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Physical Model Study of Gravel Bar Realignment Designs for the Bow River near Louise Bridge

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Executive Summary

This report describes a three-dimensional physical model study conducted by the Ocean, Coastal and River Engineering Research Centre of the National Research Council of Canada (NRC-OCRE), in partnership with the City of Calgary, and in close collaboration with the University of Ottawa. The study was focused on evaluating potential gravel bar realignment designs to address flood hazard concerns for a section of the Bow River near Louise Bridge (i.e. 10th Street).

A three-dimensional model of an approximately 1 km reach of the Bow River was designed and constructed using an undistorted geometric scale of 1:30 in NRC-OCRE's research laboratory in Ottawa, Ontario, Canada. The upstream boundary of the study reach was located near Mewata Bridge (i.e. 14th Street) and the downstream boundary was located a short distance downstream of the Bow River Pathway bridge. The model included a mobile bed composed of scaled granular material, permitting an investigation of hydrodynamic conditions as well as geomorphic change.

The model was calibrated by simulating a scaled, simplified representation of the 2013 flood event and comparing the modelled geomorphic change to the geomorphic change observed in the field. Following the calibration process, the model was used to evaluate three conceptual bar realignment designs under a range of extreme flow conditions including the 10-, 35-, and 100-year return period flows. This report summarizes the physical modelling test program, including an overview of the methodology and key findings, in support of the City of Calgary's flood risk management objectives for the study reach as well as an improved understanding of nature-based solutions.

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1 Introduction

National Research Council Canada (NRC) is Canada's leading research and technology organization and, through its Ocean, Coastal and River Engineering (OCRE) Research Centre, operates one of the world's largest and most advanced hydraulics laboratories dedicated to applied research in coastal and river engineering. The NRC-OCRE laboratory (formerly known as the NRC Canadian Hydraulic Centre) has a strong track record of applied research concerning the design and performance of many types of marine and hydraulic infrastructure, including dams, diversions, spillways, breakwaters, groynes, and revetments.

NRC-OCRE was commissioned by the City of Calgary to conduct a physical model study of conceptual gravel bar realignment designs for a reach of the Bow River in the vicinity of Louise Bridge (i.e. 10th Street). The work is part of a broader initiative, led by the City of Calgary, focused on applying environmentally-conscious bar realignment strategies to manage flood risk, while enhancing habitat and recreational opportunities. The scope of NRC-OCRE's work included the design, construction, and outfitting of a three-dimensional physical model of an approximately 1 km reach of the Bow River. The test program was comprised of two initial test series, focused on assessing model skill and calibrating parameters, as well as three subsequent test series, focused on assessing conceptual bar realignment designs under a range flow conditions.

The project was co-funded by the NRC's Climate Resilient Built Environment initiative, in support of delivering the Government of Canada's Adaptation Action Plan, and towards achieving commitments under the National Adaptation Strategy. Findings from the project contributed to objectives focused on improving scientific understanding and practical application of nature-based solutions to manage flood and erosion risks in riverine environments. The project was conducted in collaboration with the University of Ottawa, who led complementary numerical modelling tasks to inform the physical model design and the test program.

2 Project Background

2.1 Study Site

The Bow River flows through the City of Calgary in a general northwest-to-southeast direction. The headwaters are situated in the Rocky Mountain range, beginning at Bow Lake which receives meltwater from Bow Glacier. The Bow River terminates, by name, at the confluence with the Oldman River in south-central Alberta where the combined flows form the beginning of the South Saskatchewan River.

This study was focused on an approximately 1 km reach of the Bow River in the vicinity of downtown Calgary (Figure 1). The upstream boundary of the study site was located approximately 50 m downstream of Mewata Bridge (i.e. 14th Street) and the downstream boundary was located approximately 130 m downstream of the Light Rail Transit (LRT) bridge that is located immediately downstream of Louise Bridge. The surrounding floodplain is heavily urbanized and there are many anthropogenic factors that influence hydraulics and sediment processes within the channel and along the riverbanks including bank armoring, pedestrian pathways, culverts, bridge piers, and bridge abutments. Historic, anthropogenic interventions at the study site between the late 1800's and the current day are documented by KCBL (2017b), wherein they describe the construction and removal of a dam that once partially spanned the Bow River near Louise Bridge as well as past channel filling works.

2.1.1 The 2013 Flood Event

The 2013 river flooding in Alberta was one of the costliest natural disasters in Canadian history (Reuters, 2021). The event was largely driven by large quantities of surface runoff in the headwaters owing to a combination of precipitation, snowmelt, rain-on-snow, and frozen soil conditions (Teufel et al., 2017). High river flows contributed to flood- and erosion-driven damages in the Bow River basin, including significant flooding in downtown Calgary. The flood event mobilized fluvial sediments in the Bow River, causing considerable geomorphic changes in the channel. In particular, the flood event led to sedimentation in the vicinity of Louise Bridge, creating a large, mid-channel gravel bar which continues to grow and stabilize over time as vegetation colonizes the sediment. Gravel bars were present in the vicinity of Louise Bridge immediately prior to the flood event, but were much smaller in size and more segmented compared to the bar that was formed by the flood event which has persisted to the current day. The flood event also contributed to the growth of pre-existing gravel bars along the north side of the river near the natural park space known as Poppy Plaza. Figure 1 shows satellite imagery of the study site before and after the flood event as well as a recent image of the study site from 2021. Figure 2 shows pre- and post-flood bathymetry and Figure 3 shows the change in elevation between the pre- and post-flood bathymetry. The pre- and post-flood bathymetry data shown in Figure 2 were provided to NRC-OCRE by the City of Calgary; the pre-flood bathymetry was derived from river surveys conducted in 2010 (Golder Associates, 2011) and the post-flood bathymetry was derived from topo-bathymetric LIDAR collected in October, 2020 (City of Calgary, 2021)

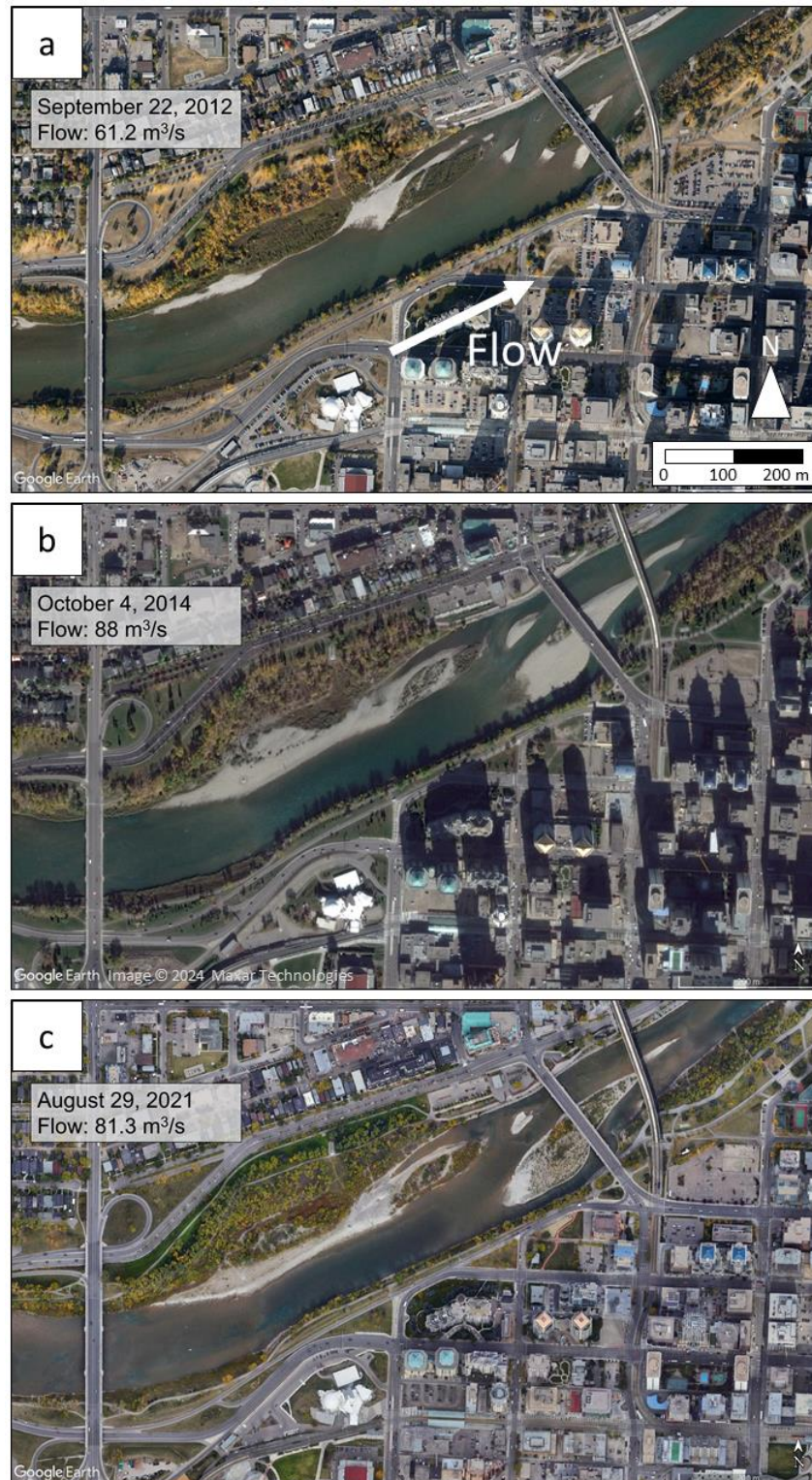


Figure 1. Satellite images of the study site (a) before and (b) after the 2013 flood event, as well as (c) a more recent satellite image from 2021.

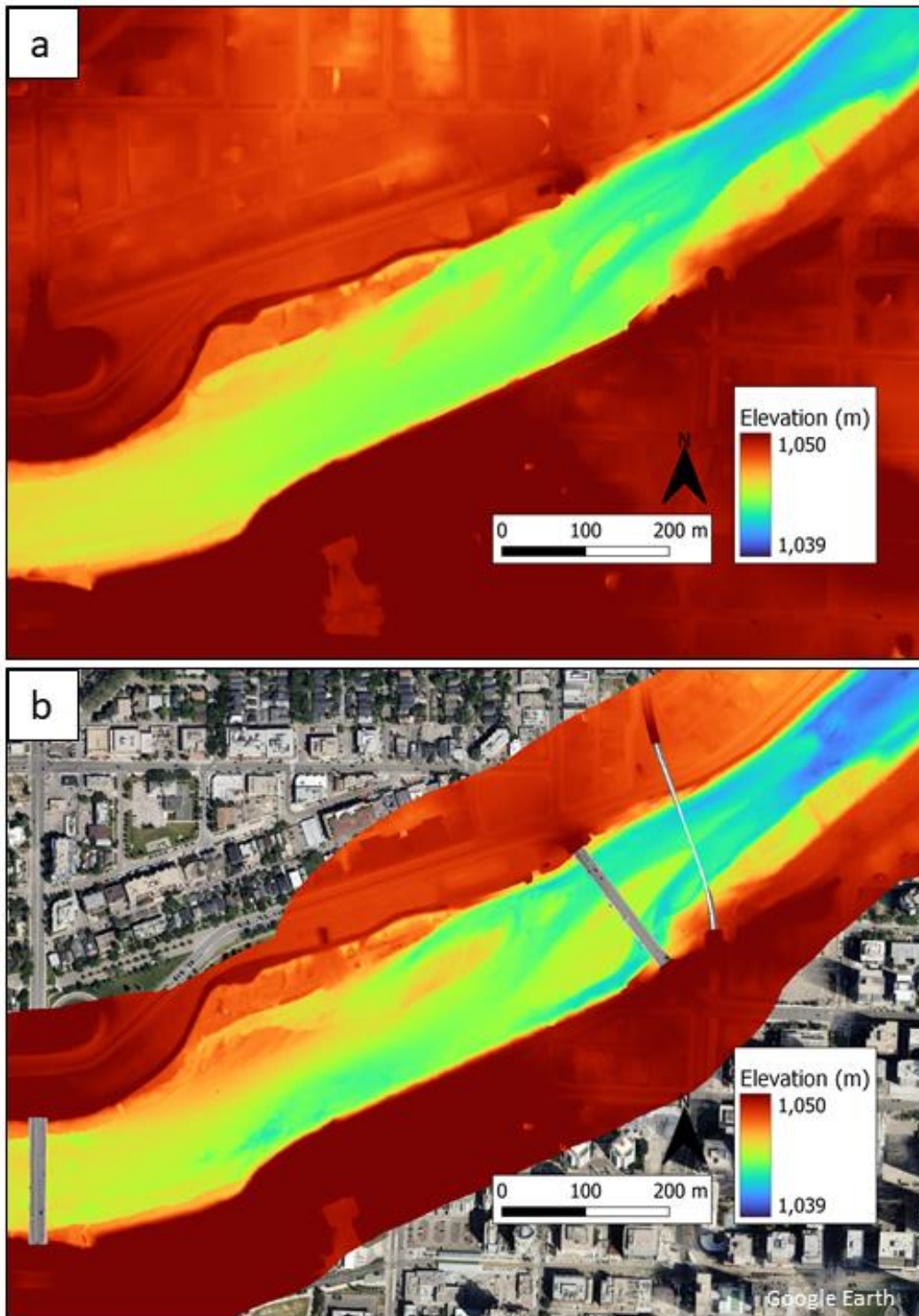


Figure 2. (a) Pre-flood and (b) post-flood bathymetry (City of Calgary, 2021; Golder Associates, 2011).

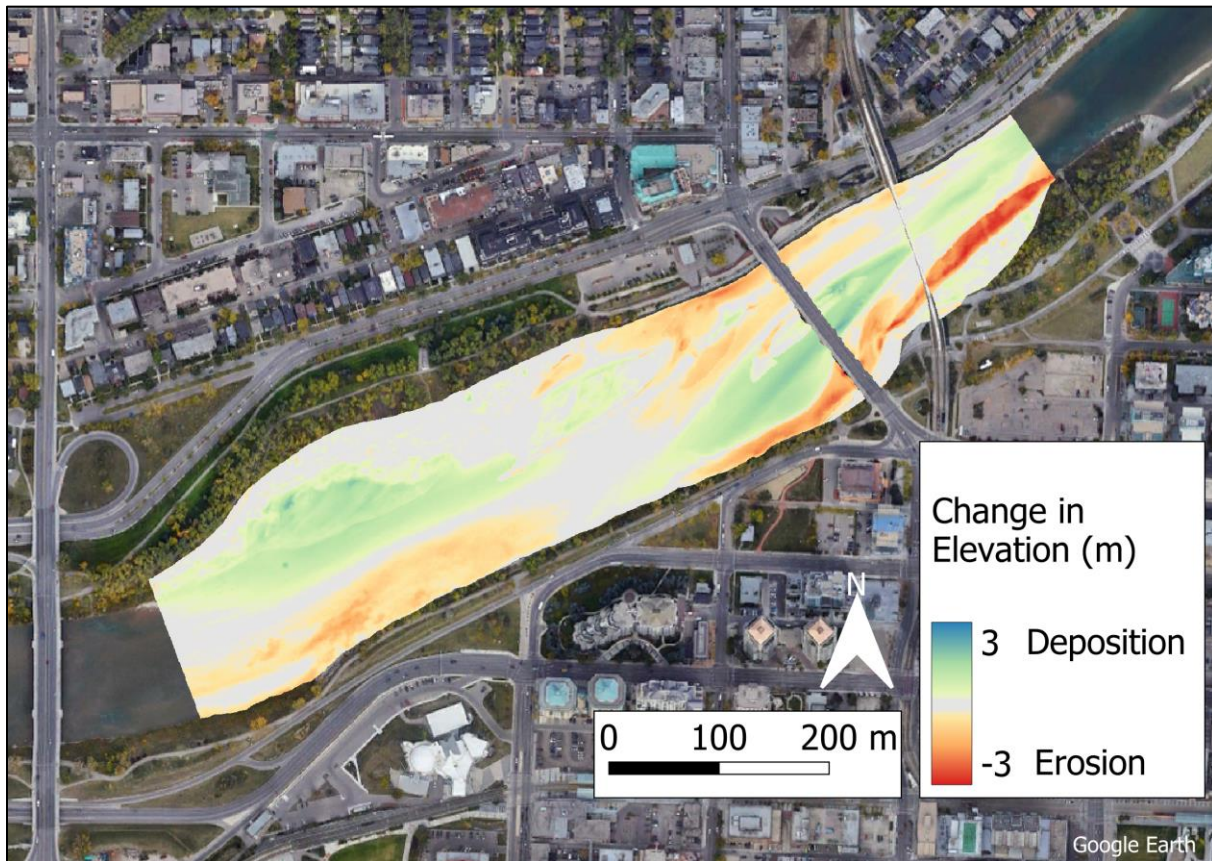


Figure 3. Change in bathymetry caused by the 2013 flood event, derived from City of Calgary (2021) and Golder Associates (2011). Positive values represent an increase in elevation. Changes in elevation between -0.15 m and 0.15 m are shown in grey.

2.1.2 Flood Hazard and Project Motivation

Sedimentation caused by the 2013 flood event has increased blockage in the vicinity of Louise Bridge, constricting flow and elevating flood hazard for the surrounding area (KCBL, 2017b). If left unaddressed, it is anticipated that vegetation will continue to colonize the gravel bars, further reducing flow conveyance and exacerbating flood risk (KCBL, 2017b; Rood et al., 2019). The City of Calgary is investigating options to alleviate the flood risk caused by the gravel bars while being mindful of impacts to natural system functions and pursuing potential opportunities to improve habitat and recreational co-benefits. KCBL (2017b) presented a conceptual design for the study area that involves reshaping the gravel bars to alleviate the blockage and increase flow capacity in the study site. The conceptual design also includes features aimed at improving aquatic habitat. KCBL's conceptual design involves strategic relocation of excavated sediments elsewhere in the Bow River system to minimize disruption of the sediment balance. A complete description of KCBL's conceptual design is presented in a technical report, where they estimate average annual damage reductions of \$716,000 and \$1,358,000 for direct and indirect damages, respectively (KCBL, 2017b). KCBL (2017b) estimated an expected lifespan of 27 years based on the assumption that a flood event with an annual exceedance probability (AEP) of 2.5% (i.e. a 40-year return period event) would mobilize enough sediment that the design would no longer provide its intended function.

The conceptual design proposed by KCBL (2017b) set the stage for additional research to optimize design features and assess performance in more detail. Yu et al. (2022) developed a two-dimensional numerical,

morphodynamic model to assess the flood risk reduction benefits provided by three potential gravel bar management approaches for the study site: a “do-nothing” approach where the study site is left unaltered from its current state, a “bar removal” approach where the gravel bars are removed or dredged from the channel, and a “bar realignment” approach where gravel is redistributed within the channel without significantly changing the total volume of the sediment in the study site. The numerical modelling indicated that the “bar removal” option would provide the greatest flood risk reduction benefit (Yu et al., 2022). However, sediment dredging is generally destructive to aquatic habitat and can also disrupt the sediment balance, potentially leading to undesirable channel incision and other morphologic changes that may elevate risk to surrounding infrastructure (Brown et al., 1998; Damseth et al., 2024; Kondolf, 1997). Morphologic changes caused by gravel extraction can propagate upstream and downstream of the site of extraction (Kondolf, 1997). Understanding that intervention is necessary to address the immediate flood risk caused by the gravel bars, the “bar realignment” concept evaluated by Yu et al. (2022), similar to the design concept proposed by KCBL (2017b), aims to address flood risk while minimizing negative environmental and morphological consequences that would otherwise be expected if a dredging approach were adopted. Compared to dredging, a bar realignment approach could provide a more nature-based solution by allowing sediment transport processes to persist more naturally while integrating design features to enhance (or minimize negative impacts to) habitat and recreation co-benefits. Model results from Yu et al. (2022), indicated that flood risk reduction could be achieved using a “bar realignment” approach, albeit, more modest than the “bar removal” approach. However, analyzing long-term morphologic response, Yu et al. (2022) indicated that a “bar realignment” approach may have superior design longevity compared to a “bar removal” approach.

Owing to the high visibility of the study site and the sensitivity of the highly urbanized floodplain, the City of Calgary retained NRC-OCRE to evaluate potential gravel bar realignment strategies using a physical modelling approach. A well-designed physical model can provide reasonably accurate information on morphologic change and can help improve confidence in, and address the uncertainties of, past numerical modelling. The physical modelling program was designed to evaluate the impact of the conceptual bar realignment designs on flow hydraulics (i.e. water levels and flow velocities) as well as geomorphic change during flood events. In addition to findings from past desktop analyses and numerical modelling, findings from the physical model will help the City of Calgary understand and anticipate the performance of potential bar realignment designs and support informed decision making.

2.2 Terminology for Site and Design Features

The terminology shown in Table 1 was used to assist with descriptions of the site and design features. Features of the study site are labelled in Figure 4.

Table 1. Terminology

Term	Description
Louise Bridge	The 10 th Street Bridge
LRT Bridge	The two-deck bridge downstream of Louise Bridge. The lower deck supports pedestrian foot traffic and the upper deck supports the LRT system.
Main Bar	The mid-channel gravel bar that formed under Louise Bridge during the 2013 flood event
Poppy Plaza	The park space along the north bank of the river between Mewata Bridge and Louise Bridge.
Recreation channel	The existing and proposed channel between the main bar and the south bank where potential recreational enhancements are being considered, including a recreational wave
Habitat channel	The proposed channel along the north bank of the river in the vicinity of Poppy Plaza
Habitat channel crossover	The proposed channel joining the main channel to the habitat channel near the downstream end of Poppy Plaza
South embankment	The proposed embankment along the south bank of the river
The landing	The portion of the south bank near the downstream end of the recreation channel
River Sediment	The natural sediment of the Bow River at the study site
Riprap	Class II riprap with $D_{50} = 500$ mm
Bioengineering	Bank protection strategies that employ vegetation
Bioengineering stone	Granular material that was used to represent bioengineering (i.e. vegetation) with $D_{50} = 200$ mm
Transition cobble	Transition material placed between riprap and bioengineering

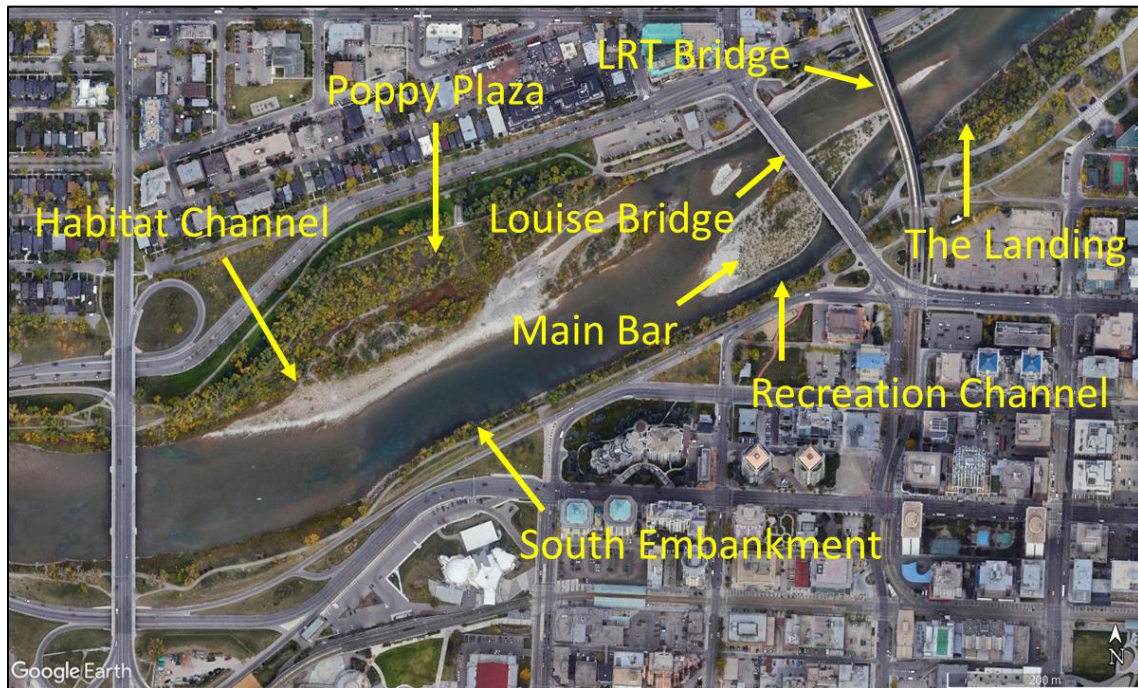


Figure 4. Study site features labelled over a satellite imagery basemap. Note that the location of the proposed habitat channel is indicated but is not visible in the satellite imagery because it is a feature of the conceptual designs.

2.3 Proposed Realignment Designs

The City of Calgary provided NRC-OCRE with three conceptual bar realignment designs. The conceptual designs were developed using an iterative process. Preliminary physical modelling results were shared with the City of Calgary during the course of the test program as they became available and these preliminary results were used to inform subsequent designs. In total, three conceptual designs were tested under a range of flow conditions. Each design aimed to manage flood risk while minimizing environmental impacts and potentially improving habitat and recreation. The City of Calgary retained engineering services from Matrix Solutions Inc. (Matrix) to develop conceptual designs to be tested in the physical modelling program. Conceptual design drawings are shown in Section 10 (Appendix A).

2.3.1 Design #1

The first conceptual design included the following channel modifications and features:

- Narrowing of the Louis Bridge Bar
- Excavation of the landing
- Excavation to construct a habitat channel in the vicinity of Poppy Plaza
- Excavation to construct a habitat channel crossover in the vicinity of Poppy Plaza
- Redistribution of excavated sediment along the south embankment
- Installation of riprap to protect the nose of the south embankment, the nose of the main bar, the downstream end of the main bar, and a portion of the south bank of the recreation channel
- Installation of bioengineering materials above the vegetation trim line to protect newly constructed features that are composed of redistributed river sediment

The initial design focused on addressing objectives pertaining to flood risk, habitat, and recreation using a “light armoring” approach to alleviate potential disturbance of the natural sediment balance. Riprap was specified in discrete locations to enhance the stability of specific features. Elsewhere, the channel was composed of natural river sediments, protected only by vegetation or bioengineering above the vegetation trimline. Riprap was specified at the nose of the south embankment, the nose of the main bar, the portion of the main bar downstream of Louise Bridge, and along the south bank of the recreation channel where a potential recreational wave is being considered by the City of Calgary.

2.3.2 Design #2

The second conceptual design was developed following an assessment of preliminary physical modelling results for Design #1 and shared many design similarities. The following list summarizes key differences between Design #1 and Design #2:

- Additional riprap was specified along a portion of the north bank of the habitat channel
- The habitat channel crossover proposed in Design #1 was removed
- The design specified redistribution of excavated sediment in the former location of the potential habitat channel crossover, increasing the continuity of the bar adjacent to Poppy Plaza and reinforcing the separation between the main channel and the habitat channel
- Additional riprap was specified on the nose of the south embankment
- The shape of the south embankment, formed from redistributed sediment, was modified
- The main bar was modified to conform to a more uniform shape, instead of an undulating shape
- The nose of the main bar was reshaped
- Additional riprap was specified along the main bar at the intersection with the Louise Bridge pier
- Bioengineering was specified at the crest of the main bar downstream of Louise Bridge as opposed to riprap
- The design specified that riprap voids should be filled with river sediment
- An additional surface treatment “transition coble” was specified in various locations adjacent to riprap armouring
- Built infrastructure was specified in the recreation channel to reflect the presence of potential recreational wave infrastructure, including a concrete hump downstream of Louise Bridge, a riprap apron downstream of the hump, and a riprap sill at the exit of the recreation channel
- Three repelling groynes were specified along the south embankment, each surrounded by a reinforced apron to reflect connection to a natural clay layer at the base of the groynes
- One attracting groyne was specified near the entrance of the habitat channel

2.3.3 Design #3

The third conceptual design was developed following an assessment of preliminary physical modelling results for Design #1 and Design #2 and shared many design similarities. The following list summarizes key differences between Design #2 and Design #3:

- The shape of the main bar was modified to be narrower than the bar specified in Design #2 and to accommodate a potential helicopter landing pad upstream of Louise Bridge
- The shape of the south embankment, formed from redistributed sediment, was modified and is narrower than the south embankment specified for Design #2
- The two upstream-most repelling groynes along the south bank specified in Design #2 were removed
- Additional riprap armouring was specified along the south embankment, beginning at the nose of the embankment and extending approximately 290 m downstream
- Built infrastructure in the recreation channel was modified; the crest elevation of the concrete hump was lowered and the riprap sill was removed

3 The Physical Model

3.1 NRC-OCRE Large Area Basin and High Discharge Flume

The Bow River physical model was situated in NRC-OCRE's Large Area Basin (LAB). The rectangular multi-use LAB facility is located indoors, has interior dimensions of 50 m-long by 30 m-wide, can accommodate water depths of up to 1.4 m, and can be filled and drained quickly via high-capacity pumps. The basin is easily accessible to heavy construction equipment, and the water is supplied from storage reservoir sumps beneath the basin floor. For riverine physical models, like the Bow River model, the LAB is connected to another physical modelling facility – the High Discharge Flume (HDF).

NRC-OCRE's HDF is used to simulate a wide range of open channel flows and study problems in rivers, canals and other inland waterways. The facility is 10 m x 2.7 m x 1.4 m deep and is made of stainless steel. The flume provides a steady flow in an open channel with continuously adjustable discharge from 0 to 1.6 m³/s. The discharge is controlled by two pumps, one with a variable pitch. For this model study, the HDF was connected to the LAB and provided the input flows upstream of the model river. A schematic showing the HDF and the LAB in NRC-OCRE's Ottawa laboratory is shown in Figure 5.

