is overtopped, which occurs at approximate river discharges 3,250 m^3/s for the 2015 design and 1,750 m^3/s for the 2007 design.

It should be noted that the Froehlich (2008) method to determine breach parameters are based on observations made on a range of typical earth embankment dam failures. The SWCRR would not fall into a typical dam category given it would have a much longer base width to embankment height ratio and the top of the embankment would be paved. Given this, the breach formation times presented in Table 4.4 are considered to be conservative.

4.4 Impact of Road Embankment Failure on Glenmore Dam Infrastructure

4.4.1 Glenmore Dam and Reservoir Structures at Risk

Glenmore Dam and Reservoir consists of a concrete gravity dam, a headpond, Glenmore Causeway, the reservoir, and the Southeast Dyke, as shown on Figure 1. The structure most vulnerable to a breach in the road embankment and the resulting pulse of additional discharge is the Southeast Dyke, to the west of the Haysboro subdivision.

The Southeast Dyke has an overall length of approximately 914 m and a maximum height of 6.7 m. The design crest elevation of the dyke is approximately 1081.3 m however there is a 300 m long section where the crest elevation is at 1080.44 m. There is some riprap protection, primarily comprised of similarly sized large rocks placed on a portion of the upstream slope however the extent is unknown (KCB 2013). Portions of the upstream and downstream slopes have also become heavily vegetated.

The Glenmore Causeway has a minimum road elevation of 1081.92 m and the predicted water level upstream of the causeway during a 3,500 m^3/s inflow discharge is 1081.45 m (assuming the Southeast Dyke has already overtopped). Glenmore Dam also has sufficient discharge capacity to pass an inflow discharge of 3,500 m^3/s (which would be less at the dam as the Southeast Dyke would have overtopped). The dam would not, therefore, be affected by the pulse of water, unless the road embankment failure occurred at the time of the peak inflow discharge at which point peak reservoir levels would be higher, spillway discharge would be higher, and the causeway would be further surcharged, but not overtopped. Also, at this high inflow, a significant discharge will be passing over the Southeast Dyke.

Analysis of a range of reservoir inflow hydrographs with a peak discharge of 3,500 m^3/s indicated that the Southeast Dyke would overtop at a reservoir inflow of between 2,500 – 2,800 m3/s. The hydrograph shapes were varied to assess the impact of volume of runoff on the overtopping discharge and it was found that the peakier the hydrograph (i.e. with less runoff volume), the higher the discharge that initiates overtopping of the Southeast Dyke.

4.4.2 Impact of a Breach in the 2015 Design on the Southeast Dyke

The SWCRR embankment in the 2015 design overtops at a discharge of approximately 3,400 m^3/s. Assuming overtopping triggers embankment failure, then when the road overtops, the Southeast Dyke would already have overtopped at a discharge of approximately 2,500 to 2,800 m^3/s and could have failed. In view of this, breach of the road embankment would not affect the timing of the Southeast Dyke overtopping or failing.

Should the road embankment failure occur at the same time as the peak reservoir inflow; however, then flood extent downstream of an already overtopped or failed Southeast Dyke would increase. The likelihood of this taking place, however, is dependent on many variables including:

- Reservoir inflow hydrograph shape;
- Timing of road embankment failure and rapidity of its failure;
- Timing of Southeast Dyke failure and rapidity of failure;
- Reservoir levels at the time of road embankment failure and the potential for the reservoir to attenuate the embankment breach flood wave; and
- The magnitude of the new PMF upstream of Glenmore Dam (i.e. it is possible that the new PMF will be less than 3 400 m^3/s, in which case the 2015 road embankment design might not overtop).

Based on the above, we consider that there is likely a high probability that should the road embankment fail during an extreme event, there will be a second peak discharge downstream of an already failed Southeast Dyke that will be smaller than the first peak. To be conservative, however, and from a public safety perspective (i.e. evacuation plan), it is recommended that the City develop flood inundation maps downstream of the Southeast Dyke assuming the worst-case scenario of dyke breach and road embankment breach occurring at the same time. The City could consider undertaking a comprehensive routing and dambreak analysis for the Southeast Dyke to complement this analysis and get a better understanding of the risk associated with failure of the Southeast Dyke.

4.4.3 Impact of a Breach in the 2007 Design on the Southeast Dyke

The road embankment in the 2007 design overtops at a discharge of approximately 1,600 m^3/s and the peak discharge resulting from embankment failure is approximately 250 m^3/s. Given that the Southeast Dyke overtops at an inflow between 2,500 and 2,800 m^3/s, it is highly likely that a failure of the 2007 road will have no impact on the Southeast Dyke.

4.5 Potential Impact of Failure of Stormwater Ponds

The top of the dykes around the stormwater ponds is 1085 m, which would overtop during an Elbow River discharge of 1,500 m^3/s. It is possible that the ponds would wash out as the flood recedes, though the resulting floodwave would be very small compared to a river discharge of 1,500 m^3/s.

The design water level for the ponds is 1083 m, see Drawing *10S-RD-SK03c* in Appendix III, which is 3 m higher than the invert elevation of the realigned Elbow River. Assuming there is 1.0 m depth in the river when the design flood recedes, then there could be a 2.0 m head differential across the pond berms for a prolonged period. There is a low piping risk under this scenario which can be significantly reduced with suitable foundation preparation and dyke fill material (i.e. no fine sand or silts under or in the dyke).

Riprap protection for the pond dykes is shown on the drawings in Appendix III. The riprap is keyed into the ground, but AT should determine a suitable minimum elevation of the riprap protection that takes into account erosion depth and potential long term morphological changes. In addition, as is current AT practice, regular monitoring is required to identify signs of erosion on all structures associated with the SWCRR and appropriate action taken when required.

5 POSSIBLE IMPACTS OF BRIDGE CROSSING ON ELBOW RIVER MORPHOLOGY AND HYDRAULICS

5.1 Current Elbow River Morphology

The understanding of the morphology of the Elbow River relies heavily on analysis of a chronological series of air photographs spanning the period between 1924 and 2014 that were provided by the City, as listed in Table 5.1. River discharges on the date the airphotos were taken were obtained from Water Survey of Canada (WSC), Alberta Environment, and the City. Discharges downstream of the dam provide some general indication of likely upstream discharges at times when the upstream discharge was not recorded.

Table 5.1 Air Photograph Dates and Corresponding River Discharges

Year	Airphoto Dates[1]	River Discharge (m^3/s)	
		Upstream of Glenmore Dam[2]	Downstream of Glenmore Dam[3]
1924 – 26	October	5.89 – 9.41	5.89 – 8.41
1948	August 31	Not recorded	9.26
1979	November 7	Not recorded	1.47
1997	September	5.54 – 6.86	3.41 – 5.03
2004	November 11	Not recorded	6.34
2012	September 23	6.27	3.55
2013	August 11	15.7	16.1
2014	October 2 – 4	12.20 – 10.00	9.88 – 10.8

Notes:
1. In cases where the date of the photography is reported as more than one day, the range of river discharges during that period is shown in the table.
2. Upstream discharges based on: WSC Station No. 05BJ010, Elbow River below Glenmore Dam (for 1924 – 26, before the construction of the dam); WSC Station No. 05BJ005, Elbow River above Glenmore Dam (1933 – 1977); and WSC Station No. 05BJ010, Elbow River at Sarcee Bridge (1978 – present).
3. Downstream discharges based on: WSC Station No. 05BJ010, Elbow River below Glenmore Dam.

This analysis incorporates the results of concurrent work being conducted for the Calgary Rivers Morphology and Fish Habitat Study (KCB, in preparation; MMA 2015). This morphology study assessed how channel conditions have been affected by processes such as development activities, river regulation, flood history and changes in sediment transport or riparian vegetation. The review of channel change over time has been employed as an initial screening methodology to identify areas with existing or developing hydrotechnical hazards.

The Elbow River upstream of Glenmore Dam has historically been a highly active channel, as illustrated on the historical Elbow River movement on Figure 26. Elbow River immediately upstream of Glenmore Reservoir has formed a meandering to anastomosing gravel-bedded channel flowing within a wide valley flat. The channel is laterally mobile and meander bends are subject to both progression and cut-off. The radii of the tightest bends are typically in the order of 100 m.

Historical air photos indicate that sizeable floods in 1923, 1929, 1932, 2005 and 2013 (Figure 27 and Table 5.2) resulted in significant coarse-textured sediment movement which was associated with substantial channel shifting. Within the short reach of channel shown on Figure 26, three avulsions are apparent, including one at river km 21.5, a second at river km 19.7, and a third at river km 18.9 (river km are measured upstream from the Bow River confluence). The avulsion at river km 19.7 occurred sometime between the 1924-26 airphotos and the 1948 airphotos, likely during the 1929 or 1932 floods. The other two occurred between the 2004 and 2012 airphotos. Interestingly, the avulsion at km 18.9 occurred not during the 2005 flood, as might be expected, but in 2006 (D. Blakely, pers. comm.) when the peak discharge was unexceptional (see Figure 27). However, even during periods without major flood events, channel movement and meander progression are evident, for example in the vicinity of river km 20.1 and river km 19.3 as shown on Figure 26.

Table 5.2 Major Flood Events on the Elbow River upstream of Calgary

Year	Maximum Daily Discharge (m^3/s)	Maximum Instantaneous Discharge (m^3/s)
2013	484	1240
1929	382	N/A
1923	331	N/A
1932	311	N/A
2005	268	338
1915	239	N/A
2008	220	N/A
1995	213	N/A

The 2006 avulsion downstream of the proposed crossing cut off a 2.1 km long meander bend, reducing the channel length between river km 19.1 and 18.3 by 1.3 km. The resulting channel gradient of 0.0020 in that reach is slightly steeper than the overall river slope of 0.0018 between river km 25.0 and 18.0. The steepened slope downstream of the crossing can be expected to result in a tendency for channel incision and/or lateral movement at the crossing as the river readjusts.

5.2 Implications for the Proposed Bridge Crossings

5.2.1 Long Term Geomorphic Stability

The historically mobile Elbow River channel will tend to continue to be mobile in the future, and that tendency can be expected to increase with increasing frequency of extreme weather events that are expected to occur with climate change (Intergovernmental Panel on Climate Change 2014). Meander bends will continue to migrate downstream, and avulsions will occur across meander bends, producing oxbow lakes. The river could occupy any portion of the valley bottom over the long term.

Construction of the highway embankment and the fixed bridge opening will interrupt these natural processes. The existing tendencies for channel movement in the natural channel will be exacerbated by the recent downstream cutoff as well as the highway construction, likely resulting in increased channel mobility upstream and downstream of the crossing.

On the downstream side of the crossing, the immediate impact of the channel realignment is likely to be increased channel erosion and movement to approximately river km 19.2. Repercussions of those morphological changes will likely include increased channel mobility downstream as far as the upstream extent of the reservoir, around river km 18.0. The increased mobility is expected to consist of similar phenomena as have occurred in the past (downstream meander bend progression and cutoffs), but likely at a somewhat faster rate than has occurred historically. A faster rate of downstream meander bend progression may result in additional maintenance requirements and costs.

Design of the channel diversion, road embankments, bridge abutments and stormwater ponds should recognize that the river upstream of the crossing could occupy any point across the width of the valley, and that robust erosion protection will be required to ensure the integrity of the facilities.

5.2.2 Scour and Erosion Potential of Riverbanks, Realigned River Channel and Road Embankments

The proposed channel realignment results in a channel length of 1.49 km from river km 20.5 to 19.4, an increase in channel length of 35%. The channel gradient of 0.0017 through the crossing is already somewhat flatter than the general river gradient of 0.0020, and the proposed diversion will reduce that slope to 0.0011 along the realignment. Considering the relatively steep reach downstream as the result of the avulsion at 18.9, the downstream reach of the realigned channel will tend to incise, and velocities through the bridge opening (Table 3.8) may increase.

Sediment will tend to deposit in the upstream reach of the realigned channel, increasing the likelihood of a channel avulsion from the upstream channel (e.g. at river km 21.0) to the realigned channel. Even without that effect, channel avulsion from river km 21 into the proposed channel realignment would be expected sometime between 50 and 100 years if bend migration continued at historical rates.

Based on this assessment of the channel stability, it is concluded that the potential for scour and erosion in the downstream half of the proposed channel realignment is high. Scour and erosion should be expected through and downstream of the bridge opening. In addition, it is likely that channel avulsion will take place upstream of the bridge.

5.3 Sediment Accumulation Potential

The highly mobile channel upstream of the crossing is associated with large amounts of sediment movement. For some time after construction, sediment is likely to accumulate in the upstream half of the relatively low-gradient channel realignment, reducing sediment transport downstream. In the longer term, the channel slope upstream of the bridge crossing will adapt, through aggradation, incision, lateral movement, and avulsion, to restore sediment movement to the bridge opening. After that occurs, sediment arriving at the bridge is initially likely to be carried through the opening because of the steeper river gradient downstream. Therefore sediment accumulation is unlikely to be a significant problem for the bridge opening over the short or medium term. However, in the long term, future changes to the river channel upstream and downstream could eventually change local slopes and result in sediment accumulation in the bridge opening.

5.4 Morphological Considerations and Recommendations for Design and Monitoring

The following conclusions and suggestions are provided for consideration by AT and the crossing designers.

- Channel mobility in this reach of the Elbow River is naturally high. Attempting to prevent or control that mobility would be difficult and would have morphological consequences upstream and downstream. Therefore, the design should accept and expect river mobility as much as possible. The design philosophy should be to protect the infrastructure (i.e. road embankment, bridge piers and abutments, and stormwater ponds) to an appropriate level, rather than to attempt to control the river.
- Over the long term, the river slope in the vicinity of the crossing will likely tend toward the general reach slope of 0.002. Using design slopes significantly flatter or steeper than the general reach slope will result in accelerated channel movement and increased erosion and sediment deposition.
- The channel realignment should avoid bends tighter than a 100 m radius to avoid increasing the potential for erosion, sedimentation and avulsion.
- The design of erosion protection around the stormwater ponds, including the minimum elevation of the protection, should consider the potential changes in river position and profile, including the likely future avulsion from river km 21 to the realigned channel.
- Similarly, the size and depth of the proposed riprap erosion protection for the abutments should take into consideration potential morphological changes to the river bed upstream and downstream of the bridge. Those changes will likely include a steeper hydraulic gradient and, therefore, higher velocities through the bridge than are shown in Table 3.7, and some channel incision through the bridge opening.
- AT should monitor the migration of meanders within approximately a kilometer upstream and downstream of the proposed bridge crossing annually after each flood season.

6 CONCLUSIONS

The following conclusions can be drawn from the work:

- The 2015 conceptual bridge design has a higher discharge capacity than the 2007 design as the top of road elevation is between 1.4 m and 1.8 m higher and the bridge opening is over 40% larger than the 2007 design (400 m^2 rather than the 570 m^2).
- The bridges (both 2007 and 2015) have the hydraulic capacity to convey the new (post-2013 flood) 1:100 year event with a minimum 1.0 m freeboard between predicted water level and estimated bridge low chord for the majority of the bridge, though there is less than 1.0 m freeboard towards the southern end of the bridge in the 2007 design.
- The 2007 road design overtops at a discharge between 1,600 – 1,700 m^3/s, which corresponds to a return period of between the 1:400 and 1:500 year flood events. The 2015 road design overtops at a discharge between 3,400 – 3,500 m^3/s, which is greater than the current Probable Maximum Flood (PMF) estimate for Glenmore Dam of 2,550 m^3/s.
- Seepage analysis indicates that piping failure of the road embankment fill or foundation is highly unlikely. In addition, it is estimated that the embankment would take between 4 to 8 weeks for steady state seepage conditions to form through the embankment, significantly longer than the expected duration of a flood event (which could be up to one week).
- Embankment instability due to rapid draw down is considered unlikely due to the relatively short duration of a flood event.
- Overtopping failure of the 2015 road embankment results in peak breach outflow discharge of 600 m^3/s. The equivalent for the 2007 design is 250 m^3/s. The time to full breach formation is estimated to be between 2.5 to 4.0 hours, which is considered a conservatively fast estimate of breach formation time.
- The lower portion of the Southeast Dyke at Glenmore Reservoir could start to overtop at an approximate inflow discharge of 2,500 – 2,800 m^3/s, though this varies depending on the shape (and, therefore, the volume) of the inflow hydrograph. The City is currently assessing updated design inflow hydrographs for Glenmore Reservoir.
- The overtopping embankment failure of the 2015 design would occur at a discharge of 3,400 m^3/s, at which time the Southeast Dyke has already overtopped and potentially failed. The proposed 2015 bridge crossing does not, therefore, affect the timing of when the Southeast Dyke overtops.
- For the 2015 design, the increase in discharge over the Southeast Dyke as a result of road embankment failure is likely to be insignificant, when considering the impact of reservoir attenuation and timing of breaches. In this scenario it is likely there would be a first peak from the inflow flood and a second peak from the road embankment failure. The second peak is lower than the first but this is dependent on the shape of the inflow hydrograph.

- The 2007 design could fail when the water levels in the Glenmore Reservoir are at the crest elevation of the Southeast Dyke. In this scenario, however unlikely, the additional flood wave from the breach could overtop the Southeast Dyke. If the proposed SWCRR is designed to overtop at a discharge greater than 3,000 m^3/s, then its failure would not affect the timing of when the Southeast Dyke overtops or fails.
- The stormwater ponds upstream of the dam would be inundated during Elbow River discharges of 1,500 m^3/s (equivalent to a 1:350 year flood event) and overtopping failure of the dykes could occur once the peak flood discharge has passed and water levels in the Elbow River are receding. Dyke failure would result in the release of a flood wave with a significantly smaller peak discharge than the minimum 1,500 m^3/s Elbow River discharge required to overtop the stormwater pond dykes. In addition, failure of the dykes would occur during the receding limb of the flood hydrograph and, therefore, after peak water levels in Glenmore Reservoir. It is for these reasons that the failure of the storm ponds would not affect maximum flood levels in Glenmore Reservoir during floods exceeding the 1:350 year event.
- Provided the stormwater pond dykes are constructed of suitable water retaining fill material and the foundation has been suitably prepared (i.e. no sand or fine silt present), then there is a very low risk of piping failure of these dykes.
- Provision to incorporate a conservative debris blockage equivalent to 10% of the bridge opening is suitable for detailed design purposes. This blockage area would be 80 m^2 for the 2015 design and 40 m^2 in the 2007 design and should be applied to the flow area below the estimated water level upstream of the bridge crossing.
- Sensitivity tests indicate that with very conservative bridge expansion / contraction loss coefficients and 10% blockage, the 1:100 year water levels upstream of the bridges could increase by 0.75 m in the 2015 design and 0.52 m in the 2007 design. The 2015 design results in a larger increase in water levels due to the larger absolute blockage area of 80 m^2. There would still be a minimum freeboard of 1.0 m between the predicted 1:100 year flood levels and the bottom of the Weaselhead Road and SWCRR southbound lanes bridge girders for the 2015 design. In the 2007 road design, however, there would be no freeboard at the south end of the road bridges, which would increase the risk of partial blockage of the structure.
- The morphological assessment indicates that the river upstream and downstream of the bridge crossings is highly mobile, and could occupy any point across the floodplain, which is 1.0 to 1.2 km wide in the vicinity of the bridge. There is, therefore, the possibility that movement of the river will result in erosive attack on the road embankment and the dykes around the stormwater ponds which may result in increased maintenance costs.
- The proposed channel realignment will flatten an already flat local gradient, promoting channel avulsion upstream. Even without that effect, channel avulsion from river km 21 (approximately 150 m upstream of the proposed river realignment) into the proposed channel realignment would be expected sometime between 50 and 100 years if historical rates of movement continued.

- Channel incision is likely in the downstream portion of the proposed channel realignment, through the bridge opening. Sediment accumulation is unlikely to be a significant problem for the bridge opening over the short or medium term, though could be an issue in the long term. Channel movement downstream of the crossing is likely to increase.
- The hydraulic model of the 2007 design predicts a high exit velocity from the bridges and realigned Elbow River. Velocities through the bridge are of the order of 3.5 m/s, but the exit velocity is predicted to be 5.3 m/s during the 1:100 year flood event. The velocities through the 2015 bridge design are typically 3.5 m/s. Morphological changes could result in increased design velocities through the bridge. It is noted that the hydraulic assessments were based on conceptual designs of the 2007 and 2015 bridges and may not reflect the eventual transition from the realigned river channel to the existing Elbow River. This transition and required erosion protection should be evaluated in further phases of design.
- Neither the proposed 2007 or 2015 designs affect water levels in the Discovery Ridge subdivision for any inflow up to the simulated 3,500 m^3/s discharge.

7 RECOMMENDATIONS

The following recommendations are drawn from the analysis:

- Channel mobility in this reach of the Elbow River is naturally high. Attempting to prevent or control that mobility would be difficult and would have morphological consequences upstream and downstream. Therefore the design should accept and expect river mobility as much as possible. The design philosophy should be to protect the infrastructure (i.e. road embankment, bridge piers and abutments, and stormwater ponds) to an appropriate level, rather than to attempt to control the river.
- The design of the stormwater ponds, bridge abutments, and bridge piers should consider potential changes in river position and profile, including the likely future avulsion from river km 21 to the realigned channel, likely erosion through the bridge opening, and possible future migration of the upstream channel to any point across the width of the valley.
- AT should monitor the migration of meanders within approximately a kilometer upstream and downstream of the proposed bridge crossing annually after each flood season. In addition, the City should monitor the migration of meanders within the City limits at the same monitoring frequency.
- The hydrotechnical assessments of the SWCRR bridges were based on the 2007 and 2015 conceptual-level bridge and road designs provided by Alberta Transportation. Upon completion of the final design, the conclusions of this report should be confirmed by the City to verify that the final crossing configuration (i.e. bridge span, river realignment, and minimum road elevation) does not have adverse impacts to the Glenmore Dam, Glenmore Trail SW Causeway, and the Southeast Dyke.

8 CLOSING

This report is an instrument of service of Klohn Crippen Berger Ltd. The report has been prepared for the exclusive use of The City of Calgary (Client) and Alberta Transportation for the specific application to the Assessments of Elbow River Upstream Bridge Structures Impact on Glenmore Dam. The report's contents may not be relied upon by any other party without the express written permission of Klohn Crippen Berger. In this report, Klohn Crippen Berger has endeavoured to comply with generally-accepted professional practice common to the local area. Klohn Crippen Berger makes no warranty, express or implied.

KLOHN CRIPPEN BERGER LTD.

November 24, 2015
Rob Cheetham, P.Eng.
Senior Civil Engineer

APEGA Permit to Practice No. P09196

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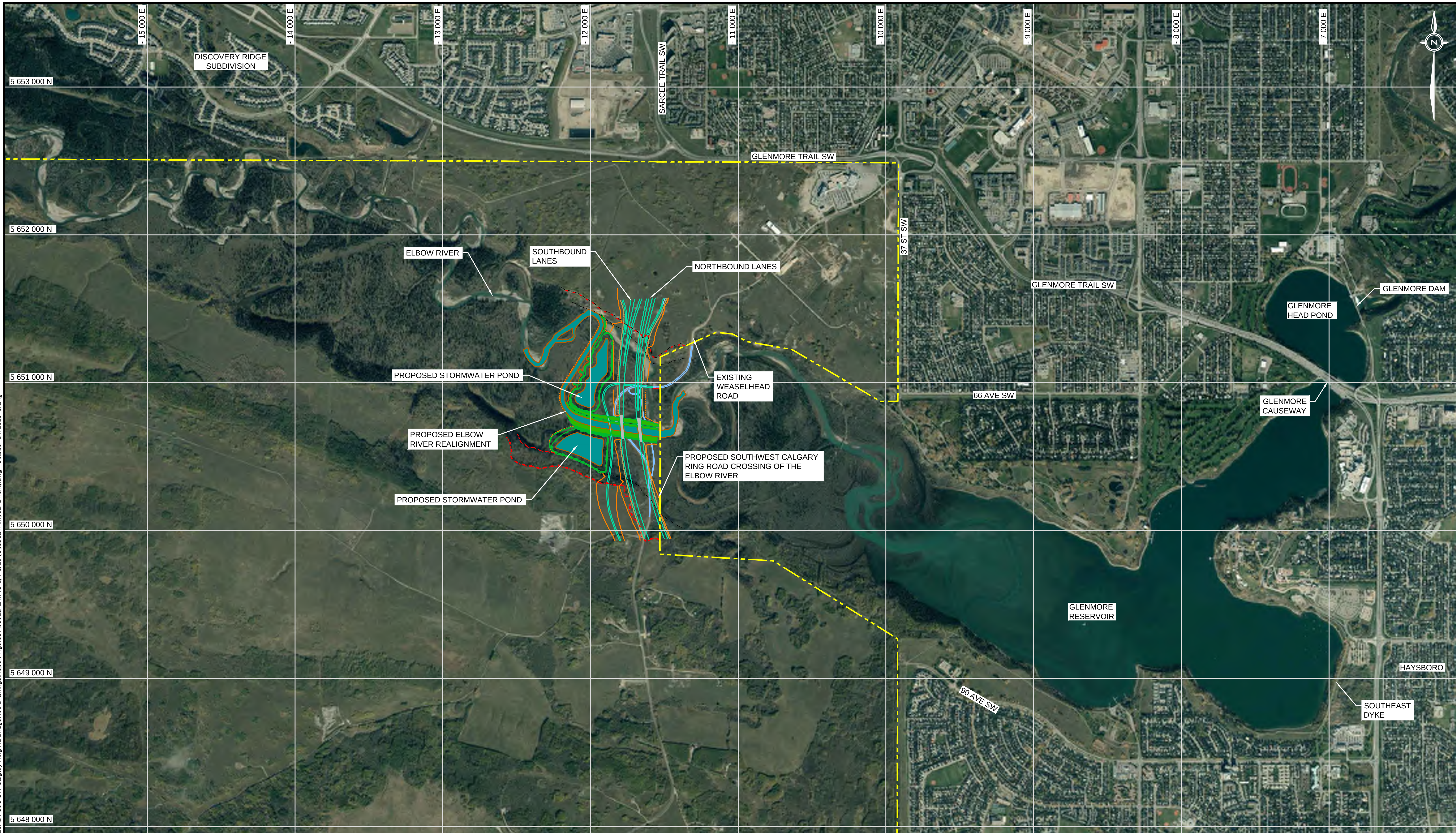
FIGURES



\\int.klohn.com\Proj\Data\A\CGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 1, 7 & 20 (Upstream Impoundment).dwg October 14, 2015 slang

NOTES:

1. SWCRR IMPOUNDMENT UP TO ELEVATION 1090 SHOWN.
2. CONTOUR INTERVAL 1 m.
3. 2015 DESIGN LAYOUT SHOWN.
4. 2013 FLOOD ORTHO PHOTO PROVIDED BY THE CITY OF CALGARY.