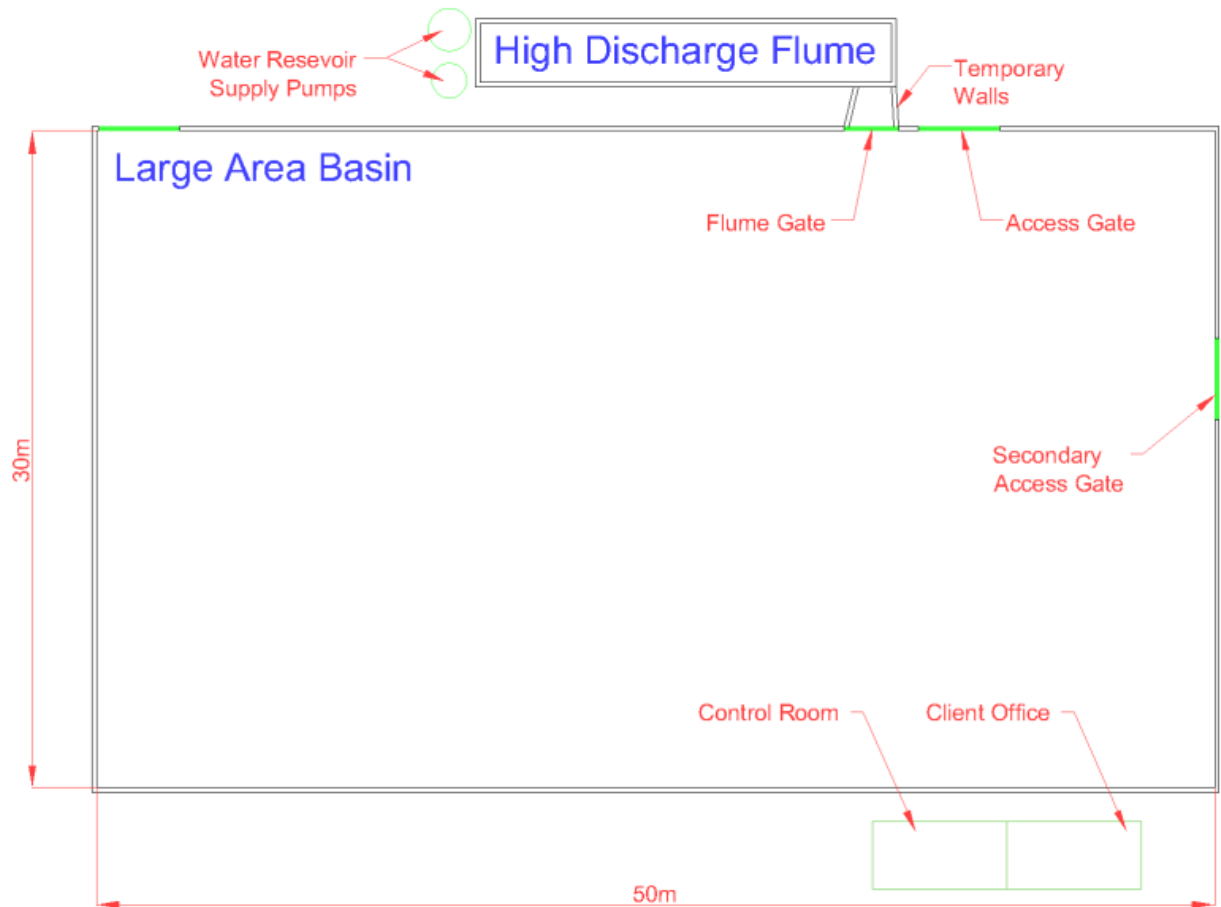


Figure 5. Schematic of NRC-OCRE's Large Area Basin and High Discharge Flume facilities (all dimensions in model-scale).

3.2 Scaling

The selection of a suitable geometric scale is a key element in the design of any physical model. Although many factors are considered, the decision hinges on establishing a compromise between two important and conflicting requirements:

- Minimizing scale effects, particularly with respect to the simulation of morphodynamic processes, by selecting the largest model-scale (MS) possible; and
- Working within constraints related to the dimensions and capabilities of key equipment such as the test facility, the pumps used to generate flows, and the sensors used to monitor conditions in the model.

The Bow River model was designed and constructed as an undistorted model with a length scale of 1:30. This scale represents a reasonable compromise between minimizing scaling effects while maximizing the extent of the model domain and the accuracy and realism of the morphodynamic processes being modelled. All lengths in the model (horizontal and vertical) were 30 times less than the corresponding lengths at full-scale (FS).

All aspects of the model (except for the river sediment, as described in Section 3.2.1) were designed using Froude scaling criterion, which assumes that gravitational and inertial forces are dominant in comparison to viscous forces. Froude scaling provides a set of scaling laws that dictate the proper relationship between quantities measured in the model, such as flows and velocities, and the same quantities at full-scale. These scaling laws are derived from the similarity of the Froude number, which represents the ratio between gravitational and inertial forces. Setting the scale factor for the Froude number (λ_{Fr}) equal to unity ensures that the relative strengths of inertial and gravitational forces are the same in the model as they are at full-scale. This similarity in Froude number, combined with geometric similitude, ensures that the model provides a reliable physical representation of flow patterns and hydraulic conditions that occur at full-scale.

Scale factors for relevant parameters in the present study are presented in Table 2. Stated quantities and measurements in this report are in full-scale units unless stated otherwise.

Table 2. Scale factors for key project variables.

Variable	Scale Factor	Typical Value at Full-Scale	Corresponding Model Value
Length	$\lambda_l = 30$	10.0 m	0.333 m
Time	$\lambda_t = \sqrt{\lambda_l} = 5.5$	10.0 sec	1.83sec
Velocity	$\lambda_u = \frac{\lambda_l}{\lambda_t} = 5.5$	1.0 m/s	0.18 m/s
Acceleration	$\lambda_a = \frac{\lambda_l}{\lambda_t^2} = 1.0$	9.81 m/s ²	9.81 m/s ²
Froude Number	$\lambda_{Fr} = \frac{\lambda_u}{\sqrt{\lambda_g \lambda_l}} = 1.0$	1.0	1.0
Reynolds Number	$\lambda_{Re} = \frac{\lambda_u \lambda_l}{\lambda_\nu} = 5.5$	2,000,000	363,636
Density of water	$\lambda_{\rho_w} = 1.0$	1,000 kg/m ³	1,000 kg/m ³
Mass	$\lambda_M = \lambda_l^3 \lambda_\rho$ $= 27000 \lambda_\rho$	1 tonne	37 g

3.2.1 Scaling of Granular Material

The physical model included a fully mobile bed to support an investigation of geomorphic change in response to flow. Granular materials represented in the physical model included the native river sediment, riprap, and transition cobble. Bioengineering was also represented using granular material. Bioengineering strategies, in reality, leverage bank protection services provided by vegetation. However, live plants could not be employed in the physical model owing to practical restrictions and the challenges and uncertainties of scaling biomass. In this report, the term “riprap” or “armour stone” is used to describe the Class II riprap used to armour sections of the river, and the term “bioengineering stone” is used to describe the granular

material that was used to represent bioengineering (i.e. vegetation). The river sediment, riprap, transition cobble, and bioengineering stone used in the model were selected, as much as possible, to ensure that they represented the prototype materials in a realistic manner.

Table 3 summarizes the grain size distribution specified for the riprap and Table 4 summarizes the grain size distribution for the native sediment in the vicinity of the study site based on field observations. The grain size distributions shown in Table 4 represent the subsurface material; a more complete summary of river sediment observations, including surface and subsurface materials, can be found in KCBL (2016).

Table 3. Class II riprap gradation (AMEC, 2012).

Percent Passing (%)	Grain Size (mm)
100	800
80	600
50	500
20	300

Table 4. River sediment gradations (KCBL, 2016).

Percent Passing (%)	Grain Size (mm)			
	KCBL Site C ~2.8 km upstream of Louise Bridge	KCBL Site D Located at Louise Bridge	KCBL Site E ~1.6 km downstream of Louise Bridge	KCBL Site F ~2.0 km downstream of Louise Bridge
90	74	100	94	-
65	37	67	39	151
50	28	48	24	93
15	6	10	1	24

Granular material with a full-scale D_{50} of 200 mm was used to represent the bioengineering stone based on information from AMEC (2012) which suggests that this material would provide erosion resistance similar to live willow stakes (see Table P.7 from AMEC which summarizes permissible shear stresses for various boundary types). A thin layer of bioengineering stone (with a thickness approximately equal to one to two times the D_{50}) was adopted for sections where bioengineering was specified. A thick layer of bioengineering stone (approximately 1 m thick full-scale) was adopted where transition cobble was specified.

3.2.1.1 Bed Shear Stress in Steady Flow

Granular materials on a riverbed become mobilized when the shear stress exerted by the flow on the bed, τ_o , exceeds the critical shear stress required for initiation of motion, τ_c . In steady uniform flow, a force balance (Figure 6) on a control volume yields the following:

$$\begin{aligned}
 P_1 - P_2 + W_x - F_f &= 0 & , P_1 = P_2 \text{ uniform flow} \\
 F_f &= W \sin(\alpha) \\
 \tau_o L P &= \gamma A L \sin(\alpha) \\
 \tau_o &= \gamma R S_o & , \sin(\alpha) \approx \tan(\alpha) = S_o \quad (1) \\
 & & \text{for mild slope}
 \end{aligned}$$

where P_1 and P_2 represent the hydrostatic pressure force at the upstream and downstream end of the control volume, W represents the weight of the control volume, W_x represents the component of the weight parallel to the bed, F_f represents the resistance force in the streamwise direction, α represents the angle of the bed slope, L represents the length of the control volume, P represents the wetted perimeter of the channel, γ represents the specific weight of water, A represents the cross-sectional area of the flow, R represents the hydraulic radius, and S_o represents the bed slope.

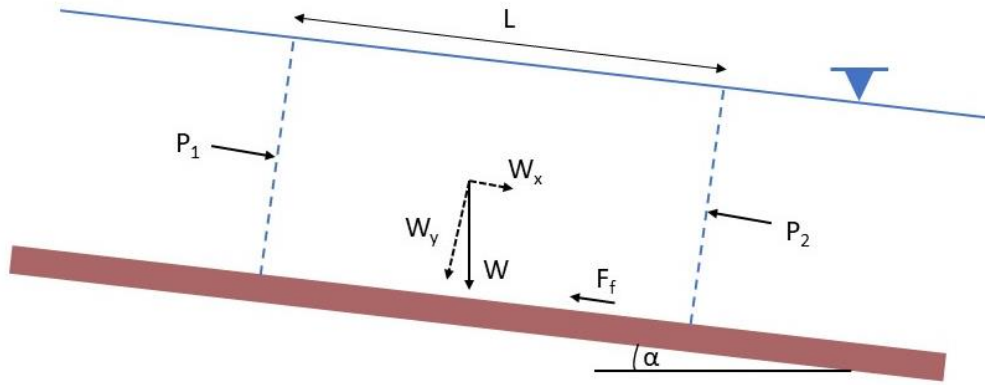


Figure 6. Free body diagram of a control volume.

Assuming a wide channel ($R \approx h$), the equation can be simplified to the depth-slope product:

$$\tau_o = \gamma h S_o \quad (2)$$

where h is the depth of the flow.

3.2.1.2 Mobility threshold for Granular Materials

The Shields parameter (often denoted as θ or τ_*) provides a means of expressing the shear stress acting on a riverbed in non-dimensional form:

$$\theta = \frac{\tau_o}{(\rho_s - \rho)gD} = \frac{u_*^2}{(S - 1)gD} \quad (3)$$

where ρ_s represents the density of the granular material, ρ represents the density of water, g represents gravitational acceleration, D represents the diameter of the granular material, S represents the specific gravity of the granular material, and u_* represents the shear velocity (Chin, 2013; van Rijn, 1984).

Shields diagram plots the threshold value of the Shields parameter (also referred to as the dimensionless critical shear stress or critical mobility parameter), θ_{cr} , as a function of the grain Reynolds number:

$$Re_* = \frac{u_{*cr} D}{\nu} = \frac{\sqrt{\frac{\tau_{cr}}{\rho}} D}{\nu} \quad (4)$$

where u_{*cr} represents the threshold shear velocity, ν represents the kinematic viscosity of water, and τ_{cr} represents the critical shear stress. However, to support practical application of the Shields diagram, a mathematical transformation is often applied to the grain Reynolds number so that the threshold Shields parameter can be plotted as a function of the nondimensional grain parameter (Soulsby & Whitehouse, 1997):

$$D_* = \left(\frac{Re_*^2}{\theta_{cr}} \right)^{\frac{1}{3}} = D \left(\frac{g(S-1)}{\nu^2} \right)^{\frac{1}{3}} \quad (5)$$

Figure 7, sourced from van Rijn (1984), illustrates initiation of motion according to Shields (1936), plotting Shields critical mobility parameter as a function of the nondimensional grain parameter. Figure 7 also provides mathematical formulas approximating Shields critical mobility for different regions of the Shields diagram (van Rijn, 1984).

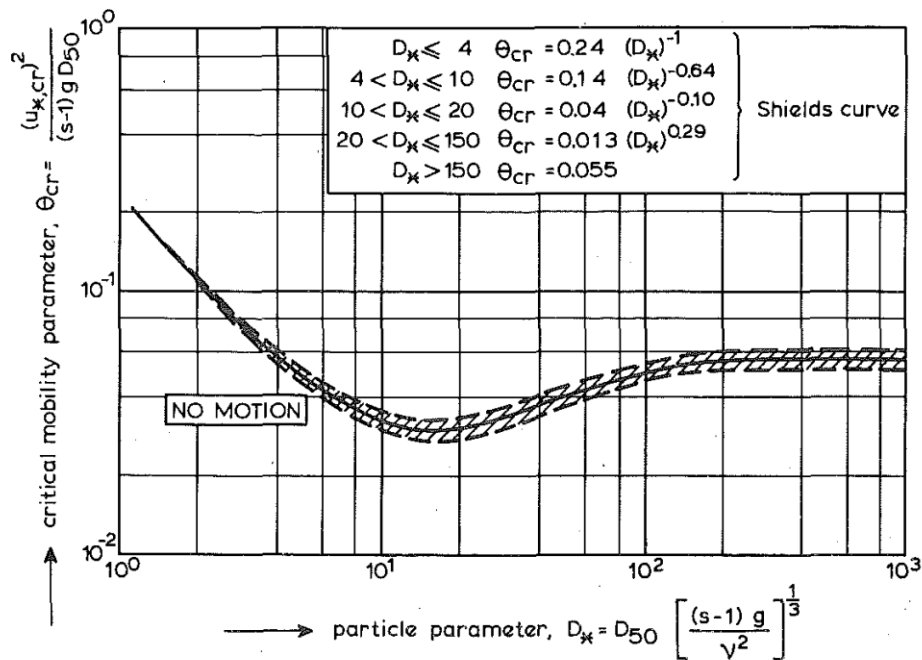


Figure 7. Initiation of motion according to Shields (van Rijn, 1984).

The Shields diagram provides fundamental context to support an understating of initiation of motion and particle mobility. The diagram indicates that a particle with particle parameter, D_* , will be mobile if the Shields parameter (or dimensionless shear stress), θ , is greater than the critical mobility parameter (or dimensionless critical shear stress), θ_{cr} .

3.2.1.3 Riprap, Bioengineering Stone, and Transition Cobble

Frostick et al. (2011) provide guidance on physical modelling and experimentation for hydraulic research. Chapter 4 of the publication, which provides guidance pertaining to sediment dynamics, indicates that bed material may be geometrically scaled according to Froude Scaling (see Section 3.2) provided that the model material has the same density and the prototype material and provided that the model material is sufficiently large. As described in Frostick et al. (2011):

The thresholds Shields parameter of rock-protection is (almost) identical in model and prototype provided that the model rock is sufficiently large that viscous effects can be ignored. However, this only holds true (to within 10%) for non-dimensional grainsize $D_* > 120$, which for quartz in fresh water at 20°C corresponds to $D > 5\text{mm}$. For smaller model rocks, and for sandy sediments, the thresholds Shields parameter varies with D_* and will generally not correspond between model and prototype.

The D_{50} grain size of the riprap and bioengineering stone described in Section 3.2.1 meet the criteria outlined by Frostick et al. (2011); applying a geometric scale of 1:30 to the riprap and bioengineering stone yields model-scale D_{50} values that exceed 5 mm in diameter. Therefore, representative (target) model-scale D_{50} values for the riprap and bioengineering stone were determined through geometric scaling, and suitable materials were identified from existing laboratory resources and a local quarry. As described in Section 3.2.1, the transition cobble was composed of the same material as the bioengineering stone. Gradations for the model riprap and bioengineering stone are summarized in Table 5 and Table 6. The gradations are presented graphically in Figure 8 and Figure 9, alongside scaled target grain sizes.

Table 5. Model riprap gradation.

Percent Passing (%)	Applied MS Grain Size (mm)	Equivalent FS Grain Size (mm)
100	25	750
86.1	19	570
48.8	16	480
18.4	13.2	396
2.9	9.5	285
2	4.75	142.5

Table 6. Model bioengineering stone gradation.

Percent Passing (%)	Applied MS Grain Size (mm)	Equivalent FS Grain Size (mm)
100	12.5	375
98.4	9.5	285
49.8	6.7	201
14.7	4.75	142.5
5.4	2.36	70.8
3.8	1.18	35.4
3.2	0.6	18
2.5	0.3	9
2	0.15	4.5
1.48	0.075	2.25

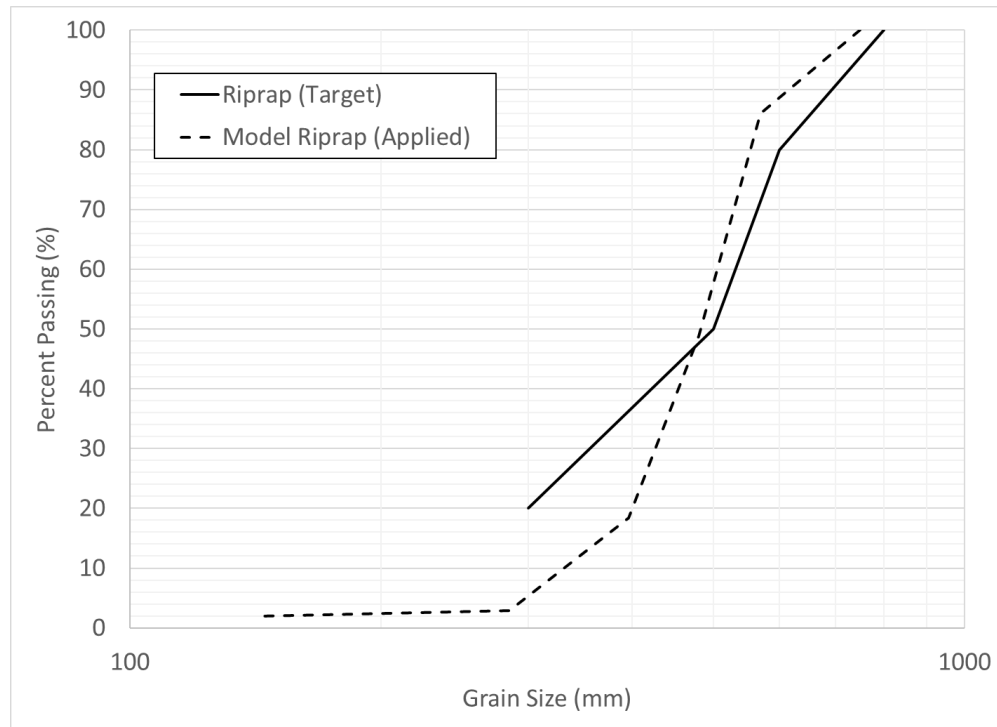


Figure 8. Target and model riprap gradations, presented in full-scale.

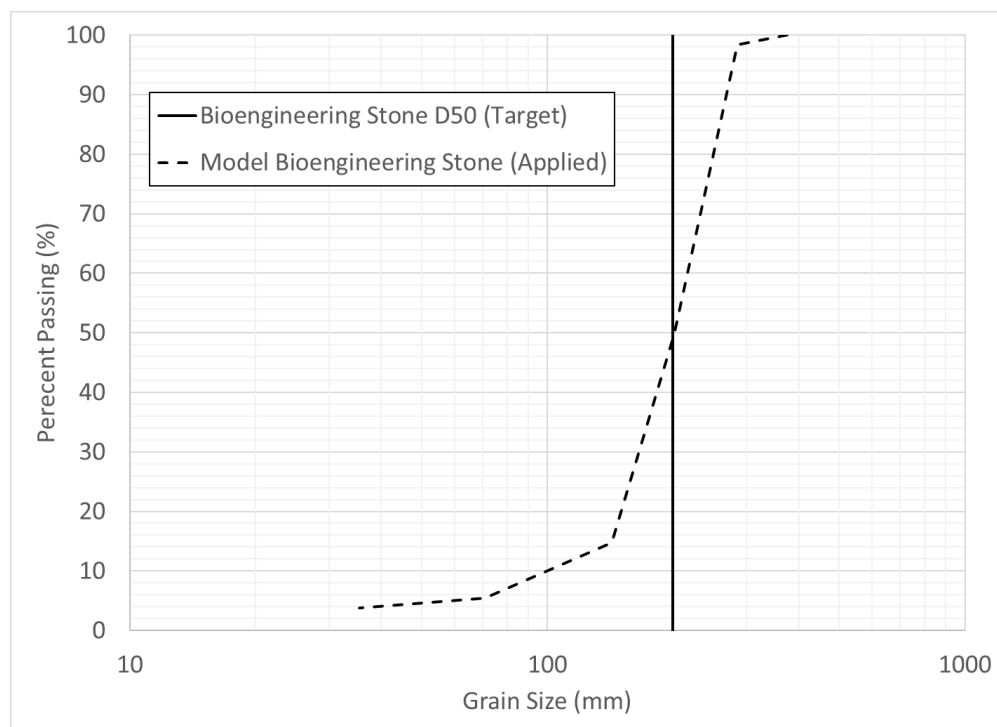


Figure 9. Target bioengineering stone D₅₀ and model bioengineering stone gradation, presented in full-scale.

3.2.1.4 River Sediment

The prototype river sediment gradations presented in Table 4 (KCBL, 2016) provided information on the range and distribution of grain sizes in the vicinity of the study site, and were used to inform the selection of a representative model-scale grain size distribution. Applying the simplified geometric scaling approach (as described in Section 3.2.1.3) resulted in a model-scale river sediment that fell outside of the range recommended by Frostick et al. (2011) where geometric scaling could be applied acceptably. Therefore, additional effort was required to compute target model-scale grain size distributions from the full-scale prototype distributions shown in Table 4.

Geometric scaling was applied to the prototype river sediment (Table 4) to establish preliminary estimates for target model-scale grain size distributions using the 1:30 scale factor. For each prototype grain size, the mobility of the prototype grain size and the mobility of the geometrically scaled grain size were evaluated based on the excess dimensionless shear stress ($\theta_{excess} = \theta - \theta_{cr}$). Bed shear (τ_o) at prototype- and model-scale was computed using the depth slope product (Eq. (2)). When computing bed shear, slope was set equal to the reach average bed slope ($S_o = 0.0018$ m/m) and depth was set equal to the approximate flow depth during the 2013 flood event ($h_{prototype} = 6.4$ m; $h_{model} = 0.21$ m). Reach average bed slope was sourced from KCBL (2017a) and depth was approximated from Yu et al. (2022) and confirmed through communication with the City of Calgary and the University of Ottawa. Owing to uncertainty regarding flow depth at the study site during the 2013 flood, the sensitivity of depth estimates on grain size scaling was evaluated for depths ranging from 5m and 7m, and it was found that grain size scaling was not very sensitive to these values. The dimensionless particle parameter (D_*) was computed for each prototype and geometrically scaled grain size using Eq. (5), and corresponding values for dimensionless critical shear stress (θ_{cr}) were computed using Shields diagram (Figure 7). The Shields parameter (θ) was computed for

each prototype and geometrically scaled grain size using Eq. (3). Using the geometrically scaled grain sizes as an initial estimate, the model-scale grain sizes were iteratively adjusted to minimize the difference between $\theta_{excess,prototype}$ and $\theta_{excess,model}$. The adjusted model-scale grain sizes represent the target model-scale grain sizes. This approach ensures that all of the target sediment fractions have a similar mobility in the model as in the prototype. In most cases, the adjusted target model-scale grain sizes needed to be increased slightly from the initial grain size estimates computed from geometric scaling. Figure 10 provides a visual example of the scaling process for the river sediment at Louise Bridge (i.e. Site D in KCBL (2016)).

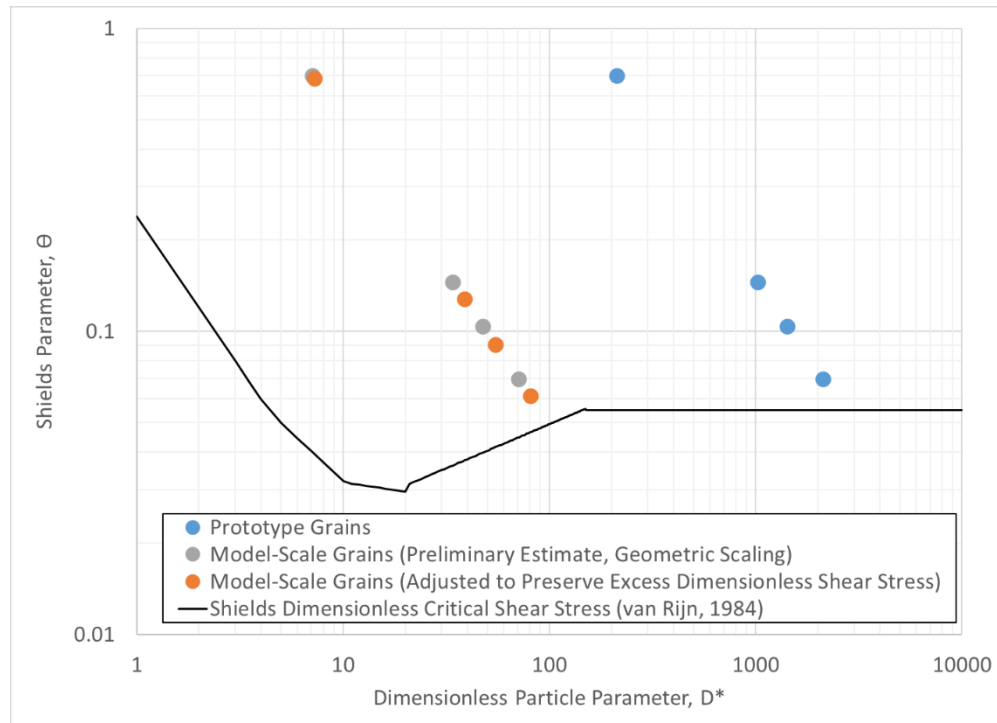


Figure 10. Example of the scaling process for the river sediment using grain size data from KCBL (2016) Site D.

The scaling process was applied to each gradation presented in Table 4 to develop a range of target model-scale grain size distributions. This information was used to guide the selection of a suitable material from a local quarry. Figure 11 shows the gradation of the model river sediment, plotted alongside the target gradations.

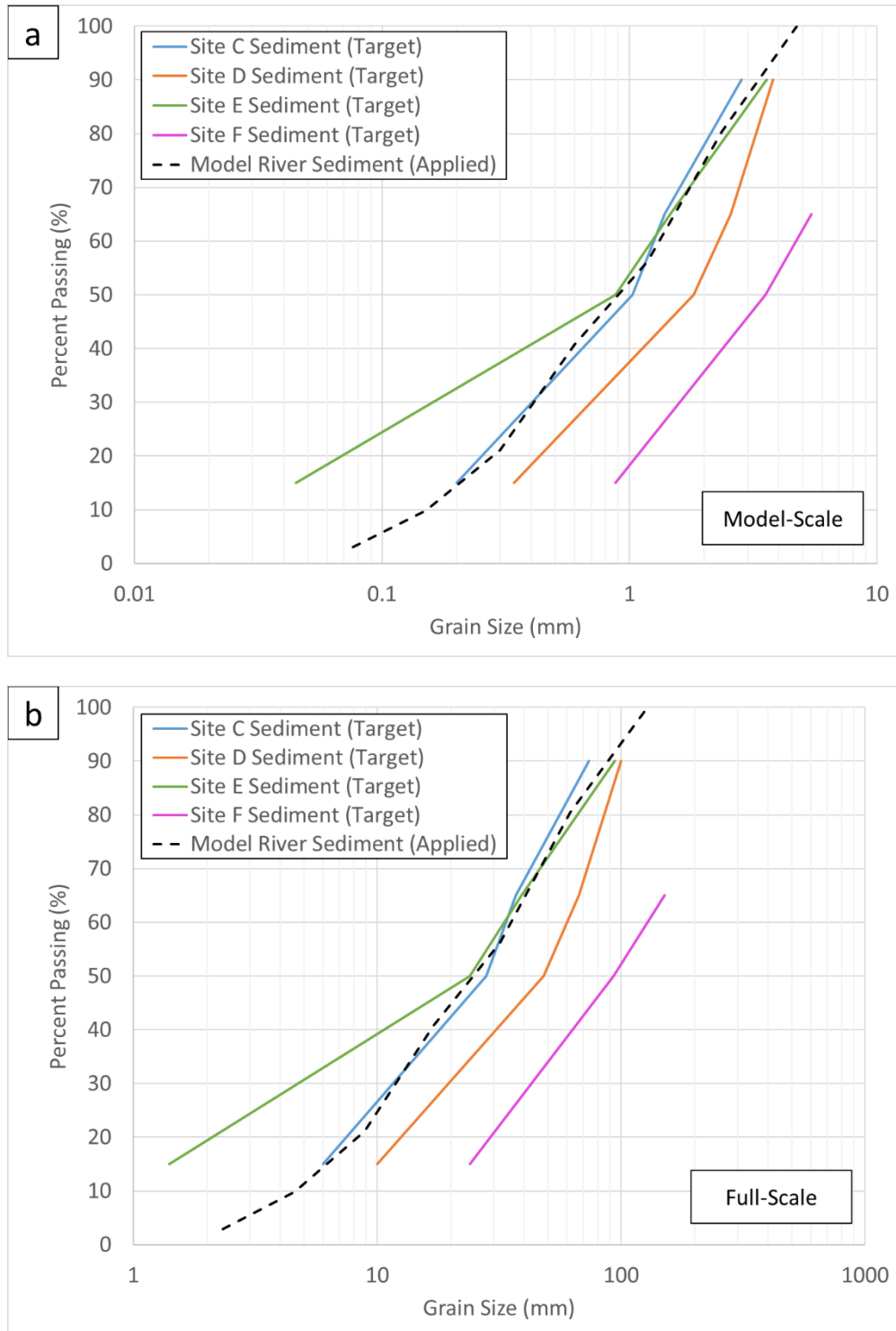


Figure 11. Target and model river sediment shown in (a) model-scale and (b) full-scale.

3.3 Model Layout and Design

As described in Section 2.1, the model domain encompassed an approximately 1 km reach of the Bow River in the vicinity of downtown Calgary. The upstream boundary of the study site was located approximately 50 m downstream of Mewata Bridge (i.e. 14th Street) and the downstream boundary was located approximately 130 m downstream of the Bow River Pathway Bridge. The model was constructed in NRC-OCRE's LAB facility with flow input from the HDF facility (Section 3.1). The model included two general sections: a mobile bed constructed from river sediment and other prescribed materials (e.g. riprap) and immobile river banks and floodplain spaces constructed from concrete.

The mobile bed included all spaces within the study area where geomorphic change during large flood events could reasonably be expected as well as locations where topographic modifications were expected in accordance with the conceptual bar realignment designs. River banks that are protected by existing armouring were not included in the mobile bed (e.g. portions of the Bow River Pathway adjacent to the river). The extent of the mobile bed was determined in consultation with the City of Calgary. The mobile bed was underlain by a flat, smooth bed of concrete gently sloped, at 0.003 m/m, in an upstream-to-downstream direction. The thickness of the mobile bed on top of the concrete bed varied throughout the model domain. The elevation of the concrete bed underlying the mobile bed (and, therefore, the thickness of the mobile bed) was selected such that the maximum scour depths anticipated from numerical modelling by Yu et al. (2022) could be fully accommodated by the mobile bed, without scour down to the concrete.

The immobile banks represent the area outside of the mobile bed. The immobile banks were contoured to faithfully represent the real bathymetry/topography up to the highest expected water levels during a 100-year flood. The highest faithfully represented elevation contour near the upstream end of the model was 1050.5 m and the highest faithfully represented elevation contour near the downstream end of the model was 1048.0 m on the north bank and 1048.5 m on the south bank. The maximum expected water level during a 100-year flood is approximately 1050.0 m and 1048.0 m near the upstream and downstream end of the model, respectively (KCBL, 2016). Beyond the faithfully represented elevation contours described above, the concrete banks were sloped to an artificially high plateau of 1055 m. These elevated plateaus provided additional freeboard to ensure no transfer of water between the model space and the basin, provided a dry area to support the installation of instrumentation infrastructure (e.g. beams, cantilevered arms), and provided walkable platforms on either bank of the model to support personnel access during testing.

Upstream flow conditions were controlled by NRC-OCRE's HDF facility which is outfitted with a variable pitch pump that draws water from a sump reservoir beneath the basin floor. Flows entering the north end of the LAB were guided toward the upstream boundary of the model reach using flow guides. Unrealistic vortices near the upstream boundary were dampened using perforated screens to promote, as much as possible, an even flow distribution perpendicular to the boundary. Downstream water levels were controlled by an adjustable mechanical weir. Flow spilling over the weir was returned to the sump reservoir.

Figure 12 displays an overview of the model in NRC-OCRE's LAB, Figure 13 shows an example cross section of the model, and Figure 14 shows photographs of the physical model after construction with and without a mobile bed installed. When projected into real coordinates, the positive x direction shown in Figure 12 is 44.39° counterclockwise from east or 45.61° clockwise from north (i.e. flow velocity in the positive x direction in the model represents flow velocity 44.39° counterclockwise from east in reality).

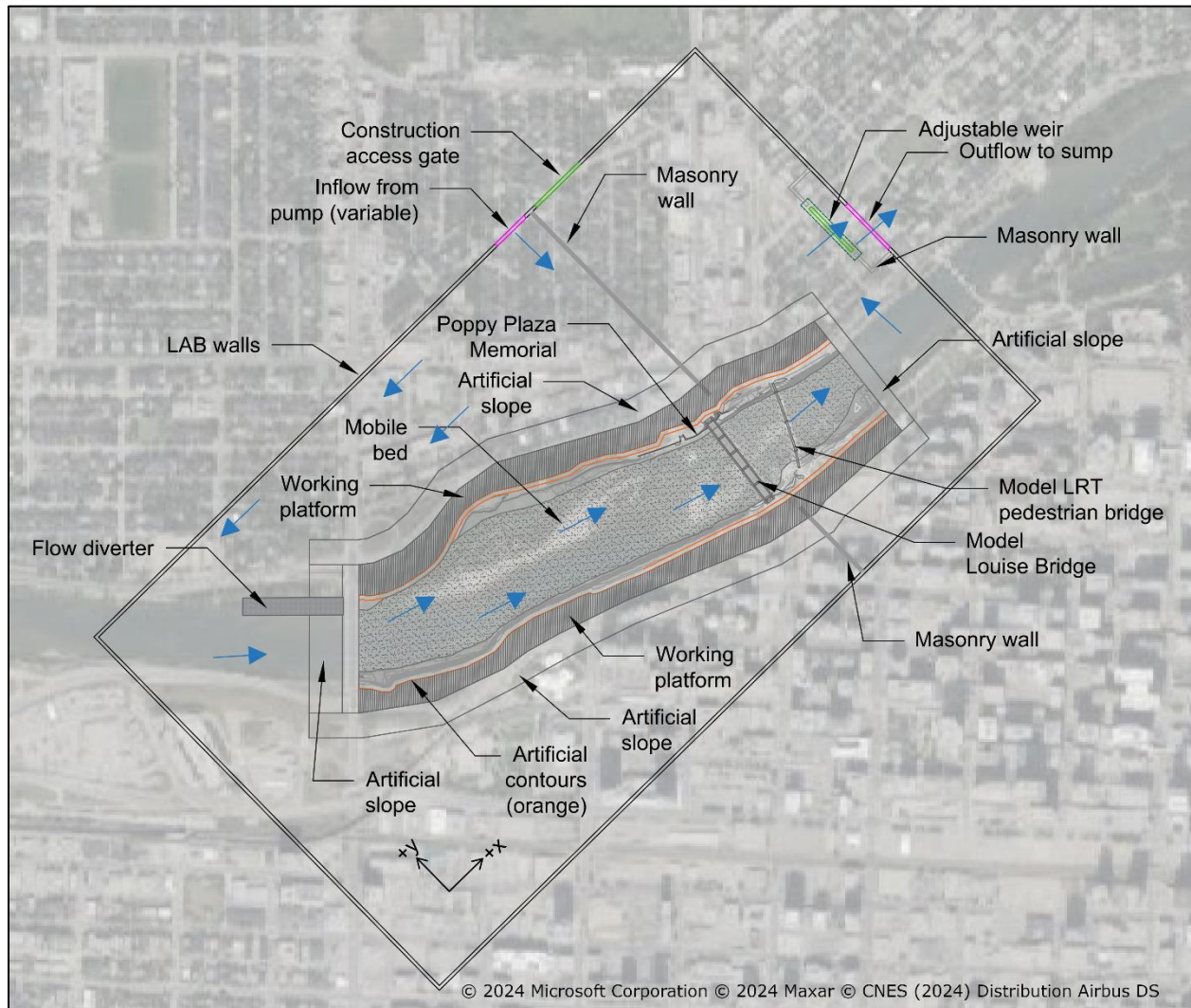


Figure 12. Overview of the physical model projected onto a satellite imagery basemap.

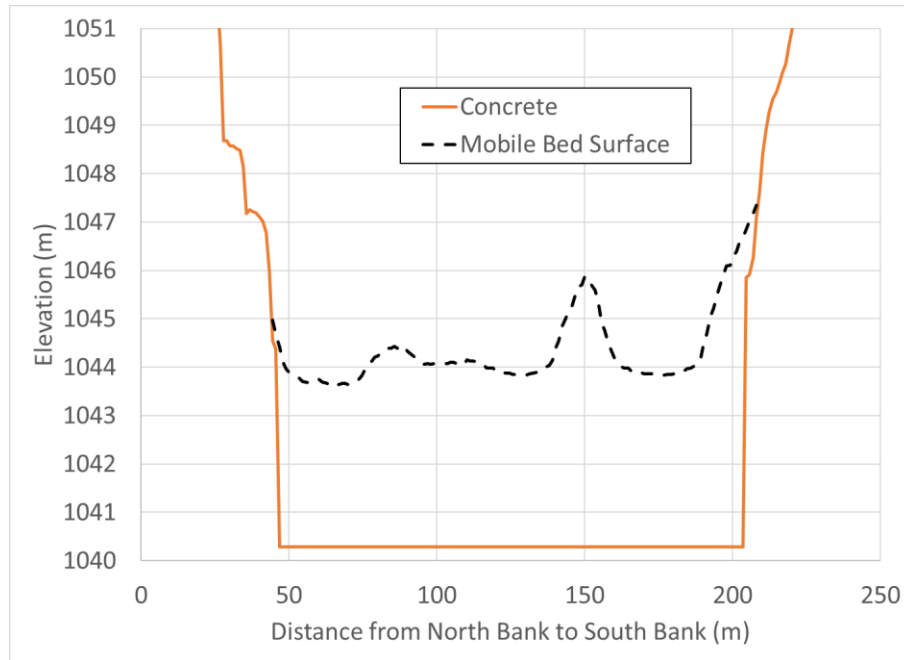


Figure 13. Example cross-section of the as-built initial bathymetry for conceptual design #1 near the nose of the main bar. The main bar approximately coincides with distance = 150 m along the x-axis.



Figure 14. Photographs of the physical model after construction; a) and b) show the model without a mobile bed installed and c) and d) show the model with a mobile bed installed.

3.4 Model Construction

3.4.1 Concrete Bathymetry and Topography

The concrete bathymetry was constructed using a series of Oriented Strand Board (OSB) templates, installed at 1 m intervals (MS) throughout the test section. The templates were used to represent cross-sections of the river bed and banks outside of the mobile bed (described in Section 3.4.2). The plywood templates were shaped to conform to 0.5 m (FS) contours derived from topo-bathymetric LIDAR data, surveyed in 2020, provided by the City of Calgary (City of Calgary, 2021). The templates were cut to an accuracy of ± 2 mm (MS) and were supported on the basin floor by a grid of pads levelled to within ± 1 mm (MS). Heavy construction equipment was used to fill the template with clear gravel once secured on top of the levelled pad. The gravel was placed such that the final surface rested approximately 5 cm (MS) below the top edge of the templates. Grout was then placed on top of the gravel base and power screeded to match the elevations of the templates. In areas of the model where there were steep slopes or complex bathymetry, the grout was screeded by hand. Once partially cured, hand or mechanical trowelling was applied to the concrete at the entrance of the model, the exit of the model, and underlying the mobile bed to achieve a smooth finish. A broom finish was applied to concrete portions representing exposed portions of the river bed, bank, and floodplain spaces to achieve a rough finish that is more representative of the natural hydraulic roughness. Photographs showing some of the stages of the bathymetry construction are shown in Figure 15.

This construction technique has been developed and perfected at NRC-OCRE over the past 30 years. The skilled technicians undertaking this work have a well-proven track record of reproducing complex bathymetries with a high level of precision and accuracy. The resulting bathymetry is generally accurate to ± 5 mm (MS), which is equivalent to ± 15 cm (FS).

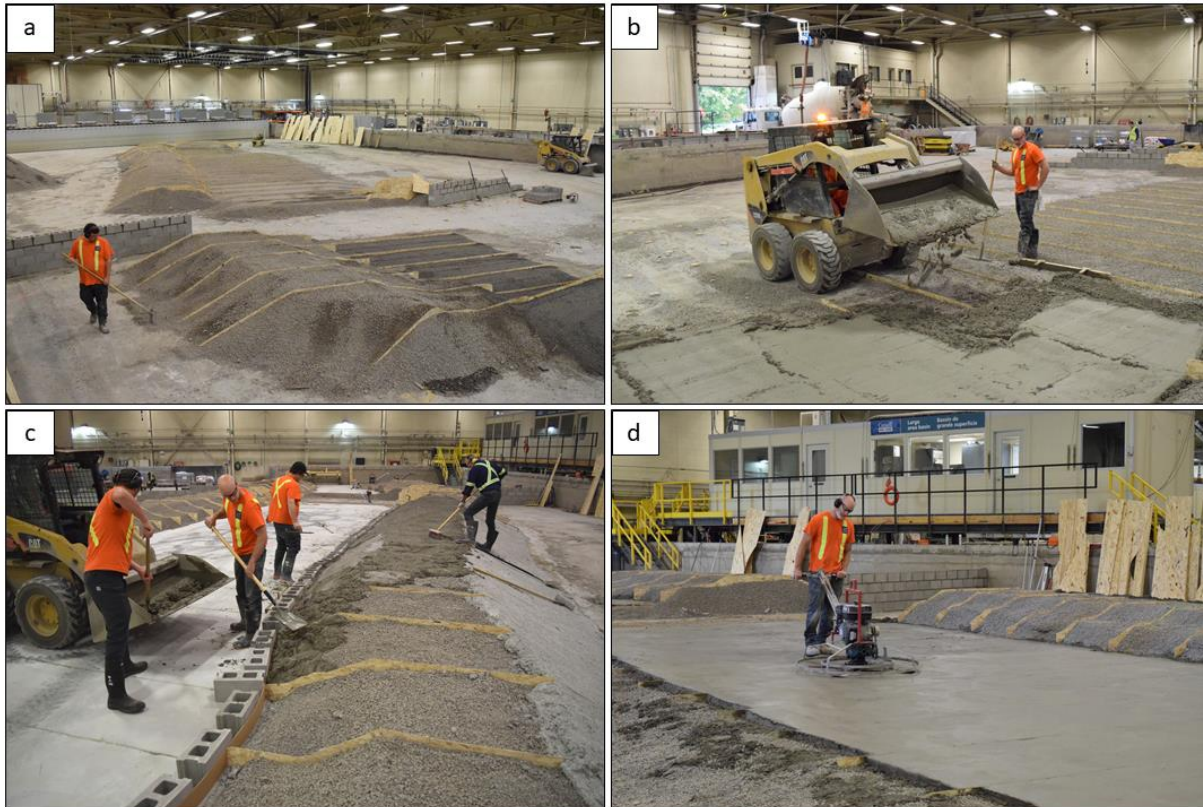


Figure 15. Various stages of the concrete bathymetry construction; a) shows OSB templates installed and backfilled with gravel, b) and c) show grout being placed on top of the gravel and template forms, c) also shows a broom finish being applied to the working platform and exposed portions of the river bed, bank, and floodplain spaces to achieve a rough finish, and d) shows the application of a mechanic trowel to achieve a smooth finish.

3.4.2 Mobile Bed – River Sediment, Riprap, Bioengineering Stone, and Transition Cobble

A total of five mobile beds were constructed and removed during the course of the test program (described in Section 4), one for each calibration test (Calibration #1 and #2), and one for each test series (Test Series A, B, and C). The mobile bed bathymetry was constructed using a series of fiberboard templates, installed at 1 m intervals (MS) throughout the test section. The template spacing for the mobile bed conformed to the template spacing for the concrete sections described in Section 3.4.1. The templates were used to represent cross-sections of the river bed. For the calibration tests, the templates were shaped to conform to 0.5 m (FS) contours derived from a 2010 river survey, provided by the City of Calgary (Golder Associates, 2011). For the conceptual design tests, the templates were shaped to conform to 0.5 m (FS) contours derived from the design drawings shown in Section 10 (Appendix A). The templates were cut to an accuracy of ± 2 mm (MS) and were supported on the concrete bed (described in Section 3.4.1) by a grid of pads levelled to within ± 1 mm (MS). Heavy construction equipment and hand tools (e.g. shovels) were used to fill the templates with river sediment once secured on top of the levelled pad. In areas where riprap, bioengineering stone, or transition cobble were prescribed, the templates were filled nearly to the top with river sediment, leaving sufficient space to apply a layer of the prescribed material. Otherwise, the templates were filled to the top with river sediment. Riprap was placed in a thin layer, approximately one particle size thick. For Test Series B and C, the voids of the riprap material were filled with river sediment (riprap voids were not filled for Calibration #1 and #2, and Test Series A). Bioengineering stone was placed in a thin

layer, approximately one to two particles thick. A thick layer of bioengineering stone (approximately 1 m thick full-scale) was placed where transition cobble was specified. The surface of the mobile bed was shaped using hand tools (e.g. rakes and hand trowels) to match the top elevation of the templates. After all of the mobile bed materials were placed and shaped, the fiberboard templates were carefully removed by pulling them straight up and out of the mobile bed. The gaps left behind by the templates were patched with additional material and smoothed with hand trowels. Photographs showing some of the stages of the mobile bed construction are shown in Figure 16.



Figure 16. Various stages of the mobile bed construction; a) shows pads installed and levelled on top of the concrete bed, b) shows fiberboard templates installed on top of the levelled pads and prepared for backfilling, c) shows an example of bioengineering stone being installed by hand near Poppy Plaza, and d) shows a portion of the final mobile bed for conceptual design #2 looking downstream from Louise Bridge; riprap material (with voids filled with river sediment) is visible on the edges of the main bar, bioengineering stone is visible on top of the main bar, and transition cobble is visible behind the right (south) pier of the LRT bridge.

3.4.3 Subsurface Clay Layer

The City of Calgary requested that additional subsurface armouring (i.e. riprap) be added along a portion of the south bank of the model to reflect a non-erodible clay layer that underlies the river sediment. This additional armouring is not shown in the design drawings in Section 10 (Appendix A). The request was based on borehole lithology data that indicate the presence of a clay layer that is exposed to the flow or is only thinly covered by river sediment in this location. Additional subsurface armouring was also requested in the vicinity of proposed design features along the south bank (e.g. groynes and revetments) where it is expected that the features will be deliberately and purposely connected to the clay layer during the construction phase. Elsewhere, the clay layer was considered to be covered by a thick layer of river

sediment (i.e. gravel) and was not reflected in the physical model. The extent and elevation of the clay layer was not firmly specified, but was determined based on guidance shown in the design drawings and through discussion with the City of Calgary and their consultants. A clay layer was not installed for the first calibration test (Calibration #1). Figure 17 shows the approximate location of the clay layers.

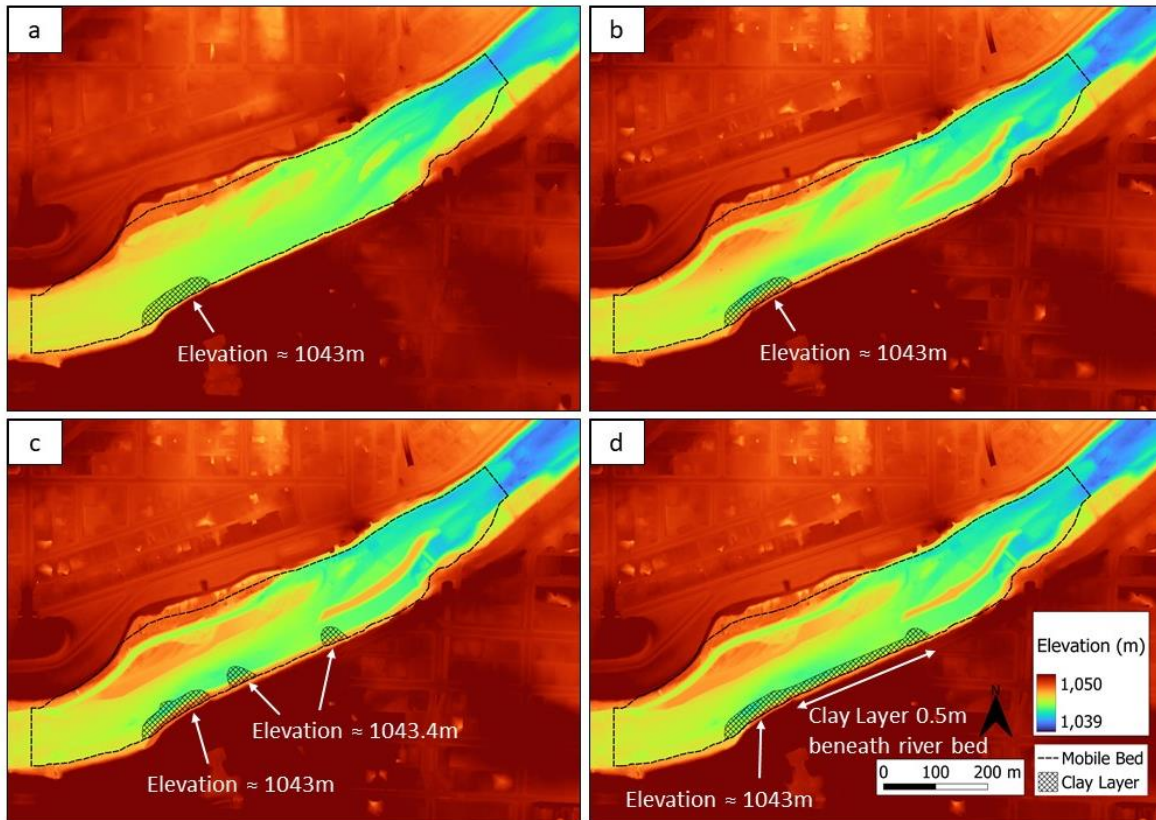


Figure 17. Model subsurface clay layers installed for a) Calibration #2 and the do-nothing test, b) Test Series A, c) Test Series B, and d) Test Series C. The bathymetry/topography data shown in a) corresponds to the pre-flood bathymetry/topography (Golder Associates, 2011) and the elevation data shown in b), c), and d) correspond to the design bathymetry/topography for the respective test series. The approximate elevation of the clay layer for each test is indicated.

3.4.4 Areas with Existing Vegetation

3.4.4.1 Calibration Tests

For the calibration tests, riprap armouring was installed in locations where the mobile bed of the model coincided with areas of established dense vegetation. It is important to note that the mobile bed was designed to be intentionally wide in some locations to accommodate potential modifications expected in the conceptual designs (e.g. portions of Poppy Plaza and the south bank downstream of the LRT bridge). These locations were included in the mobile bed so that they could be easily modified, as needed, for the subsequent conceptual design tests. However, it would have been unrealistic to represent these locations as loose river sediment during the calibration tests. In Calibration #1, riprap was placed to represent densely vegetated areas prior to the 2013 flood event. Noting that a large portion of the south bank downstream of the LRT bridge was eroded during the 2013 flood event, despite being densely vegetated, riprap placement was adjusted for Calibration #2 to allow this specific area to be more mobile. Figure 18 indicates the riprap placement for the calibration tests.

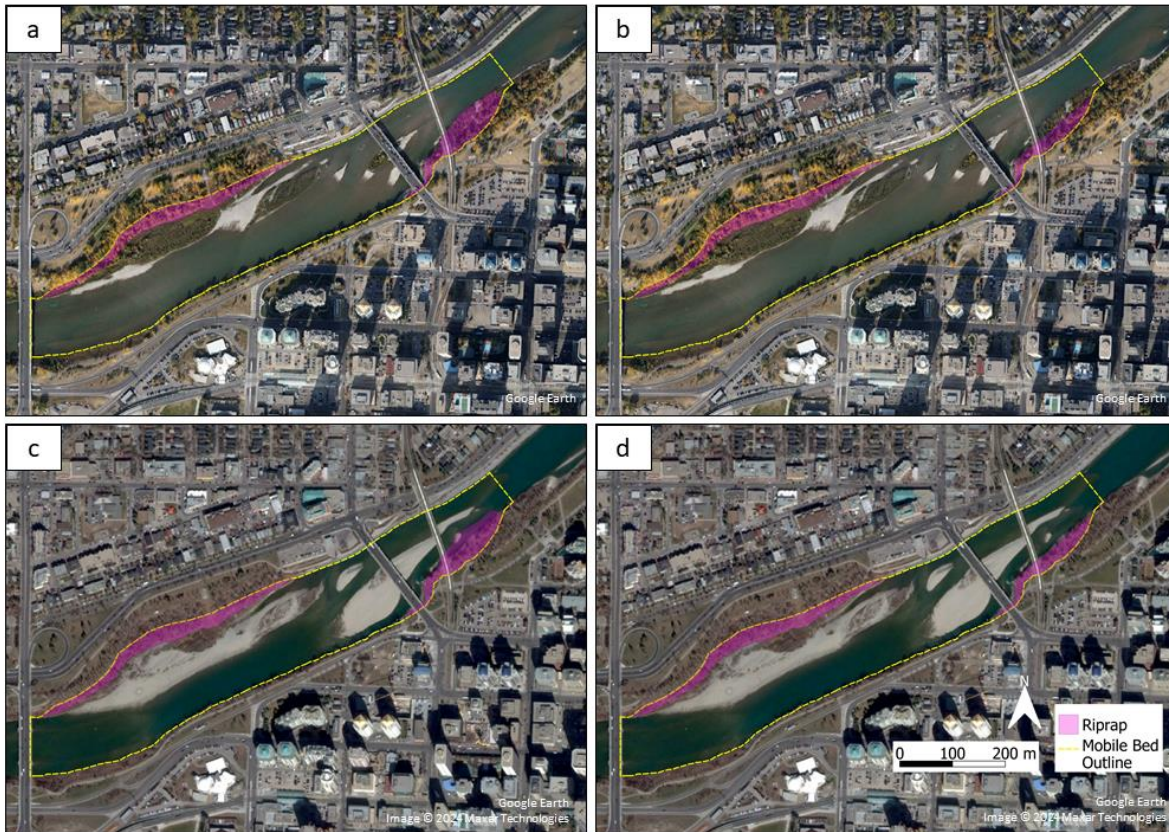


Figure 18. Riprap placement for the calibration tests. Figure a) indicates the riprap placement for Calibration #1 over a pre-flood basemap, b) indicates the riprap placement for Calibration #2 over a pre-flood basemap, c) indicates the riprap placement for Calibration #1 over a post-flood basemap, and d) indicates the riprap placement for Calibration #2 over a post-flood basemap. The pre- and post-flood basemaps are dated September, 2012 and April, 2015, respectively.

3.4.4.2 Conceptual Designs

Unless otherwise specified in the design drawings (and in addition to the bioengineering stone prescribed in the design drawings – Section 10), it was requested that bioengineering stone be placed at all locations in the mobile bed that are higher than the expected water level coinciding with a $200 \text{ m}^3/\text{s}$ flow, as it is expected that vegetation will become established in these locations. This flow approximately coincides with a water surface elevation of 1046.1 m near the upstream end of the model. This approach was adopted for Test Series A, B, and C, except for a small portion of the mobile bed for Test Series A along the south bank at the extreme downstream end of the model (i.e. the downstream end of the landing) which was represented as river sediment instead of bioengineering stone.

3.4.5 Bridges and Structures

3.4.5.1 Louise Bridge and the LRT Bridge

Louise Bridge and the LRT bridge were geometrically scaled based on information from engineering drawings and terrestrial LiDAR scans provided by the City of Calgary. The model bridge infrastructure was carefully surveyed during installation to ensure that the elevation, position, and orientation of the bridge features (e.g. the low chord elevation, pier locations, pier angles) conformed to the full-scale prototype.

For the Louise Bridge structure, the faces of the archways were constructed from rigid PVC plastic. The underside of the bridge (i.e. the arch barrel) was constructed from a flexible plastic material that could be contoured to conform to the shape of the archways. The deck of the bridge was constructed from $\frac{3}{4}$ " (MS) plywood. The sides of the piers were constructed from rigid PVC material. The noses and tails of the piers were fabricated from cedar posts that were professionally machined to conform to the shape of the full-scale prototype. The piers were fastened to a 1.5" (MS) marine plywood baseplate that was anchored to the LAB floor. Once constructed, the baseplate and pier bases were backfilled with gravel that was topped with concrete during the construction of the concrete floor described in Section 3.4.1. Figure 19 shows select construction photos of the Louise Bridge structure as well as the finished product.

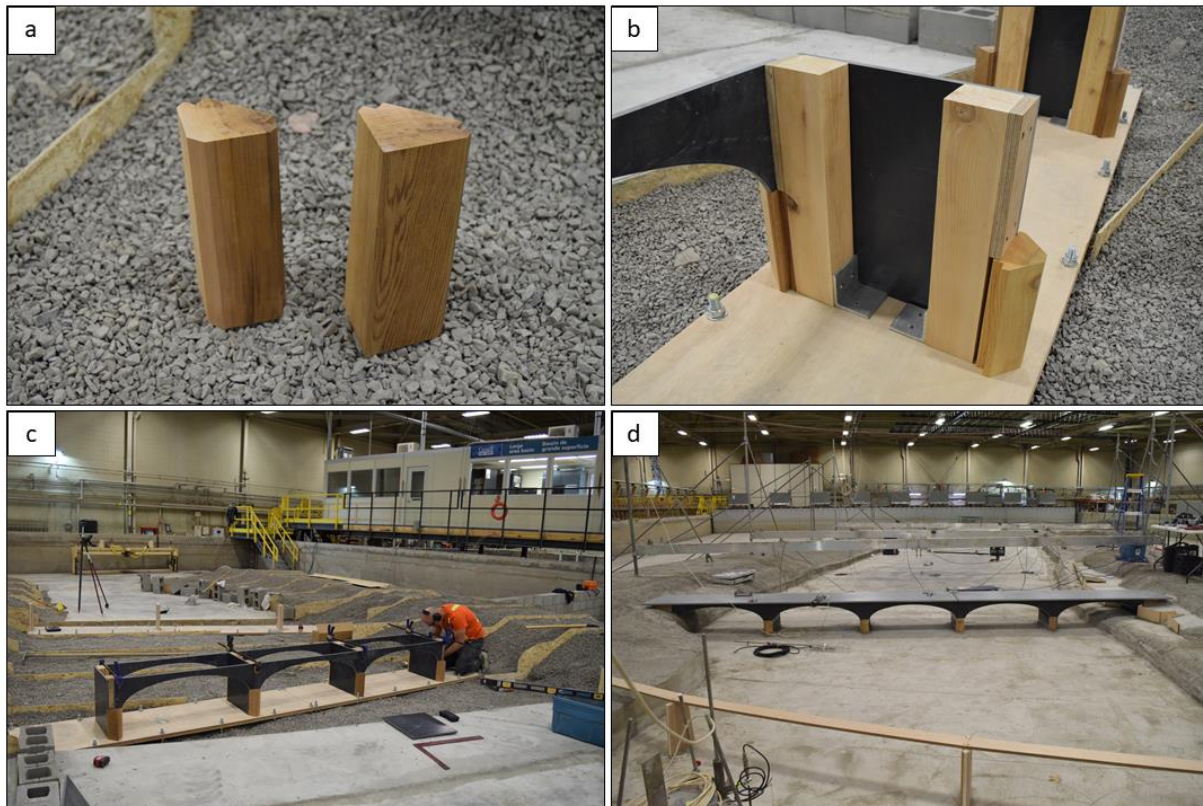


Figure 19. Louise Bridge construction; a) shows the machined nose and tail pieces of one of the piers, b) shows the composition of one of the piers, c) shows the installed piers and archway faces, and d) shows the finished Louise Bridge structure; the finished LRT bridge structure is also visible in the foreground.

The LRT bridge was constructed from both standard and marine plywood. The model structure did not include the upper deck (i.e. the bridge span that contains the LRT rail infrastructure); it was anticipated and agreed that the upper deck would not influence the flow hydraulics of the river within the range of tested flow conditions considered in the test program. The lower deck of the bridge (i.e. the pedestrian walkway) was constructed from $\frac{3}{4}$ " (MS) plywood. The piers were fabricated from laminated marine plywood boards that were professionally machined to conform to the shape of the full-scale prototype. The piers were fastened to a 1.5" (MS) marine plywood baseplate that was anchored to the LAB floor. Once constructed, the baseplate and pier bases were backfilled with gravel that was topped with concrete during the construction of the concrete floor described in Section 3.4.1. Figure 20 shows a construction photo of the LRT bridge structure as well as the finished product.