LEGEND:

CITY LIMITS

SCALE 1:25 000 0 250 500 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES

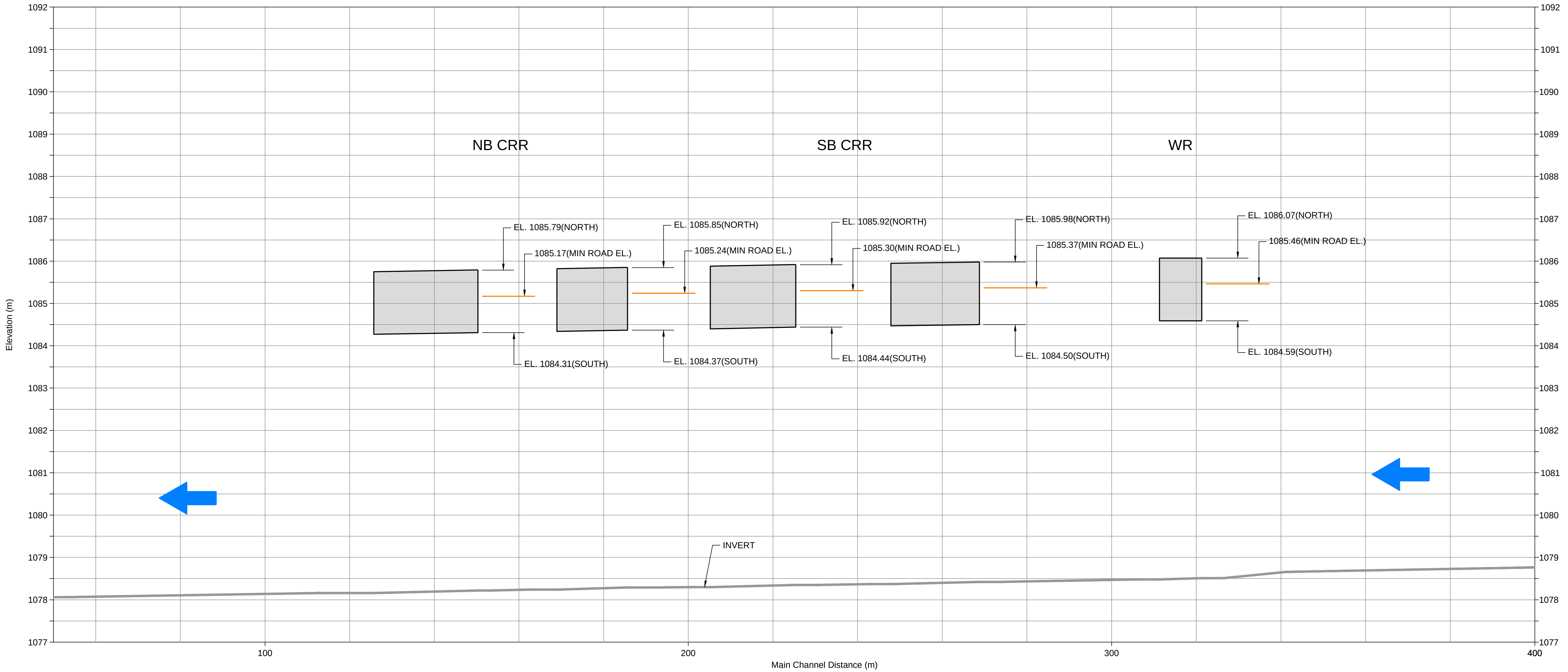
Klohn Crippen Berger

PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

TITLE: LOCATION PLAN

SCALE	PROJECT No.	FIG. No.
1:25 000	A03018A24	FIGURE 1

KCB-FIG-B-L



PROFILE

SCALE VER 1:100
HOR 1:1 000

LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

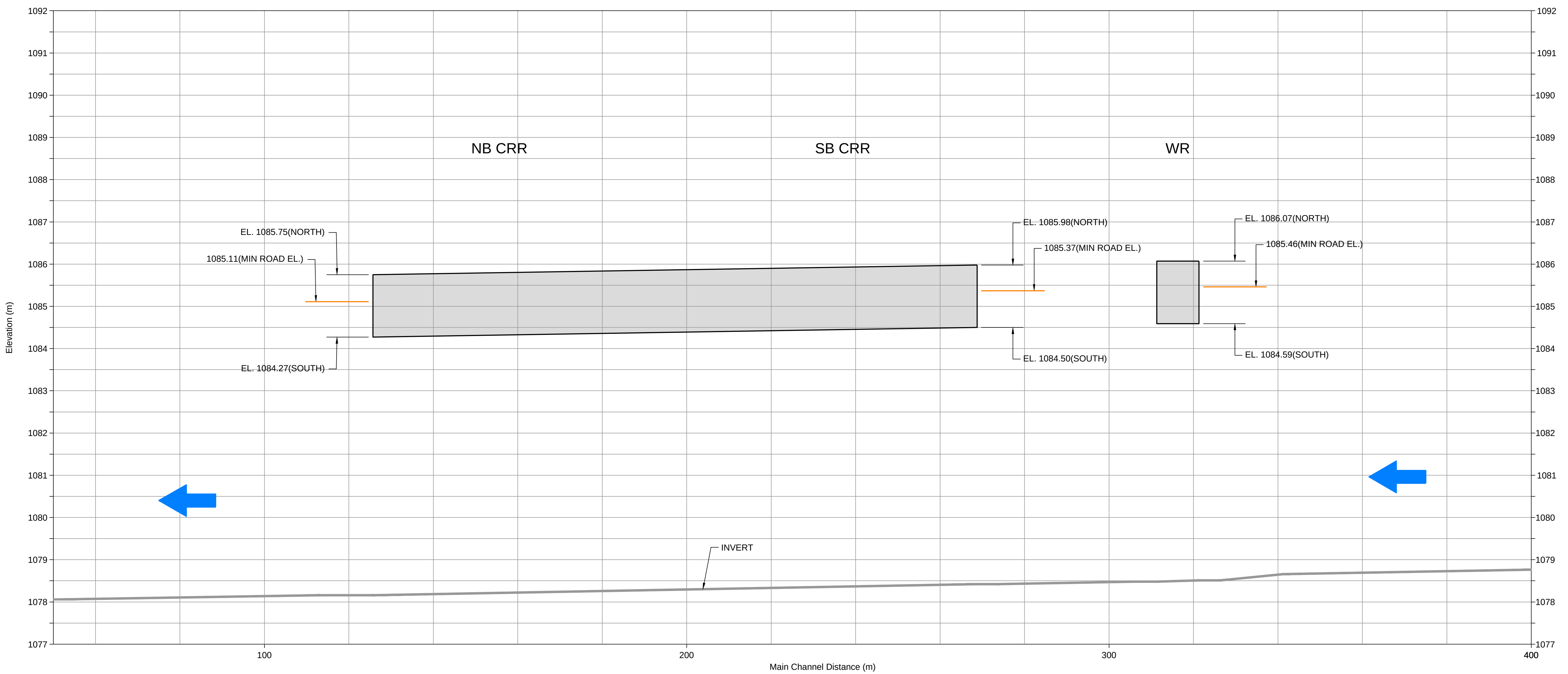
CLIENT: THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

TITLE: BRIDGE SCHEMATIC - 2007 DESIGN NB AND SB BRIDGES ASSUMED AS SEPARATE BRIDGES

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 2



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT
THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

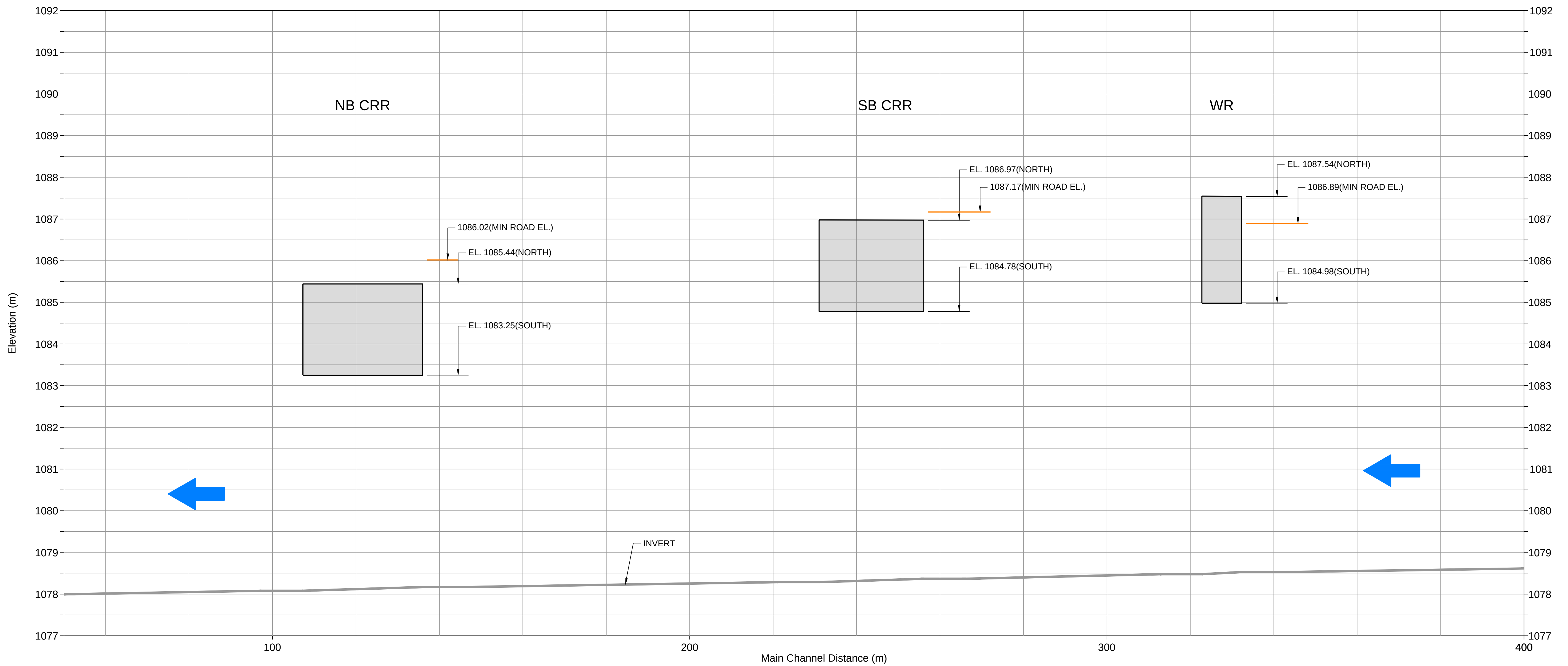
PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
BRIDGE SCHEMATIC - 2007 DESIGN
NB AND SB BRIDGES ASSUMED AS
SINGLE (PARALLEL) BRIDGE

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 3

KCB-FIG-B-L

Z:\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 2-5 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 14, 2015 vcastillo



PROFILE

SCALE VER 1:100
HOR 1:1 000

LEGEND:

NB CRR NORTHBOUND CALGARY RING ROAD
SB CRR SOUTHBOUND CALGARY RING ROAD
WR WEASELHEAD ROAD

CHANNEL INVERT
RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
MINIMUM TOP OF ROAD ELEVATION

NOTE:

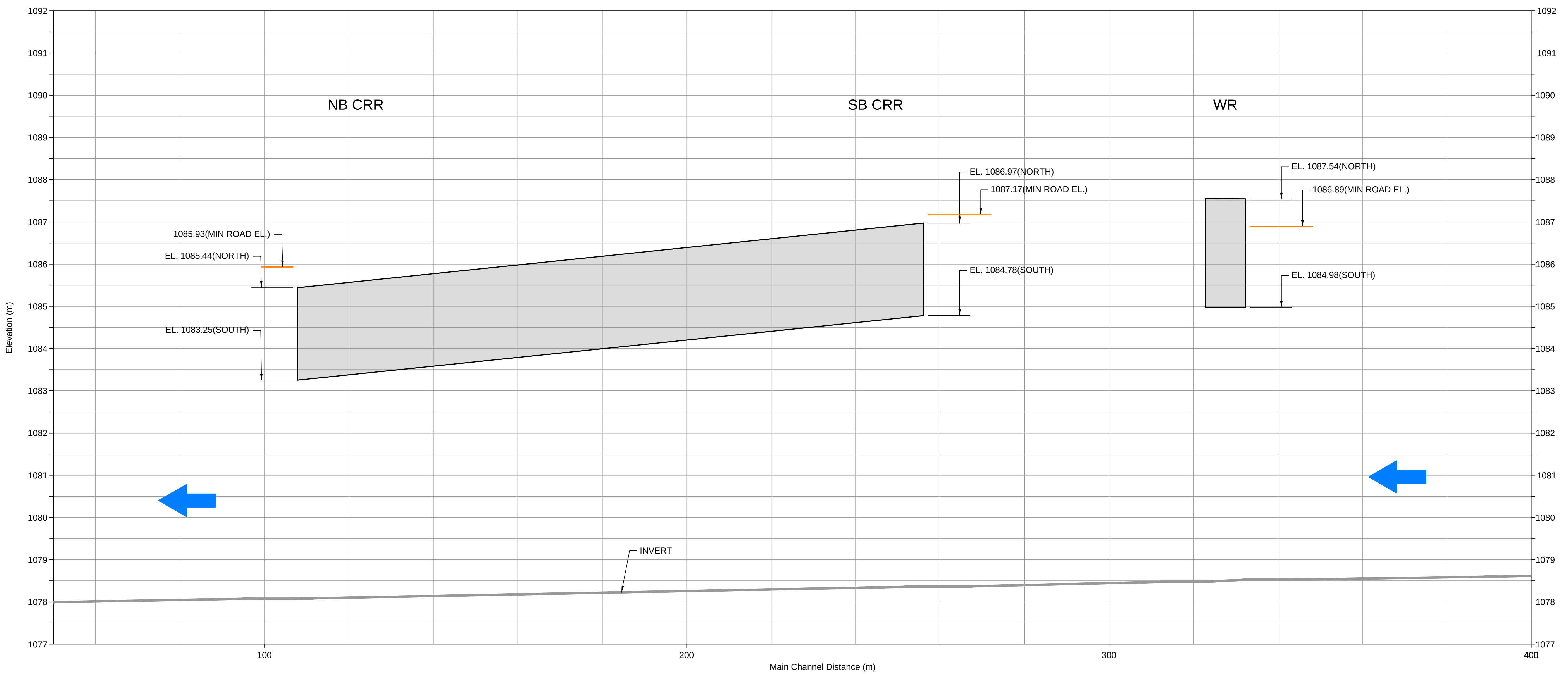
1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES	PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT
Klohn Crippen Berger	TITLE: BRIDGE SCHEMATIC - 2015 DESIGN NB AND SB BRIDGES ASSUMED AS SEPARATE BRIDGES
SCALE: AS SHOWN	PROJECT No.: A03018A24
FIG. No.: FIGURE 4	



PROFILE

SCALE VER 1:100
HOR 1:1 000

LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD

CHANNEL INVERT

RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)

MINIMUM TOP OF ROAD ELEVATION

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT
THE CITY OF CALGARY
WATER RESOURCES

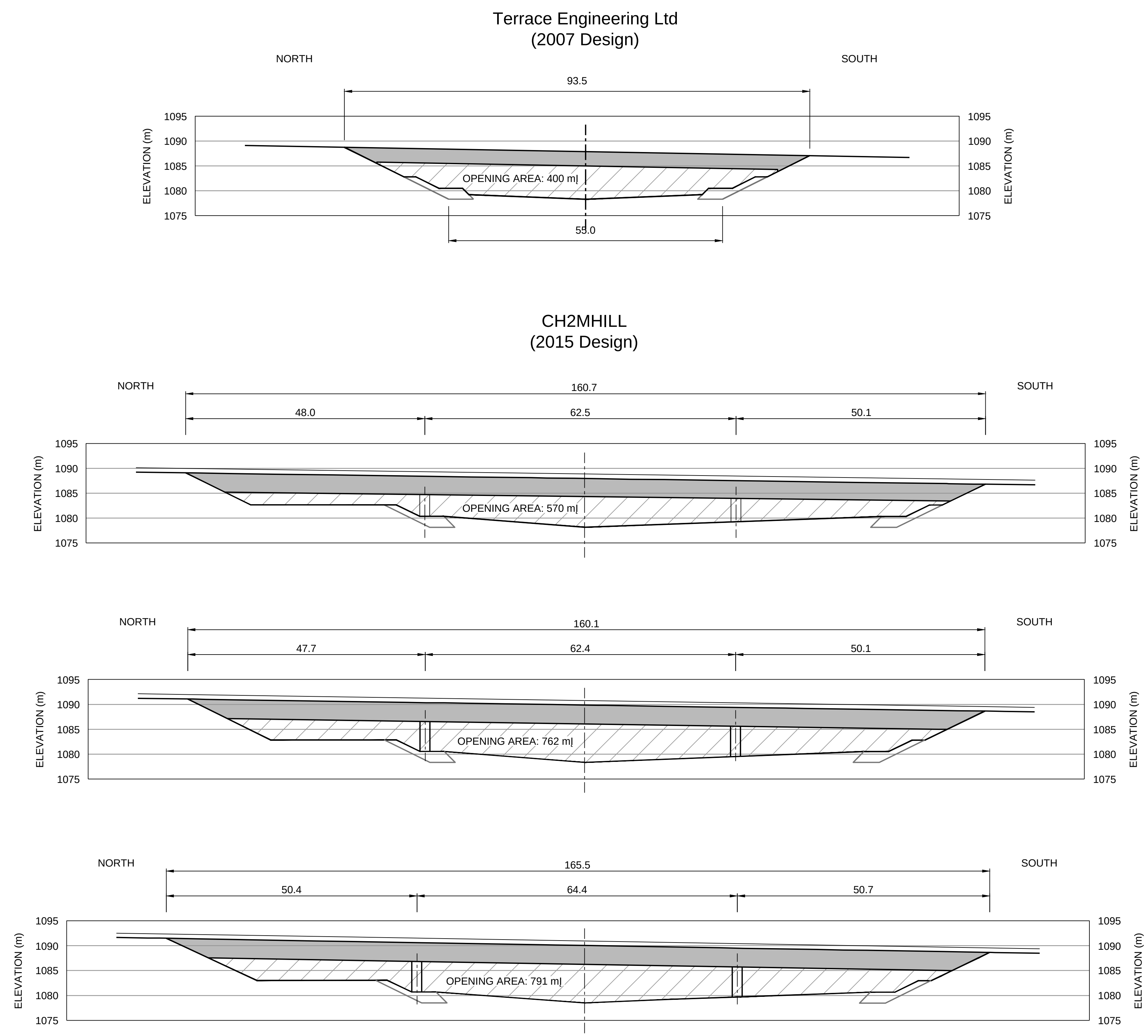
Klohn Crippen Berger

PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
BRIDGE SCHEMATIC - 2015 DESIGN
NB AND SB BRIDGES ASSUMED AS SINGLE (PARALLEL) BRIDGE

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 5

Z:\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 2-5 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 14, 2015 vcastillo



SCALE 1:1,000 0 10 20 m

LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	DECK / ROADWAY LOW CHORD
	OPENING AREA