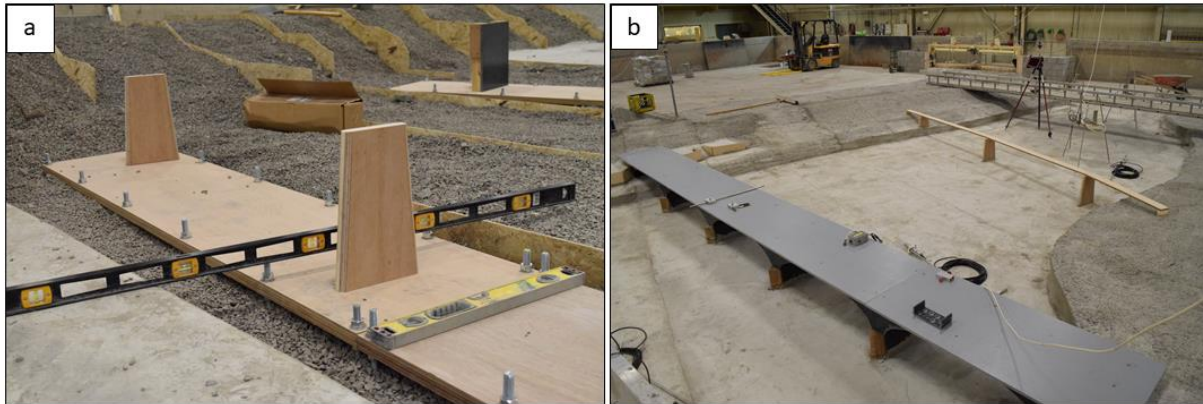


Figure 20. LRT Bridge construction; a) shows an example of the machined LRT bridge piers and the marine plywood baseplate, and b) shows the finished LRT bridge structure (background) and Louise Bridge structure (foreground).

3.4.5.2 Groynes and Recreational Wave Structures

Design #2 (i.e. Test Series B) included: three repelling groynes along the south embankment; one attracting groyne near the entrance of the habitat channel; and recreational wave infrastructure consisting of a concrete hump, a riprap apron downstream of the hump, and a riprap sill at the exit of the recreation channel. Design #3 (i.e. Test Series C) included a single repelling groyne along the south embankment and recreational wave infrastructure consisting of a concrete hump and a riprap apron downstream of the hump. For Design #2, the concrete hump protruded above the initial riverbed and had an elevation of 1043.75 m. For Design #3, the concrete hump was modified to be flush with the initial river bed elevation, equal to approximately 1042.5 m. Figure 21 shows select construction photos of the groynes and recreational wave infrastructure.

Groyne structures, the riprap apron, and the riprap sill were constructed entirely from riprap material. Groyne structures and the riprap sills were constructed using fiberboard templates, cut to scale. The templates were cut to an accuracy of ± 2 mm (MS) and were supported on the concrete floor by pads levelled to within ± 1 mm (MS). The spaces between the templates were filled with riprap material once secured on top of the levelled pads. The templates were removed prior to testing and the gaps left behind by the templates were patched with additional riprap material.

The concrete hump was constructed using fiberboard templates. The templates were cut to an accuracy of ± 2 mm (MS) and were supported on the concrete floor by pads levelled to within ± 1 mm (MS). The spaces between the templates were filled with clear gravel once secured on top of the levelled pads. The gravel was placed such that the final surface rested approximately 5 cm (MS) below the top edge of the templates. Grout was then placed on top of the gravel base and hand screeded to match the elevations of the templates. Once cured, hand trowelling was applied to the concrete to achieve a smooth finish. For Test Series C, the crest of the concrete hump constructed for Test Series B (elevation of 1043.75 m) was cut off and re-paved to conform to the new, lower elevation prescribed for Test Series C (elevation of approximately 1042.5 m).

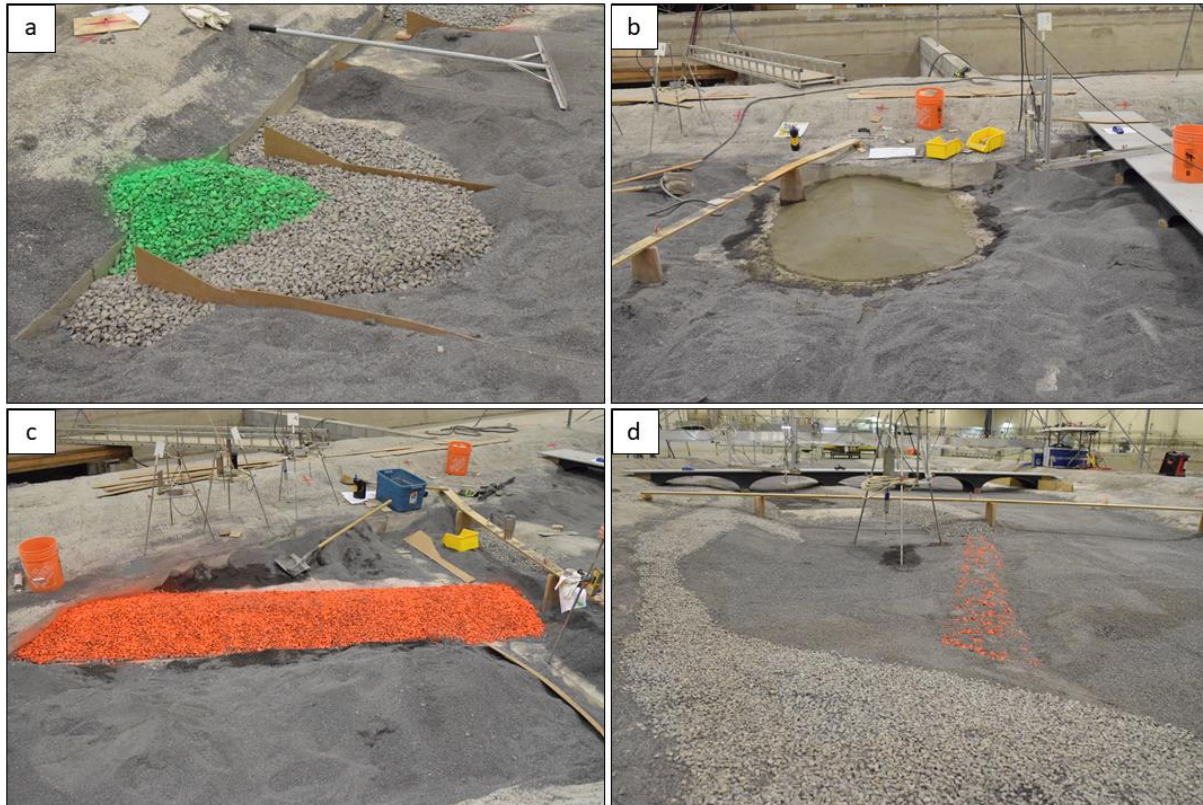


Figure 21. Example photos of other structures including a) construction of a repelling groyne (painted green) surrounded by a subsurface clay layer (represented as riprap, unpainted), b) construction of the recreational wave hump for conceptual design #2, c) construction of the riprap sill (painted orange) at the exit of the recreation channel for conceptual design #2, and d) the completed recreational wave hump (background) and sill (foreground) for conceptual design #2.

3.4.6 Water Supply and Water Level Control

A schematic showing the flow through the model is shown in Figure 12. The flow was supplied from the underground water storage reservoir (sump) and pumped into the HDF using a variable pitch pump. The flow was controlled by adjusting the vane pitch on the pump control panel. The HDF was connected to the LAB. Masonry walls were constructed to route the inflow from the HDF through the Bow River model reach (i.e. so that flows would not bypass around the outside of the model reach). The flow from the HDF entered the LAB and flooded the area upstream of the model. Once the volume of water was high enough, the upstream end of the river began to flood and the flow travelled down through the model reach. Upon exiting the model reach, flow entered a settling pool before flowing over the control weir. The tailwater elevation (at the downstream end of the model reach) was controlled by adjusting the elevation of the weir. The control weir is shown in Figure 22. Flow spilling over the weir was returned to the water reservoir sump and recirculated back to the basin inflow via the HDF.



Figure 22. Flow spilling over the control weir into the reservoir sump (photo taken from above).

3.4.7 Sediment Supply

Motorized spreaders were used to introduce sediment at the upstream boundary of the model in accordance with sediment loads estimated by the University of Ottawa (see Section 4.1). Three SnowEx SD-1400 spreaders were installed near the upstream boundary and filled with scaled river sediment material. The spreaders were mounted on steel support frames and powered by a custom-designed battery system. The physical properties of the scaled river sediment material were somewhat sensitive to moisture conditions, which could not be easily controlled in the laboratory. Consequently, a calibrated and reliable relationship could not be developed between the motor dial settings and the delivery rate of the river sediment. In lieu of calibrated dial settings, prescribed sediment loads were achieved by filling the spreaders with a known volume of river sediment before each test and carefully monitoring the spreaders to ensure that sediment was introduced into the flow in a consistent manner (i.e. ensure that 25% of the target sediment load was released after 25% of the test duration had passed, ensure that 50% of the target sediment load was released after 50% of the test duration had passed, and so on). Dial settings were adjusted, as needed, during each test and a built-in vibration feature was used occasionally. Lateral and temporal irregularity of sediment delivery may have introduced a model effect in the resulting bed morphology near the upstream boundary. Figure 23 shows the sediment spreaders positioned at the upstream boundary of the model reach. Sediment exiting the model reach at the downstream boundary was allowed to accumulate on the basin floor outside of the model reach (which was lower in elevation than the concrete floor of the model). The accumulated sediment on the basin floor did not appear to influence flows or water levels in the model reach. The accumulated sediment downstream of the model reach was cleared after each test.



Figure 23 Sediment spreaders installed at the upstream boundary of the model.

3.5 Instrumentation Systems and Data Analysis

The model was equipped with instruments for measuring water levels, velocities, and geomorphic change. Water level and velocity gauges were deployed throughout the model domain. The horizontal positions of the water level and velocity instruments were fixed for the duration of the calibration tests and also for the duration of the conceptual design tests; some minor adjustments to the horizontal positions of the instruments were made between the do-nothing test (which immediately followed Calibration #2) and Test Series A so that instrumentation was better aligned with the geometry of the conceptual designs. The vertical positions of the instruments (i.e. the elevations) were adjusted depending on the elevation of the initial bathymetry for each test series. A FARO Focus 3D laser scanner was used to measure the bathymetry of the model and record geomorphic change. Photographs and video were recorded to document each test series, including the construction process, the flow tests, and the resulting bathymetry. Figure 24 shows the instrumentation layout for the calibration tests and the do-nothing test. Figure 25 shows the instrumentation layout for the conceptual design tests.

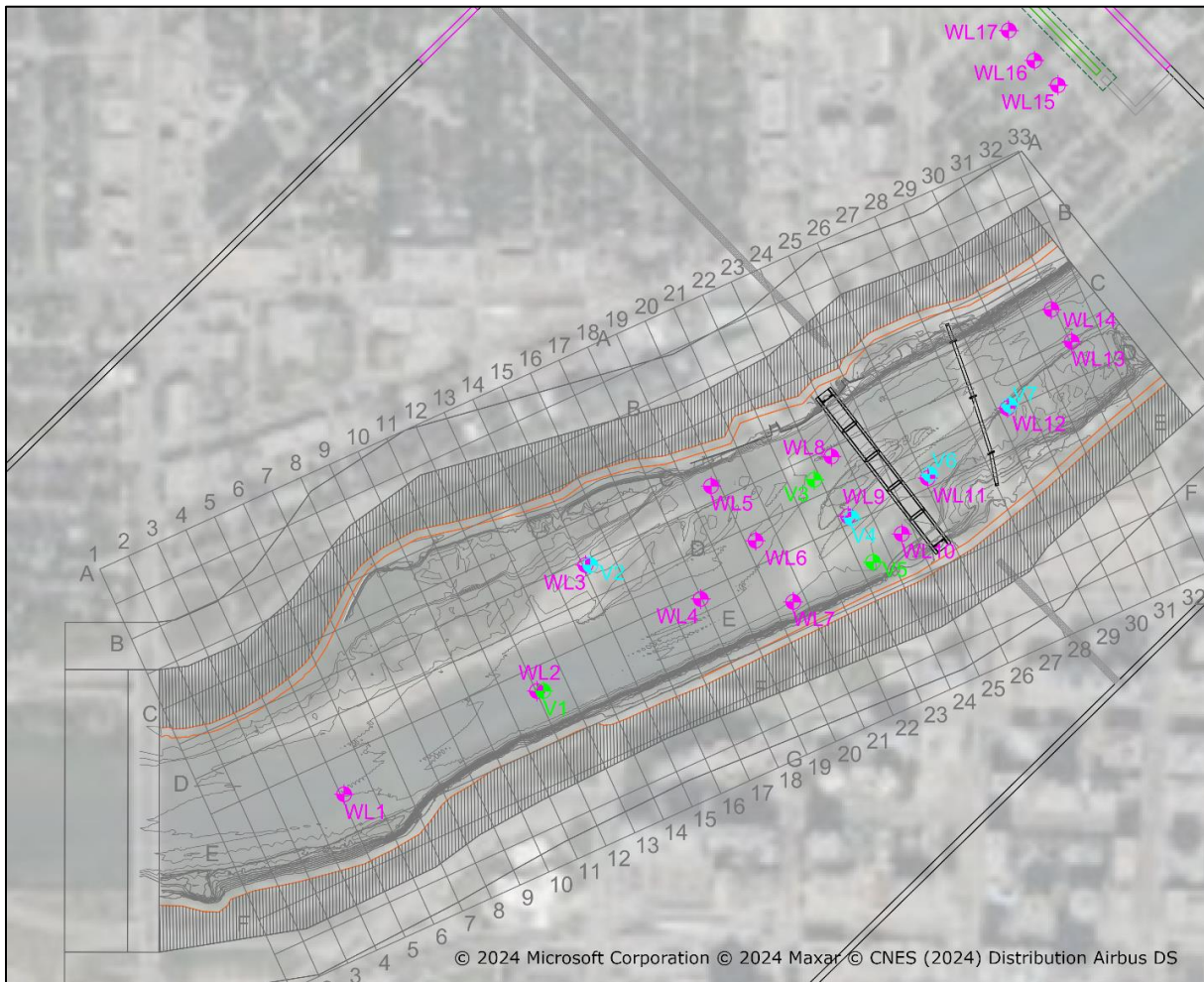


Figure 24. Instrumentation layout for the calibration tests and the do-nothing test plotted over the pre-flood bathymetry contours.

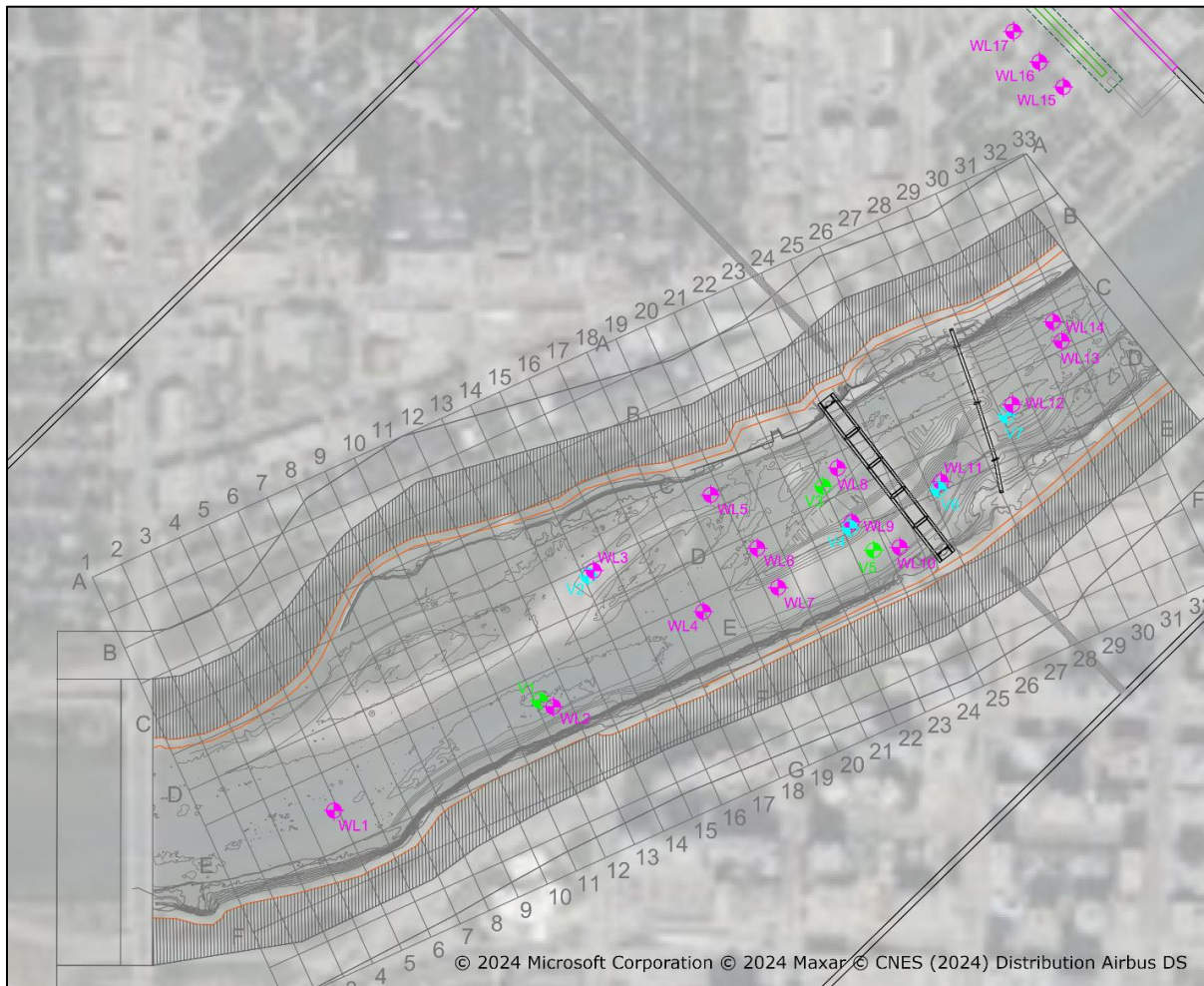


Figure 25. instrumentation layout for the conceptual design tests plotted over the bathymetry contours for conceptual design #1.

A data acquisition system converted analog voltage signals from the water level and velocity sensors deployed in the model into digital form for storage in a computer data file. Analog signals from each sensor were routed to a set of signal condition units equipped with separate amplifiers and anti-aliasing filters for each channel. From the signal conditions units, the analog signals were routed through BNC Interface units to data acquisition cards housed inside a high-speed PC. The data acquisition system was controlled with GDAC software developed at NRC-OCRE (Pelletier, 1997). GDAC creates a single binary data file for each test (*.DAQ) containing the raw data for all channels (in volts) in a multiplexed format, together with the calibration constants required to convert the raw data into meaningful engineering units. The raw data can be de-multiplexed and converted to either full-scale or model-scale engineering units for subsequent analyses.

NRC-OCRE's GEDAP software was used for data analysis of water levels, velocities, and flows. GEDAP is a general-purpose software system for the synthesis, analysis and management of laboratory data that also includes modules for real-time experiment control and data acquisition functions. Standard GEDAP time-domain analysis algorithms were applied to analyse in considerable detail the water levels, velocities and flows measured in the model.

The following parameters (among others) were obtained from time-domain analysis of the measured water level and velocity records:

Mean (μ) – average of the recorded data (water levels, velocities or flows);

Min – minimum of the recorded data (water levels, velocities or flows);

Max – maximum of the recorded data (water levels, velocities or flows);

Std Dev (σ) – standard deviation of the recorded data (water levels, velocities or flows).

Foot access on the mobile bed was restricted during each test series to minimize disturbance of the mobile bed and to facilitate high-quality observations of geomorphic change. Foot access was not permitted during each flow test, nor during breaks between each flow test within a given test series. Therefore, water level capacitance gauges (Section 3.5.1) and velocity instrumentation (Section 3.5.2) could not be adjusted from the original positioning applied over the initial bed of each test series regardless of changes in the bathymetry or flow hydraulics during the test series. The fixed vertical positions of the water level capacitance gauges and velocity instrumentation were selected as a compromise to accommodate and prioritize the water depths anticipated during the larger flow conditions, while still providing some information during the smaller flow conditions. However, some challenges arose owing to the inability to adjust the instrumentation, the uncertainty of anticipating geomorphic change and flow depth, and the compromises required to provide information over the range of tested flow conditions. In some cases, water level capacitance gauges or velocity instrumentation were not operational (not submerged in the flow) during smaller flow conditions, or became very close to, or even buried in, river sediment as testing progressed. The sensors of the velocity instrumentation were positioned to measure approximately 5 cm (MS) above the initial bed of each test.

3.5.1 Water Levels

Two different types of instruments were used to measure water levels in the model, nine capacitance-type water level gauges developed by Akamina Technologies and eight UltraLab USS 13-HF ultrasonic sensors developed by General Acoustic (Figure 26). The water level sensors were mounted at specific locations in the model using a series of beams, cantilevered arms, and tripod stands.

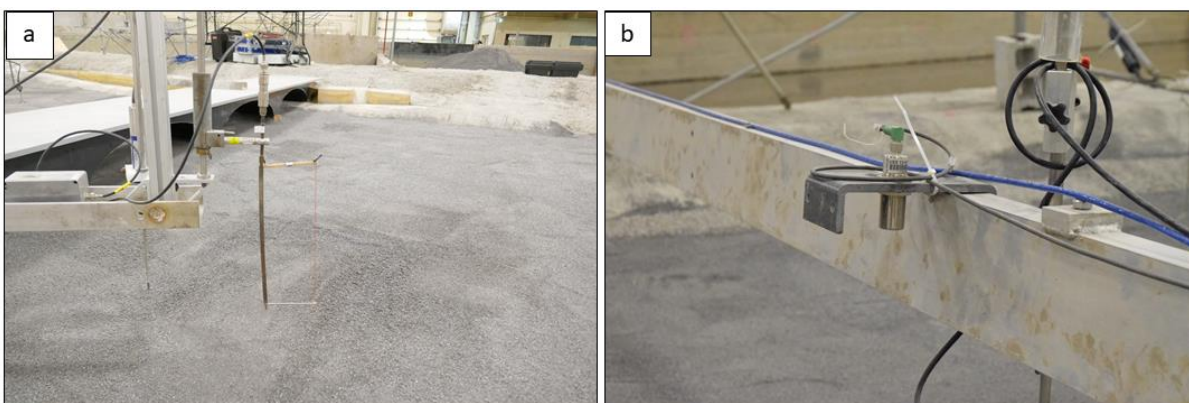


Figure 26. Example of water level instrumentation including a) a capacitance water level gauge and b) an ultrasonic acoustic sensor.

The capacitance gauges operate by sensing the change in capacitance that occurs as a portion of the insulated wire becomes submerged. The signal is directly proportional to the percentage of the wire that is wetted, regardless of whether the wetting is continuous or intermittent (as in the case of spray). This type of gauge has been shown to exhibit a highly linear and stable relationship between water level and voltage. They typically feature calibration errors of less than 0.5% over a 200 mm (MS) calibration range. This error represents an accuracy of ± 1.0 mm (MS) or ± 30 mm (FS). The capacitance water level probes were calibrated before each test series by changing their elevation with respect to a fixed water level. The calibration constants remained stable throughout the test program.

The USS 13-HF ultrasonic sensors operate by emitting an acoustic ping which is reflected by the water surface back to the sensor where the echo is received (General Acoustics, n.d.). The digital signal is proportional to the distance between the sensor and the water surface. The system features an internal measurement system that monitors local humidity and temperature, and adjusts signal interpretation accordingly to accommodate impacts on the speed of sound. The sensors feature a narrow beam angle permitting an obtainable spatial resolution of 0.18 mm at model-scale (5.4 mm FS), and have an assured accuracy of 1 mm at model-scale (30 mm FS), although the accuracy for most common applications will be substantially better. The sensors have a working range between 200 and 1200 mm (MS).

Four to six water level measurements were recorded during each 30-hour test (see Section 4.2). Each measurement involved triggering the water level instrumentation and allowing them to record for a period of 1643 seconds (equal to 300 seconds or 5 minutes, MS). Figure 27 shows an example of the water level plots generated from the study. It shows the water level time histories measured at seven water level gauges and lists some key derived parameters describing the water levels at each location. All of the water level data collected during the study were analyzed and plotted in this manner.

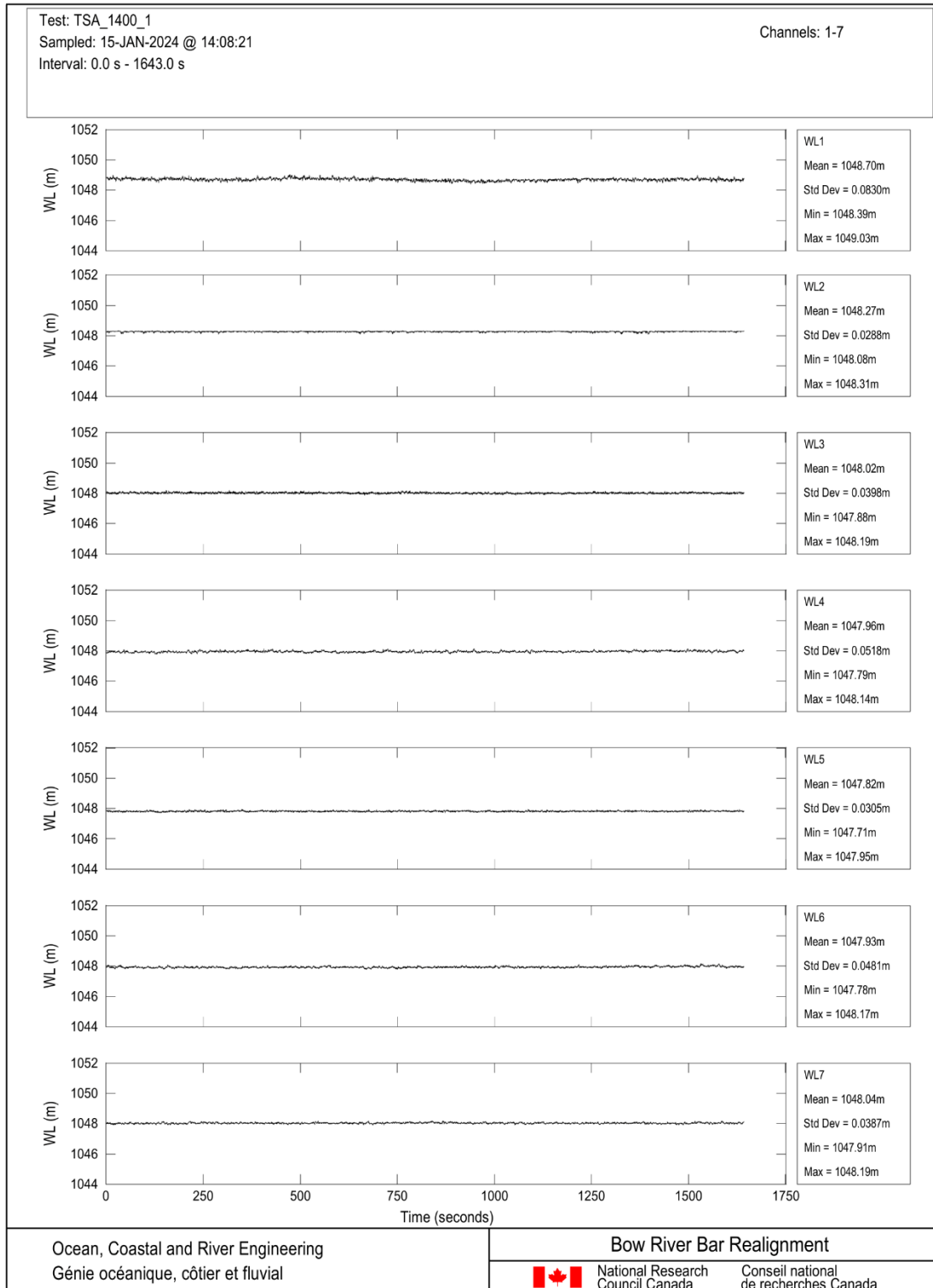


Figure 27. Example water level analysis plot from Test Series A.

3.5.2 Velocities

Two different types of instruments were used to measure velocities in the model, three 3-axis acoustic Doppler velocimeters (ADV) developed by Nortek and two 2-axis electromagnetic current meters (ECMs) developed by Alec Technologies (Figure 28). The velocity instruments were mounted at specific locations in the model using a series of beams, cantilevered arms, and tripod stands. The ADV and ECM sensors were positioned to measure approximately 5 cm model-scale (1.5 m FS) above the initial bed for each test series. The x-axes and y-axes of the instruments were oriented toward the east and north wall of the LAB, respectively. The local coordinate system is shown in Figure 12.

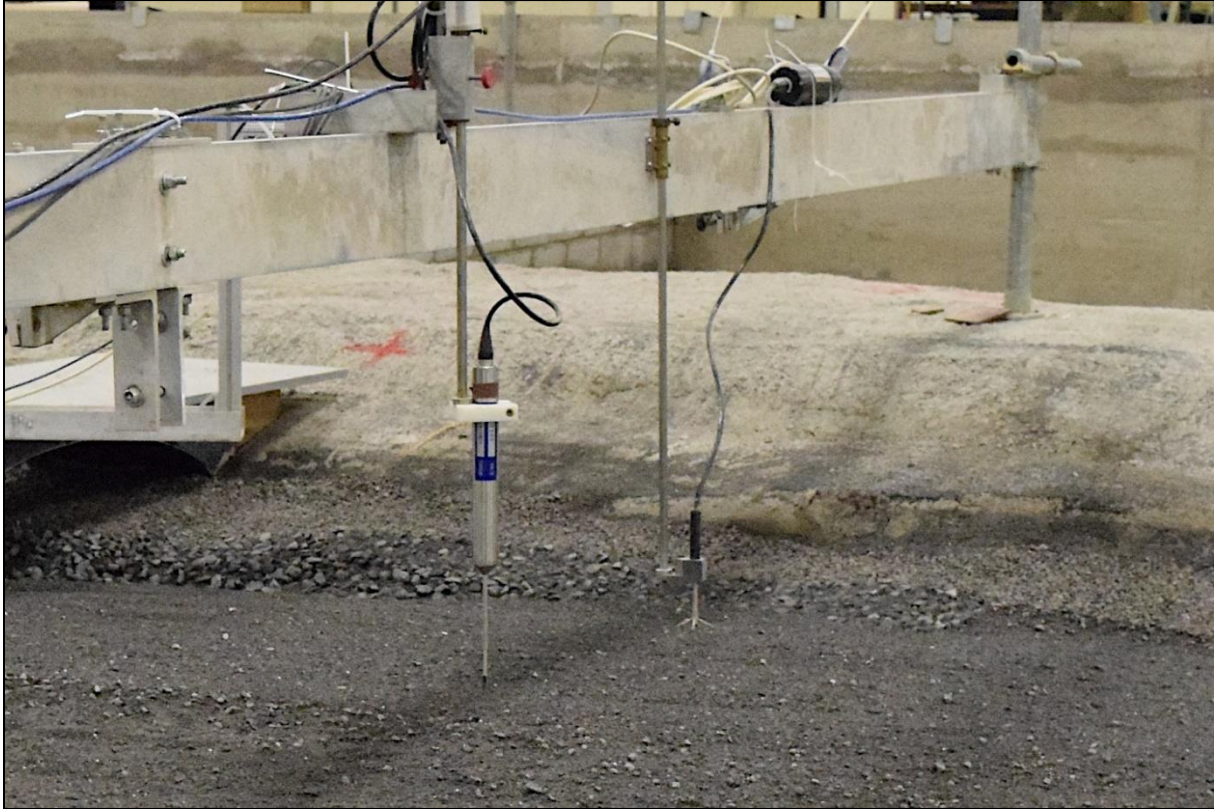


Figure 28. Example of velocity instrumentation including an electromagnetic current meter (left) and an acoustic Doppler current profiler (right).