CLIENT

THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

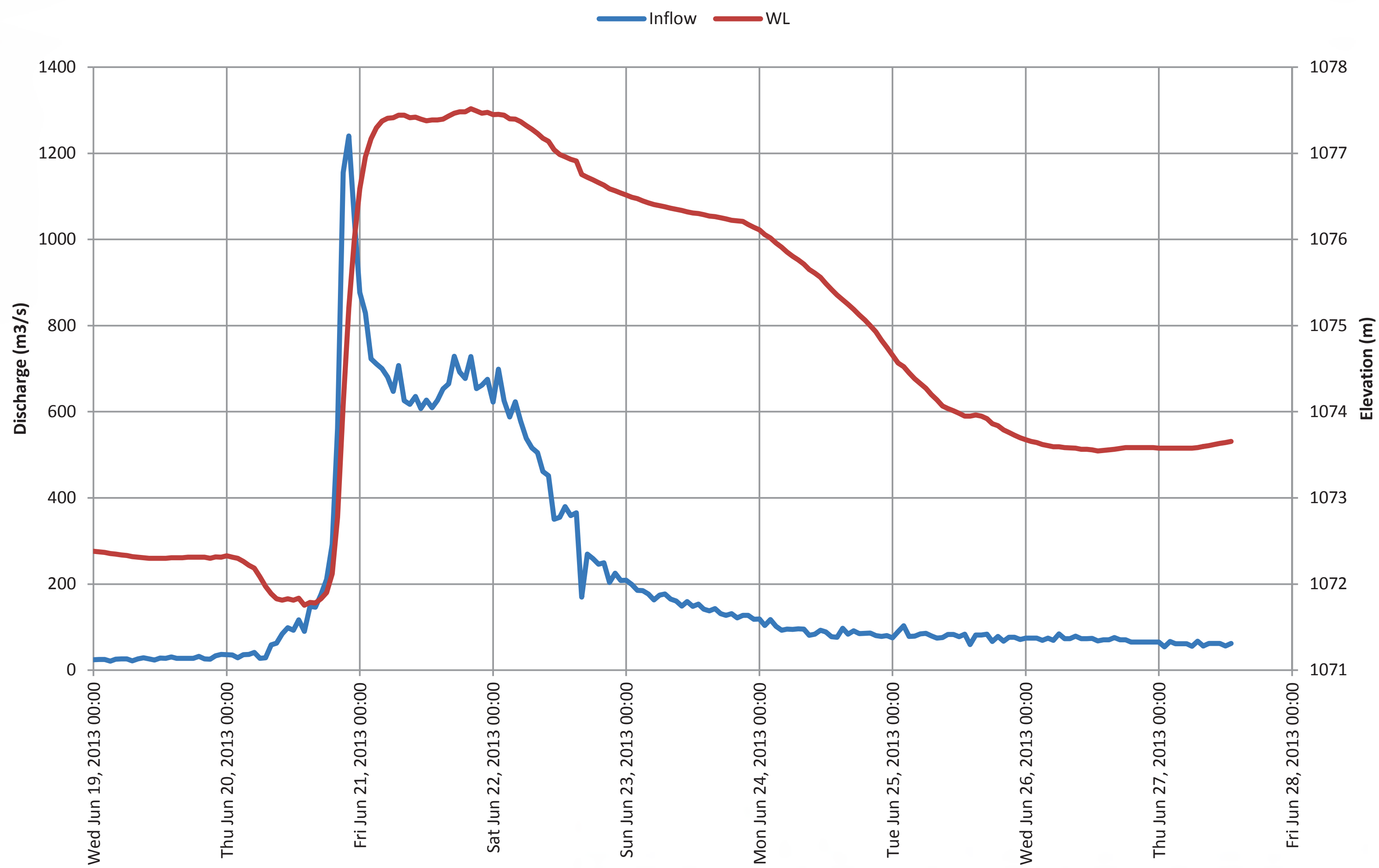
PROJECT

SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE

BRIDGE DESIGNS

SCALE	PROJECT No.	FIG. No.
1:1 000	A03018A24	FIGURE 6



NOTE: HYDROGRAPHS PROVIDED BY THE CITY OF CALGARY

CLIENT

THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

PROJECT

SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE

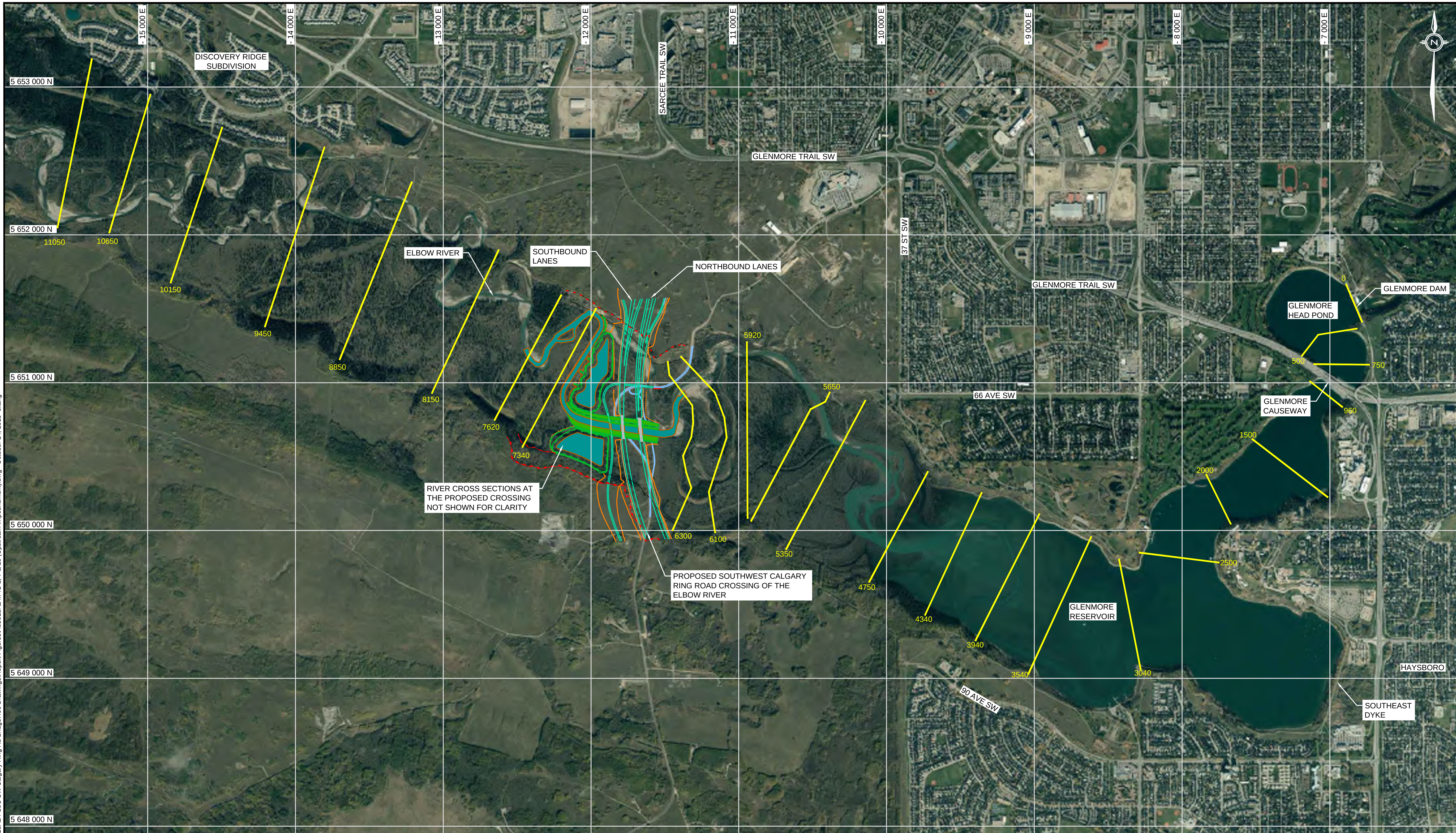
JUNE 2013 FLOOD
RESERVOIR INFLOW AND WATER LEVEL

SCALE	PROJECT No.	FIG. No.
N.T.S	A03018A24	FIGURE 7

NOTE:

1. AERIAL PHOTOGRAPH TAKEN JUNE 22, 2013 BETWEEN 8:00 am - 9:30 am.

CLIENT	PROJECT		
THE CITY OF CALGARY WATER RESOURCES	SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT		
Klohn Crippen Berger	TITLE: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT		
	SCALE: N.T.S	PROJECT No.: A03018A24	FIG. No.: FIGURE 8



NOTES:

1. SWCRR IMPOUNDMENT UP TO ELEVATION 1090 SHOWN.
2. CONTOUR INTERVAL 1 m.
3. 2015 DESIGN LAYOUT SHOWN.
4. 2013 FLOOD ORTHO PHOTO PROVIDED BY THE CITY OF CALGARY.

LEGEND:

CROSS SECTION LINE

SCALE 1:25 000 0 250 500 m

CLIENT
THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
HYDRAULIC MODEL LAYOUT

SCALE	PROJECT No.	FIG. No.
1:25 000	A03018A24	FIGURE 9

KCB-FIG-B-L

\\int.klohn.com\Proj\Data\AI\CGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 1, 7 & 20 (Upstream Impoundment).dwg October 14, 2015 slang

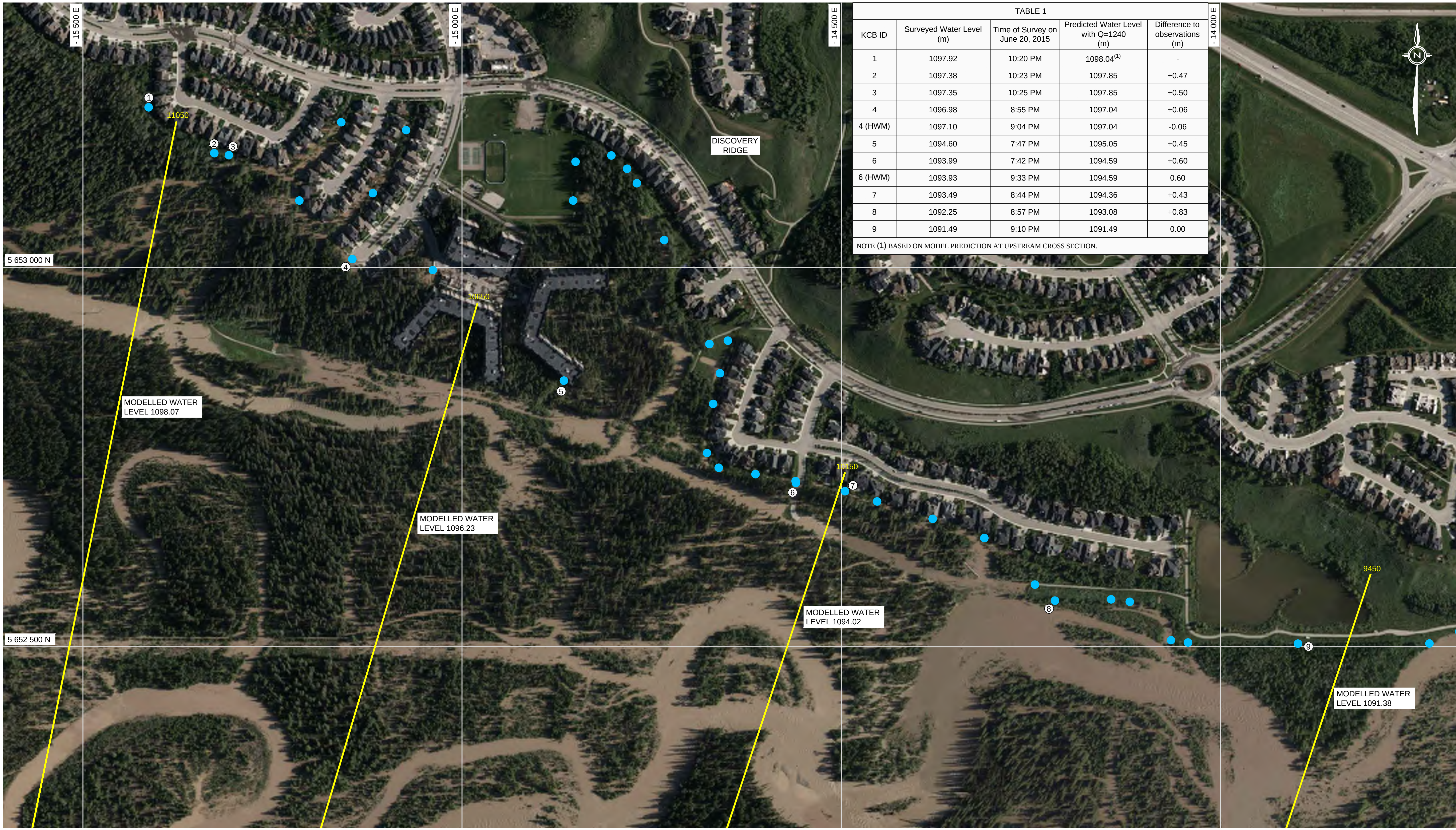


TABLE 1

KCB ID	Surveyed Water Level (m)	Time of Survey on June 20, 2015	Predicted Water Level with Q=1240 (m)	Difference to observations (m)
1	1097.92	10:20 PM	1098.04(1)	-
2	1097.38	10:23 PM	1097.85	+0.47
3	1097.35	10:25 PM	1097.85	+0.50
4	1096.98	8:55 PM	1097.04	+0.06
4 (HWM)	1097.10	9:04 PM	1097.04	-0.06
5	1094.60	7:47 PM	1095.05	+0.45
6	1093.99	7:42 PM	1094.59	+0.60
6 (HWM)	1093.93	9:33 PM	1094.59	0.60
7	1093.49	8:44 PM	1094.36	+0.43
8	1092.25	8:57 PM	1093.08	+0.83
9	1091.49	9:10 PM	1091.49	0.00

NOTE (1) BASED ON MODEL PREDICTION AT UPSTREAM CROSS SECTION.

LEGEND:

1095.2 ● ESTIMATED WATER LEVEL WITH EXISTING WEASELHEAD BRIDGE, 2007 AND 2015 SWCRR DESIGN AT Q=1,240 m³/s

● 2013 FLOOD DATA

① ● SEE TABLE 1 FOR OBSERVED WATER LEVEL

HYDRAULIC MODEL CROSS SECTION AND STATION NUMBER

NOTES:

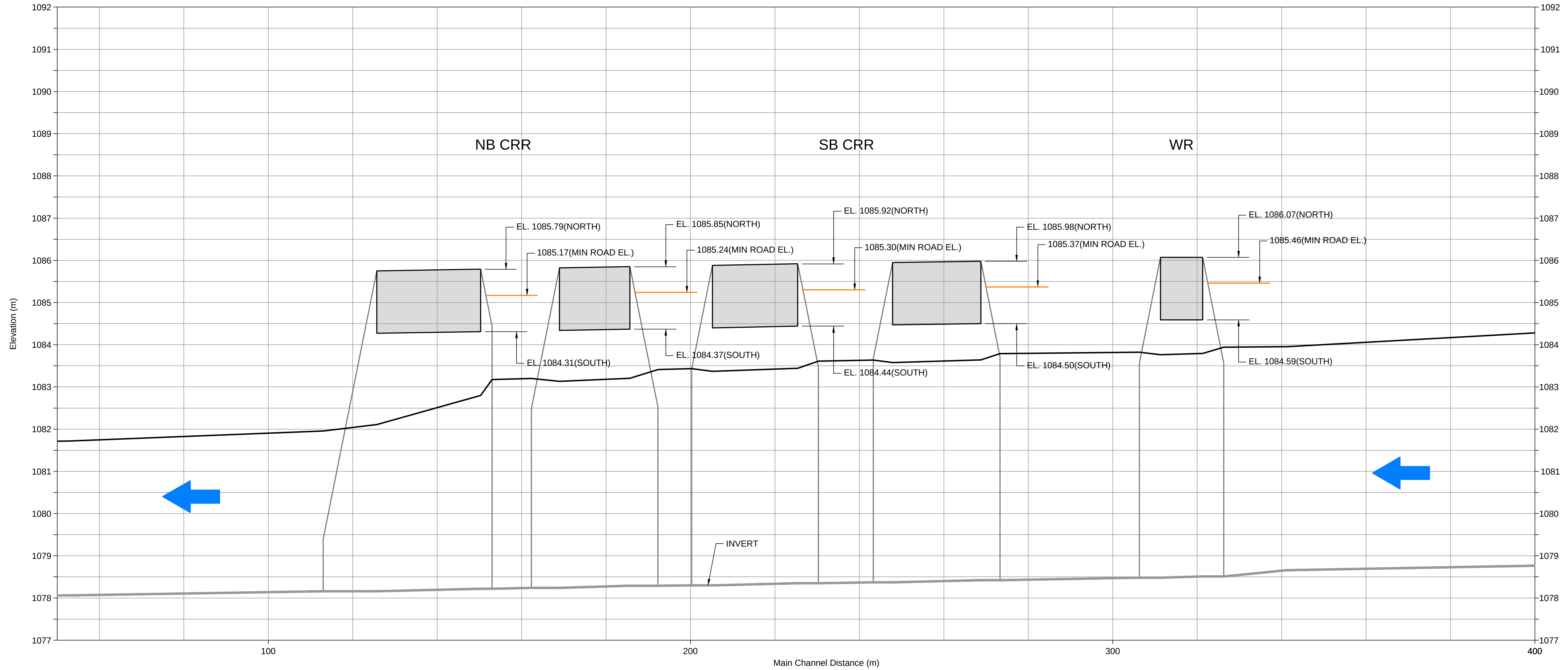
1. 2013 FLOOD ORTHO PHOTO PROVIDED BY THE CITY OF CALGARY. PHOTOGRAPH WAS TAKEN SOMETIME BETWEEN 08:00 - 09:40 ON JUNE 22, 2013, APPROXIMATELY 34 HOURS AFTER THE PEAK DISCHARGE ON THE ELBOW RIVER.

SCALE 1:5 000 0 50 100 m

CLIENT	THE CITY OF CALGARY WATER RESOURCES
	Klohn Crippen Berger
PROJECT	SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT
TITLE	COMPARISON OF PREDICTED AND OBSERVED WATER LEVELS AT DISCOVERY RIDGE DURING THE JUNE 2013 EVENT
SCALE	1:5 000
PROJECT No.	A03018A24
FIG. No.	FIGURE 10

\\int.klohn.com\Proj\Data\AI\CGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 10 (Discovery Ridge - 100yr WL).dwg October 14, 2015 sliang

KCB-FIG-B-L



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD

CHANNEL INVERT

RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)

MINIMUM TOP OF ROAD ELEVATION

1:100 YEAR WATER LEVELS

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

PROFILE

SCALE VER 1:100
HOR 1:1 000

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

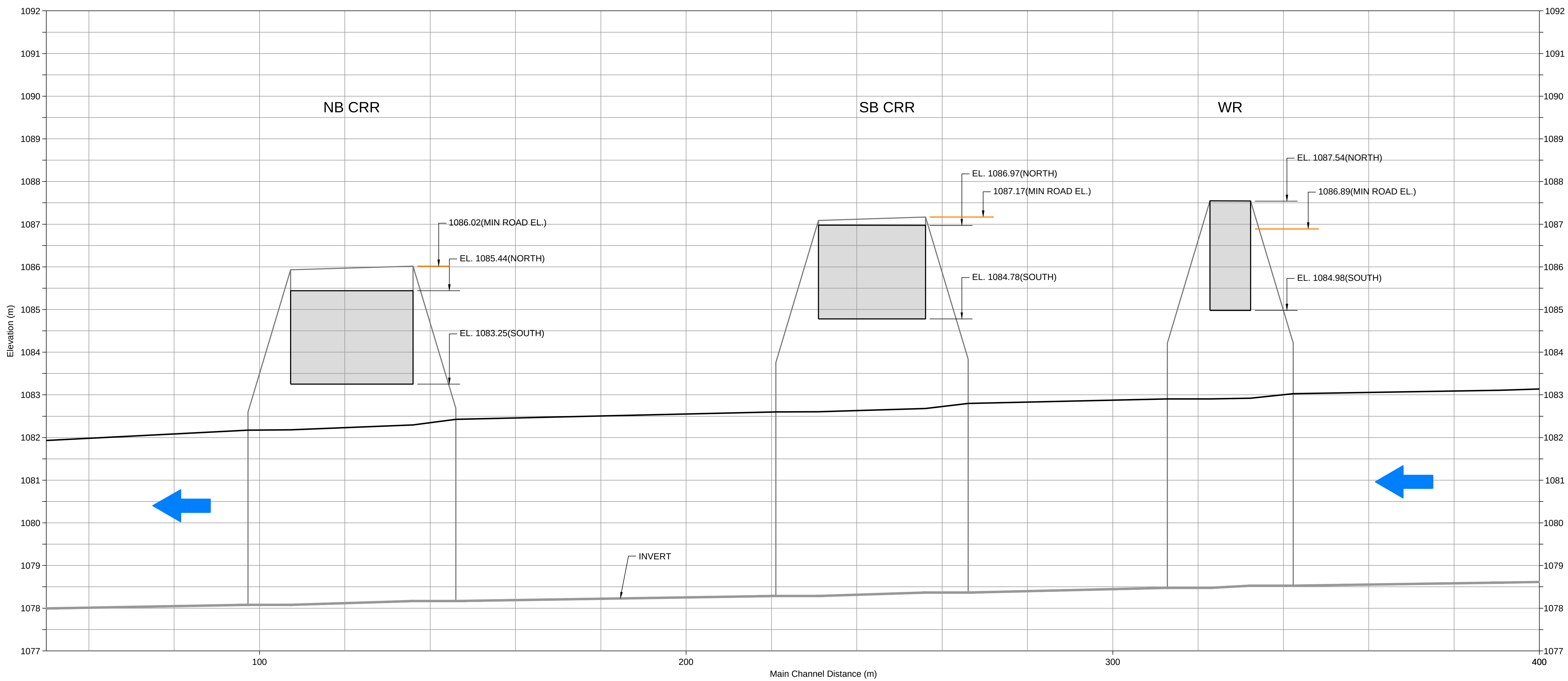
CLIENT: THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

TITLE: 1:100-YR WSEL PROFILE - 2007 DESIGN NB AND SB BRIDGES ASSUMED AS SEPARATE BRIDGES

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 11



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
(grey line)	CHANNEL INVERT
(shaded box)	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
(orange line)	MINIMUM TOP OF ROAD ELEVATION
(black line)	1:100 YEAR WATER LEVELS

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES

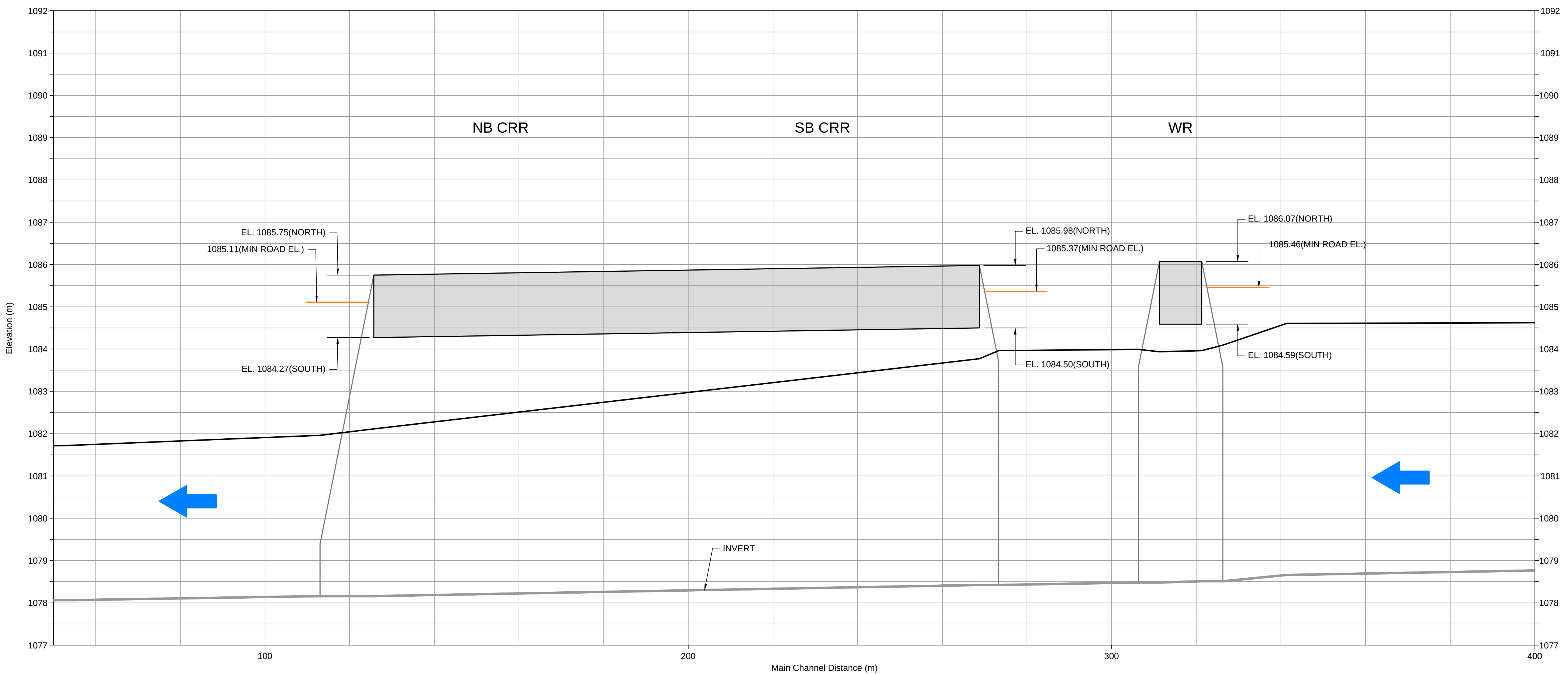
Klohn Crippen Berger

PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

TITLE: 1:100-YR WSEL PROFILE - 2015 DESIGN NB AND SB BRIDGES ASSUMED AS SEPARATE BRIDGES

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 12

\\int.klohn.com\Proj\Data\CALGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24.FIG 3-6 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 07, 2015 sliang



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION
	1:100 YEAR WATER LEVELS

PROFILE
SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

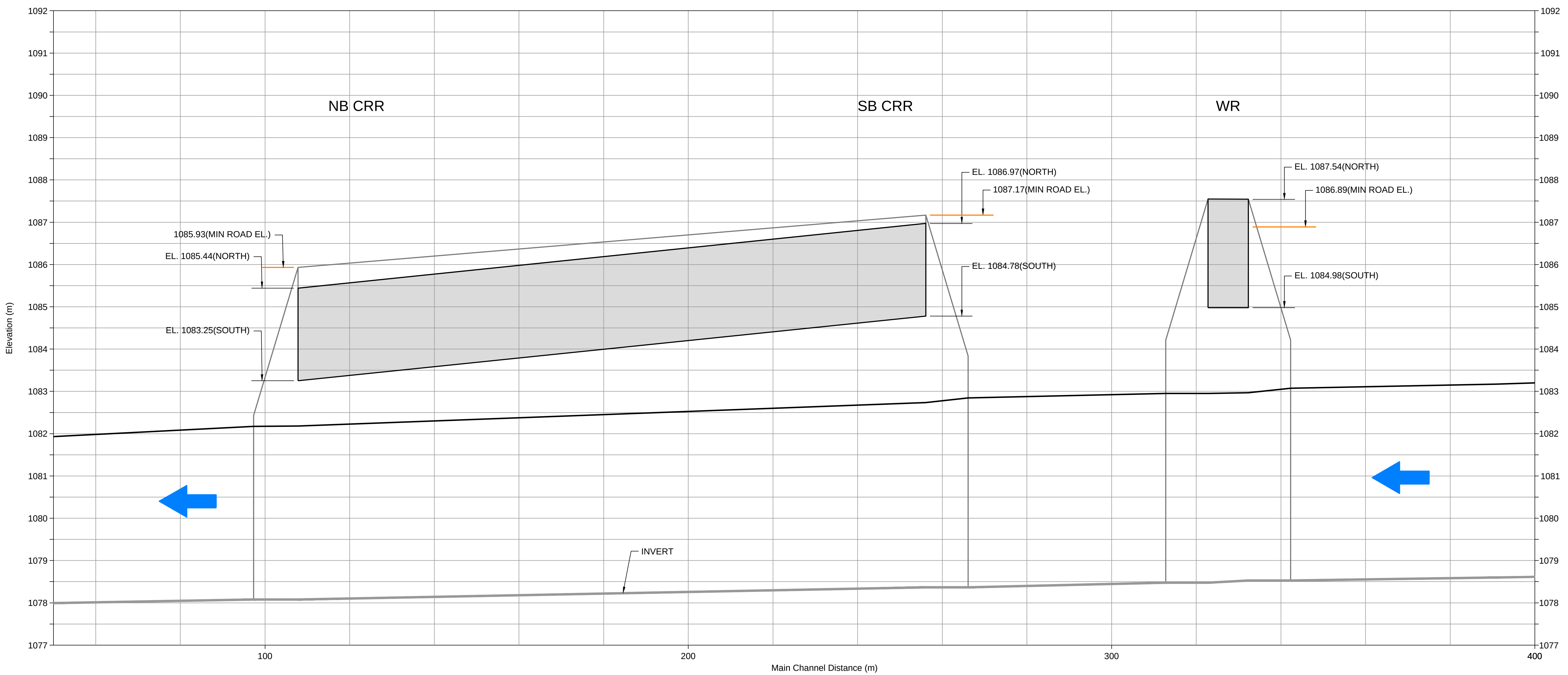
CLIENT
THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
1:100-YR WSEL PROFILE - 2007 DESIGN
NB AND SB BRIDGES ASSUMED AS
SINGLE (PARALLEL) BRIDGE