The velocity vector at a point in space can be defined as $U(t) = (u, v, w)$ where $u(t)$, $v(t)$, and $w(t)$ are the components parallel to the x-, y- and z-axes, respectively. For this study, the reported x and y velocity data (see Appendix B) can be translated to easting and northing velocity components at the study site using the following equations

$$V_E = u \cos 44.39^\circ - v \sin 44.39^\circ \quad (6)$$

$$V_N = u \sin 44.39^\circ + v \cos 44.39^\circ \quad (7)$$

where V_E represents the easting velocity at the study site, V_N represents the northing velocity at the study site, u represents the velocity in the laboratory x direction, and v represents the velocity in the laboratory y direction.

The velocity measurements obtained in this study were analyzed with the help of GEDAP programs and procedures. The various steps in the analysis were as follows:

- Convert the measured velocities into the proper coordinate system
- Remove high-frequency noise using a digital low-pass filter with a cut off frequency of 3Hz full-scale
- Compute the magnitude of the horizontal velocity vector:

$$|U_h|(t) = \sqrt{u^2(t) + v^2(t)} \quad (8)$$

- Execute a statistical analysis program to compute key statistics for the time series data
- Compile statistics and parameters into an ASCII file
- Import the ASCII file into a spreadsheet program
- Generate plots to visually illustrate the velocity data for each test

Four to six velocity measurements were recorded during each 30-hour test (see Section 4.2). Each measurement involved triggering the velocity instrumentation and allowing them to record for a period of 1643 seconds (equal to 300 seconds or 5 minutes, MS). Figure 29 shows an example of the plotted velocity data.

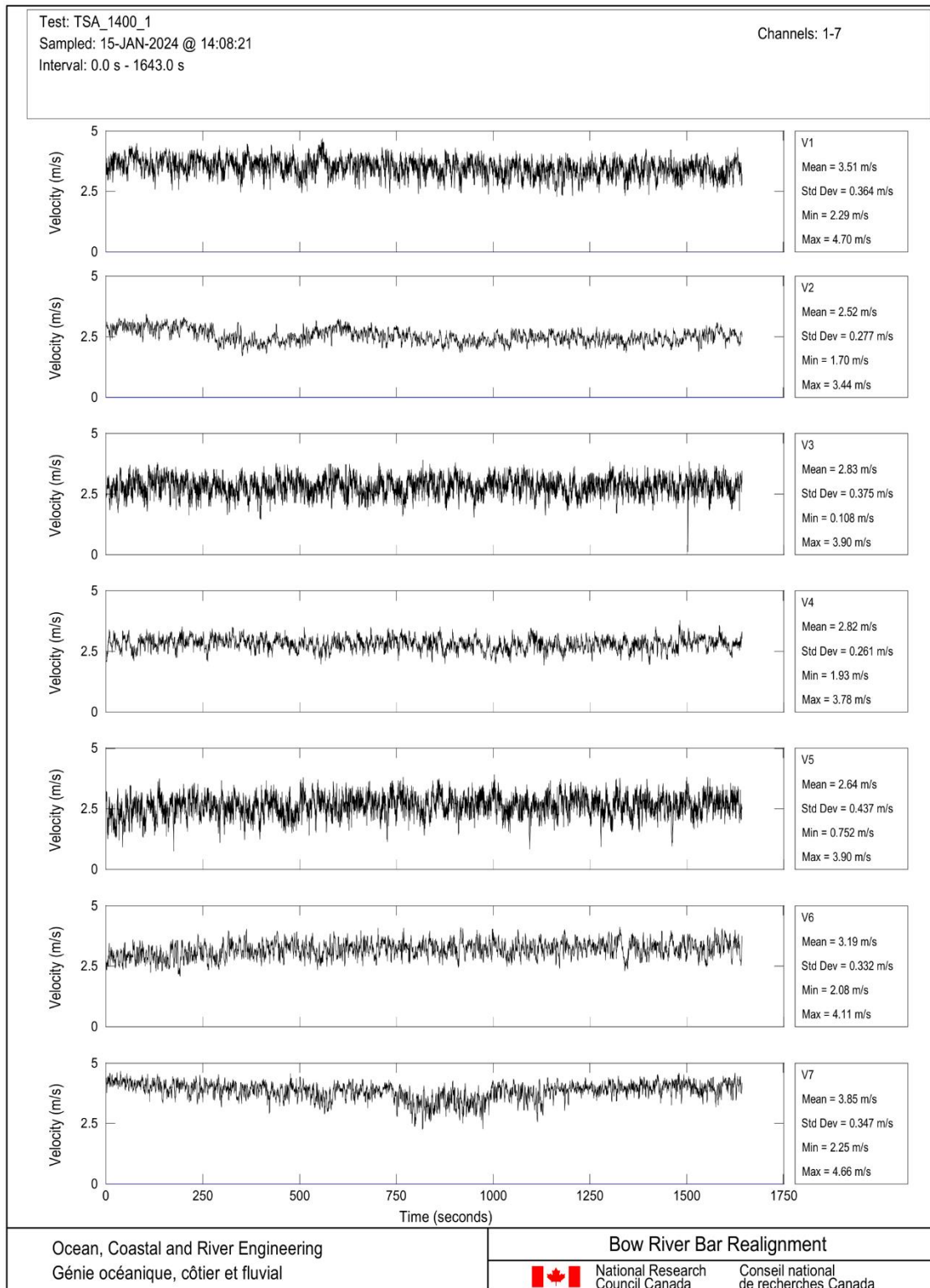


Figure 29. Example velocity analysis plot from Test Series A.

3.5.3 Discharge

The discharge through the model was measured using the three capacitance-type water level gauges positioned just upstream of the weir (see Section 3.4.6). The crest of the weir was adjusted during the test program to provide the various target tailwater elevations at the downstream boundary of the model reach specified by the University of Ottawa (for the calibration tests) or the City of Calgary (for the conceptual design tests). The discharge over the weir was estimated based on the elevation of the weir crest and the water level elevation at the weir using the sharp-crested weir equation:

$$Q = 1.84BH^{1.5} \quad (9)$$

where B is the width of the weir (109.76 m FS) and H is the height of water above the crest of the weir (as measured by the water level gauges).

3.5.4 Mobile Bed Changes

A FARO Focus 3D laser scanner (Figure 30) was used to survey the bathymetry of the model and record geomorphic change. For each test series, scans were taken before testing (i.e. the initial bed) and after each flow event, including scans of the final bed after all flow events had been run. For each bathymetry of interest, multiple scans were conducted at different locations around the model reach and were stitched together using SCENE software to compile a single, high-resolution point cloud. Each scan had a resolution (i.e. point spacing) of approximately 6.1 mm at 10 m from the scanner (MS). The point clouds were georeferenced and interpolated using Python programming to compile 1 m resolution (FS) digital elevation models (DEM). The resulting DEMs were used to interpret and analyze geomorphic change.



Figure 30. FARO Focus 3D laser scanner.

The model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition (Figure 31). The zone footprints were consistent for all of the tests and were selected to represent key features of the current-day study area and proposed design features, as follows:

- Zone 1 represents the upstream portion of Poppy Plaza and includes a large portion of the proposed habitat channel
- Zone 2 represents the upstream portion of the south embankment
- Zone 3 represents the downstream portion of Poppy Plaza and includes the downstream portion of the proposed habitat channel
- Zone 4 represents a portion of the south embankment upstream of the main bar
- Zone 5 represents the area north of the main bar, upstream of Louise Bridge
- Zone 6 represents the area south of the main bar, upstream of Louise Bridge
- Zone 7 represents the northern portion of the channel downstream of Louise Bridge
- Zone 8 represents the southern portion of the channel downstream of Louise Bridge and includes a portion of the proposed recreation channel adjacent to the landing



Figure 31. Zones used to assist with the interpretation of erosion and deposition, plotted over a post-flood satellite imagery basemap.

Bathymetry profiles were extracted at three cross-sections to provide a profile-view perspective of bathymetric change for the do-nothing test and the conceptual design tests. Figure 32 indicates the locations of the cross-sections.

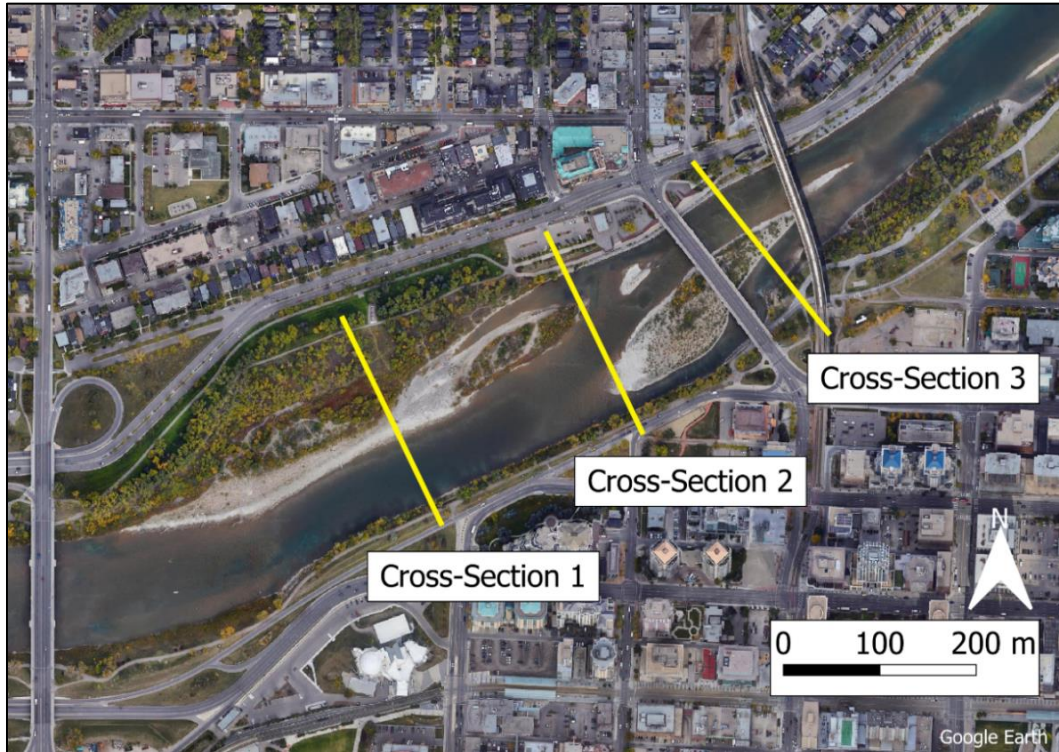


Figure 32. Cross-Sections for bathymetry profiles, plotted over a post-flood satellite imagery basemap.

3.5.5 Video and Photography

Digital photographs were taken to document all stages of the model study including construction and testing. Videos were recorded during testing as well. Photographs and videos were shared with the City of Calgary, the University of Ottawa, and the City of Calgary's consultants during the study to share progress and facilitate discussion.

4 Test Program

The test program for the Bow River physical model included 17 unique tests divided into six distinct test series (TS). Testing began on November 28, 2023 and concluded on February 29, 2024. During each test series, the model flow conditions were set at the pump, the sediment spreaders were prepared to deliver the specified sediment input rate, and the downstream weir elevation was set to produce the proper tailwater elevations. Each combination of flow and tailwater setting was assigned a unique identifier (e.g. TSA_850). A description of each test series is presented in Table 7.

Table 7. Description of test series.

Test Series	Instrumentation Setup	Description
CAL1	I-A	Calibration test focused on recreating the flow conditions and geomorphic change observed during the 2013 flood event
CAL2	I-A	Calibration test focused on recreating the flow conditions and geomorphic change observed during the 2013 flood event
Do_Nothing	I-B	Test the resulting bed after CAL2, subjected to a 100-year flow condition
TSA	I-B	Test the performance of conceptual design #1 under a 10-, 35-, and 100-year flow condition
TSB	I-B	Test the performance of conceptual design #2 under a 10-, 35-, and 100-year flow condition
TSC	I-B	Test the performance of conceptual design #3 under a 10-, 35-, and 100-year flow condition

4.1 Initial Bathymetry and Boundary Conditions

4.1.1 Calibration tests

The flow and water level conditions for the calibration tests were based on a simplified hydrograph of the 2013 flood event, developed by the University of Ottawa. The morphodynamic model developed by Yu et al. (2022) was employed to evaluate two simplified three-stage hydrographs. The three stages of each hydrograph were designed to represent the rising, peak, and falling stages of the 2013 flood event. To support operational feasibility for the physical model, each simplified hydrograph was designed such that each stage could be completed in approximately 5.5 hours (MS), which is the maximum amount of testing time possible in a single work day, and a full-hydrograph could be completed within three testing days. One simplified hydrograph (H1) was designed to preserve the total cumulative flood discharge of the event as accurately as possible within the condensed testing window, but did not accurately represent the flow discharge of each individual flood stage. The other simplified hydrograph (H2) was designed to accurately represent the flow discharge for each stage, but did not preserve to total cumulative discharge of the event. The details of the two simplified hydrographs are summarized in Table 8 and the hydrographs are plotted in Figure 33.

Table 8. Details of the simplified hydrographs considered for the calibration tests.

Hydrograph	Hydrograph Details						Total duration (hour)	Cumulative discharge (m³/s * h)
2013 flood hydrograph (unsteady)							204	183304.6
H1	Stage	Rising	Transition	Peak	Transition	Falling	92	135075.0
	Discharge (m³/s)	1150	unsteady	1750	unsteady	1500		
	Duration (hour)	30	1	30	1	30		
H2	Stage	Rising	Transition	Peak	Transition	Falling	92	97560.0
	Discharge (m³/s)	568	unsteady	1750	unsteady	852		
	Duration (hour)	30	1	30	1	30		

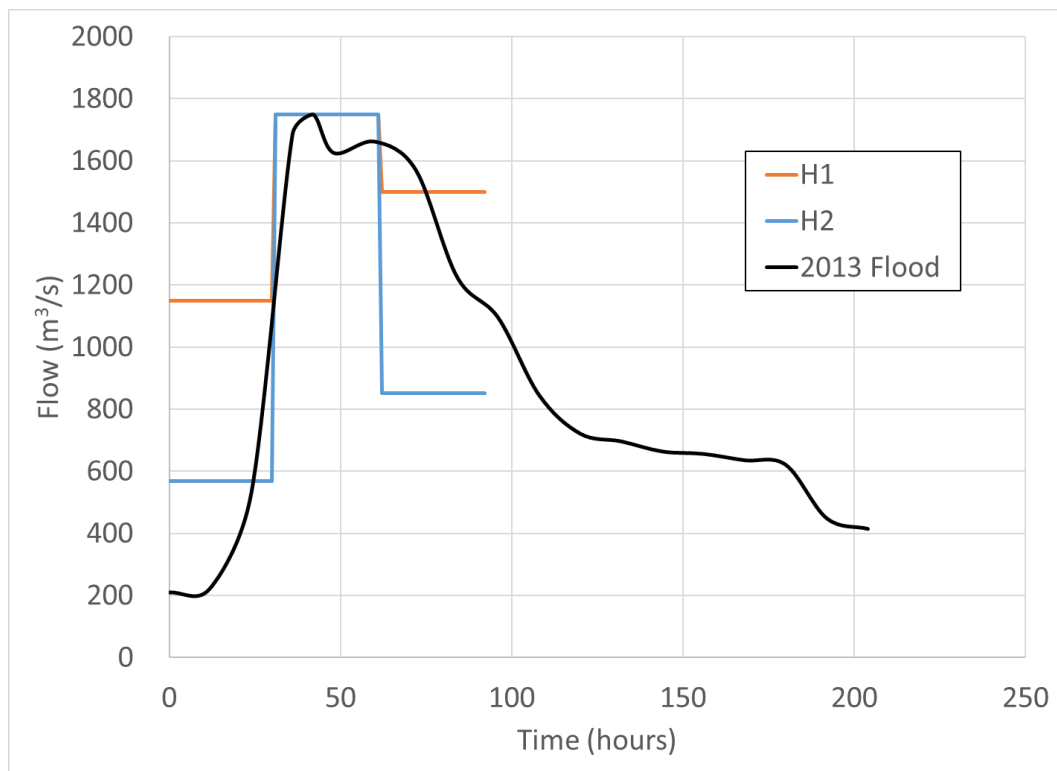


Figure 33. Simplified three-stage hydrographs, H1 and H2, plotted alongside the actual 2013 flood hydrograph.

Each simplified hydrograph was simulated using the numerical model described in Yu et al. (2022) and was initiated from the pre-flood condition. Other model parameters, such as roughness and sediment sizes, were directly adopted from the calibrated and validated numerical model. Bedload transport data were not available for the 2013 flood event. Therefore, a Neumann boundary condition (Gustafson & Abe, 1998) was applied at the numerical model boundaries to set the input/output bedload transport rate. This condition specifies that the bedload entering or leaving the model boundaries will be near-perfectly adapted to the local flow conditions (Deltares, 2020), which is mostly the case in the Bow River based on the historical gauged data from the large flood events from the year 1972 to 1974 (Water Survey of Canada, 2013). In this way, the input bedload rates for three flood stages were determined.

The modelled post-flood bathymetry (from the numerical model) was compared with the measured 2013 bathymetry from Golder Associates (2015). The mean absolute error was computed between the modelled bathymetry for each simplified hydrograph and the measured post-flood bathymetry. The mean absolute error for H1 and H2 was equal to 0.57 m and 0.54 m, respectively. Therefore, H2 was selected to inform the boundary conditions for the physical model calibration tests, including the upstream flow condition, the downstream water level condition, and the sediment input condition.

For the physical model, the initial bathymetry for the calibration tests was based on the 2010 river survey, provided by the City of Calgary (Golder Associates, 2011). In addition to hydrograph H2 shown in Figure 33, Calibration #2 was run for an additional 30 hours (5.5 hours MS) at the 850 m³/s flow condition to evaluate the potential impact of a longer hydrograph tail on the bathymetry.

4.1.2 Conceptual Design Tests

The flow and water level conditions for the do-nothing test and the conceptual design tests (TSA, TSB, and TSC) were provided by the City of Calgary and correspond to the values shown in the Government of Alberta's Flood Awareness Map Application (Government of Alberta, n.d.). The tested flows and water levels reflected estimates for the 10-year, 35-year, and 100-year flow conditions (approximately equal to 850, 1400, and 2100 m³/s at full-scale, respectively). The sediment input rates for each flow condition were provided by the University of Ottawa based on results from the numerical model described in Section 4.1.1.

For the do-nothing test, it was determined that the resulting bathymetry from Calibration #2 (CAL2) was generally representative of the current-day bathymetry, providing an opportunity to understand how the existing study site might respond to a major flood event without any intervention to realign the gravel bars. Therefore, for the do-nothing test, the resulting bathymetry from Calibration #2 was modified slightly and then subjected to a 100-year flow condition for 30 hours (5.5 hours MS). The modification involved digging a side channel that was approximately 1.0 to 1.5 m deep and approximately 6 m wide along the south bank near Louise Bridge to better represent the actual bathymetry in this location (Figure 34). The length of the constructed side channel was approximately 180 m, beginning approximately 120 m upstream of Louise Bridge. The side channel extended downstream, underneath of Louise Bridge, toward the landing. This side channel, which is present in reality, was not naturally formed during Calibration #1 or Calibration #2. However, this discrepancy may be explained by potential interpolation errors present in the initial bathymetry. A detailed assessment of the discrepancy is provided in Section 5.1.



Figure 34. Bathymetry modification for the do-nothing test.

For the conceptual design tests, the initial bathymetry conformed to the design drawings described in Section 2.3 and shown in Section 10 (Appendix A). Each design was subjected to a 10-, 35-, and 100-year flow condition, in that order. Each flow condition was tested for a duration of 30 hours (5.5. hours MS). The mobile bed was not reset between flow conditions (i.e. the resulting bathymetry after the 10-year flow condition represented the starting bathymetry for the 35-year flow condition, and the resulting bathymetry after the 35-year flow condition represented the starting bathymetry for the 100-year flow condition). The model was drained between each flow condition so that the bathymetry could be scanned. After the scanning was completed, the model was refilled to prepare for the next flow condition. Draining and filling between tested flow conditions was conducted slowly and carefully to minimize artificial disturbance of the bathymetry.

4.2 Test Program Summary

Table 9 summarizes the test program for the physical model including target and actual boundary condition parameters for flow and tailwater (TW) elevation.

Table 9. Test program summary.

Test ID	TW Estimate (Government of Alberta, n.d.)	TW Estimate [Yu et al. (2022) Model]	Target TW	Target Flow (Government of Alberta, n.d.)	Target Sediment Rate [Yu et al. (2022) Model]	Duration	Actual Flow	Actual TW
Units	(m)	(m)	(m)	(m ³ /s)	(kg/s)	hours	(m ³ /s)	(m)
CAL1_600	-	1045.58	1045.58	600	103.92	30	534.93	1045.58
CAL1_1750	-	1047.74	1047.74	1750	453.55	30	1834.42	1048.14
CAL1_850	-	1046.09	1046.09	850	87.00	30	824.18	1046.53
CAL2_600	-	1045.58	1045.58	600	103.92	30	602.92	1046.05
CAL2_1750	-	1047.74	1047.74	1750	453.55	30	1789.65	1048.13
CAL2_850	-	1046.09	1046.09	850	87.00	30	920.86	1046.58
CAL2_850b	-	1046.09	1046.09	850	87.00	30	884.59	1046.50
Do_Nothing	-	1047.74	1047.74	2100	390.72	30	2100.97	1048.69
TSA_850	1045.83	1046.28	1045.83	850	87.00	30	863.82	1046.30
TSA_1400	1046.91	1047.07	1046.91	1400	142.22	30	1399.77	1047.30
TSA_2100	1047.74	1047.92	1047.74	2100	390.72	30	2175.61	1048.61
TSB_850	1045.83	1046.28	1045.83	850	87.00	30	865.93	1046.03
TSB_1400	1046.91	1047.07	1046.91	1400	142.22	30	1470.83	1047.21
TSB_2100	1047.74	1047.92	1047.74	2100	390.72	30	2086.20	1048.25
TSC_850	1045.83	1046.28	1045.83	850	87.00	30	871.51	1046.33
TSC_1400	1046.91	1047.07	1046.91	1400	142.22	30	1472.82	1047.20
TSC_2100	1047.74	1047.92	1047.74	2100	390.72	30	2125.53	1048.12

5 Results and Discussion

The following section summarizes the results of the test program. A complete summary of the results is available in the electronic archive (Appendix B) which includes;

- Measured water levels and velocities
- Measured bathymetry data (i.e. digital elevation files)
- Photographs and videos

5.1 Calibration #1

Calibration #1 (CAL1) was focused on recreating the flow conditions and geomorphic change observed during the 2013 flood event. A scaled representation of the pre-2013 flood bathymetry (Golder Associates, 2011) was constructed in the LAB and was subjected to the simplified 2013 flood hydrograph described in Section 4.1.1. The skill of the physical model was assessed by comparing the resulting bathymetry, and the general patterns of erosion and deposition, to post-flood bathymetry data. Two post-flood bathymetry resources were available: the 2020 topo-bathymetric LIDAR presented in Figure 2 (City of Calgary, 2021) and a 2013 bathymetric survey (Golder Associates, 2015). As described in Section 3.5.4, the model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition.

The error of the modelled bathymetry (i.e. the difference between the resulting bathymetry elevation and the measured bathymetry elevation) was quantified using root-mean-square error (*RMSE*):

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (y_i - \hat{y}_i)^2}{n}} \quad (10)$$

where y_i represents the modelled elevation at pixel i (i.e. the elevation measured in the physical model), $\hat{y}_i$ represents the field observed elevation at pixel i , and n represents the number of pixels in the assessed area (i.e. the number of pixels in each zone). The *RMSE* values were computed using a 1 m (FS) pixel size.

The accuracy of the as-built initial bathymetry was also assessed by computing the *RMSE* using the pre-flood bathymetry from Golder Associates (2011) as a baseline. It is important to interpret the *RMSE* of the as-built initial bathymetry in the context of the spatial resolution of the construction methodology. As described in Section 3.4.2 the templates that guided the construction of the mobile bed were cut to an accuracy of ± 2 mm (MS) and levelled with an accuracy of ± 1 mm (MS). The installed templates had a vertical resolution of 0.5m (FS) and a streamwise horizontal resolution of 30m (FS). The *RMSE* values for the as-built initial bathymetry were computed using a 1 m (FS) pixel size which is finer than the resolution of the construction approach. Therefore, the *RMSE* of the as-built initial bathymetry represents a combination of construction error and interpolation error, and should not be interpreted solely as construction error.

Figure 35 displays the as-built initial bathymetry for CAL1. Figure 36 displays the difference between the as-built initial bathymetry and the target pre-flood bathymetry from Golder Associates (2011). Figure 37 displays the model bathymetry after CAL1. Figure 38 displays the difference between the model bathymetry after CAL1 and the post-flood bathymetry from City of Calgary (2021). Figure 39 displays the change in the model bathymetry after CAL1 and can be compared directly to Figure 3 to further assess model skill. Computed error values are summarized in Table 10.

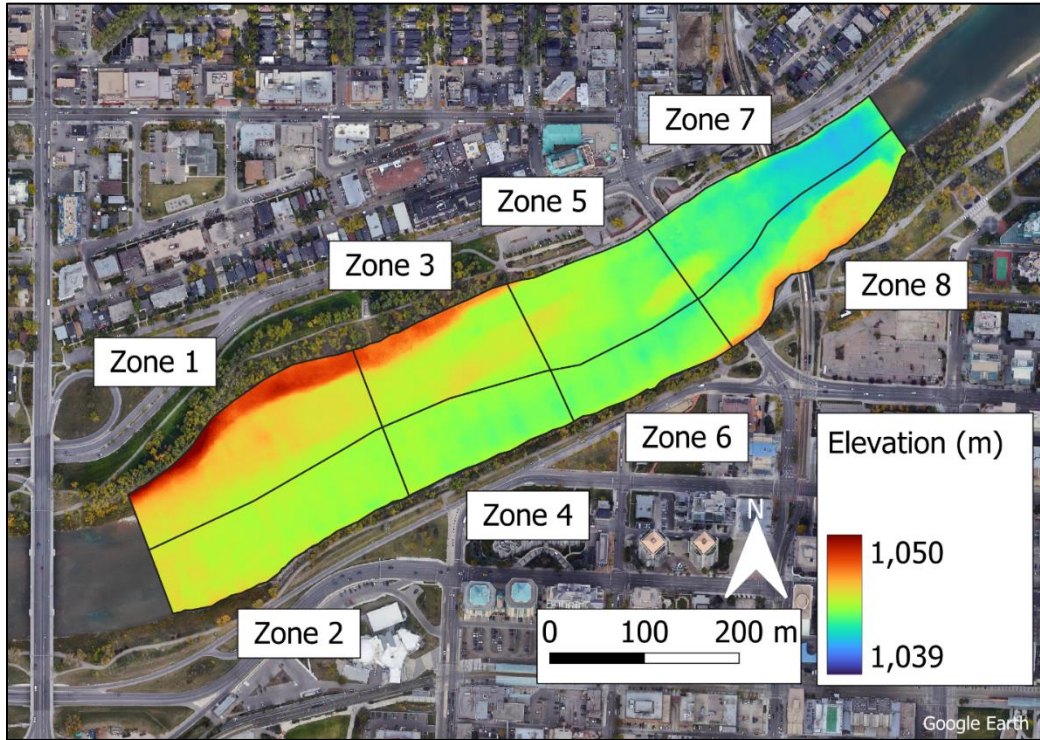


Figure 35. As-built initial bathymetry for CAL1. Zone names conform to Table 10.



Figure 36. As-built initial bathymetry for CAL1 minus the target pre-flood bathymetry from Golder Associates (2011). Differences in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 10.

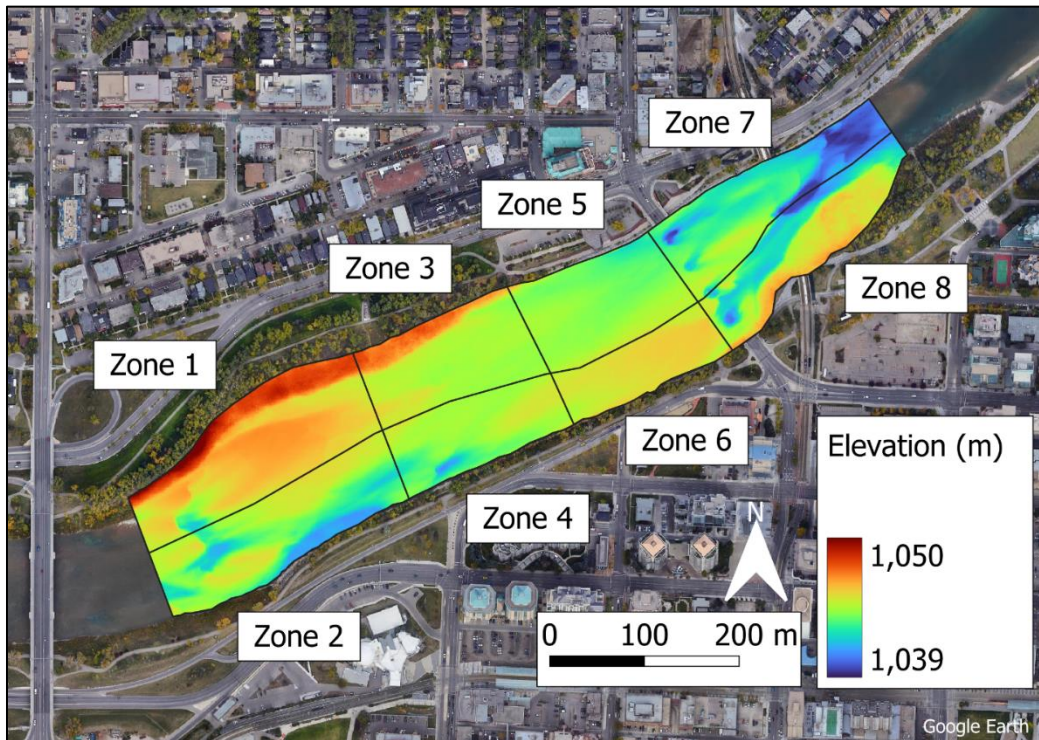


Figure 37. Bathymetry after CAL1. Zone names conform to Table 10.