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 13



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION
	1:100 YEAR WATER LEVELS

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT
THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

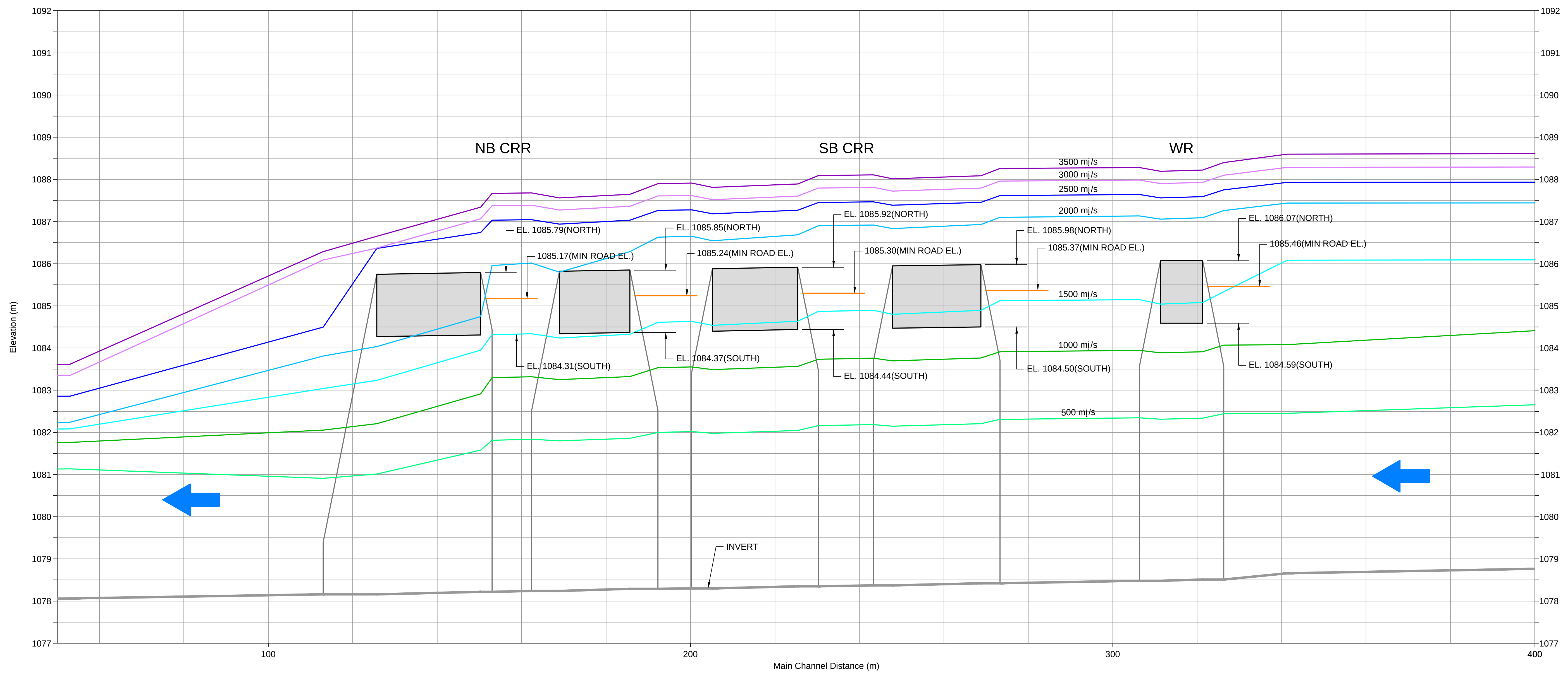
PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
1:100-YR WSEL PROFILE - 2015 DESIGN
NB AND SB BRIDGES ASSUMED AS
SINGLE (PARALLEL) BRIDGE

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 14

KCB-FIG-B-L

\\int.klohn.com\Proj\Data\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24.FIG 3-6 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 07, 2015 sliang



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION

RIVER DISCHARGE:

- 3500 m³/s
- 3000 m³/s
- 2500 m³/s
- 2000 m³/s
- 1500 m³/s
- 1000 m³/s
- 500 m³/s

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT
THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

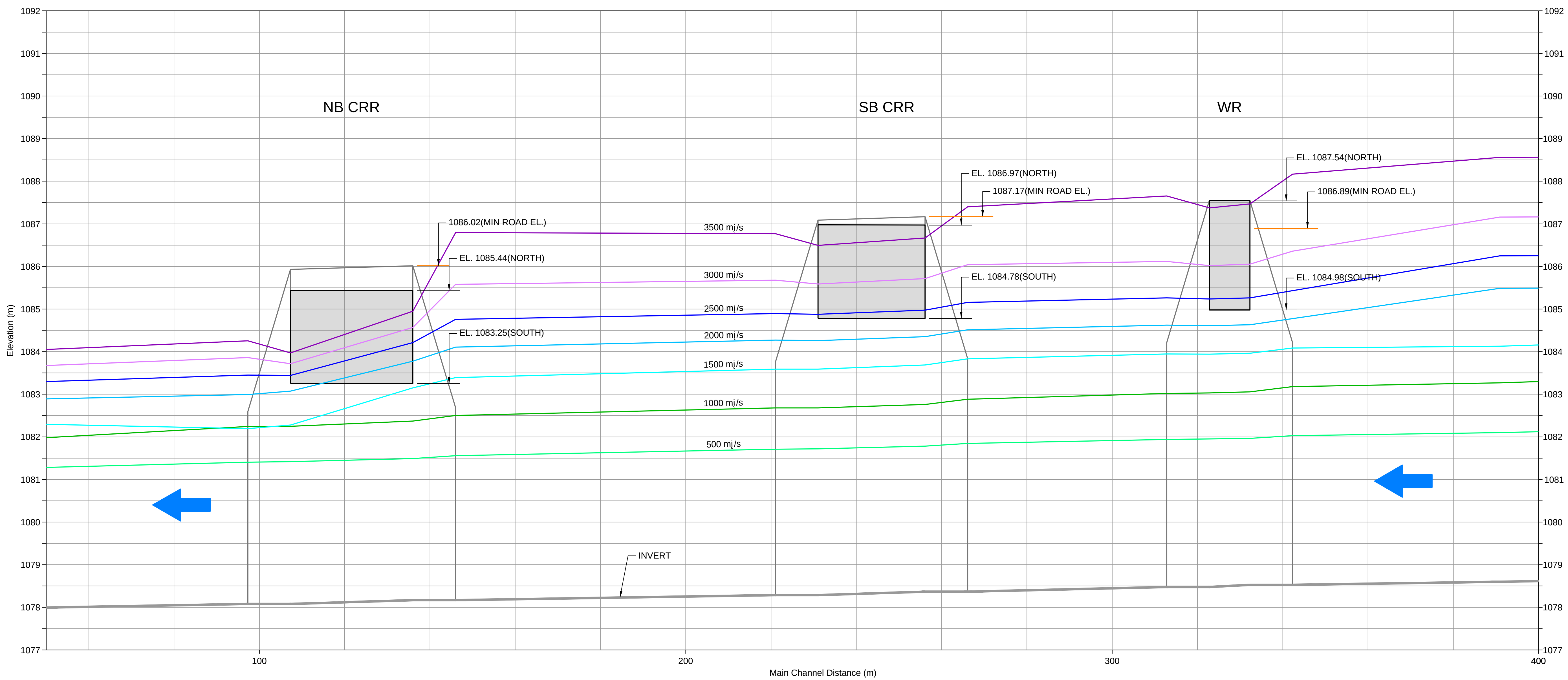
PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
WSEL PROFILES - 2007 DESIGN
NB AND SB BRIDGES ASSUMED AS
SEPARATE BRIDGES

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 15

\\int.klohn.com\Proj\Data\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24.FIG 3-6 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 07, 2015 sliang

KCB-FIG-B-L



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION

RIVER DISCHARGE:

- 3500 m^3/s
- 3000 m^3/s
- 2500 m^3/s
- 2000 m^3/s
- 1500 m^3/s
- 1000 m^3/s
- 500 m^3/s

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

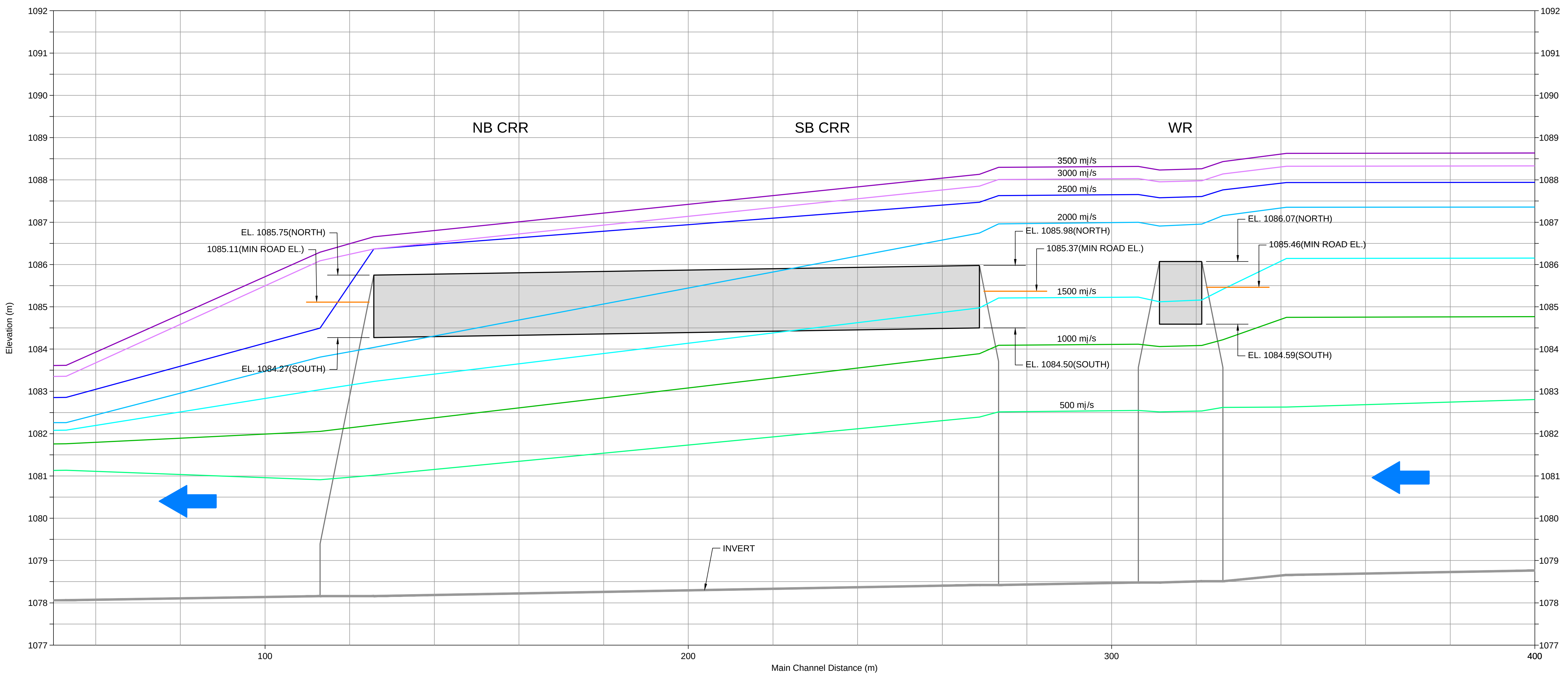
CLIENT
THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
WSEL PROFILES - 2015 DESIGN
NB AND SB BRIDGES ASSUMED AS
SEPARATE BRIDGES

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 16



LEGEND:

NB CRR	NORTHBOUND CALGARY RING ROAD
SB CRR	SOUTHBOUND CALGARY RING ROAD
WR	WEASELHEAD ROAD
	CHANNEL INVERT
	RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
	MINIMUM TOP OF ROAD ELEVATION

RIVER DISCHARGE:

- 3500 m³/s
- 3000 m³/s
- 2500 m³/s
- 2000 m³/s
- 1500 m³/s
- 1000 m³/s
- 500 m³/s

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT
THE CITY OF CALGARY
WATER RESOURCES

Klohn Crippen Berger

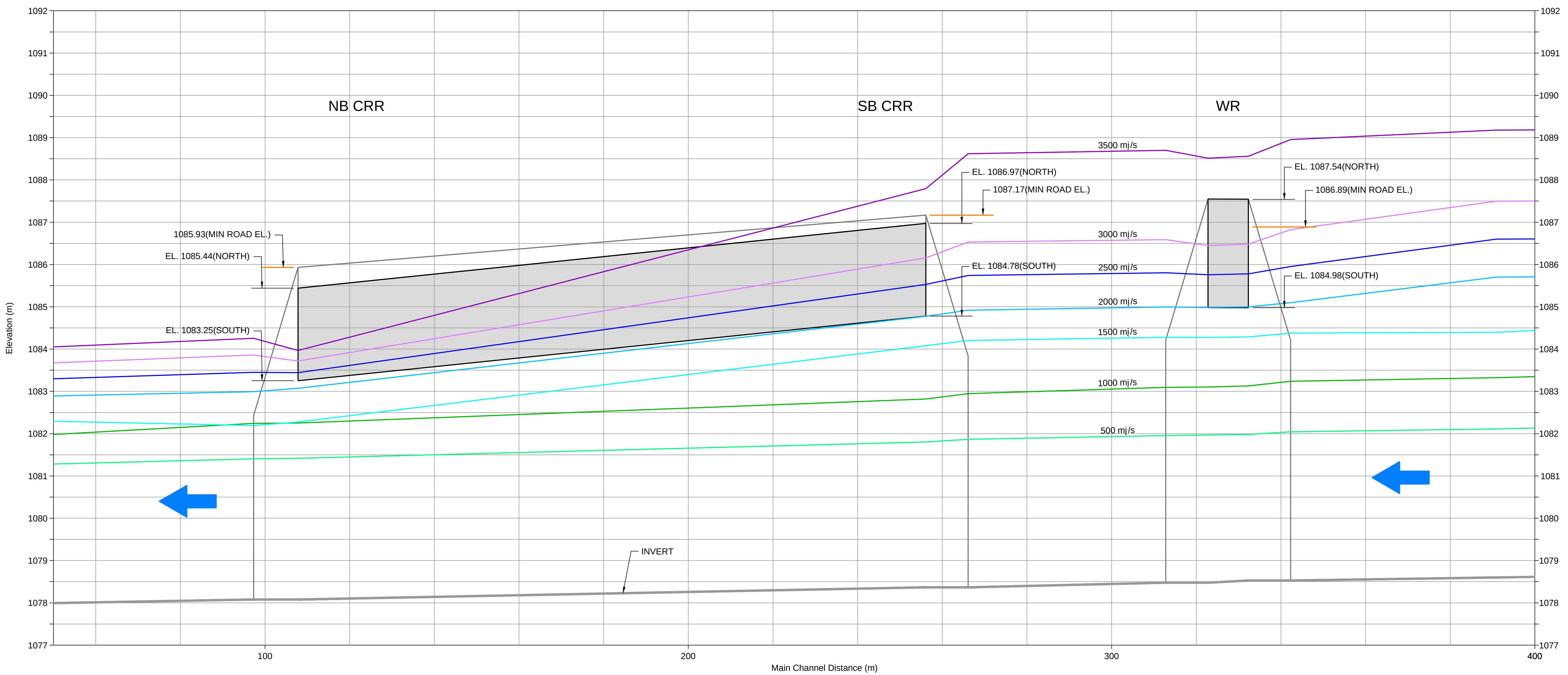
PROJECT
SW CALGARY RING ROAD BRIDGE
HYDRAULIC ASSESSMENT

TITLE
WSEL PROFILES - 2007 DESIGN
NB AND SB BRIDGES ASSUMED AS
SINGLE (PARALLEL) BRIDGE

SCALE	PROJECT No.	FIG. No.
AS SHOWN	A03018A24	FIGURE 17

\\int.klohn.com\ProjData\CGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 3-6 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 07, 2015 sliang

KCB-FIG-B-L



LEGEND:

NB CRR — NORTHBOUND CALGARY RING ROAD
SB CRR — SOUTHBOUND CALGARY RING ROAD
WR — WEASELHEAD ROAD
CHANNEL INVERT
RANGE OF DECK / ROADWAY LOW CHORD ELEVATIONS (SEE NOTE 1)
MINIMUM TOP OF ROAD ELEVATION

RIVER DISCHARGE:

3500 m³/s
3000 m³/s
2500 m³/s
2000 m³/s
1500 m³/s
1000 m³/s
500 m³/s

PROFILE

SCALE VER 1:100
HOR 1:1 000

NOTE:

1. THE SOUTHERN END OF THE BRIDGE IS LOWER THAN THE NORTHERN END OF THE BRIDGE.

HEC-RAS BASELINE MODEL PARAMETERS:

1. CHANNEL MANNING'S n =0.03, FLOODPLAIN MANNING'S n =0.15
2. CONTRACTION COEFFICIENT: 0.1, EXPANSION COEFFICIENT: 0.3
3. NO BLOCKAGE

VER SCALE 1:100 0 1 2 m
HOR SCALE 1:1 000 0 10 20 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

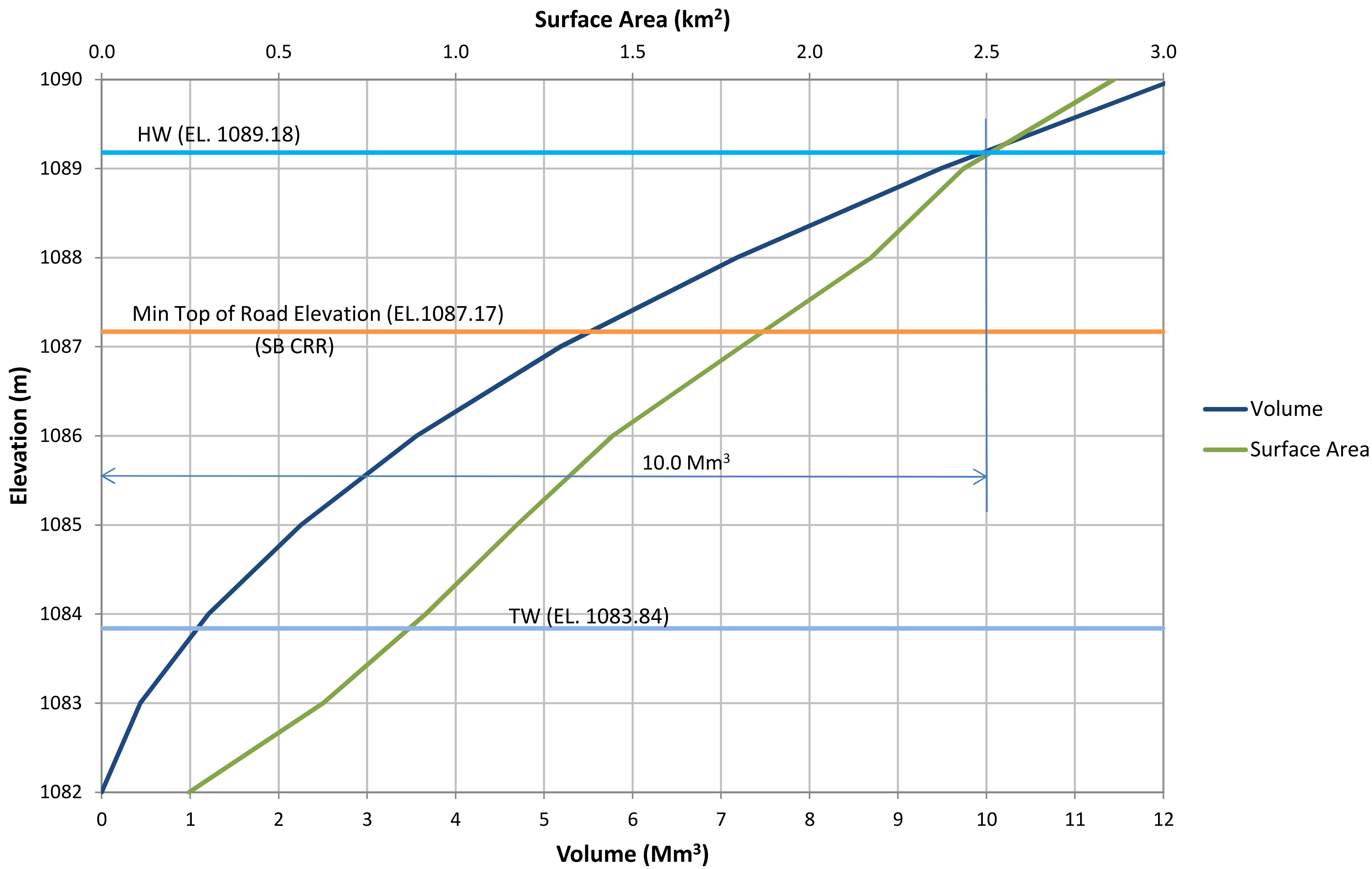
PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

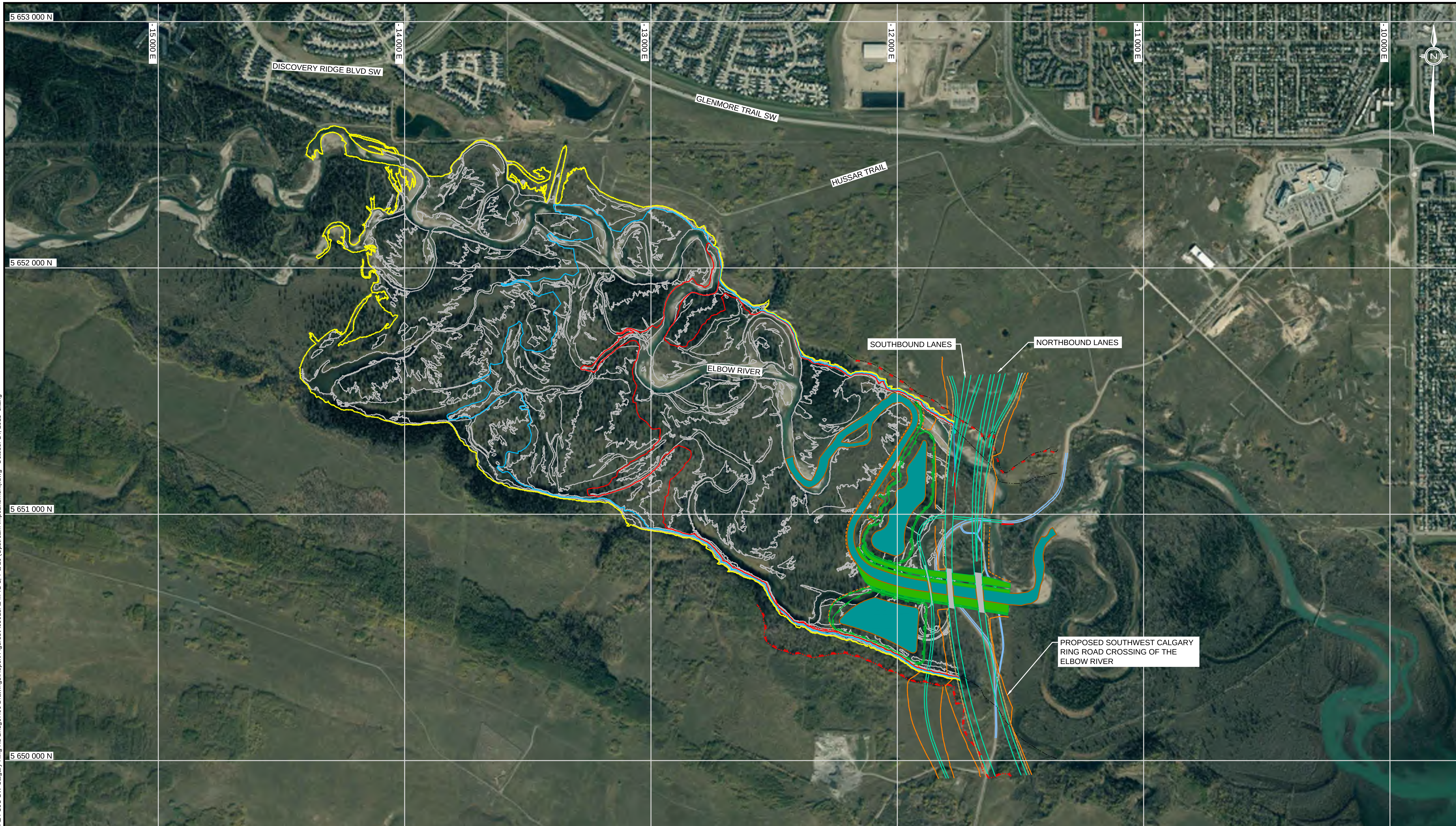
TITLE: WSEL PROFILES - 2015 DESIGN NB AND SB BRIDGES ASSUMED AS SINGLE (PARALLEL) BRIDGE

SCALE: AS SHOWN | PROJECT No.: A03018A24 | FIG. No.: FIGURE 18

\\int.klohn.com\Proj\Data\CGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24.FIG 3-6 & 11-18 (HEC-RAS - Bridge Profiles).dwg October 07, 2015 sliang

Figure 19
SWCRR Upstream Storage - Stage/Area/Volume Curves - 2015 Design





\\int.klohn.com\Proj\Data\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\Report Figures\A03018A24 FIG 1, 7 & 20 (Upstream Impoundment).dwg October 14, 2015 slang

NOTES:

1. SWCRR IMPOUNDMENT UP TO ELEVATION 1090 SHOWN.
2. CONTOUR INTERVAL 1 m.
3. 2015 DESIGN LAYOUT SHOWN.
4. 2013 FLOOD ORTHO PHOTO PROVIDED BY THE CITY OF CALGARY.