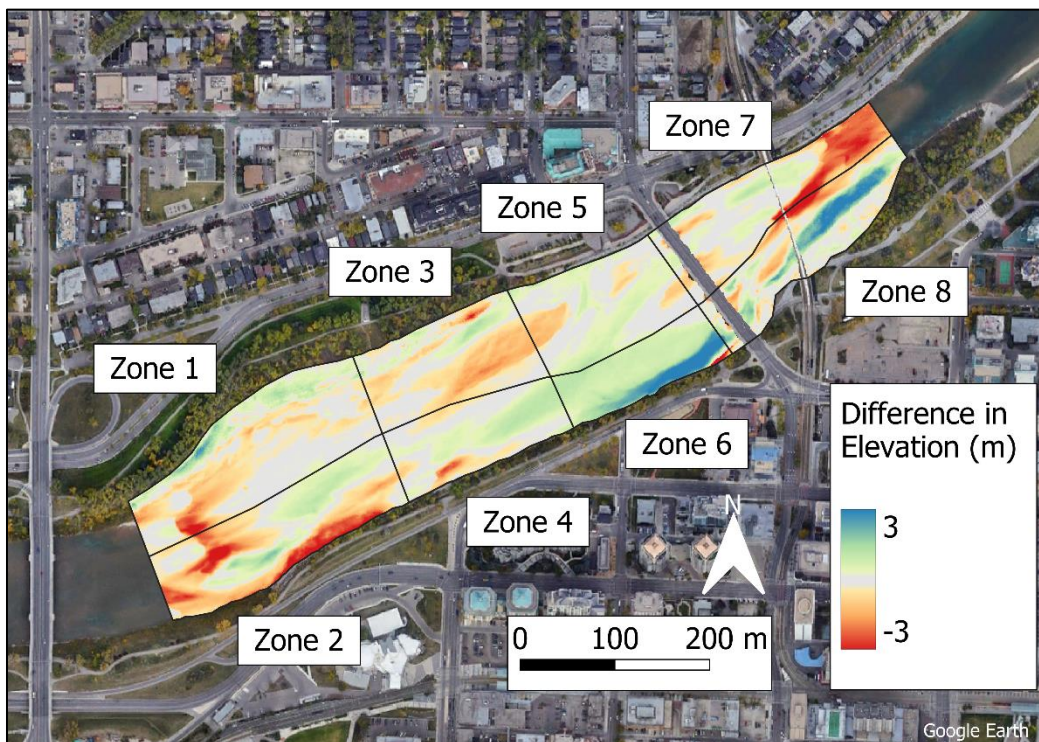


Figure 38. Bathymetry after CAL1 minus the post-flood bathymetry from City of Calgary (2021). Differences in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 10.

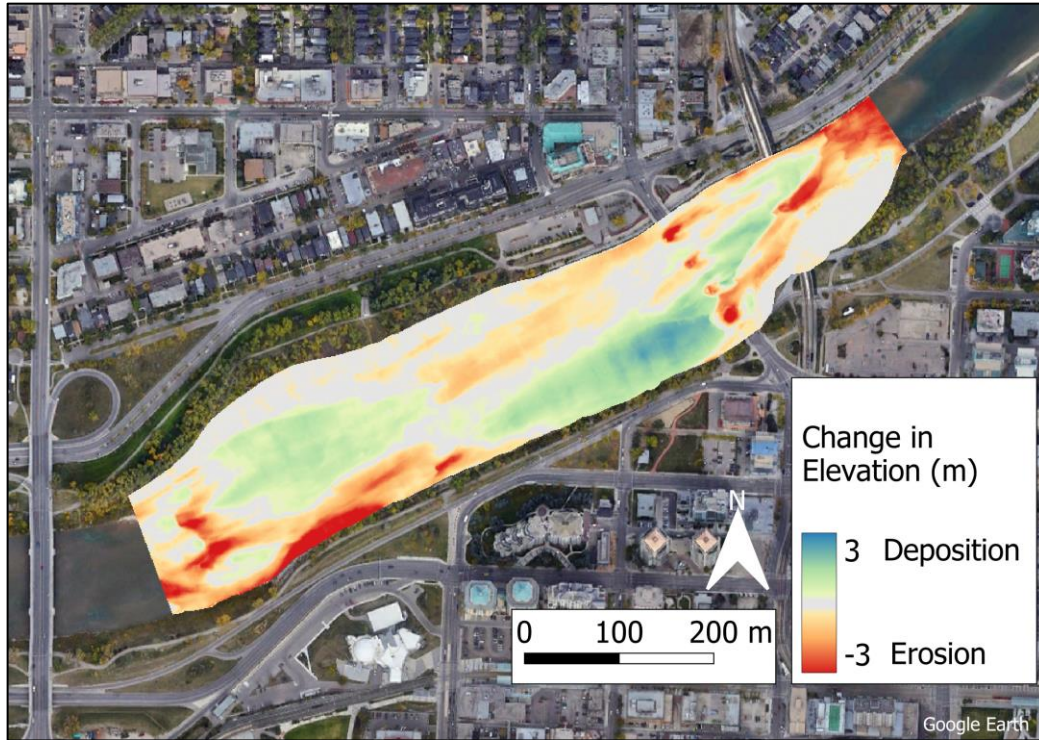


Figure 39. Change in bathymetry after CAL1. Changes in elevation between -0.15 m and 0.15 m are shown in grey.

Table 10. Error summary for CAL1.

Zone	RMSE of As-Built Initial Bathymetry Compared to 2010 Survey (m)	RMSE of Final Bathymetry Compared to 2013 ADCP Survey (m)	RMSE of Final Bathymetry Compared to 2020 LIDAR (m)
1	0.41	0.73	0.71
2	0.18	1.39	1.21
3	0.39	0.73	0.74
4	0.19	0.75	0.60
5	0.25	0.58	0.51
6	0.27	1.37	1.52
7	0.41	1.25	1.24
8	0.52	1.39	1.33
Overall	0.35	1.06	1.01

The overall *RMSE* for the as-built initial bathymetry was equal to 0.35 m. Of the eight zones evaluated, zone 2 had the smallest error (*RMSE* = 0.18 m) and zone 8 had the greatest error (*RMSE* = 0.52 m). The *RMSE* values for the final bathymetry based on the 2020 LIDAR data were similar to those based on the 2013 ADCP survey. Using the 2020 LIDAR data as a baseline, the final modelled bathymetry had an overall *RMSE* equal to 1.01 m. Of the eight zones evaluated, zone 5 had the smallest error (*RMSE* = 0.51 m) and zone 6 had the greatest error (*RMSE* = 1.52 m). Based on visual inspection, the geomorphic change

observed in the model was generally similar to the geomorphic change caused by the 2013 flood event. Notably, the physical model successfully reproduced the general shape and extent of the main bar. In addition, the physical model successfully reproduced the general shape and extent of: the deposition observed near Poppy Plaza, the erosion observed near the south embankment, and the erosion observed north of the main bar in the vicinity of Louise Bridge.

The physical model did not successfully reproduce the erosion observed along the south bank downstream of Louise Bridge. As described in Section 3.4.4.1, the armouring installed in this location was modified for CAL2 in an effort to address this discrepancy. In addition, the model overexaggerated the erosion depth in the vicinity of the south embankment and created unrealistic erosion in the vicinity of the upstream boundary, the downstream boundary, and the bridge piers. Adjustments were made in CAL2 in an effort to address these discrepancies as well. The results for CAL2 are described in Section 5.2.

The physical model did not successfully reproduce the erosion observed along the south bank upstream, and in the vicinity, of Louise Bridge as shown in Figure 3. However, it is possible that the erosion observed in this location, based on field data, may be overexaggerated or unrealistic. Figure 3 indicates the change in elevation based on the 2020 LIDAR data (City of Calgary, 2021) and the 2010 river survey (Golder Associates, 2011). Golder Associates (2011) explain that their data product was “interpolated based on [their] surveyed cross-sections and zig-zag lines ... [and] the bathymetric surface was merged into a TIN with other topographic data”. The survey point data from Golder Associates (2011), provided by the City of Calgary, indicate that the survey track did not cover the right (south) side of the channel in the vicinity of Louise Bridge (Figure 40), suggesting that elevations in this location were interpolated. The bathymetry from Golder Associates (2011) indicate a generally flat bathymetry along the south bank in the vicinity of Louise Bridge (see Figure 2a). However, pre-flood satellite imagery suggests that a deep side channel was present in this location prior to the 2013 flood event (Figure 1). It is possible that this channel is not well represented in the interpolated pre-flood bathymetry from Golder Associates (2011) owing to a sparsity of data in this location. If the pre-flood bathymetry in this location was deeper than indicated by Golder Associate (2011), it may explain the discrepancy observed in the physical model.

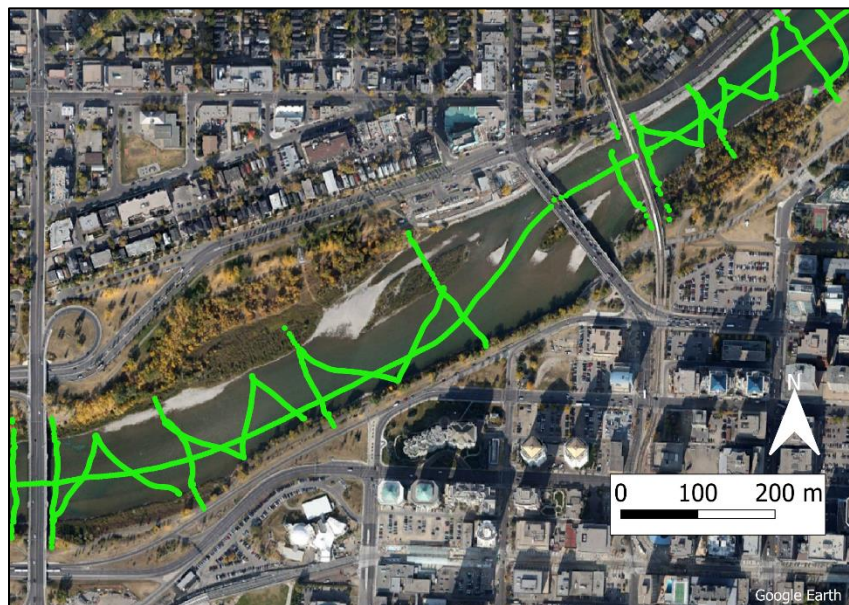


Figure 40. Survey track for the pre-flood bathymetry (Golder Associates, 2011) plotted over a pre-food satellite imagery basemap.

Figure 41 shows a photo of the model during the peak flow of CAL1 and Figure 42 shows select photos of the model after CAL1.



Figure 41. Photo during CAL1_1750.



Figure 42. Photos of the model after CAL1; a) standing at the south embankment looking downstream, b) standing near the north abutment of the LRT bridge, looking upstream, c) standing at Poppy Plaza, looking downstream, and d) standing at Poppy Plaza, looking upstream.

5.2 Calibration #2

Similar to Calibration #1, Calibration #2 (CAL2) was focused on recreating the flow conditions a geomorphic change observed during the 2013 flood event. The calibration approach applied for CAL2 was consistent with the approach described for CAL1 in Section 5.2.

Several adjustments were made to the model in preparation for, and during, CAL2 in an effort to address the discrepancies identified in CAL1 (see Section 5.1). These adjustments included:

- Modifying the riprap armouring installed on the landing as described in Section 3.4.4.1
- Adding a subsurface clay layer (represented as riprap) near the upstream end of the south embankment as described in Section 3.4.3
- Adding a narrow strip of riprap material at the upstream and downstream boundaries
- Adding riprap material around the base of the Louise Bridge piers and the LRT bridge piers
- Lowering the concrete bed at the downstream boundary of the model to improve control of the tailwater elevation

The results for CAL2 presented in this section refer to the results observed after subjecting the model to the three-stage hydrograph H2 described in Section 4.1.1 (i.e. not including the extended hydrograph tail). Figure 43 displays the as-built initial bathymetry for CAL2. Figure 44 displays the difference between the as-built initial bathymetry and the target pre-flood bathymetry from Golder Associates (2011). Figure 45 displays the model bathymetry after CAL2. Figure 46 displays the difference between the model bathymetry after CAL2 and the post-flood bathymetry from City of Calgary (2021). Figure 47 displays the change in the model bathymetry after CAL2 and can be compared directly to Figure 3 to further assess model skill. Computed error values are summarized in Table 11.

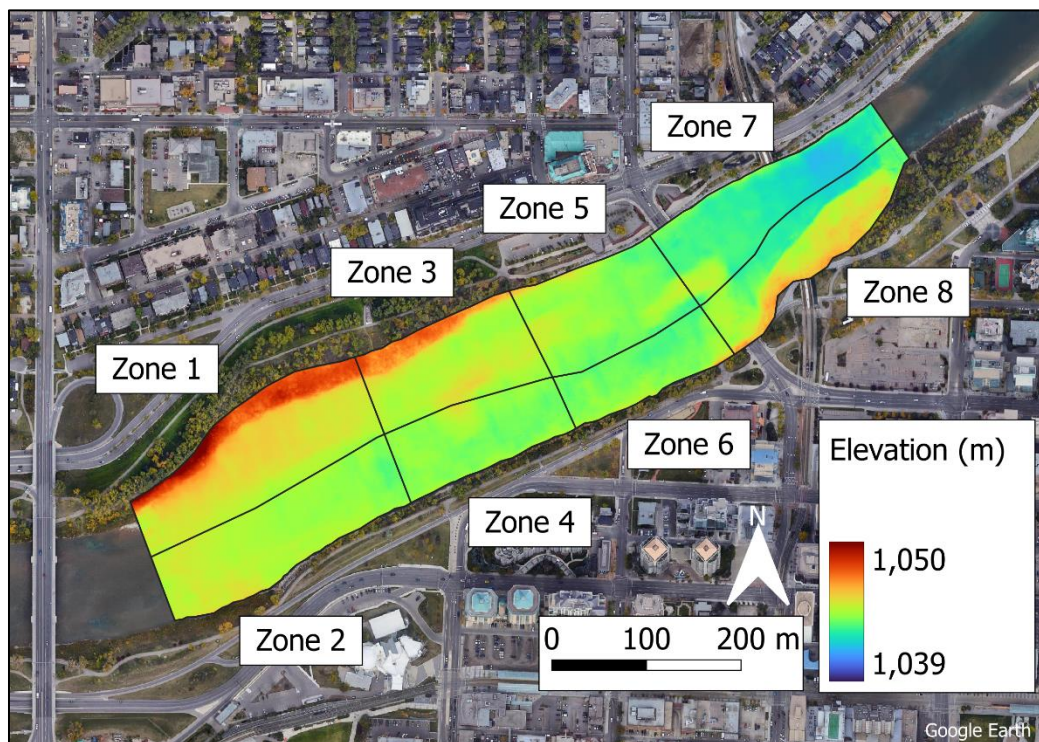


Figure 43. As-built initial bathymetry for CAL2. Zone names conform to Table 11.

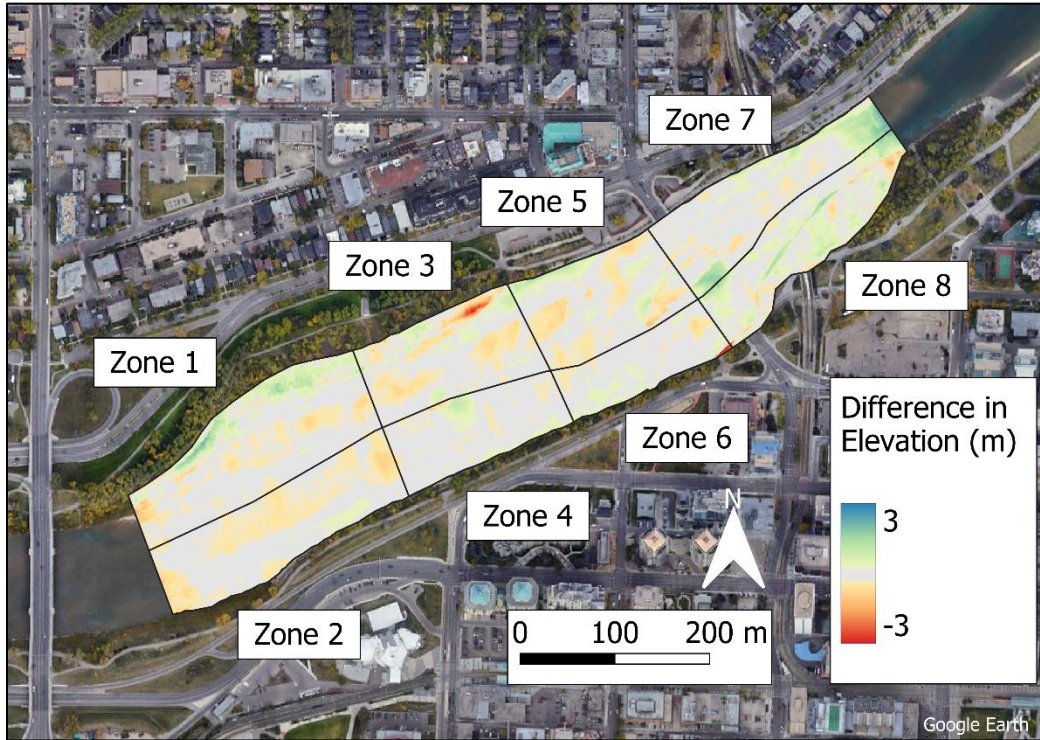


Figure 44. As-built initial bathymetry for CAL2 minus the target pre-flood bathymetry from Golder Associates (2011). Differences in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 11.

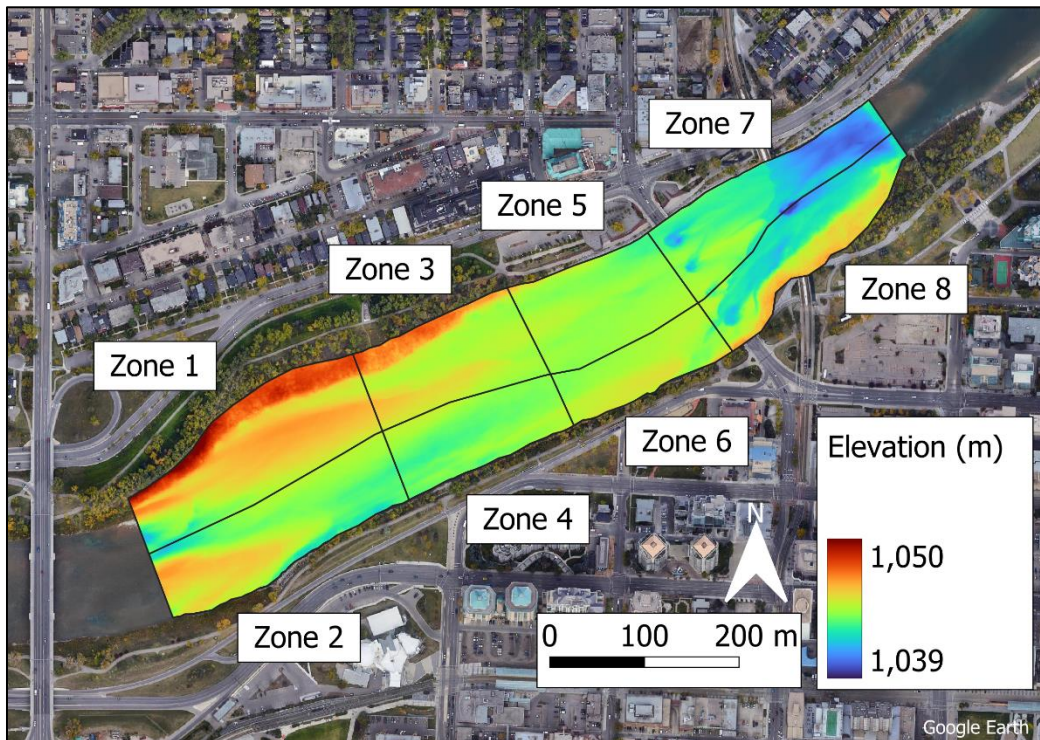


Figure 45. Bathymetry after CAL2. Zone names conform to Table 11.

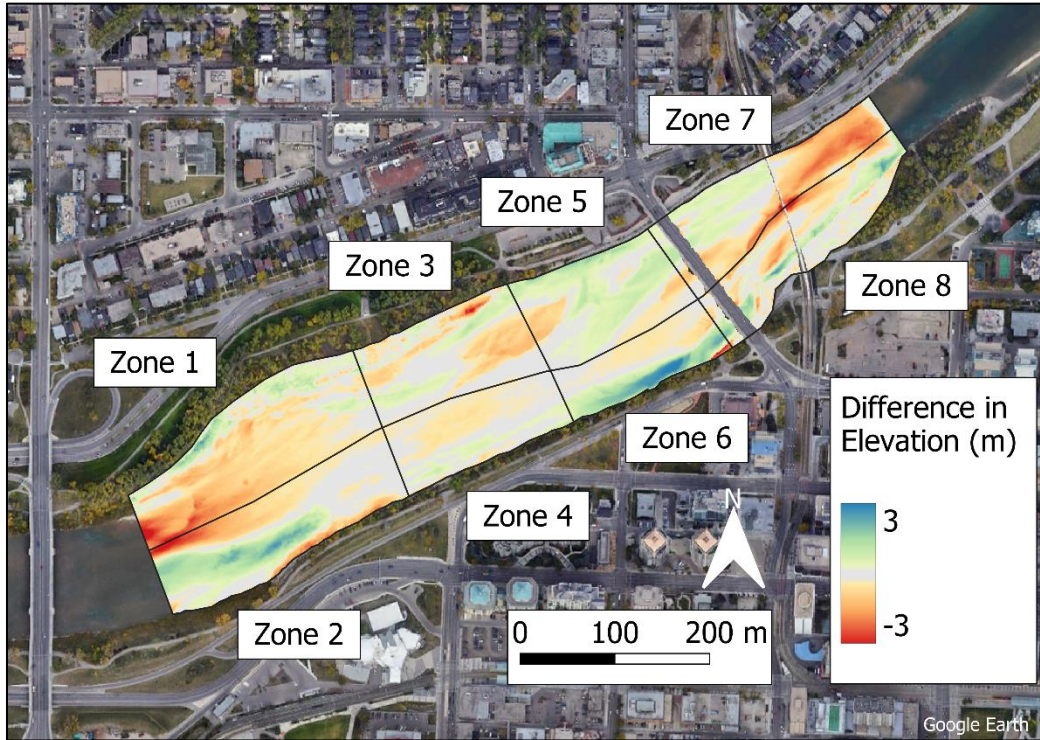


Figure 46. Bathymetry after CAL2 minus the post-flood bathymetry from City of Calgary (2021). Differences in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 11.

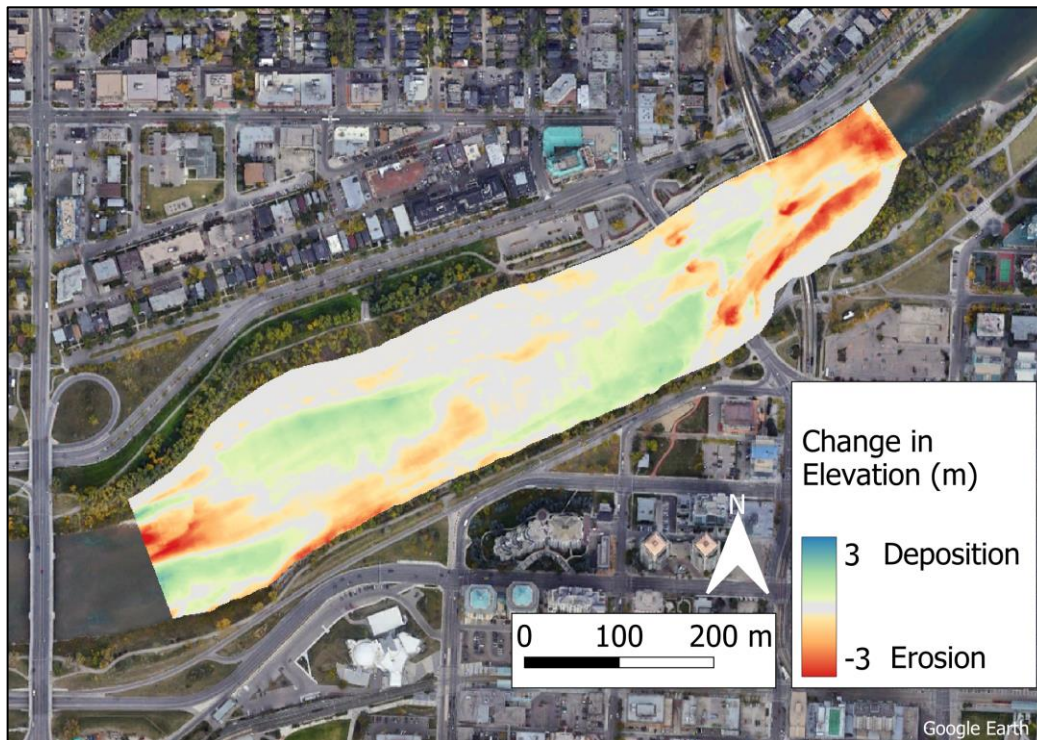


Figure 47. Change in bathymetry after CAL2. Changes in elevation between -0.15 m and 0.15 m are shown in grey.

Table 11. Error summary for CAL2.

Zone	RMSE of As-Built Initial Bathymetry Compared to 2010 Survey (m)	RMSE of Final Bathymetry Compared to 2013 ADCP Survey (m)	RMSE of Final Bathymetry Compared to 2020 LIDAR (m)
1	0.41	0.91	0.89
2	0.29	1.02	0.98
3	0.43	0.55	0.57
4	0.24	0.46	0.32
5	0.36	0.44	0.62
6	0.31	0.90	1.08
7	0.54	1.02	1.01
8	0.49	0.86	0.80
Overall	0.40	0.82	0.82

The overall *RMSE* for the as-built initial bathymetry was equal to 0.40 m. Of the eight zones evaluated, zone 4 had the smallest error (*RMSE* = 0.24 m) and zone 7 had the greatest error (*RMSE* = 0.54 m). The *RMSE* values for the final bathymetry based on the 2020 LIDAR data were similar to those based on the 2013 ADCP survey. Using the 2020 LIDAR data as a baseline, the final modelled bathymetry had an overall *RMSE* equal to 0.82 m. Of the eight zones evaluated, zone 4 had the smallest error (*RMSE* = 0.32 m) and zone 6 had the greatest error (*RMSE* = 1.08 m). However, as described in Section 5.1, discrepancies in zone 6 may be explained, in part, by potential interpolation errors in the available field data. Based on visual inspection, and similar to CAL1, the geomorphic change observed in CAL2 was generally similar to the geomorphic change caused by the 2013 flood event. Notably, the physical model successfully reproduced the general shape and extent of the main bar. In addition, the physical model successfully reproduced the general shape and extent of the deposition observed near Poppy Plaza and the erosion observed near the south embankment. Visually, CAL2 did not reproduce the erosion observed north of the main bar in the vicinity of Louise Bridge as well as CAL1.

CAL2 successfully reproduced the erosion observed along the south bank downstream of Louise Bridge (i.e. the landing), which was not well reproduced in CAL1, suggesting that the modification of the riprap in this location improved the skill of the model. In addition, the erosion depth near the upstream end of the south embankment appeared to be more realistic in CAL2, compared to CAL1, suggesting that the introduction of the clay layer improved the skill of the model. The introduction of riprap around the bases of the bridge piers appeared to improve model skill in these locations. However, the introduction of riprap at the upstream and downstream boundaries did not appear to have a significant, local impact on model skill. The model adjustments that were introduced for CAL2 resulted in an overall improvement in model skill. Therefore, these adjustments were maintained for the subsequent tests focused on the conceptual designs, unless the design drawings specified otherwise. Figure 48 shows select photos of the model after CAL2.

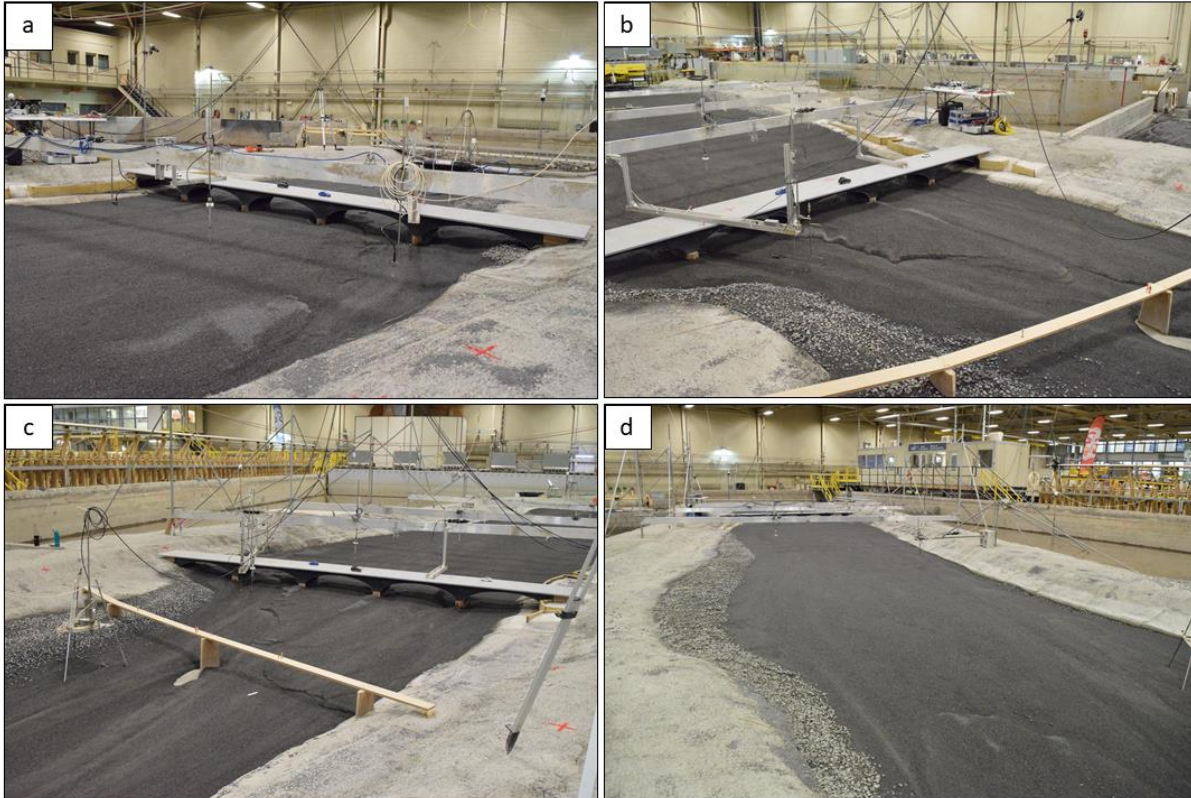


Figure 48. Photos of the model after CAL2; a) standing at the south embankment, looking downstream, b) standing near the south abutment of the LRT bridge, looking upstream, c) standing near the north side of the downstream boundary, looking upstream, and d) standing near the north side of the upstream boundary, looking downstream.

As described in Section 4.1.1, in addition to the simplified 2013 flood hydrograph, the model was run for an additional 30 hours (5.5 hours MS) at the 850 m³/s flow condition to evaluate the potential impact of a longer hydrograph tail on the bathymetry (see test CAL2_850b in Table 9). This additional flow test did not cause any significant geomorphic change.

5.3 Do-Nothing Test

The do-nothing test was focused on evaluating the resulting bathymetry after CAL2 under a 100-year flow condition. As described in Section 4.1.2, the resulting bathymetry from CAL2 (including the extended hydrograph tail described in Section 4.1.1) was generally representative of the current-day bathymetry, providing an opportunity to understand how the existing study site might respond to a major flood event without any intervention to realign the gravel bars. A small modification was made to the resulting bathymetry from CAL2 prior to running the do-nothing test. As described in Section 4.1.2, a small side channel was dug along the south bank to better represent the actual bathymetry in this location. This side channel, which is present in reality, but was not reproduced well in the calibration tests, is described in detail in Section 5.1.

As described in Section 3.5.4, the model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition. Figure 49 displays the initial bathymetry for the do-nothing test and includes labels for each zone. Table 12 and Table 13 summarize the average water levels and flow velocities observed during the 100-year flow condition, respectively.

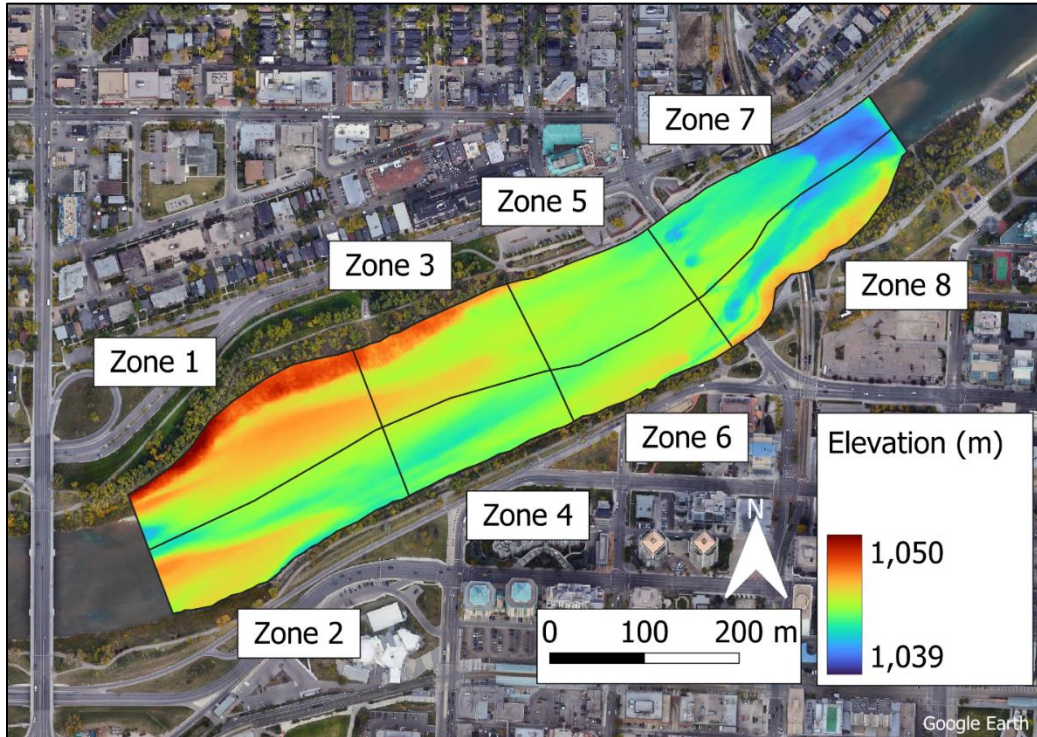


Figure 49. Initial bathymetry for the do-nothing test. This represents the resulting bathymetry following CAL2 (including the extended hydrograph tail test CAL2_850b) plus the introduction of a small side-channel along the south bank near Louise Bridge.

Table 12. Average water levels for the do-nothing test.

Water Level Gauge	Average Water Level (m)
WL1	1049.93
WL2	1049.68
WL3	1049.25
WL4	1049.42
WL5	1048.89
WL6	1049.31
WL7	1049.77
WL8	1049.13
WL9	1049.27
WL10	1049.34
WL11	1049.73
WL12	1048.98
WL13	1048.68
WL14	1048.71

Table 13. Average horizontal velocities for the do-nothing test.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	1.82
V2	3.36
V3	2.87
V4	3.27
V5	-
V6	3.44
V7	3.98

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1049.25 m. Therefore, the average water level was approximately 1.25 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The water level in the vicinity of the LRT bridge did not exceed the bottom chord of the bridge, which has an elevation of 1049.8 m. Figure 50 and Figure 51 show select photos of Louise Bridge and the LRT bridge, respectively, during the do-nothing test. The average velocity on the north side of the main bar approaching Louise Bridge was equal to 2.87 m/s. The average velocity near the south embankment was equal to 1.82 m/s.

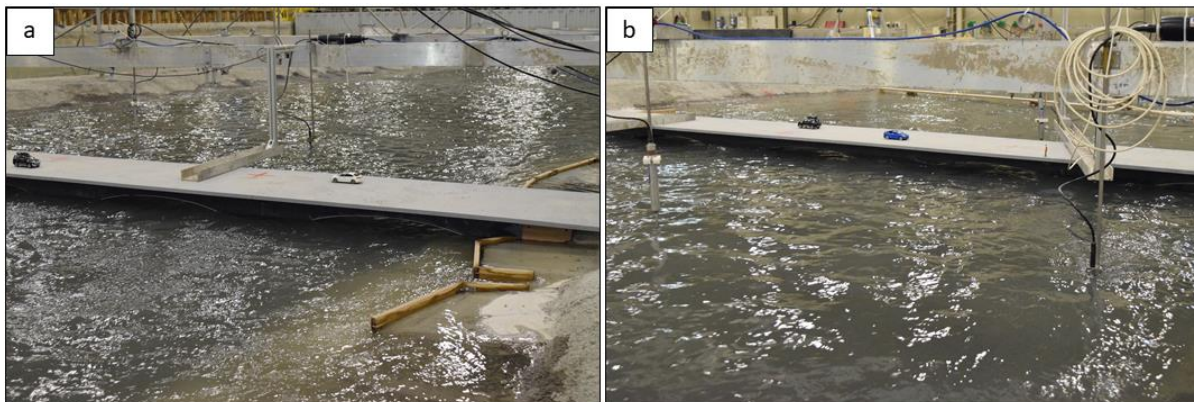


Figure 50. Flow near Louise Bridge during the do-nothing test; a) standing on the north bank downstream of Louise Bridge, looking upstream and b) standing on the south embankment, looking downstream.

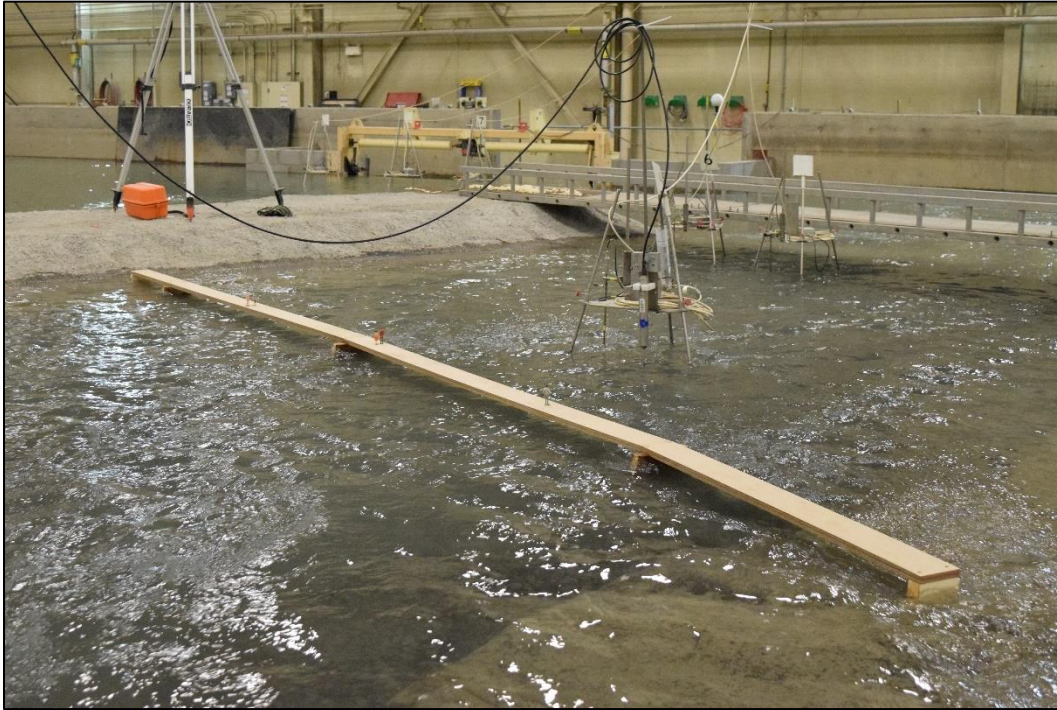


Figure 51. Flow near the LRT bridge during the do-nothing test (standing on the south bank, looking across the channel and downstream).

Following the do-nothing test, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 52 displays the resulting bathymetry and Figure 53 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 14 (positive values indicate deposition and negative values indicate erosion).

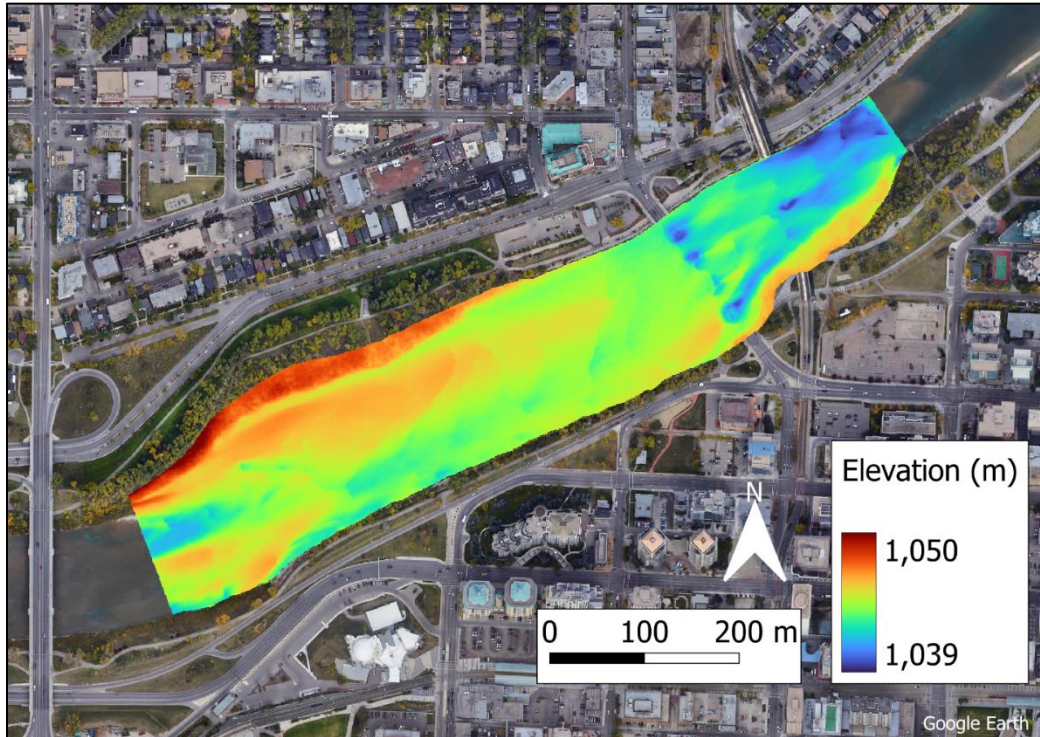


Figure 52. Bathymetry after the do-nothing test.

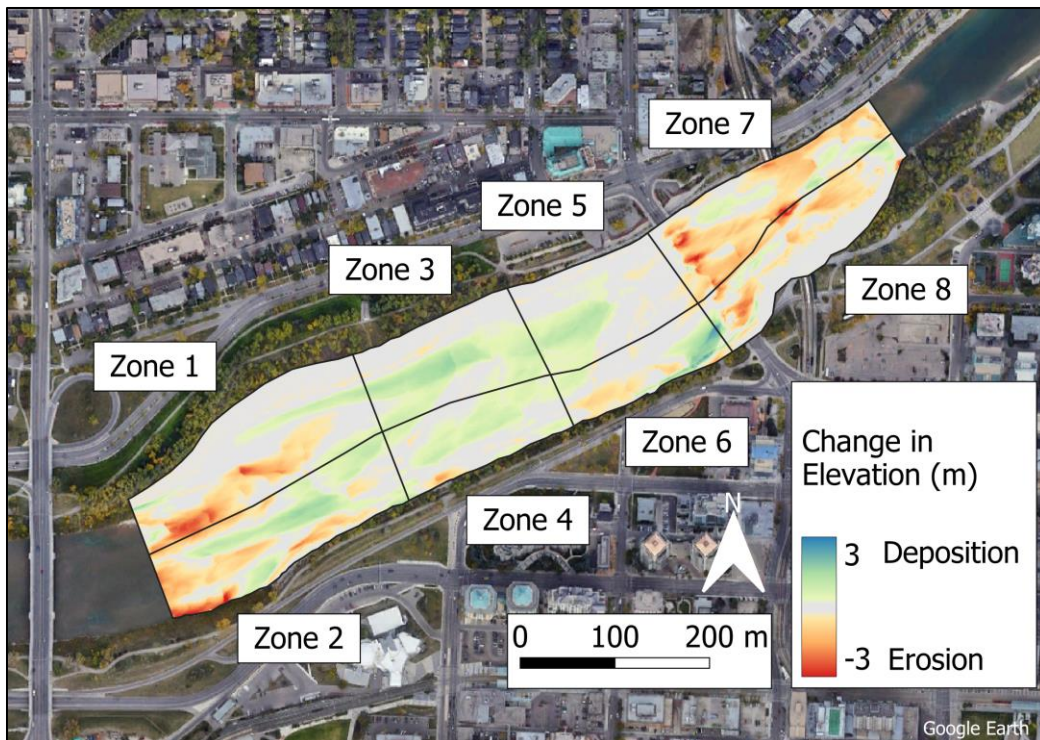


Figure 53. Change in bathymetry after the do-nothing test. Changes in elevation between -0.15m and 0.15m are shown in grey. Zone names conform to Table 14.

Table 14. Deposition and erosion by zone after the do-nothing test.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	24996	-2.65	1.76	-0.19	-4805
2	21318	-3.39	1.72	-0.03	-644
3	16635	-0.63	1.75	0.40	6644
4	13397	-1.70	1.27	0.27	3556
5	16295	-0.75	1.42	0.19	3105
6	10164	-1.80	2.56	0.14	1394
7	16833	-3.05	1.39	-0.47	-7909
8	15869	-3.46	2.67	-0.24	-3888
Overall	135508	-3.46	2.67	-0.02	-2547

Localized deposition was observed on the south side of the main bar upstream of Louise Bridge (zone 6), suggesting that the main bar would continue to grow towards the south bank during a 100-year flood if no action is taken to modify the site. This observation agrees with the findings from Yu et al. (2022) who concluded that deposition would continue to occur on the right (south) side of the gravel bar if no action is taken based on numerical modelling. Yu et al. (2022) also concluded that deposition would continue to occur near the nose of the main bar, and this was generally observed in the physical model as well (e.g. the deposition near the upstream portion of zones 5 and 6). Downstream of Louise Bridge, results from Yu et al. (2022) indicated that erosion would occur along the north side of the main bar and deposition would occur along the right side. The physical model results indicated a generally erosive environment downstream of Louise Bridge on both sides of the main bar, although some localized deposition was observed immediately south side of the main bar. The physical model results indicated that net deposition would occur on both the north and south side of the channel upstream of the main bar (zones 3 and 4). Figure 54 shows a photo of the model during the do-nothing test and Figure 55 shows select photos of the model after the do-nothing test.



Figure 54. Photo of the model during the do-nothing test (standing near the north side of the downstream boundary, looking upstream).

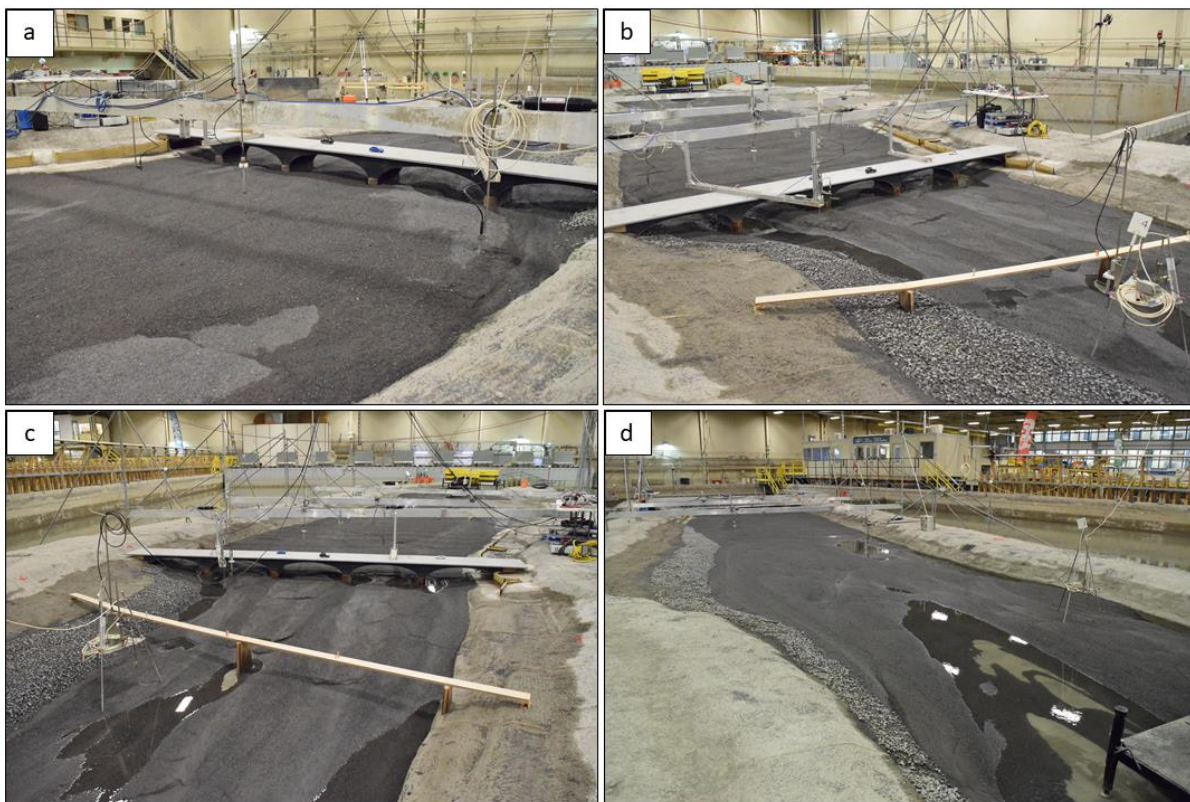


Figure 55. Photos of the model after the do-nothing test; a) standing at the south embankment, looking downstream, b) standing near the south side of the downstream boundary, looking upstream, c) standing near the north side of the downstream boundary, looking upstream, and d) standing near the north side of the upstream boundary, looking downstream.

5.3.1 Cross-Sections

Bathymetry profiles were extracted at three cross-sections as described in Section 3.5.4. Figure 56, Figure 57, and Figure 58 show the bathymetry profiles for the do-nothing test for Cross-Section 1, 2, and 3, respectively.

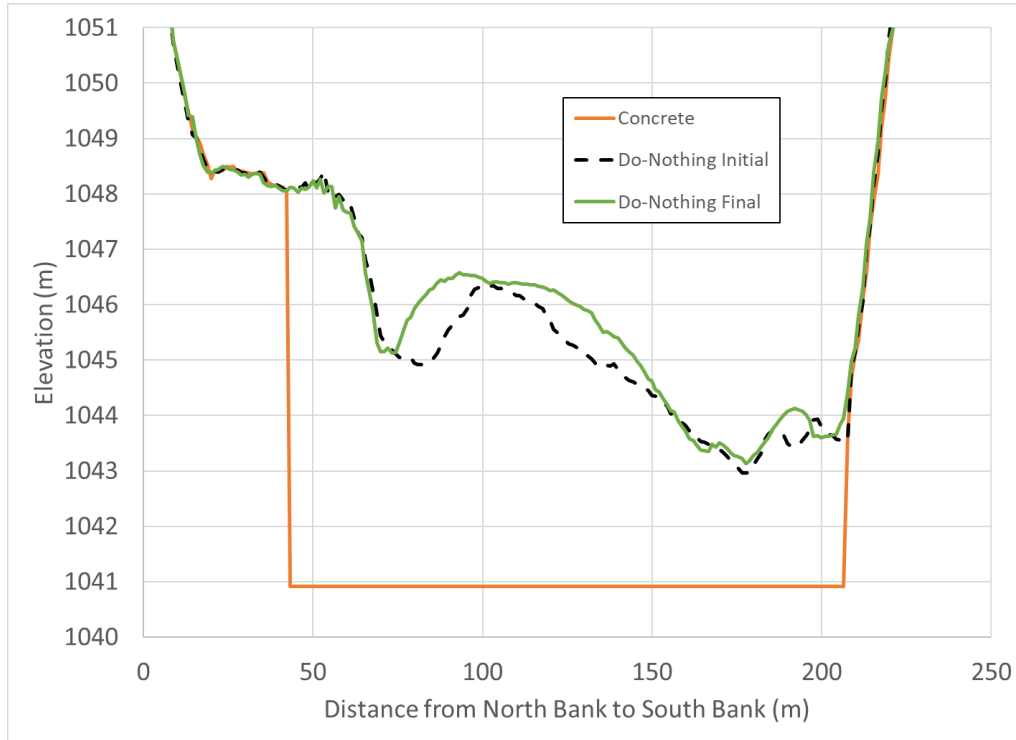


Figure 56. Bathymetry profiles for the do-nothing test at Cross-Section 1.

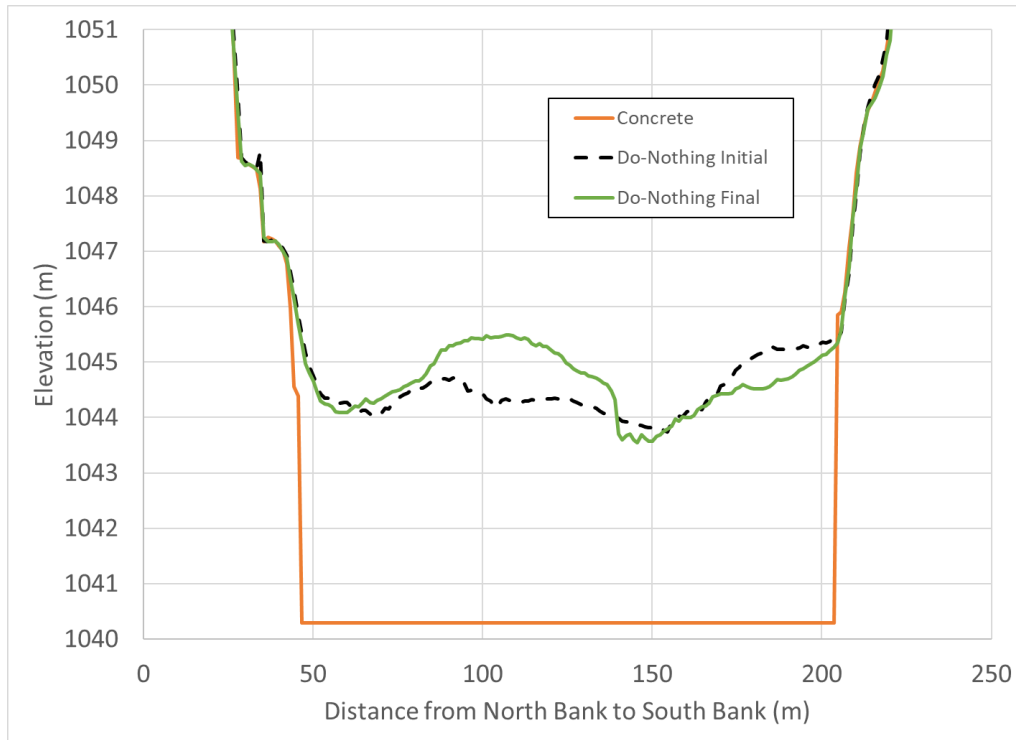


Figure 57. Bathymetry profiles for the do-nothing test at Cross-Section 2.

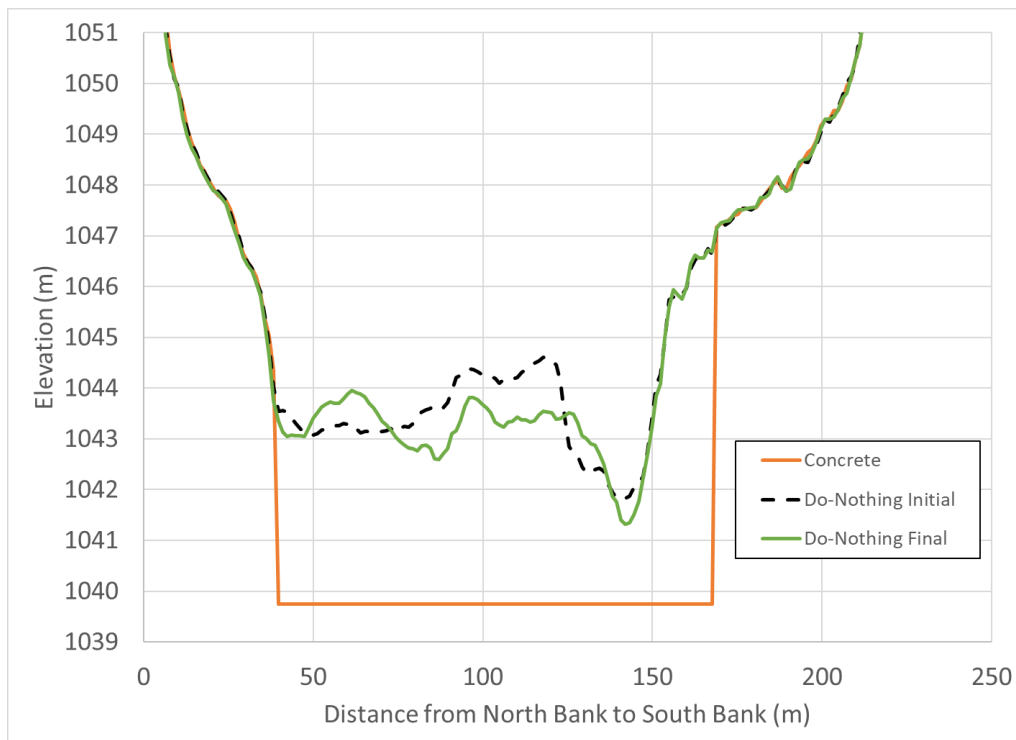


Figure 58. Bathymetry profiles for the do-nothing test at Cross-Section 3.

5.4 Test Series A – Design #1

Test Series A (TSA) was focused on evaluating Conceptual Design #1 under the 10-, 35-, and 100-year flow condition. The following subsections summarize observed water levels, flow velocities, and geomorphic change. As described in Section 3.5.4, the model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition. Figure 59 displays the as-built initial bathymetry for TSA and includes labels for each zone. Figure 60 displays the difference between the as-built initial bathymetry for TSA and the design bathymetry prescribed by Matrix Solutions Inc (see Appendix A).

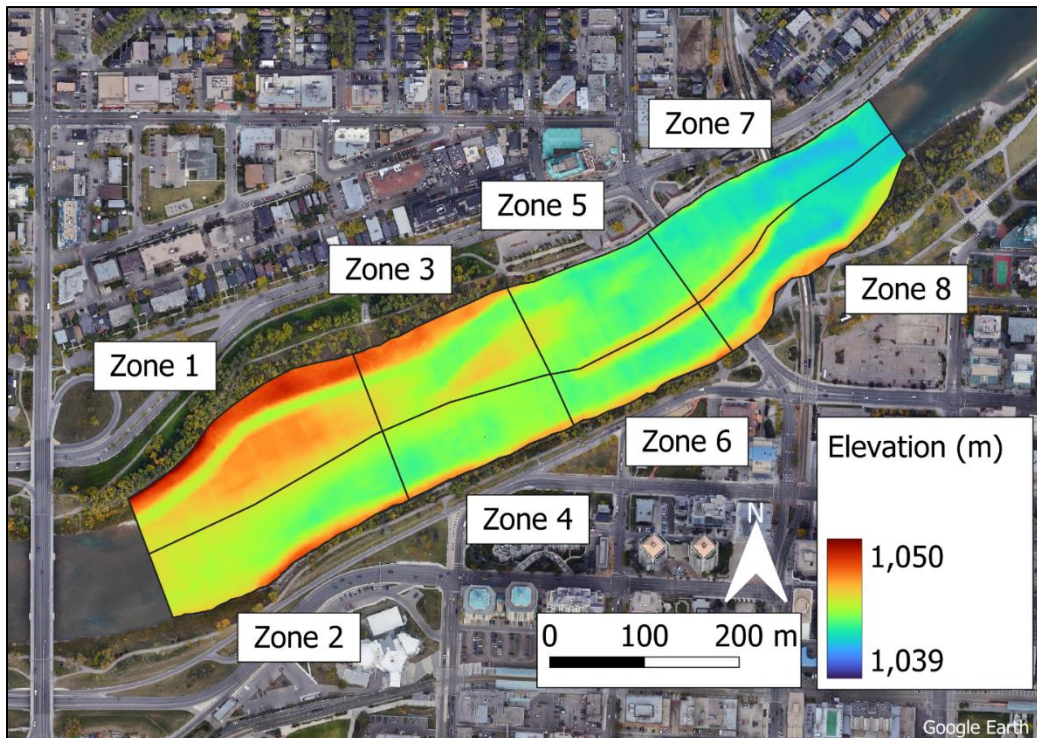


Figure 59. As-built initial bathymetry for TSA.