LEGEND:

- EL. 1087.2 CONTOUR
- EL. 1085.4 CONTOUR
- EL. 1090.0 CONTOUR

SCALE 1:15 000 0 150 300 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES	PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT
Klohn Crippen Berger	TITLE: SWCRR UPSTREAM IMPOUNDMENT
SCALE: 1:15 000	PROJECT No.: A03018A24
FIG. No.: FIGURE 20	

KCB-FIG-B-L

Figure 21 Breach Scenarios for the 2015 Road Crossing Design

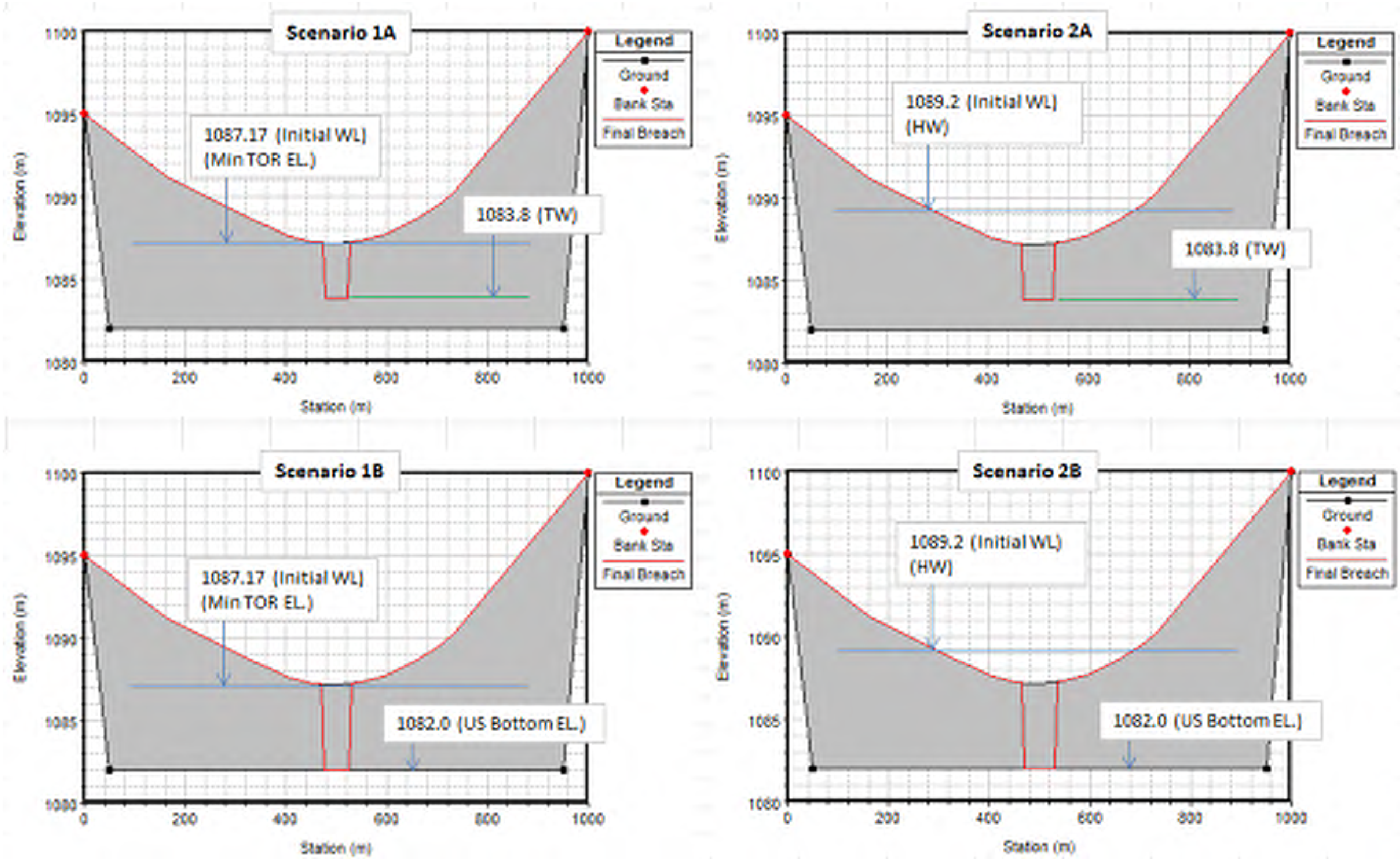


Figure 22
SWCRR Breach Discharge Hydrographs (Overtopping Failure) - 2015 Design

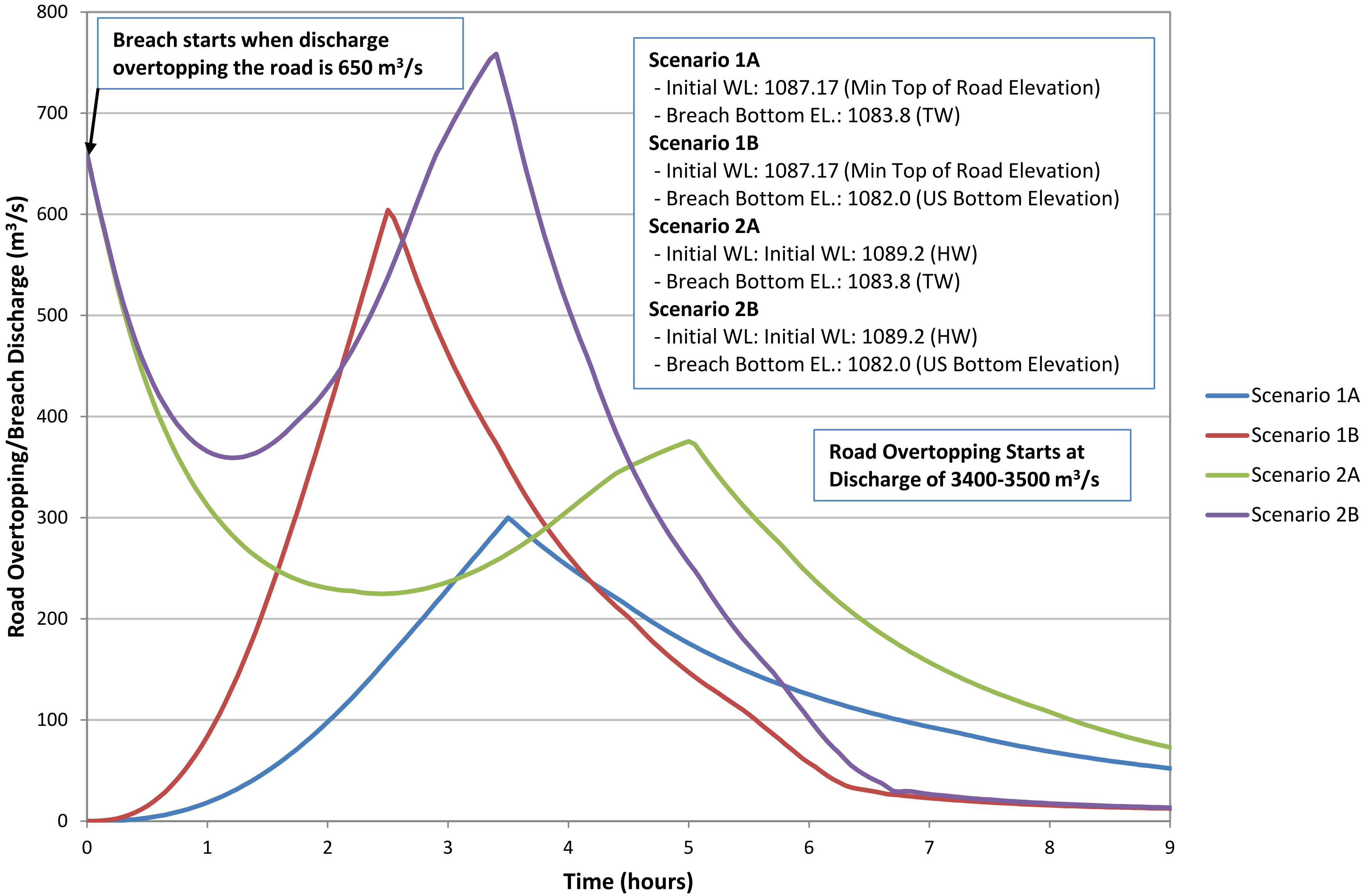


Figure 23
SWCRR Upstream Storage - Stage/Area/Volume Curves - 2007 Design

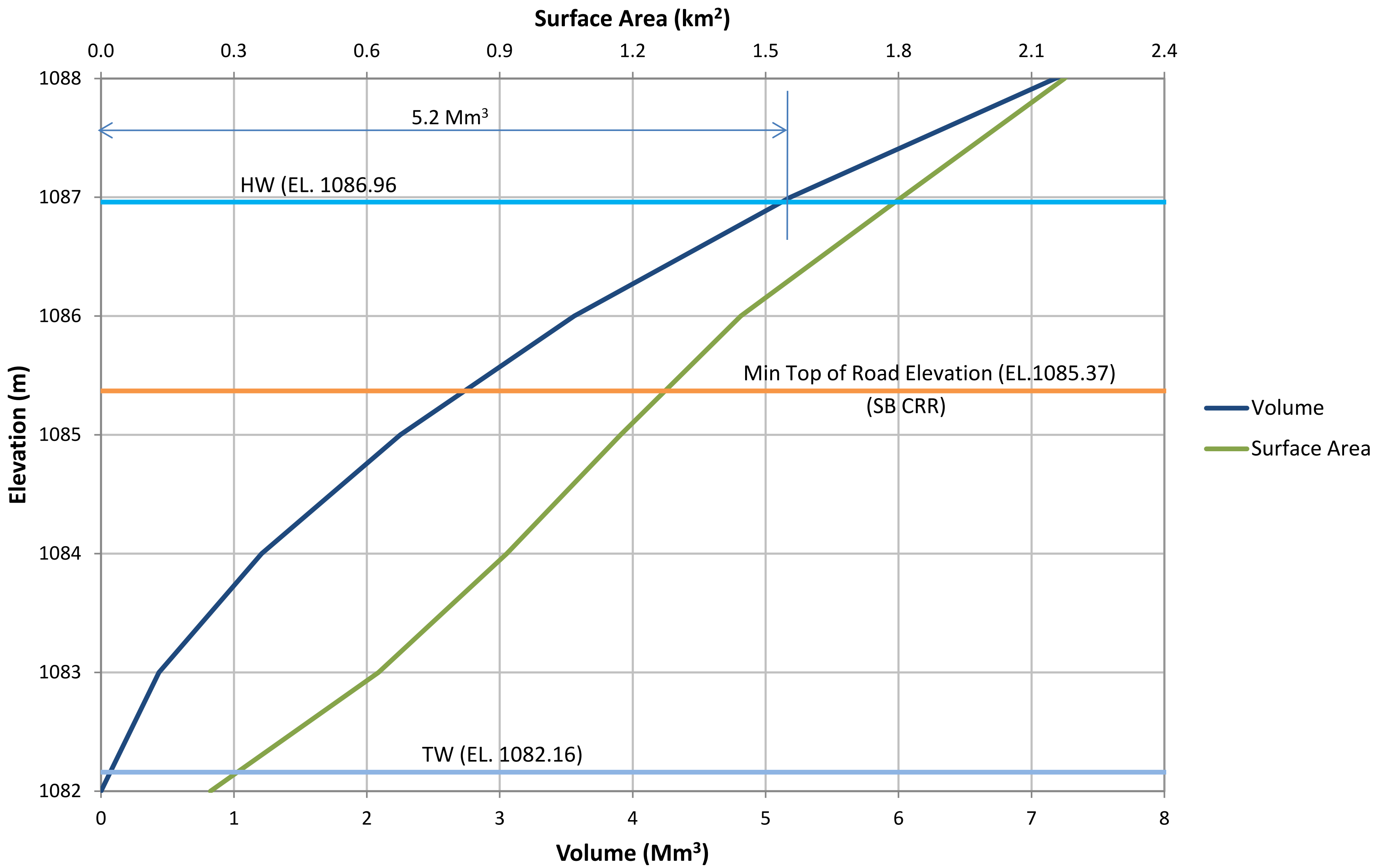


Figure 24 Breach Scenarios for the 2007 Road Crossing Design

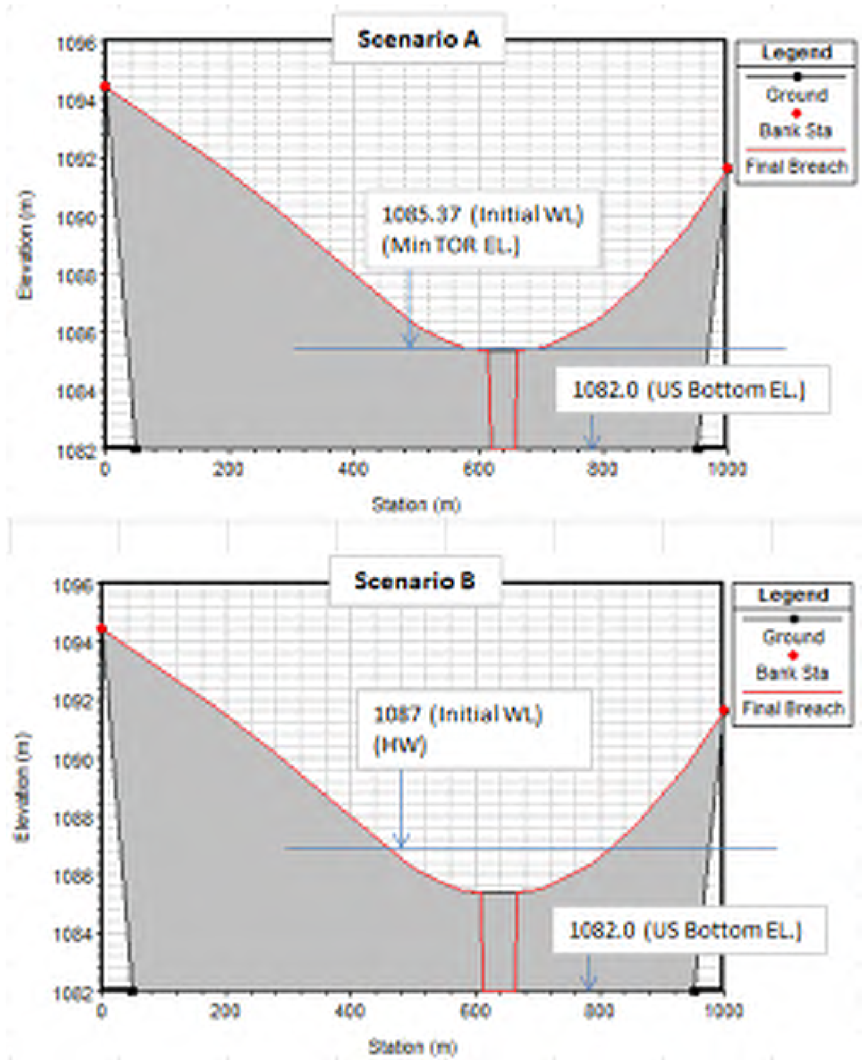
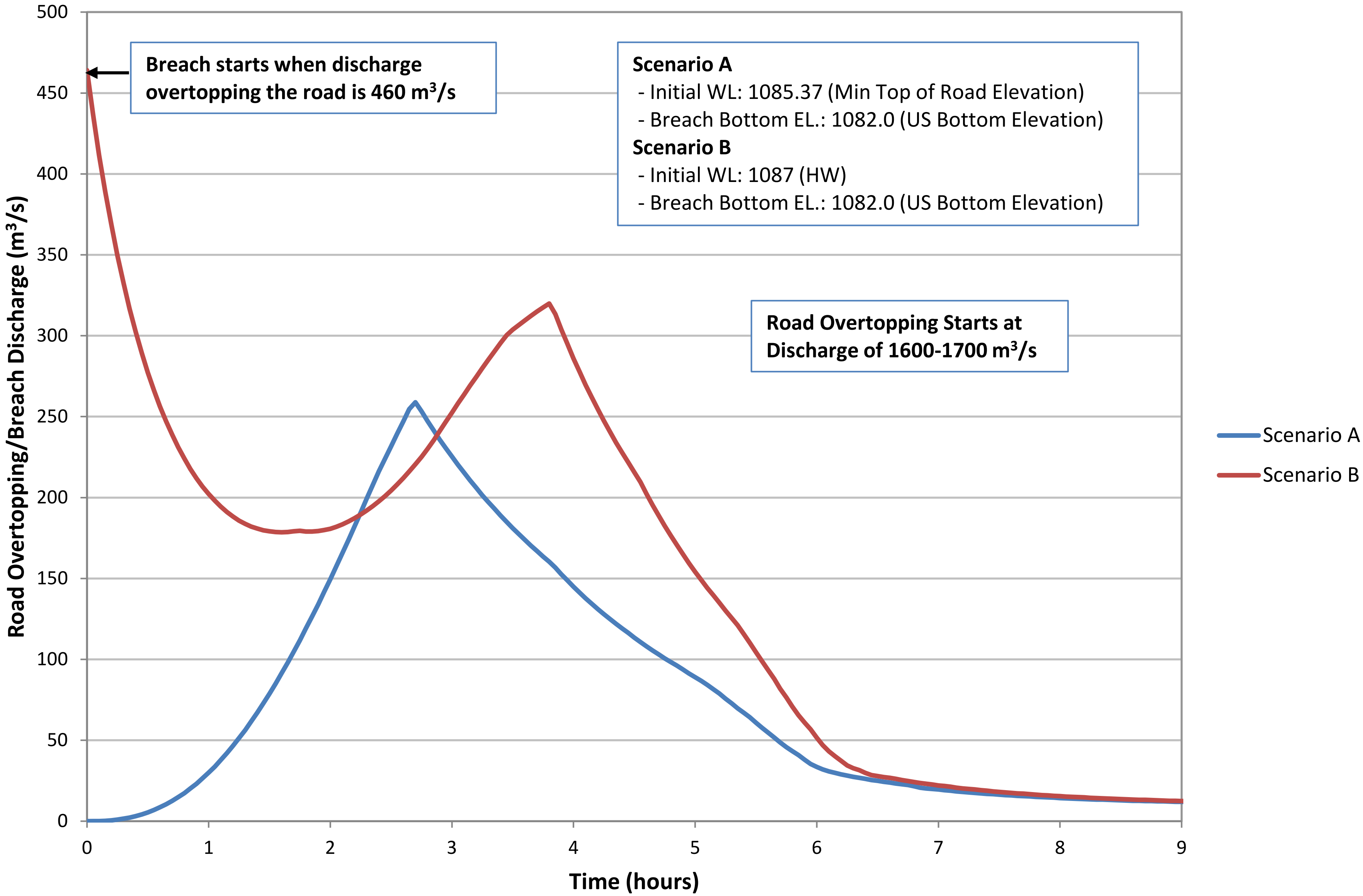
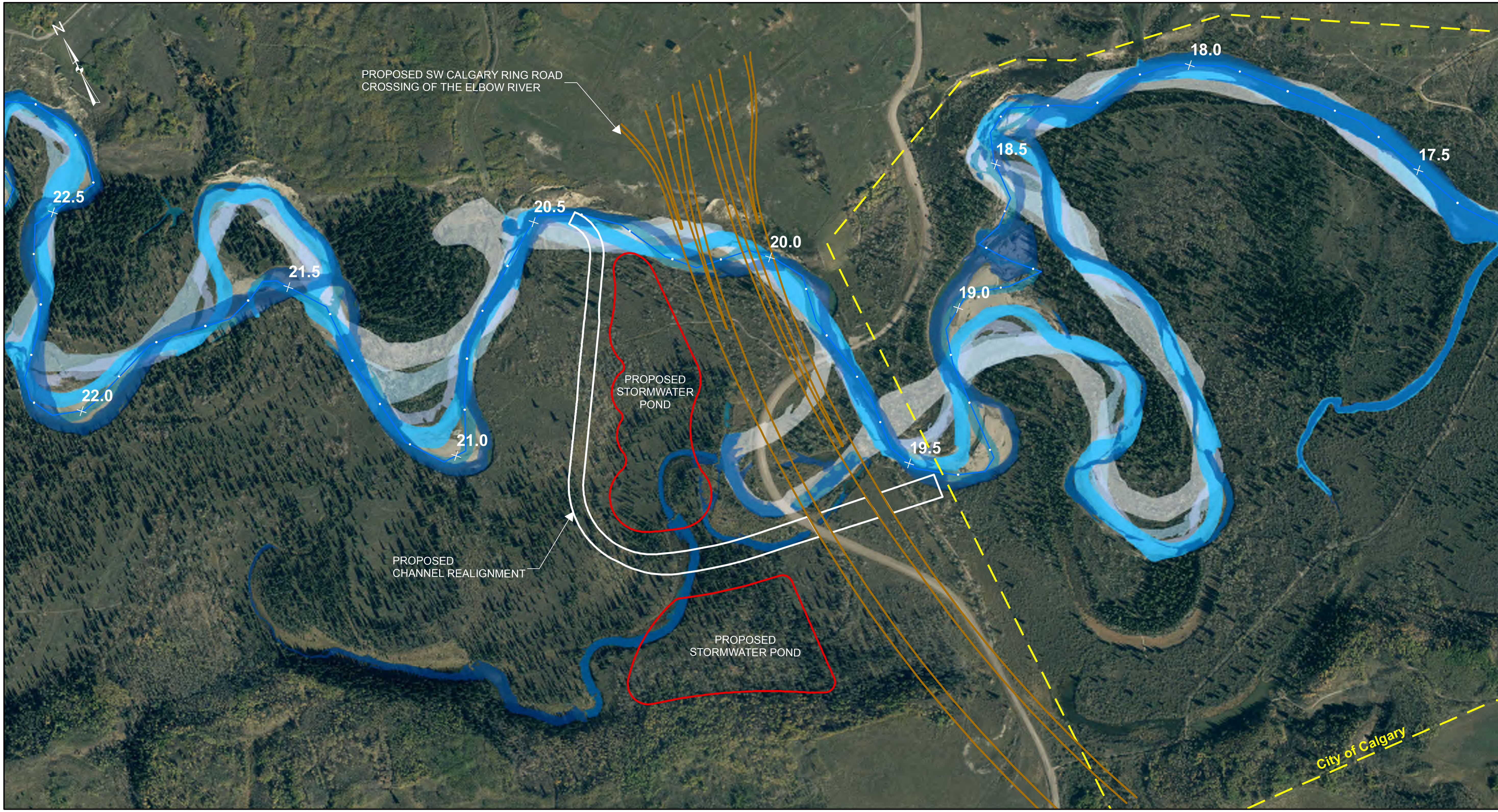


Figure 25
SWCRR Breach Discharge Hydrographs (Overtopping Failure) - 2007 Design





LEGEND:

Water Surfaces

	1924-26		1962		1997		2012		CITY LIMITS
	1948		1979		2007		2013	×	RIVER km
								·	0.1 km MARKER

NOTES:

1. PROPOSED DESIGN FROM CH2M HILL (2015).
2. WATER SURFACE DELINEATIONS PROVIDED BY THE CITY OF CALGARY.

1:7,500 0 250 m

CLIENT: THE CITY OF CALGARY WATER RESOURCES

Klohn Crippen Berger

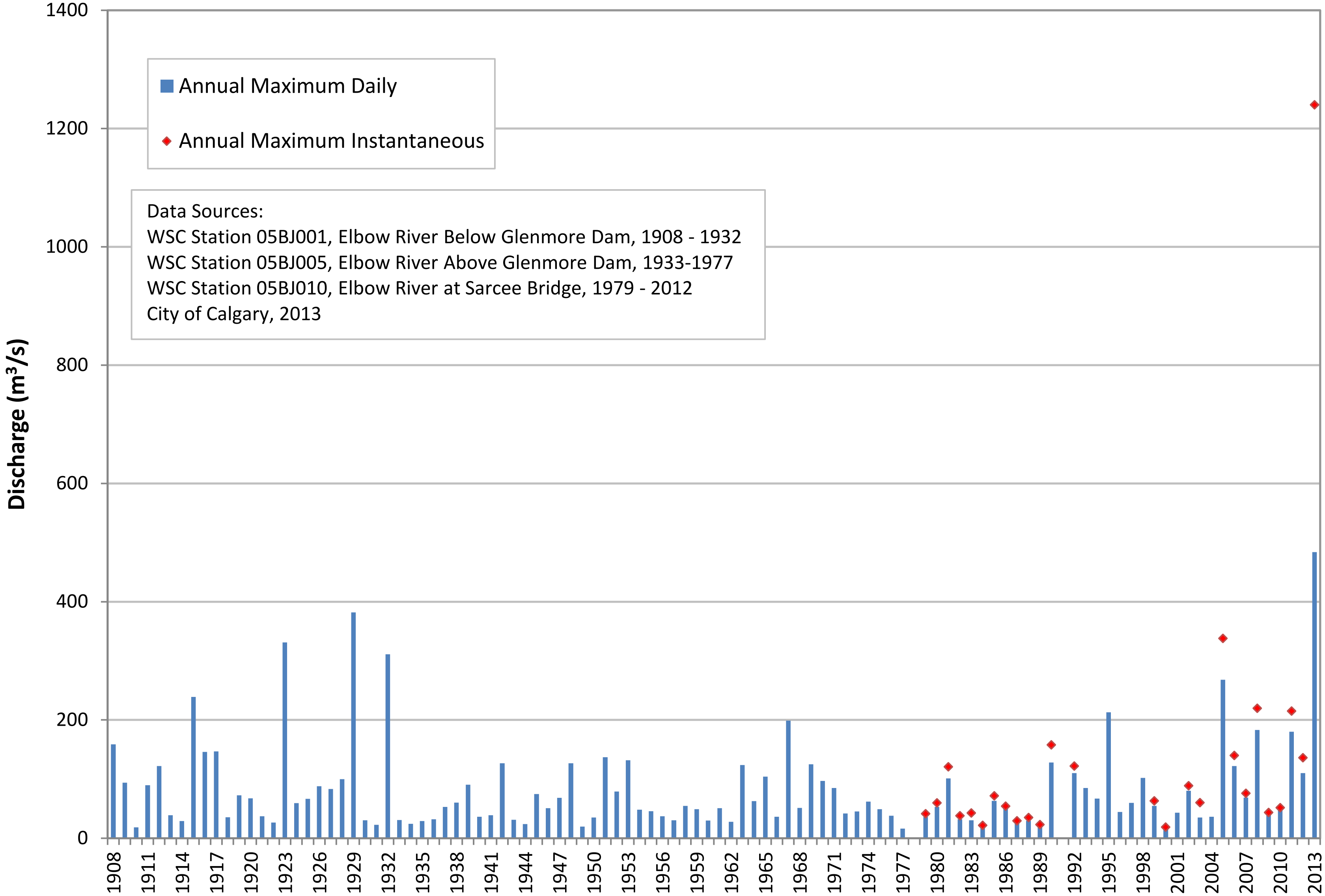
PROJECT: SW CALGARY RING ROAD BRIDGE HYDRAULIC ASSESSMENT

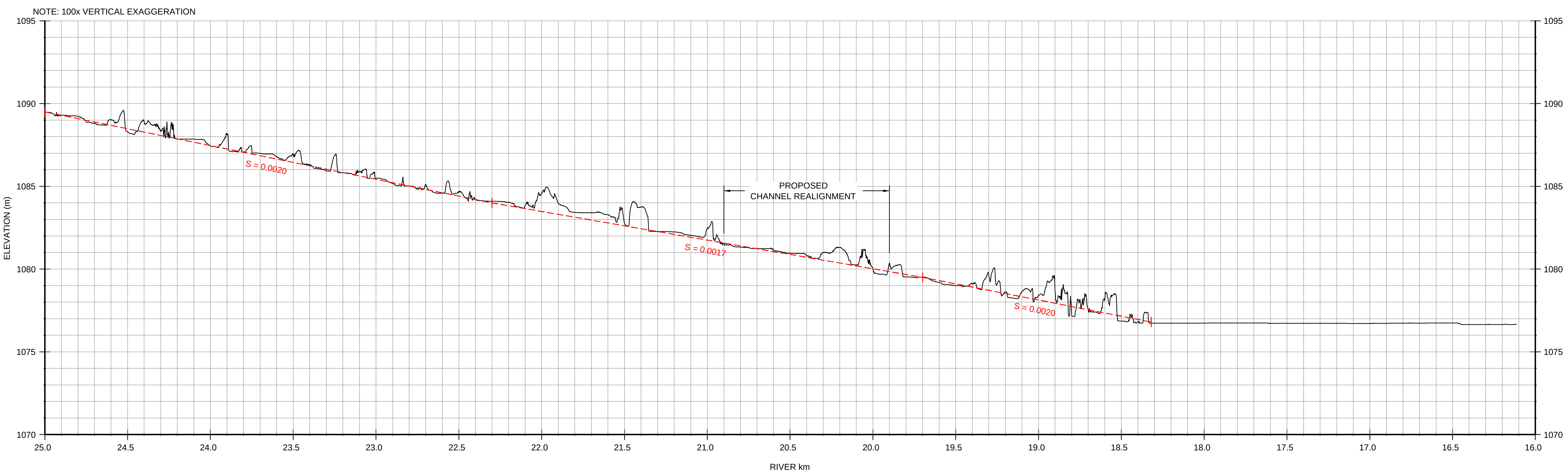
TITLE: HISTORICAL ELBOW RIVER MOVEMENT

SCALE	PROJECT No.	FIG No.
1:7,500	A03018A24	26

File: Z:\ACGY\Alberta\A03018A24 CoC SW Calgary Ring Rd Bridge\400 Drawings\GIS\A03018A24 FIG 26 Historical Elbow River Movement.mxd Date: October 14, 2015 Time: 11:26:06 AM Creator: vcastillo

Figure 27
Elbow River at Stoney Trail
Flood History





NOTES:

1. LONGITUDINAL PROFILE DRAWN USING LIDAR DATA PROVIDED BY THE CITY. THIS DATA WAS CAPTURED BETWEEN SEPTEMBER 9 AND OCTOBER 7, 2013.

1:25 000 0 250 500 m

CLIENT: THE CITY OF CALGARY	PROJECT: CITY OF CALGARY SW CALGARY RING ROAD BRIDGE
Klohn Crippen Berger	TITLE: ELBOW RIVER PROFILE NEAR PROPOSED BRIDGE

SCALE	PROJECT No.	FIG. No.
1:25 000	A03018A24	28

APPENDIX I

MD Rocky View Flood Maps (AGRA 1996)

NOTES:

1. BASE MAPS WERE PROVIDED BY ALBERTA ENVIRONMENTAL PROTECTION, RIVER ENGINEERING BRANCH.

2. THE FOLLOWING TABLE PRESENTS THE COMPUTED WATER LEVELS FOR THE 2, 5, 10, 25, 50 AND 100 YEAR FLOODS UNDER NATURAL CONDITIONS.

Cross Section No.	2 Year Flood	5 Year Flood	10 Year Flood	25 Year Flood	50 Year Flood	100 Year Flood
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

3. THE 10 AND 100 YEAR FLOOD RISK BOUNDARIES ALONG THE ELBOW RIVER WERE DELINEATED USING THE COMPUTED WATER LEVELS UNDER NATURAL CONDITIONS. THE 50 YEAR FLOOD RISK BOUNDARIES WOULD BE CLOSE TO THOSE OF THE 100 YEAR FLOOD. THE MAJORITY OF THE 2 YEAR FLOOD WATER WOULD BE CONTAINED IN THE MAIN RIVER CHANNEL.

LEGEND

FLOOD RISK LIMIT OF THE 100 YEAR FLOOD
FLOOD RISK LIMIT OF THE 10 YEAR FLOOD
8 CROSS-SECTION NUMBER AND LOCATION
BRIDGE
FLOW DIRECTION

KEY MAP

SCALE 1:5 000

M.D. OF ROCKYVIEW

Canada-Alberta FLOOD DAMAGE REDUCTION PROGRAM

Alberta ENVIRONMENT

REV.	DATE	REVISION DESCRIPTION	DWG.	APPR.
1	[illegible]	ISSUED FOR FINAL REPORT	[illegible]	[illegible]
0	DEC./95	ISSUED FOR DRAFT REVIEW	[illegible]	[illegible]

Alberta
ENVIRONMENTAL PROTECTION
RIVER ENGINEERING BRANCH TECHNICAL SERVICES

PERMIT TO PRACTICE

AGRA
Earth & Environmental Limited

ELBOW RIVER–M.D. OF ROCKYVIEW
FLOOD RISK MAPPING STUDY

FLOOD FREQUENCY MAP
SHEET 6 OF 8

DESIGNED	CHECKED	DATE	JOB No.	DRAWING NO.
DRAFTED	APPROVED	SCALE AS SHOWN	SHEET No.	1444-136

NOTES:

1. BASE MAPS WERE PROVIDED BY ALBERTA ENVIRONMENTAL PROTECTION, RIVER ENGINEERING BRANCH.

2. THE FOLLOWING TABLE PRESENTS THE COMPUTED WATER LEVELS FOR THE 2, 5, 10, 25, 50 AND 100 YEAR FLOODS UNDER NATURAL CONDITIONS.

Cross Section No.	Computed Water Levels (m)					
	2 Year Flood	5 Year Flood	10 Year Flood	25 Year Flood	50 Year Flood	100 Year Flood
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

3. THE 10 AND 100 YEAR FLOOD RISK BOUNDARIES ALONG THE ELBOW RIVER WERE DELINEATED USING THE COMPUTED WATER LEVELS UNDER NATURAL CONDITIONS. THE 50 YEAR FLOOD RISK BOUNDARIES WOULD BE CLOSE TO THOSE OF THE 100 YEAR FLOOD. THE MAJORITY OF THE 2 YEAR FLOOD WATER WOULD BE CONTAINED IN THE MAIN RIVER CHANNEL.

LEGEND

FLOOD RISK LIMIT OF THE 100 YEAR FLOOD
FLOOD RISK LIMIT OF THE 10 YEAR FLOOD
CROSS-SECTION NUMBER AND LOCATION
FENCE
FLOW DIRECTION

REV.	DATE	REVISION DESCRIPTION	CHK.	APPR.
		ISSUED FOR FINAL REPORT		
		ISSUED FOR DRAFT REVIEW		

Alberta
ENVIRONMENTAL PROTECTION
RIVER ENGINEERING BRANCH TECHNICAL SERVICES

AGRA
Earth & Environmental Limited

PROJECT: ELBOW RIVER—M.D. OF ROCKYVIEW FLOOD RISK MAPPING STUDY

FLOOD FREQUENCY MAP
SHEET 7 OF 8

DATE: OCTOBER 1995
DRAWING NO. 1444-57

SCALE 1:5 000

M.D. OF ROCKYVIEW

KEY MAP

Projection: 3° Transverse Mercator, Scale factor 0.9999 at Reference Meridian 114°

Contour interval 1 METRE

Elevations in metres above mean sea level, North American Datum, 1983

Canada-Alberta
FLOOD
DAMAGE
REDUCTION
PROGRAM

Environment Canada
Environnement Canada

Alberta
ENVIRONMENT

NOTES:

1. BASE MAPS WERE PROVIDED BY ALBERTA ENVIRONMENTAL PROTECTION, RIVER ENGINEERING BRANCH.

2. THE FOLLOWING TABLE PRESENTS THE COMPUTED WATER LEVELS FOR THE 2, 5, 10, 25, 50 AND 100 YEAR FLOODS UNDER NATURAL CONDITIONS.

Cross Section No.	Computed Water Levels (m)						Cross Section No.	Computed Water Levels (m)					
	2 Year Flood	5 Year Flood	10 Year Flood	25 Year Flood	50 Year Flood	100 Year Flood		2 Year Flood	5 Year Flood	10 Year Flood	25 Year Flood	50 Year Flood	100 Year Flood
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

3. THE 10 AND 100 YEAR FLOOD RISK BOUNDARIES ALONG THE ELBOW RIVER WERE DELINEATED USING THE COMPUTED WATER LEVELS UNDER NATURAL CONDITIONS. THE 25 YEAR FLOOD RISK BOUNDARIES WOULD BE CLOSE TO THOSE OF THE 100 YEAR FLOOD. THE MAJORITY OF THE 2 YEAR FLOOD WATER WOULD BE CONTAINED IN THE MAIN RIVER CHANNEL.

LEGEND

FLOOD RISK LIMIT OF THE 100 YEAR FLOOD

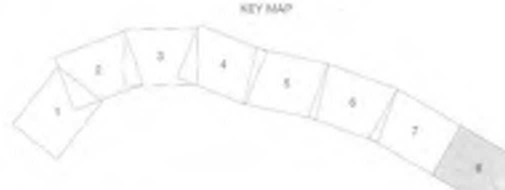
FLOOD RISK LIMIT OF THE 10 YEAR FLOOD

8 CROSS-SECTION NUMBER AND LOCATION

BRIDGE

FLOW DIRECTION

KEY MAP



SCALE 1:5 000

Contour interval 1 METRE

Elevations in metres above mean sea level, North American Datum, 1983

Depression contour

M.D. OF ROCKYVIEW

REV.	DATE	REVISION DESCRIPTION	ENG.	APPR.
1		ISSUED FOR FINAL REPORT	D.L.	
0		ISSUED FOR DRAFT REVIEW	D.L.	

Alberta
ENVIRONMENTAL PROTECTION
RIVER ENGINEERING BRANCH TECHNICAL SERVICES

PERMIT TO PRACTICE
AGRA Earth & Environmental Limited

AGRA
Earth & Environmental Limited

PROJECT
ELBOW RIVER–M.D. OF ROCKYVIEW
FLOOD RISK MAPPING STUDY

FLOOD FREQUENCY MAP
SHEET 8 OF 8

DESIGNED:	CHECKED:	DATE: OCTOBER 1995	JOB No.	DRAWING NO.
DRAFTED:	APPROVED:	SCALE: AS SHOWN	SHEET No.	1444–11B

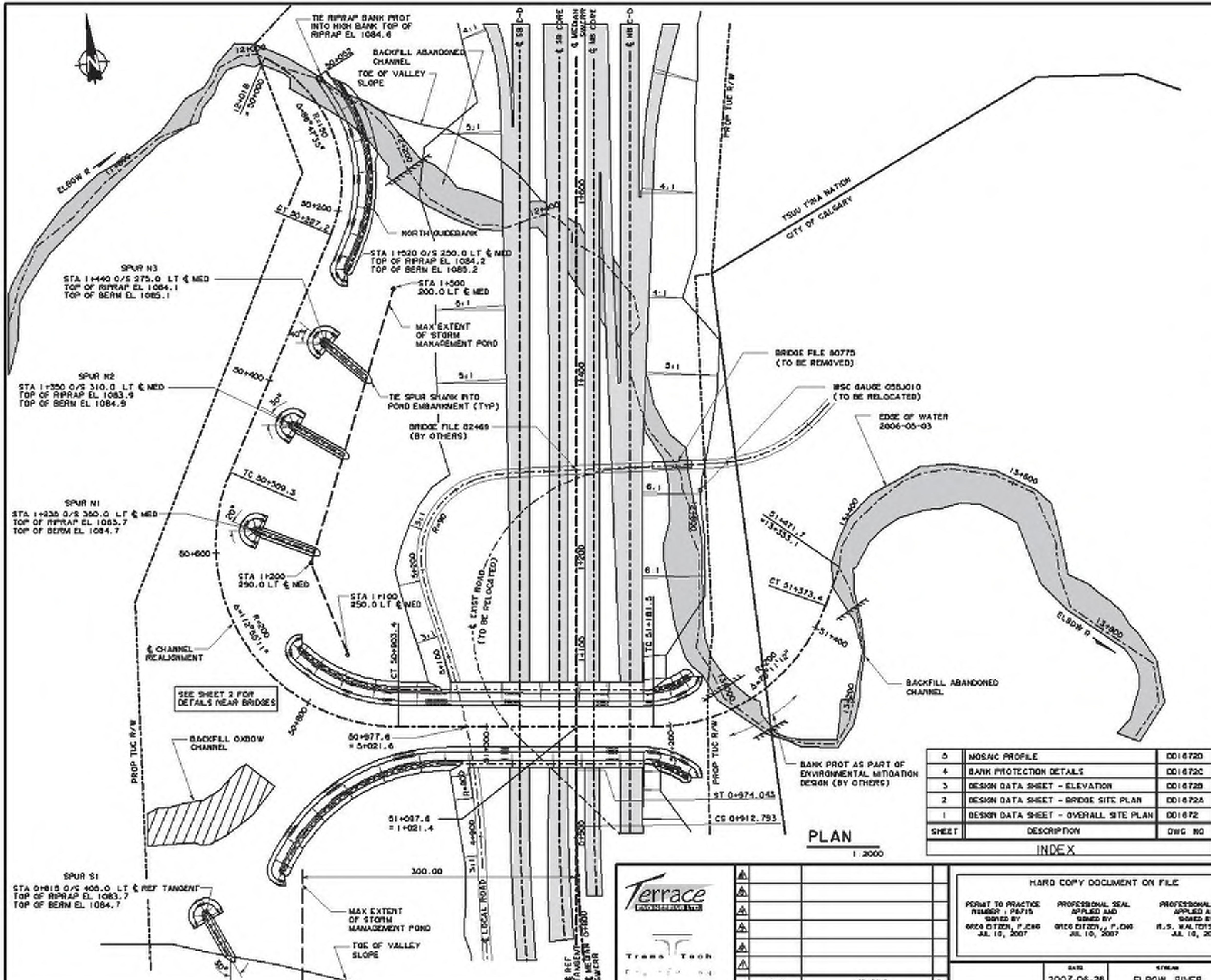
Canada-Alberta
FLOOD
DAMAGE
REDUCTION
PROGRAM

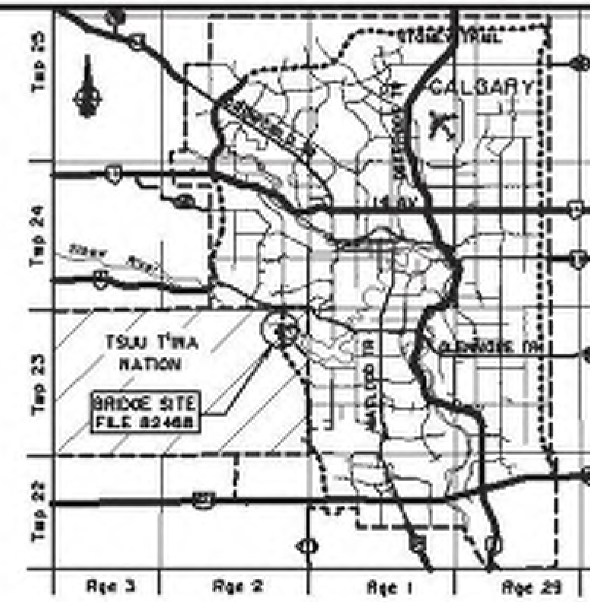
Environment Canada
Environnement Canada

Alberta
ENVIRONMENT

APPENDIX II

2007 Bridge Design Drawings





SITE MAP

HIGHWAY 1 - 216 — 1:250,000
HIGHWAY 500 - 986
LOCAL ROAD
FUTURE RING ROAD

SURVEY INFORMATION

- SURVEY, ROADWAY AND LEGAL INFORMATION PROVIDED BY TRANSTECH ENGINEERING. SURVEY BY TRONNES SURVEYS, JAN - MAY 2006, 3TM 114° NAD83
- GRID 26+648.653 = N 5650681.696 E -11741.723 = GROUND 1+000.000 COMBINED SCALE FACTOR = 0.999728, SCALE BASE POINT = 0.0,0.0
- STA 1+000 ℄ MED SWCRR PERPENDICULAR INTERSECT STA 5+000 ℄ LOCAL ROAD = STA 102+707.135

HYDROMETRIC DATA

- DRAINAGE AREA 1220 km^2
- DESIGN DISCHARGE 700 m^3/s
- MEAN VELOCITY FOR DESIGN DISCHARGE THROUGH PROPOSED OPENING ESTIMATED TO BE 3.1 m/s
- ESTIMATED STREAM SLOPE = 0.0017 m/m
- FLOWING ICE CONDITIONS AS PER CSA CAN/CSA-S6-00, SEC 3.12.2.1 APPEAR TO MEET SITUATION "b" AT EL 1080.7 (℄ MEDIAN) WITH AN EFFECTIVE ICE THICKNESS OF 1.0 m

NOTES

- ALL DIMENSIONS, STATIONS AND ELEVATIONS ARE GIVEN IN METRES IN GROUND COORDINATES UNLESS NOTED OTHERWISE. THE ROADWAY PROFILE AND MOSAIC ARE SHOWN IN GRID.
- ROAD DESIGN STANDARD:
  RFD-110 (SWCRR)
  UAU-60 (LOCAL ROAD)
- ROADWAY AND BRIDGE DRAINAGE TO BE DIRECTED TO STORM MANAGEMENT PONDS (DESIGN BY OTHERS)
- ESTIMATED QUANTITY OF CLASS 3 RIPRAP 30,000 m^3
- FOR ADDITIONAL INFORMATION RELATING TO ROADWAY DESIGN, GEOTECHNICAL INPUT, CHANNEL REALIGNMENT DESIGN, ENVIRONMENTAL MITIGATION, AND OTHER ISSUES REFER TO:
  - SWCRR ENVIRONMENTAL ASSESSMENT REPORT BY AMEC EARTH & ENVIRONMENTAL
  - SWCRR FUNCTIONAL PLANNING STUDY REPORT BY TRANSTECH ENGINEERING

  ALL DISCLAIMERS IN THESE REPORTS ARE APPLICABLE. ANY DISCREPANCY BETWEEN THESE REPORTS (OR ANY FUTURE REPORTS/AGREEMENTS) AND THIS DRAWING SET MUST BE CONSIDERED AND RESOLVED DURING BRIDGE DESIGN
- INFORMATION SHOWN ON THESE DRAWINGS IS FOR PRELIMINARY DESIGN DATA FOR THIS SPECIFIC PROJECT AND IS FOR THE BENEFIT OF THE CLIENT ONLY. THESE DRAWINGS ARE NOT TO BE USED FOR CONSTRUCTION.

INDEX

SHEET	DESCRIPTION	DWG NO
5	MOSAIC PROFILE	DD1672D
4	BANK PROTECTION DETAILS	DD1672C
3	DESIGN DATA SHEET - ELEVATION	DD1672B
2	DESIGN DATA SHEET - BRIDGE SITE PLAN	DD1672A
1	DESIGN DATA SHEET - OVERALL SITE PLAN	DD1672

Terrace Engineering Ltd.

TransTech Engineering

REV	DATE	REVISIONS	BY

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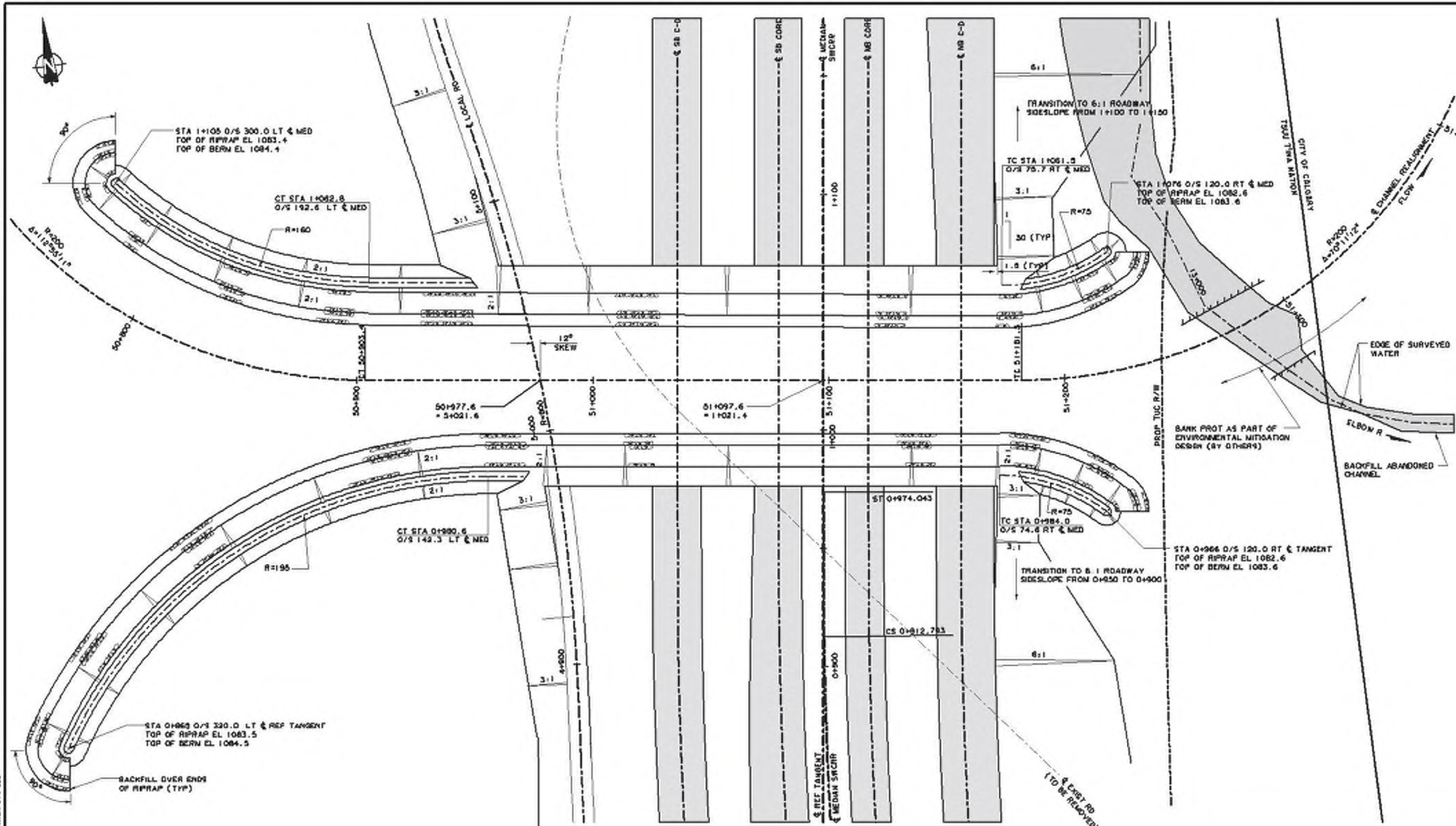
Alberta INFRASTRUCTURE AND TRANSPORTATION

ELBOW RIVER BRIDGES ON SW CALGARY RING RD (SWCRR)
DESIGN DATA SHEET OVERALL SITE PLAN

DATE	STREAM	LOCATION	HIGHWAY	FILE	SHEET	DRAWING
2007-06-26	ELBOW RIVER	SE35-23-2-5	20L06	82468	1 of 5	DD1672



PLAN

1:750

Terrace Engineering Ltd.

Trans Tech Engineering

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PROFESSIONAL SEAL APPLIED AND SIGNED BY R.S. WALTERS, P.ENG JUL 10, 2007

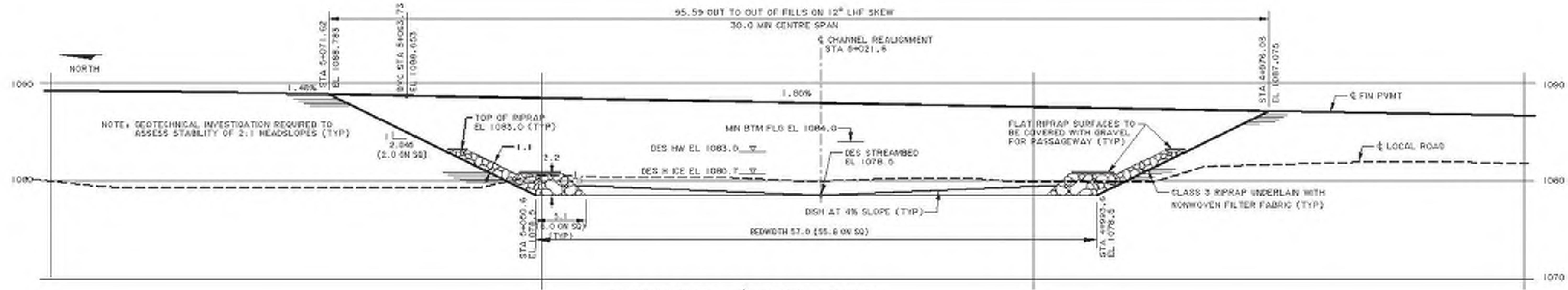
Alberta INFRASTRUCTURE AND TRANSPORTATION

ELBOW RIVER BRIDGES ON SW CALGARY RING RD (SWCRR)

DESIGN DATA SHEET BRIDGE SITE PLAN

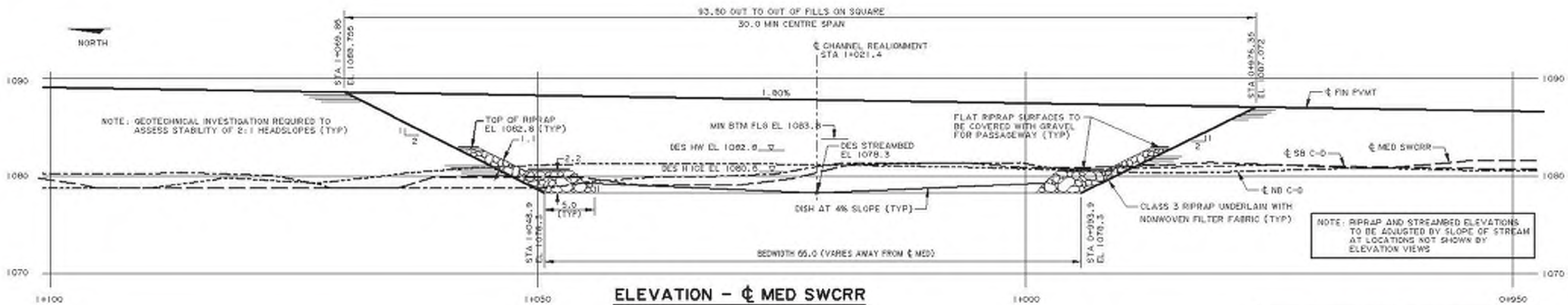
DATE	STREAM	LOCATION	HIGHWAY	FILE	SHEET	DRAWING
2007-06-26	ELBOW RIVER	SE35-23-2-5	20106	82468	2 of 4	DD1672A

FILE: TERRACE07\ELBOW.DWG DRAWN BY: GEG



ELEVATION – ℄ LOCAL ROAD

LOOKING D/S 1:200

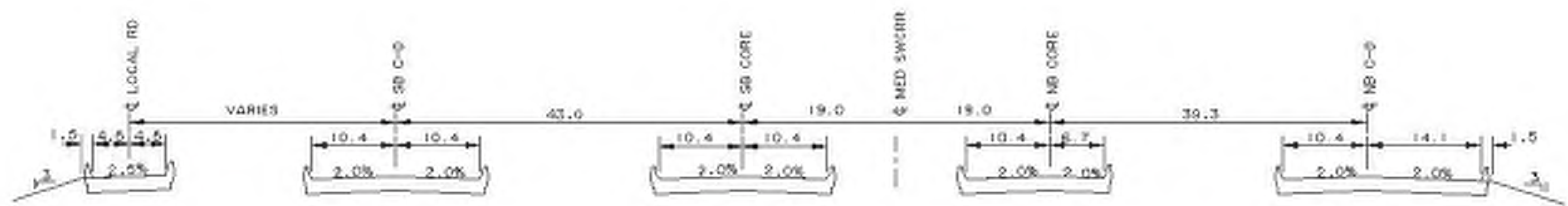


NOTE: RIPRAP AND STREAMBED ELEVATIONS TO BE ADJUSTED BY SLOPE OF STREAM AT LOCATIONS NOT SHOWN BY ELEVATION VIEWS

ELEVATION – ℄ MED SWCRR

LOOKING D/S 1:200

	TOP OF RIPRAP	DESIGN STREAMBED	BEDWIDTH
℄ SB C-D	1082.9	1078.4	55.4
℄ SB CORE	1082.8	1078.3	55.1
℄ NB CORE	1082.8	1078.3	54.9
℄ NB C-D	1082.7	1078.2	54.6



BRIDGE SECTION

LOOKING NORTH 1:200

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PROFESSIONAL SEAL APPLIED AND SIGNED BY GREG EITZEN, P.ENG JUL 10, 2007

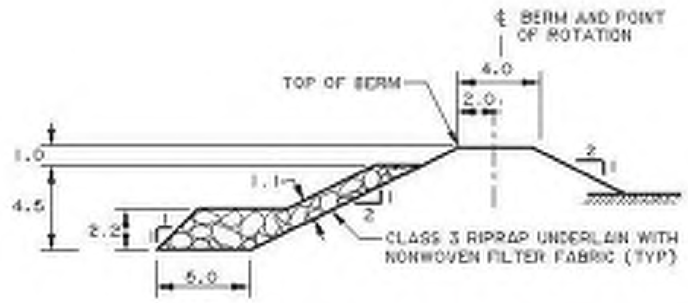
PROFESSIONAL SEAL APPLIED AND SIGNED BY R.S. WALTERS, P.ENG JUL 10, 2007

Alberta INFRASTRUCTURE AND TRANSPORTATION

ELBOW RIVER BRIDGES ON SW CALGARY RING RD (SWCRR) DESIGN DATA SHEET ELEVATION

DATE	STREAM	LOCATION	HIGHWAY	FILE	SHEET	DRAWING
2007-06-26	ELBOW RIVER	SE35-23-2-5	201.06	82468	3 of 5	DD1672B

FILE: TERRACE07\07024\DD1672B.DGN DRAWN BY: WT



SECTION–BRIDGE GUIDEBANKS

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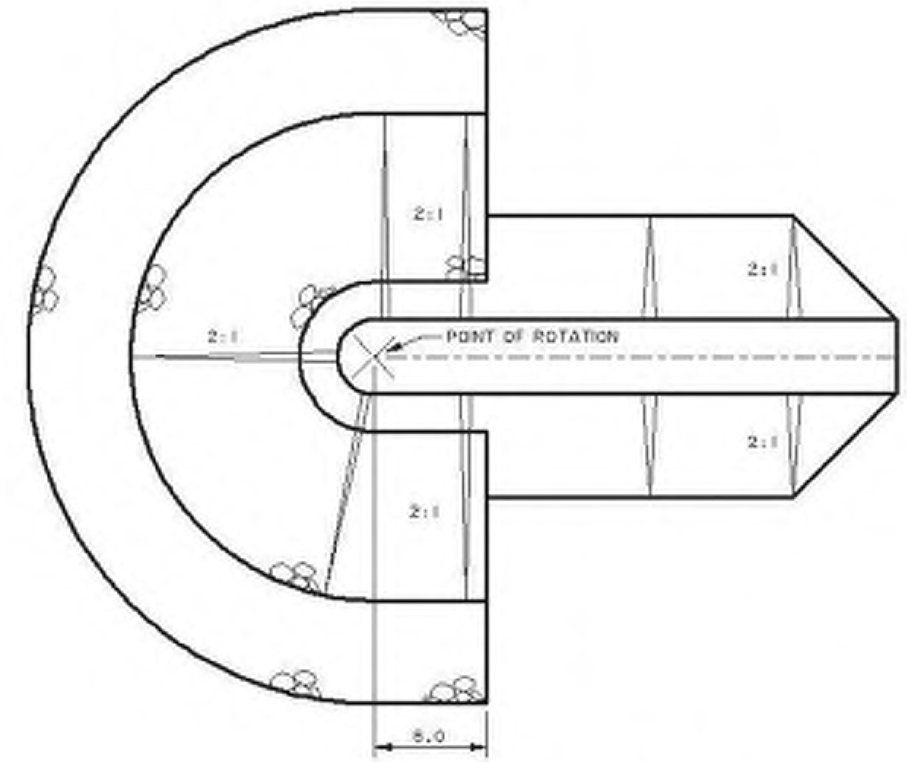

PLAN–TYPICAL SPUR

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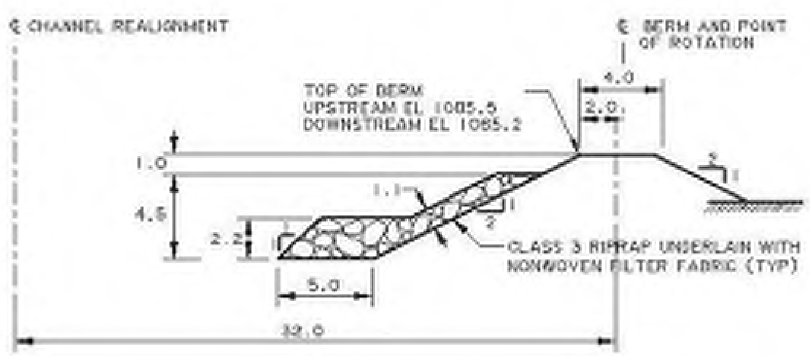

SECTION–NORTH GUIDEBANK

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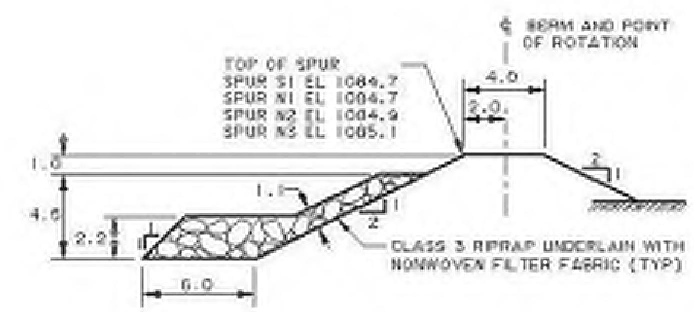

SECTION–TYPICAL SPUR

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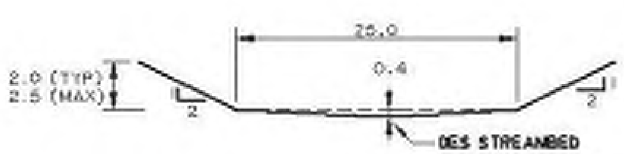

MIN CHANNEL HYDRAULIC SECTION

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Terrace ENGINEERING LTD.

Trans Tech

REV	DATE	REVISIONS	BY

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Alberta INFRASTRUCTURE AND TRANSPORTATION

ELBOW RIVER BRIDGES ON SW CALGARY RING RD (SWCRR)

BANK PROTECTION SHEET

DATE	STREAM	LOCATION	HIGHWAY	FILE	SHEET	DRAWING
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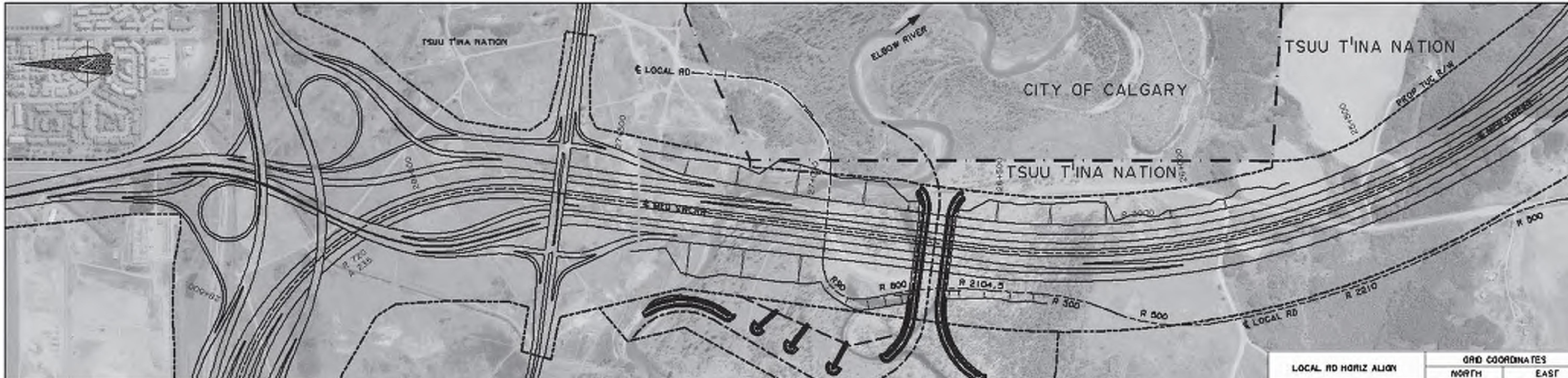

SWCRR HORIZ ALIGN	GRID COORDINATES	
	NORTH	EAST
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C.S. STA 26+651.446	5650695.204	-11752.868
S.T. STA 26+622.696	5650665.983	-11745.132
T.S. STA 27+641.797	5651668.234	-11611.270
S.C. STA 27+716.498	5651742.429	-11603.548
P.I. 25 R=720	5652375.763	-11517.256

LOCAL RD HORIZ ALIGN	GRID COORDINATES	
	NORTH	EAST
P.I. 105 R=2104.6	5650506.206	-11866.655
C.S. STA 102+567.464	5650557.611	-11861.878
S.C. STA 102+627.464	5650617.410	-11857.074
P.I. 106 R=800	5650704.706	-11848.206
C.S. STA 102+801.290	5650790.650	-11865.348
S.C./C.S. STA 102+861.290	5650849.599	-11877.509
S.C. STA 102+921.290	5650909.069	-11882.793
P.I. 107 R=90	5650998.607	-11907.655

ROADWAY AND LEGAL INFORMATION PROVIDED BY TRANSTECH ENGINEERING
INFORMATION SHOWN IS BASED ON GRID COORDINATES
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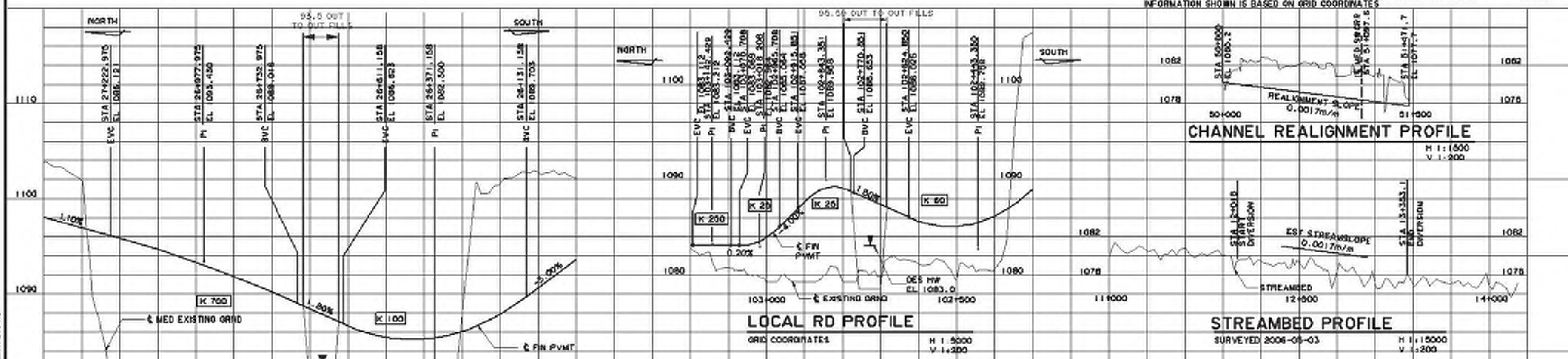

Terrace Engineering Ltd.

TransTech Engineering

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PROFESSIONAL SEAL APPLIED AND SIGNED BY R.S. WALTERS, P.ENG JUL 10, 2007

Alberta INFRASTRUCTURE AND TRANSPORTATION

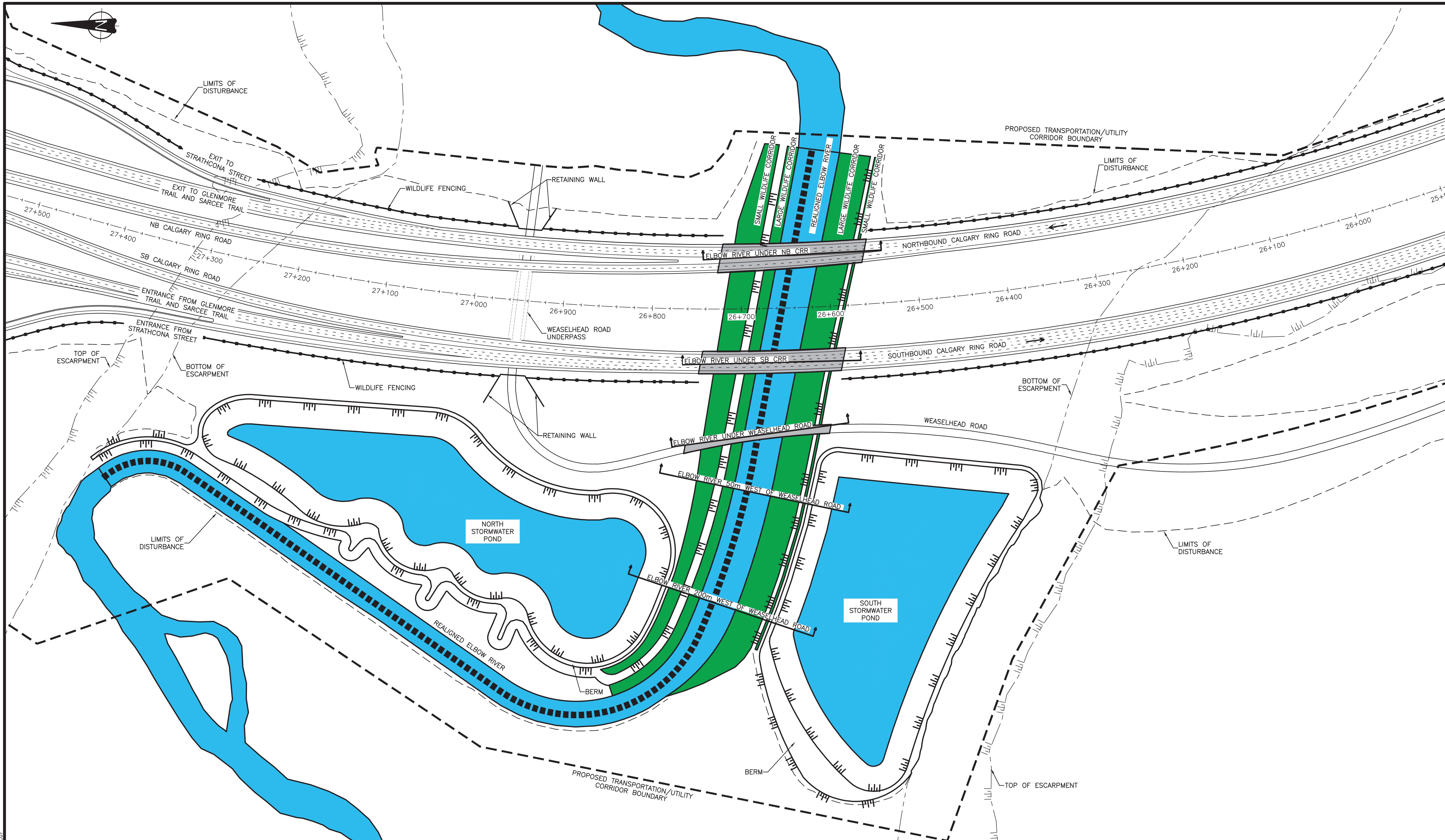
ELBOW RIVER BRIDGES ON SW CALGARY RING RD (SWCRR)

MOSAIC PROFILE

DEPARTMENT BAR CODE	DATE	STREAM	LOCATION	HIGHWAY	FILE	SHEET	DRAWING
	2007-06-26	ELBOW RIVER	SE35-23-2-5	201.06	82468	5 of 5	DD1672D

APPENDIX III

2015 Bridge Design Drawings



WILDLIFE CORRIDOR

NORMAL WATER LEVEL WETTED PERIMETER

TOP OF SLOPE

ELBOW RIVER CENTRELINE

SCALE

FOR DISCUSSION PURPOSES ONLY MARCH 20, 2015

CONSULTANT

CH2MHILL.

ISL Engineering and Land Services

JOB No. 494197

REV	DATE	REVISION	BY

DATE	LOCATION	SITE

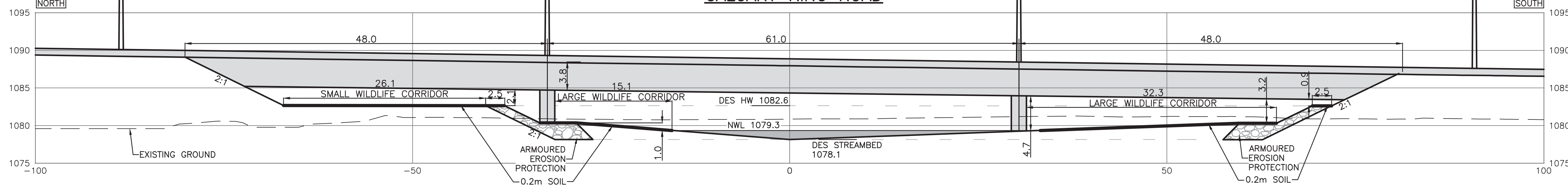
Alberta Transportation

SOUTHWEST CALGARY RING ROAD

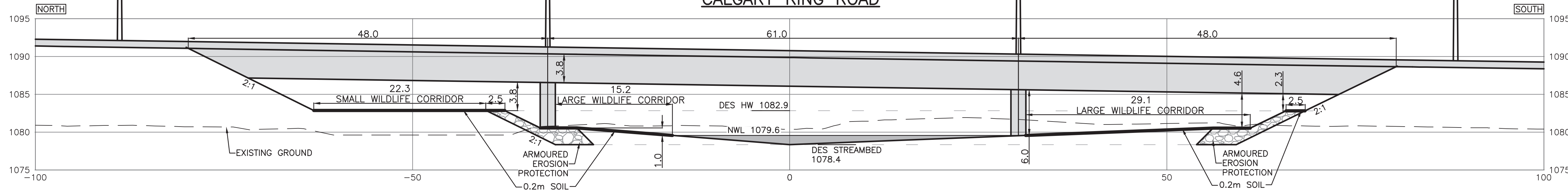
CALGARY RING ROAD OVER ELBOW RIVER & STORM PONDS PLAN

CONTRACT	HIGHWAY	SHEET	DRAWING
	HWY 201	1 of 3	10S-RD-SK03a

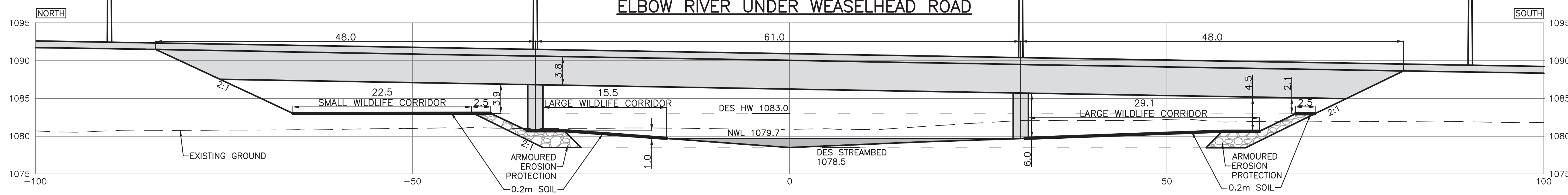
ELBOW RIVER UNDER NORTHBOUND CALGARY RING ROAD



ELBOW RIVER UNDER SOUTHBOUND CALGARY RING ROAD



ELBOW RIVER UNDER WEASELHEAD ROAD



DES – DESIGN
NWL – NORMAL WATER LEVEL
HW – HIGH WATER

SCALE

FOR DISCUSSION PURPOSES ONLY MARCH 20, 2015

CONSULTANT

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ISL Engineering and Land Services

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REV	DATE	REVISION	BY

DATE	LOCATION	SITE

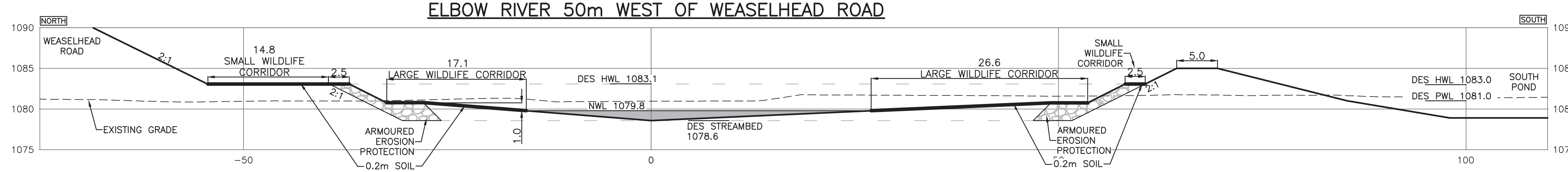
Alberta Transportation

SOUTHWEST CALGARY RING ROAD

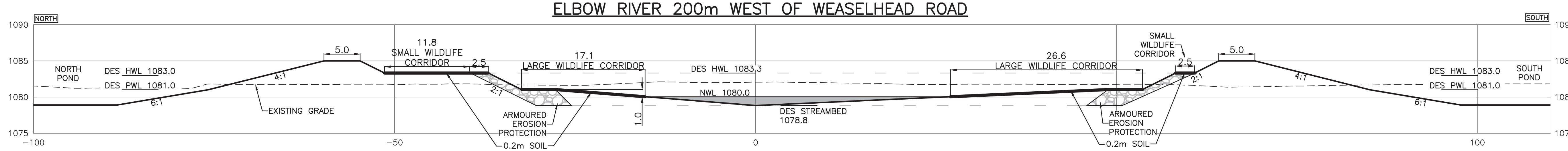
CALGARY RING ROAD OVER ELBOW RIVER & STORM PONDS SECTIONS ALONG RIVER

CONTRACT	HIGHWAY	SHEET	DRAWING
	HWY 201	2 of 3	10S-RD-SK03b

ELBOW RIVER 50m WEST OF WEASELHEAD ROAD



ELBOW RIVER 200m WEST OF WEASELHEAD ROAD



DES – DESIGN
NWL – NORMAL WATER LEVEL
HWL – HIGH WATER LEVEL
PWL – PERMANENT WATER LEVEL

SCALE

FOR DISCUSSION PURPOSES ONLY
MARCH 20, 2015

CONSULTANT

CH2MHILL.

ISL Engineering and Land Services

JOB No. 494197

REV	DATE	REVISION	BY
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DATE	LOCATION	SITE
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Alberta Transportation

SOUTHWEST CALGARY RING ROAD

CALGARY RING ROAD OVER
ELBOW RIVER & STORM PONDS
SECTIONS ALONG RIVER

CONTRACT	HIGHWAY	SHEET	DRAWING
· ·	HWY 201	3 of 3	10S-RD-SK03c

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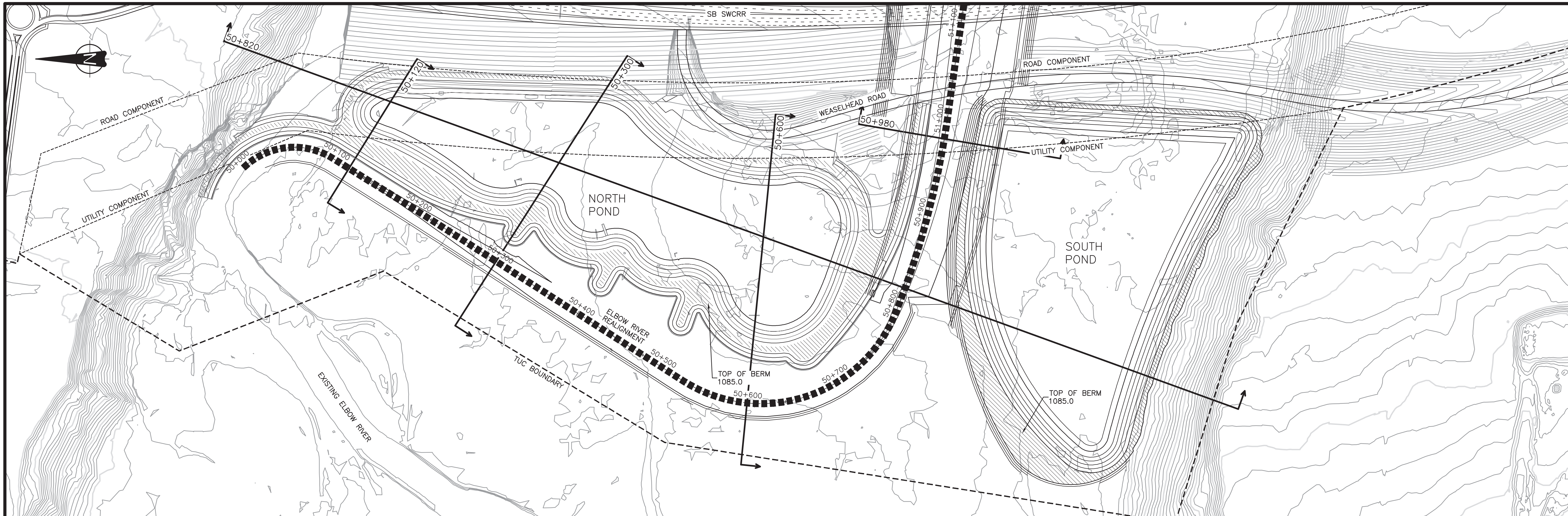

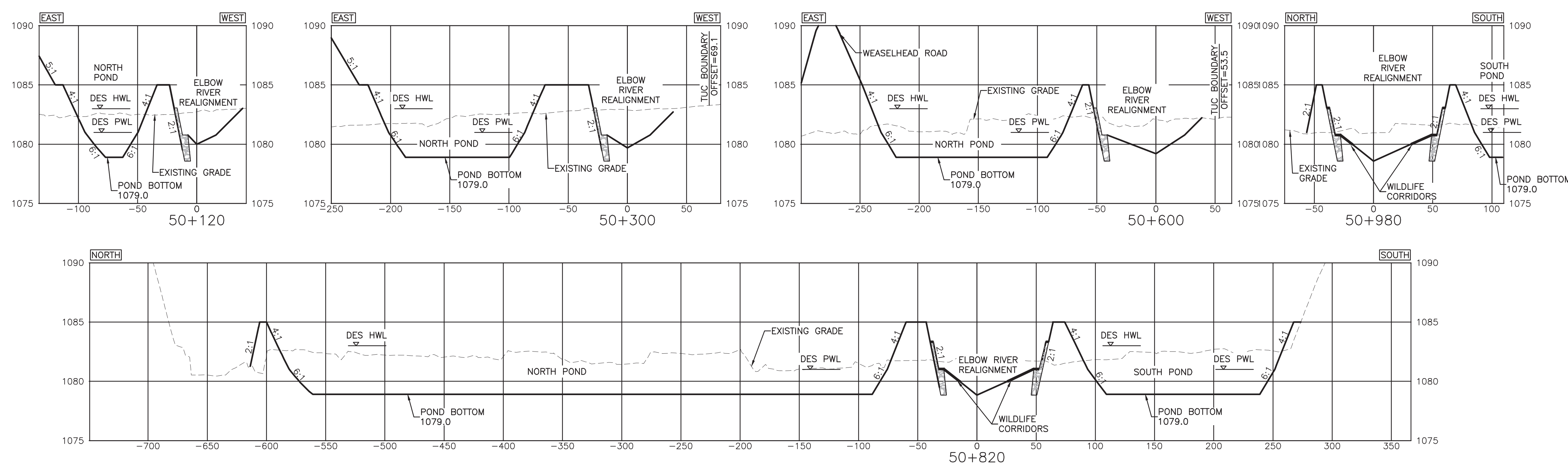

NOTES:
1. DESIGN HIGH WATER LEVEL (DES HWL) = 1083.0
2. DESIGN PERMANENT WATER LEVEL (DES PWL) = 1081.0
3. INFORMATION PROVIDED FOR DISCUSSION PURPOSES ONLY. DETAILS ARE SUBJECT TO CHANGE AS CONCEPTS ARE CONVERTED TO DETAILED DESIGN BY OTHERS.

SCALE

FOR DISCUSSION PURPOSES ONLY MARCH 5, 2015

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CH2MHILL.

ISL Engineering and Land Services

JOB No. 494197

REV	DATE	REVISION	BY

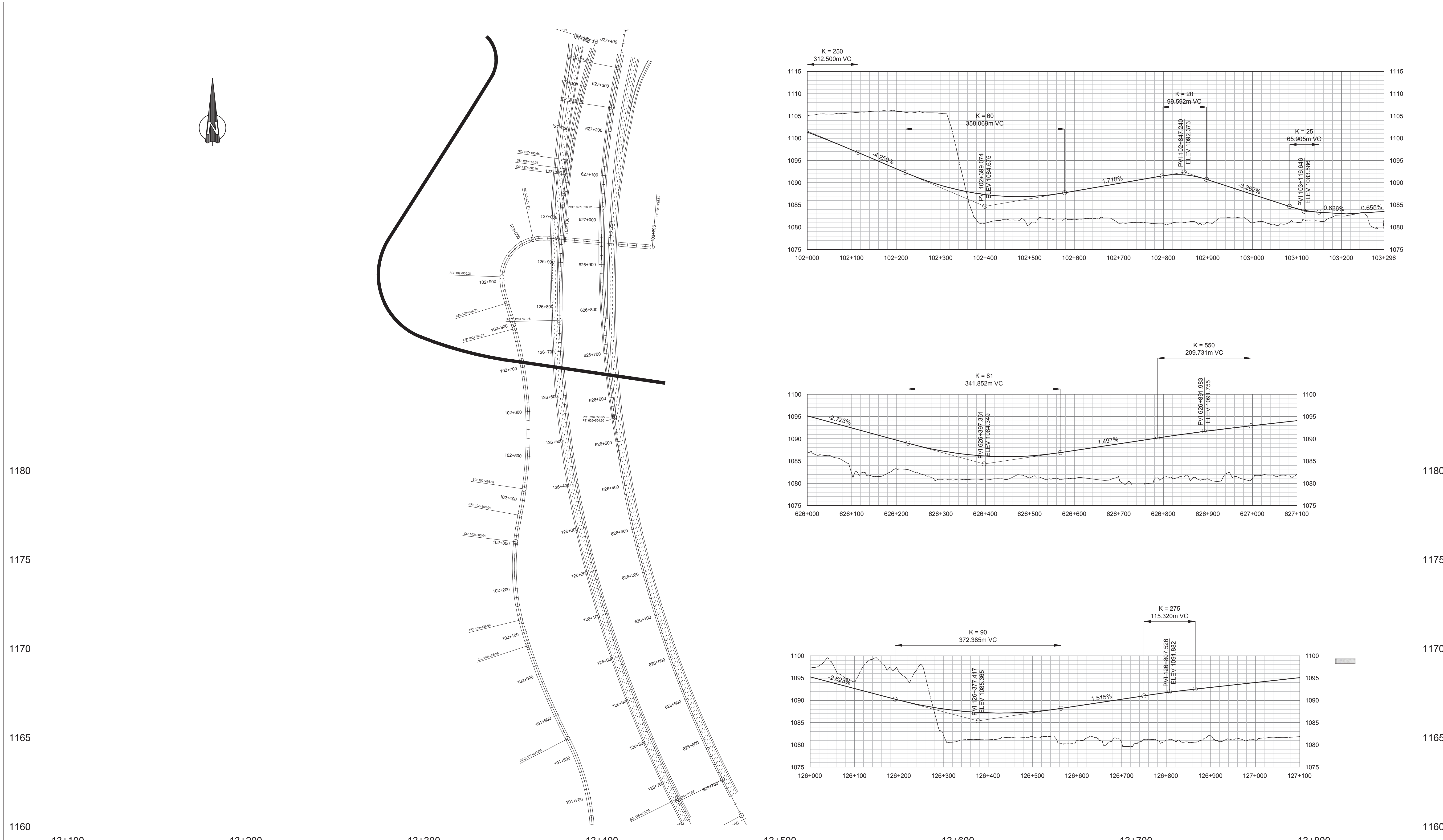
DATE	LOCATION	SITE

Alberta Transportation

SOUTHWEST CALGARY RING ROAD

ELBOW RIVER REALIGNMENT AND STORM PONDS PLAN AND SECTIONS

CONTRACT	HIGHWAY	SHEET	DRAWING
	HWY 201	1 of 2	10S-RD-SK01a



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FOR DISCUSSION PURPOSES ONLY SEPTEMBER 8, 2014

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JOB No. 494197

DATE	LOCATION	SITE

Alberta Transportation

SOUTHWEST CALGARY RING ROAD

CONTRACT	HIGHWAY	SHEET	DRAWING
	HWY 201	of	

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2015/10/29 7:18 AM By: Hernandez, Sergio

$DATE$

$FILES$

APPENDIX IV

Water Levels Surveyed During the June 2013 Flood

FLOOD DATA AROUND DISCOVERY RIDGE

KCB Number	ELEV_ERR	P_ID	POINT_NAME	PCODE_ORG	PCODE	ALIGNMENT	NORTHING	EASTING	ELEVATION	ANT_HGT	H_PREC	V_PREC	DATE	TIME	EPOCHS	SV	RMS	FROM	SURVEYOR	F_ID	FILENAME	LOCATION	RIVER	BANK
		1518	PV2024	WL	WL	ELBOW	5653506.155	-15896.35	1099.324	2	0.016	0.027	6/20/2013	9:40 PM	1	12	0.035	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		68	LD1018	WL	WL	ELBOW	5653211.207	-15413.33	1097.917	2	0.012	0.036	6/20/2013	10:20 PM	2	8	0.013	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
1		69	LD1019	WL	WL	ELBOW	5653150.647	-15326.84	1097.382	2	0.016	0.034	6/20/2013	10:23 PM	1	10	0.029	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
2		70	LD1020	WL	WL	ELBOW	5653148.25	-15307.56	1097.345	2	0.02	0.036	6/20/2013	10:25 PM	1	10	0.033	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		1516	PV2022	WL	WL	ELBOW	5653088.24	-15214.7	1097.275	2	0.015	0.028	6/20/2013	9:19 PM	1	12	0.03	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1517	PV2023	WL	WL	ELBOW	5653191.524	-15159.53	1097.213	2	0.016	0.029	6/20/2013	9:26 PM	1	14	0.04	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
3		1512	PV2018	WL	WL	ELBOW	5653011.046	-15144.82	1096.978	2	0.011	0.02	6/20/2013	8:55 PM	1	13	0.026	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
	High Water Mark						5653013.089	-15144.1	1097.102															
		1513	PV2019	HWM	HWM	ELBOW	5653013.089	-15144.1	1097.102	2	0.012	0.022	6/20/2013	9:04 PM	1	12	0.025	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1514	PV2020	WL	WL	ELBOW	5653098.006	-15117.77	1097.102	2	0.016	0.034	6/20/2013	9:08 PM	1	12	0.037	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1515	PV2021	WL	WL	ELBOW	5653181.285	-15073.9	1097.128	2	0.017	0.034	6/20/2013	9:13 PM	1	13	0.039	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1511	PV2017	WL	WL	ELBOW	5652996.694	-15038.51	1096.61	2	0.009	0.019	6/20/2013	8:50 PM	1	12	0.016	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1510	PV2016	WL	WL	ELBOW	5652850.806	-14865.82	1095.155	2	0.013	0.025	6/20/2013	8:30 PM	1	14	0.029	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1509	PV2015	WL	WL	ELBOW	5653088.409	-14853.66	1095.024	2	0.015	0.028	6/20/2013	8:20 PM	1	16	0.038	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1508	PV2014	WL	WL	ELBOW	5653139.458	-14850.47	1095.076	2	0.019	0.035	6/20/2013	8:19 PM	1	15	0.044	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
4		1507	PV2013	WL	WL	ELBOW	5653147.778	-14803.27	1095.133	2	0.024	0.036	6/20/2013	8:17 PM	4	12	0.041	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
5		1506	PV2012	WL	WL	ELBOW	5653130.135	-14782.49	1095.286	2	0.018	0.032	6/20/2013	8:11 PM	1	12	0.033	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
6		1505	PV2011	WL	WL	ELBOW	5653111.168	-14769.69	1095.208	2	0.017	0.032	6/20/2013	8:09 PM	1	11	0.03	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1504	PV2010	WL	WL	ELBOW	5653036.267	-14733.61	1094.919	2	0.019	0.036	6/20/2013	8:06 PM	4	14	0.041	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
7		1499	PV2004	WL	WL	ELBOW	5652755.357	-14677.04	1094.598	2	0.015	0.022	6/20/2013	7:47 PM	1	14	0.033	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1503	PV2008	WL	WL	ELBOW	5652899.109	-14673.95	1094.994	2	0.018	0.03	6/20/2013	7:57 PM	1	15	0.042	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1500	PV2005	WL	WL	ELBOW	5652820.136	-14669.17	1094.959	2	0.023	0.028	6/20/2013	7:50 PM	1	12	0.03	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1497	PV2002	WL	WL	ELBOW	5652736.025	-14661.63	1094.216	2	0.011	0.016	6/20/2013	7:46 PM	3	15	0.025	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1498	PV2003	WL	WL	ELBOW	5652736.037	-14661.61	1094.206	2	0.009	0.014	6/20/2013	7:46 PM	1	15	0.021	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1501	PV2006	WL	WL	ELBOW	5652860.609	-14659.99	1094.985	2	0.009	0.014	6/20/2013	7:52 PM	1	16	0.023	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1502	PV2007	WL	WL	ELBOW	5652903.658	-14649.7	1094.999	2	0.016	0.026	6/20/2013	7:54 PM	1	16	0.04	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1496	PV2001	WL	WL	ELBOW	5652727.606	-14613.13	1094.112	2	0.009	0.014	6/20/2013	7:45 PM	6	12	0.017	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
8		1495	PV2000	WL	WL	ELBOW	5652718.472	-14560.35	1093.988	2	0.011	0.017	6/20/2013	7:42 PM	1	15	0.027	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		1519	PV2025	WL	WL	ELBOW	5652715.872	-14559.84	1093.907	2	0.01	0.016	6/20/2013	9:53 PM	1	13	0.021	DART_CLGY	P.VANOTTERLOO	26	FLD2013-2.job	DISCOVERY RIDGE	ELBOW	LEFT
		66	LD1015	HWM	HWM	ELBOW	5652718.075	-14559.31	1093.931	2	0.01	0.018	6/20/2013	9:33 PM	1	16	0.026	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		51	LD1000	WL	WL	ELBOW	5652718.032	-14559.3	1093.989	2	0.009	0.014	6/20/2013	8:41 PM	6	11	0.014	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
	High Water Mark						5652718.075	-14559.31	1093.931															
9		52	LD1001	WL	WL	ELBOW	5652704.999	-14495.06	1093.493	2	0.022	0.031	6/20/2013	8:44 PM	5	12	0.036	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		53	LD1002	WL	WL	ELBOW	5652691.45	-14452.84	1093.268	2	0.028	0.032	6/20/2013	8:45 PM	1	11	0.034	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		54	LD1003	WL	WL	ELBOW	5652668.599	-14379.63	1092.927	2	0.02	0.028	6/20/2013	8:48 PM	1	10	0.031	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		55	LD1004	WL	WL	ELBOW	5652643.285	-14311.36	1092.635	2	0.014	0.022	6/20/2013	8:51 PM	2	12	0.026	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		56	LD1005	WL	WL	ELBOW	5652581.85	-14244.8	1092.513	2	0.024	0.039	6/20/2013	8:55 PM	2	13	0.047	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
10		57	LD1006	WL	WL	ELBOW	5652560.837	-14218.2	1092.248	2	0.012	0.022	6/20/2013	8:57 PM	1	14	0.026	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
11		58	LD1007	WL	WL	ELBOW	5652562.371	-14144.19	1091.93	2	0.014	0.023	6/20/2013	8:59 PM	1	15	0.032	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		59	LD1008	WL	WL	ELBOW	5652559.195	-14119.57	1091.975	2	0.014	0.026	6/20/2013	8:59 PM	1	14	0.031	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		60	LD1009	WL	WL	ELBOW	5652508.638	-14065.34	1091.777	2	0.014	0.025	6/20/2013	9:02 PM	1	15	0.033	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		61	LD1010	WL	WL	ELBOW	5652505.487	-14042.77	1091.747	2	0.013	0.021	6/20/2013	9:07 PM	1	14	0.026	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
12		62	LD1011	WL	WL	ELBOW	5652504.07	-13897.92	1091.492	2	0.017	0.022	6/20/2013	9:10 PM	1	12	0.021	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		63	LD1012	WL	WL	ELBOW	5652504.346	-13724.58	1090.13	2	0.011	0.02	6/20/2013	9:13 PM	1	16	0.028	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		64	LD1013	WL	WL	ELBOW	5652519.994	-13685.26	1090.121	2	0.011	0.02	6/20/2013	9:14 PM	1	14	0.023	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT
		65	LD1014	WL	WL	ELBOW	5652505.072	-13642.92	1090.074	2	0.012	0.02	6/20/2013	9:15 PM	1	15	0.026	DART_CLGY	L.DAVIS	3	LD DISCOVERY WL.job	DISCOVERY RIDGE	ELBOW	LEFT

HIGH WATER MARK DATA

PCODE_OR	PCODE	ALIGNMENT	STN_PERP	STN_NUM	OS_PERP	STN_SRC	NORTHING	EASTING	ELEVATION	ANT_HGT	H_PREC	V_PREC	DATE	TIME	EPOCHS	SV	RMS	FROM	SURVEYOR	F_ID	FILENAME	LOCATION	RIVER	BANK
HWM	HWM	ELBOW					5653013.09	-15144.1	1097.102	2	0.012	0.022	6/20/2013	9:04 PM	1	12	0.025	DART_	P.VANOTTERLOO	26	FLD2013-2.jo	DISCOVERY RI	ELBOW	LEFT
HWM	HWM	ELBOW					5652718.08	-14559.31	1093.931	2	0.01	0.018	6/20/2013	9:33 PM	1	16	0.026	DART_	L.DAVIS	3	LD DISCOVER	DISCOVERY RI	ELBOW	LEFT

	Northing	Easting
Sarcee Trail Crossing	5650563	-10216
Upstream Glenmore Reservoir	5650043	-9502

Easting range	-9502	to	-16000
Northing Range	5650043	to	56536000