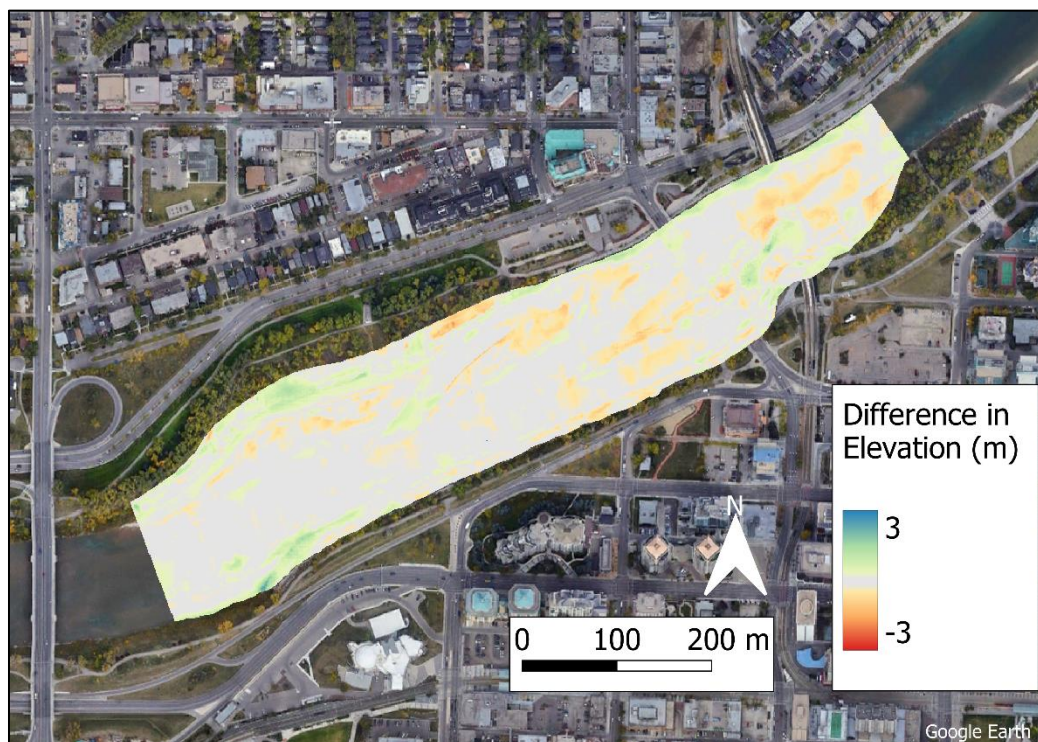


Figure 60. As-built initial bathymetry for TSA minus the design bathymetry prescribed by Matrix Solutions Inc. Differences in elevation between -0.15 m and 0.15 m are shown in grey.

5.4.1 10-Year Flow Condition (850 m³/s)

Conceptual Design #1 was, first, subjected to a 10-year flow condition with a flow equal to 850 m³/s for a duration of 30 hours. Table 15 and Table 16 summarize the average water levels and flow velocities observed during the 10-year flow condition, respectively.

Table 15. Average water levels for TSA_850

Water Level Gauge	Average Water Level (m)
WL1	1047.90
WL2	1047.30
WL3	1046.97
WL4	1046.87
WL5	1046.66
WL6	1046.76
WL7	1046.87
WL8	1046.63
WL9	1046.76
WL10	1046.73
WL11	1046.53
WL12	1046.41
WL13	1046.29
WL14	1046.30

Table 16. Average horizontal velocities for TSA_850

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.43
V2	1.81
V3	2.33
V4	-
V5	2.56
V6	3.15
V7	3.62

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1046.71 m. Therefore, the average water level was approximately 3.79 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 2.33 m/s and 2.56 m/s, respectively. The average velocity near the south embankment was equal to 3.43 m/s.

Following the 10-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 61 displays the resulting bathymetry and Figure 62 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 17 (positive values indicate deposition and negative values indicate erosion).

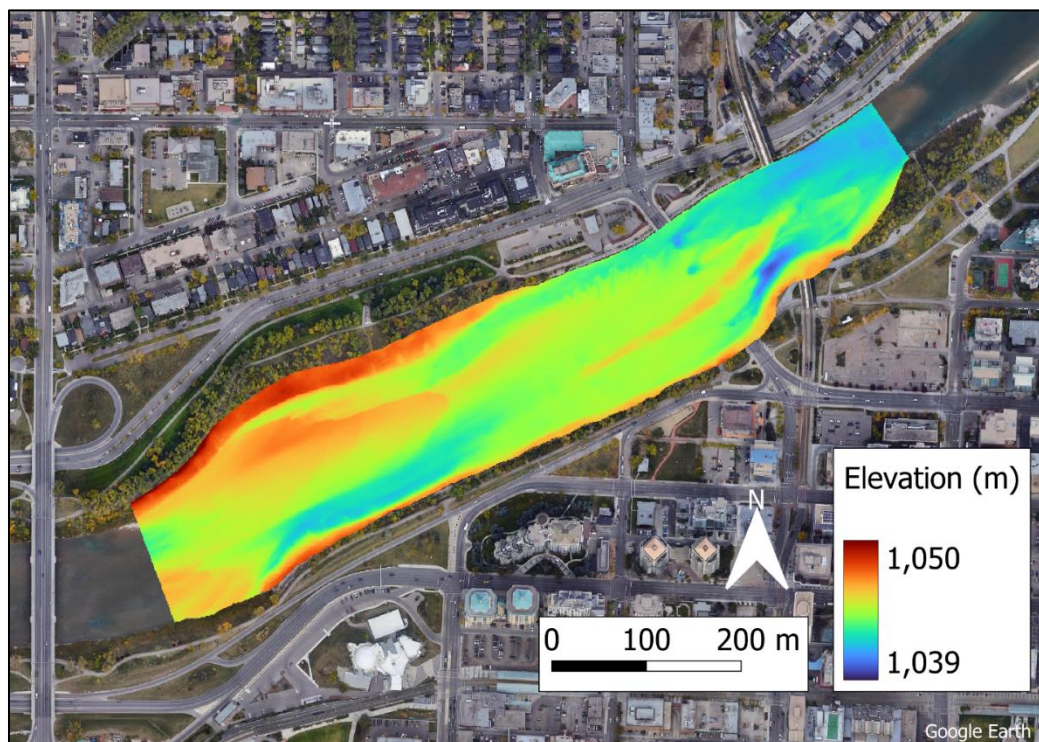


Figure 61. Bathymetry after TSA_850.

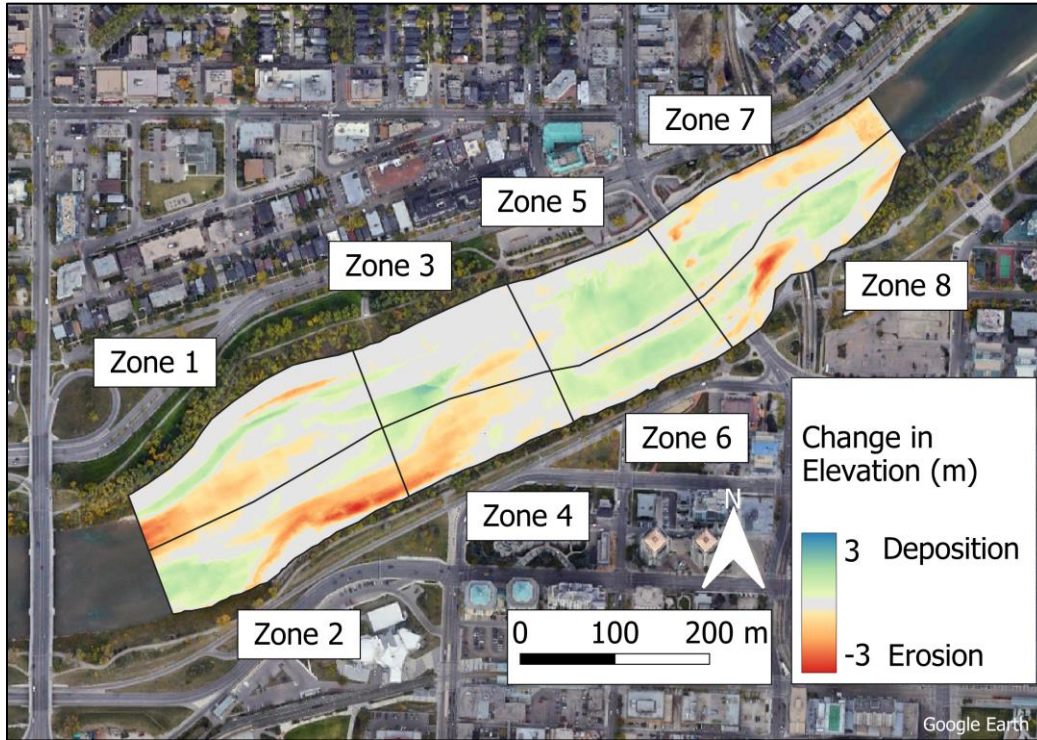


Figure 62. Change in bathymetry after TSA_850. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 17.

Table 17. Deposition and erosion by zone after TSA_850.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25014	-2.25	1.85	-0.11	-2759
2	21388	-2.92	1.48	-0.19	-4050
3	16638	-1.65	2.09	0.11	1747
4	13436	-5.51	1.00	-0.29	-3894
5	16303	-0.94	1.60	0.44	7217
6	10204	-0.91	1.49	0.45	4626
7	16851	-2.73	1.53	-0.10	-1669
8	15876	-2.71	1.78	-0.01	-230
Overall	135710	-5.51	2.09	0.01	988

The scan data indicated localized sediment deposition in the habitat channel, as well as deposition in the habitat channel crossover. Some localized erosion was observed along the north edge of the habitat channel. Net erosion was observed along a large portion of the south embankment, upstream of the main bar (zones 2 and 4). Deposition was generally observed north and south of the main bar upstream, and underneath, of Louise Bridge (zones 5 and 6, and the upstream portions of zones 7 and 8). Localized scour

was observed in the recreation channel downstream of Louise Bridge, adjacent to the landing. A localized area of increased flow turbulence was observed immediately downstream of the armoured main bar nose, creating surface water undulations, and potentially contributing to some of the erosion observed along the main bar. Sediment input at the upstream boundary was concentrated on the south side of the boundary. It is believed that this approach was, in part, responsible for the scour observed on the northern portion of the boundary and the deposition observed on the southern portion of the boundary. Sediment introduction was more evenly distributed across the length of the upstream boundary for subsequent tests. Figure 63 shows a photo of the model during TSA_850, Figure 64 shows a close-up photo of the turbulence downstream of the armoured main bar nose, and Figure 65 shows select photos of the model after TSA_850.



Figure 63. Photo of the model during TSA_850.

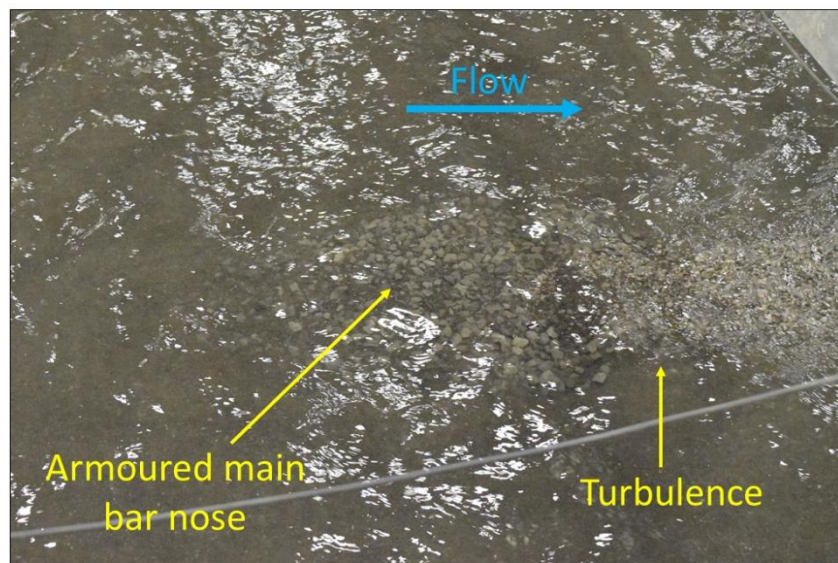


Figure 64. Photo of turbulence downstream of the armoured main bar nose.



Figure 65. Photos of the model after TSA_850; a) standing at Poppy Plaza, looking across the channel and downstream, b) standing near the south side of the downstream boundary, looking upstream, c) standing near the north abutment of the LTR bridge, looking upstream, and d) standing at Poppy Plaza, looking upstream.

5.4.2 35-Year Flow Condition (1400 m³/s)

Following the 10-year flow, Conceptual Design #1 was subjected to a 35-year flow condition with a flow equal to 1400 m³/s for a duration of 30 hours. Table 18 and Table 19 summarize the average water levels and flow velocities observed during the 35-year flow condition, respectively.

Table 18. Average water Levels for TSA_1400.

Water Level Gauge	Average Water Level (m)
WL1	1048.79
WL2	1048.34
WL3	1048.00
WL4	1047.97
WL5	1047.81
WL6	1047.91
WL7	1048.01
WL8	1047.71
WL9	1047.87
WL10	1047.79
WL11	1047.56
WL12	1047.48
WL13	1047.29
WL14	1047.31

Table 19. Average horizontal velocities for TSA_1400.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.40
V2	2.40
V3	2.77
V4	3.06
V5	2.52
V6	3.09
V7	3.68

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1047.79 m. Therefore, the average water level was approximately 2.71 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 2.77 m/s and 2.52 m/s, respectively. The average velocity near the south embankment was equal to 3.40 m/s.

Following the 35-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 66 displays the resulting bathymetry and Figure 67 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 20 (positive values indicate deposition and negative values indicate erosion).

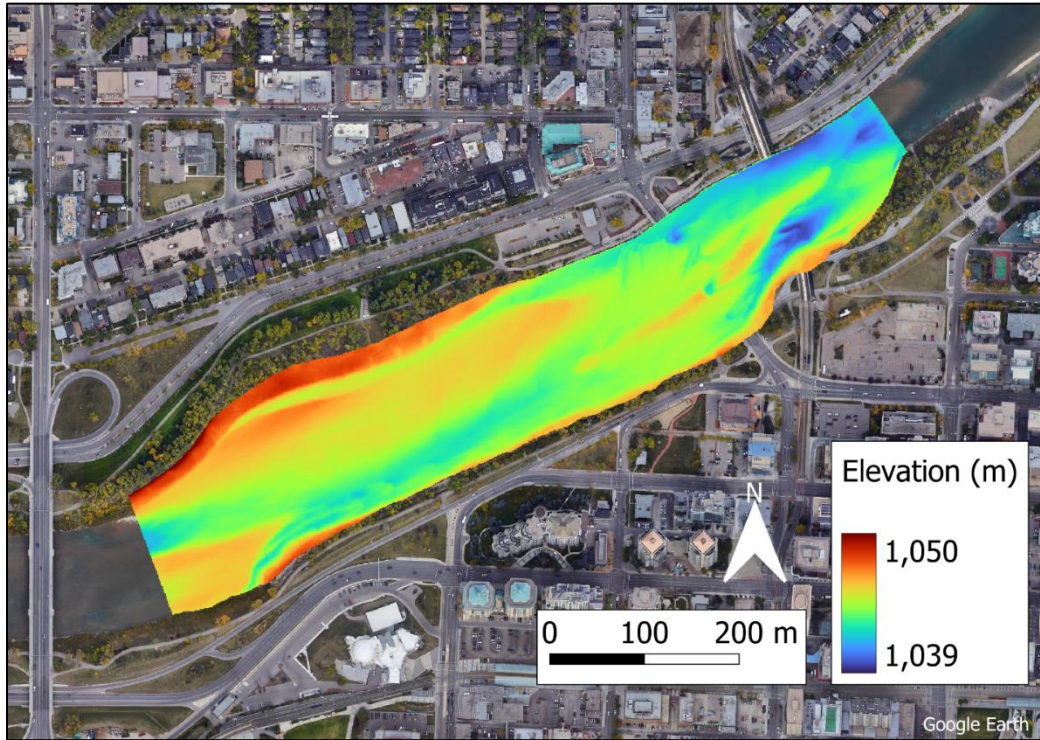


Figure 66. Bathymetry after TSA_1400.

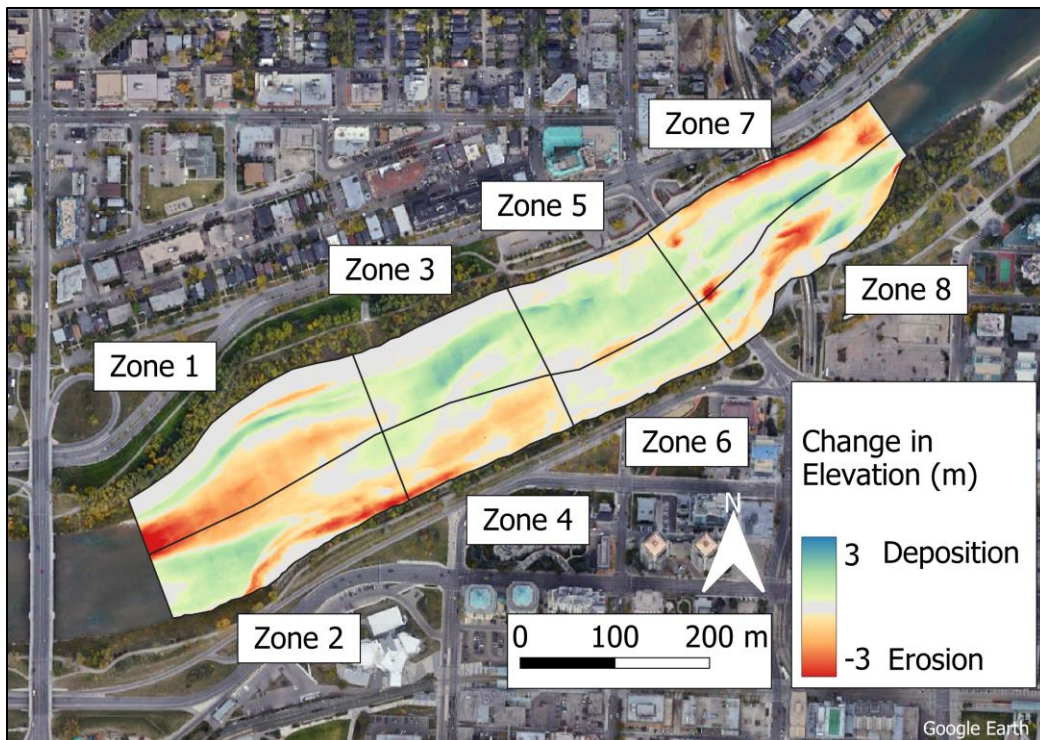


Figure 67. Change in bathymetry after TSA_1400 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 20.

Table 20. Deposition and erosion by zone after TSA_1400.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	24952	-3.69	2.19	-0.42	-10456
2	21394	-3.39	1.77	-0.16	-3508
3	16585	-0.83	2.14	0.54	8925
4	13438	-6.43	0.94	-0.40	-5314
5	16260	-1.37	2.11	0.55	9024
6	10207	-1.13	1.58	0.37	3738
7	16783	-3.77	2.51	-0.21	-3523
8	15906	-3.64	2.54	-0.02	-250
Overall	135527	-6.43	2.54	-0.01	-1363

Similar to TSA_850, localized net deposition was observed in the habitat channel and the habitat channel crossover, and localized net erosion was observed along the north bank of the habitat channel. The patterns of erosion and deposition observed along the south embankment, and north and south of the main bar, were generally similar to the patterns observed in TSA_850. Compared to the initial bathymetry, net erosion was observed along the south embankment upstream of the main bar (zones 2 and 4), and net deposition was observed north and south of the main bar upstream of Louise Bridge (zones 5 and 6). Localized erosion was observed immediately downstream of the armoured nose of the main bar, resulting in a failure of the bar in this location (Figure 69a). Visual observations suggested that this failure was initiated by the turbulence observed in TSA_850 (Figure 64) and exacerbated by the larger flow of TSA_1400. Localized erosion was also observed where the main bar intersects Louise Bridge, which coincides with a bridge pier. The scour adjacent to the landing appeared to be larger than that observed in TSA_850. Figure 68 shows a photo of the model during TSA_1400 and Figure 69 shows select photos of the model after TSA_1400.



Figure 68. Photo of the model during TSA_1400.

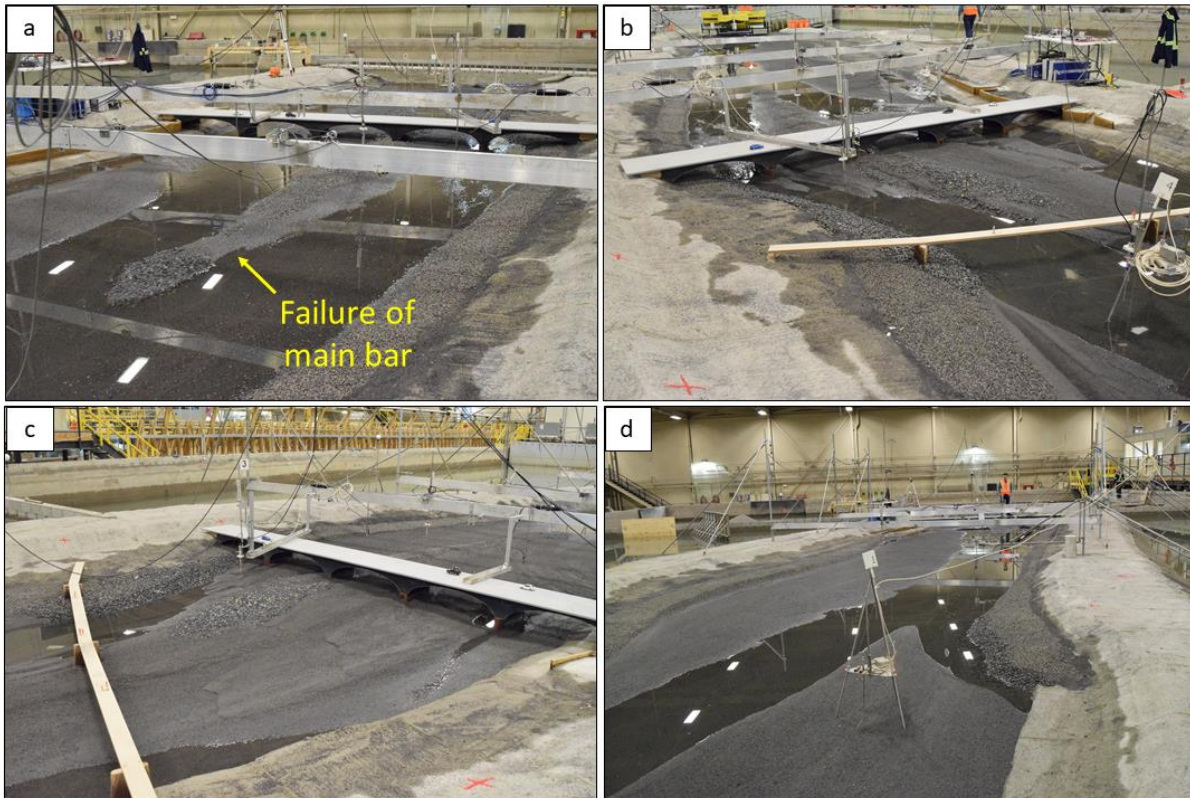


Figure 69. Photos of the model after TSA_1400; a) standing at the south embankment, looking downstream, b) standing on the south side of the downstream boundary, looking upstream, c) standing near the north abutment of the LRT bridge, looking across the channel and upstream, d) standing at the south side of the upstream boundary, looking downstream.

5.4.3 100-Year Flow condition (2100 m³/s)

Following the 35-year flow, Conceptual Design #1 was subjected to a 100-year flow condition with a flow equal to 2100 m³/s for a duration of 30 hours. Table 21 and Table 22 summarize the average water levels and flow velocities observed during the 100-year flow condition, respectively.

Table 21. Average water levels for TSA_2100.

Water Level Gauge	Average Water Level (m)
WL1	1050.03
WL2	1049.52
WL3	1049.29
WL4	1049.28
WL5	1048.99
WL6	1049.16
WL7	1049.29
WL8	1048.97
WL9	1049.11
WL10	1049.09
WL11	1048.88
WL12	1048.84
WL13	1048.59
WL14	1048.63

Table 22. Average horizontal velocities for TSA_2100.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.17
V2	2.74
V3	2.70
V4	3.67
V5	2.91
V6	3.37
V7	4.03

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1049.06 m. Therefore, the average water level was approximately 1.44 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 2.70 m/s and 2.91 m/s, respectively. The average velocity near the south embankment was equal to 3.17 m/s.

Following the 100-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 70 displays the resulting bathymetry and Figure 71 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 23 (positive values indicate deposition and negative values indicate erosion).

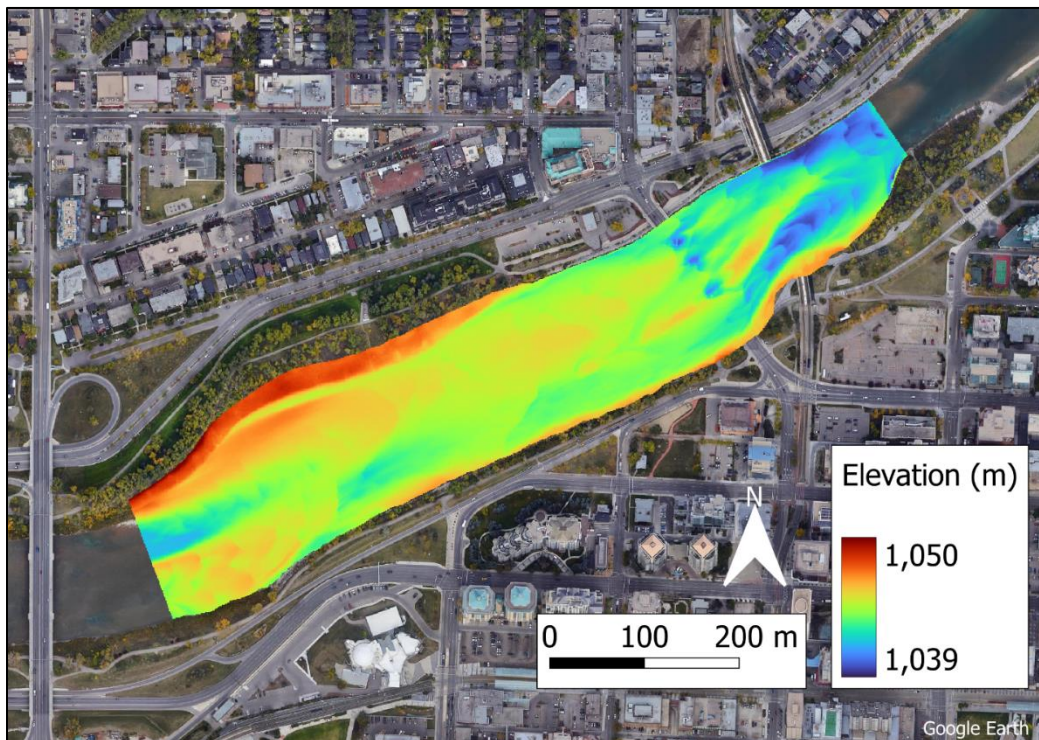


Figure 70. Bathymetry after TSA_2100.

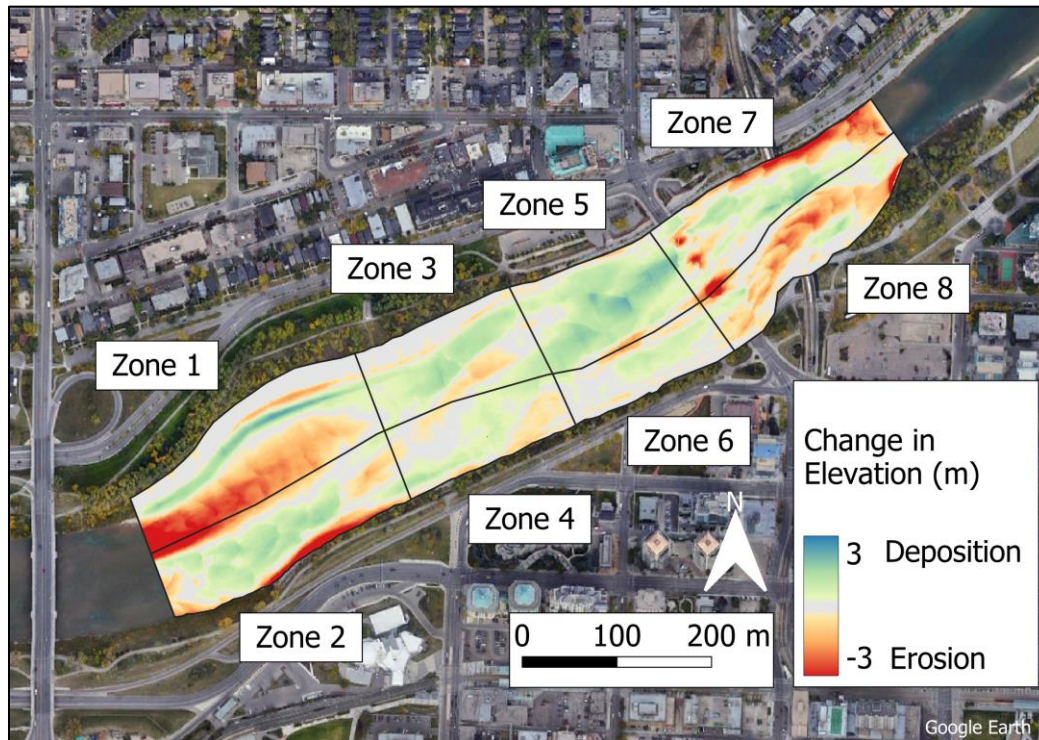


Figure 71. Change in bathymetry after TSA_2100 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 23.

Table 23. Deposition and erosion by zone after TSA_2100.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	24985	-3.83	2.49	-0.57	-14135
2	21278	-4.73	1.74	-0.12	-2537
3	16633	-1.29	1.78	0.32	5345
4	13373	-5.02	1.56	0.12	1578
5	16290	-1.87	2.37	0.78	12718
6	10140	-1.84	1.13	0.12	1206
7	16854	-5.13	2.10	-0.24	-4059
8	15827	-5.24	1.74	-0.50	-7845
Overall	135380	-5.24	2.49	-0.06	-7730

The patterns of erosion and deposition observed in TSA_2100 were generally similar to those observed in TSA_1400. Localized net deposition was observed in the habitat channel and the habitat channel crossover, and localized erosion was observed along the north edge of the habitat channel. Erosion along the south embankment appeared to be more localized to the embankment nose and the areas immediately adjacent to the embankment nose compared to the previous TSA tests. The crest of the embankment nose

was eroded resulting in failure of the feature. Unlike TSA_850 and TSA_1400, net deposition was observed at the mid-section of the south embankment (zone 4). Similar to the previous TSA tests, net deposition was observed in the areas north and south of the main bar, upstream of Louise Bridge (zones 5 and 6). Localized erosion immediately downstream of the main bar nose was greater than that observed in the previous TSA test, exacerbating the failure of the bar in this location. Localized erosion in the vicinity of bridge piers, and adjacent to the landing was greater than that observed in TSA_850 and TSA_1400. Figure 72 shows a photo of the model during TSA_2100 and Figure 73 shows select photos of the model after TSA_2100.



Figure 72. Photo of the model during TSA_2100.



Figure 73. Photos of the model after TSA_2100; a) standing on Louise Bridge, looking upstream at the main bar, b) standing at the north side of the downstream boundary, looking upstream, c) standing at Poppy Plaza, looking upstream, d) standing near the south side of the upstream boundary, looking downstream.

5.4.4 Test Series A Cross-Sections

Bathymetry profiles were extracted at three cross-sections as described in Section 3.5.4. Figure 74, Figure 75, and Figure 76 show the bathymetry profiles for all tests in TSA for Cross-Section 1, 2, and 3, respectively.

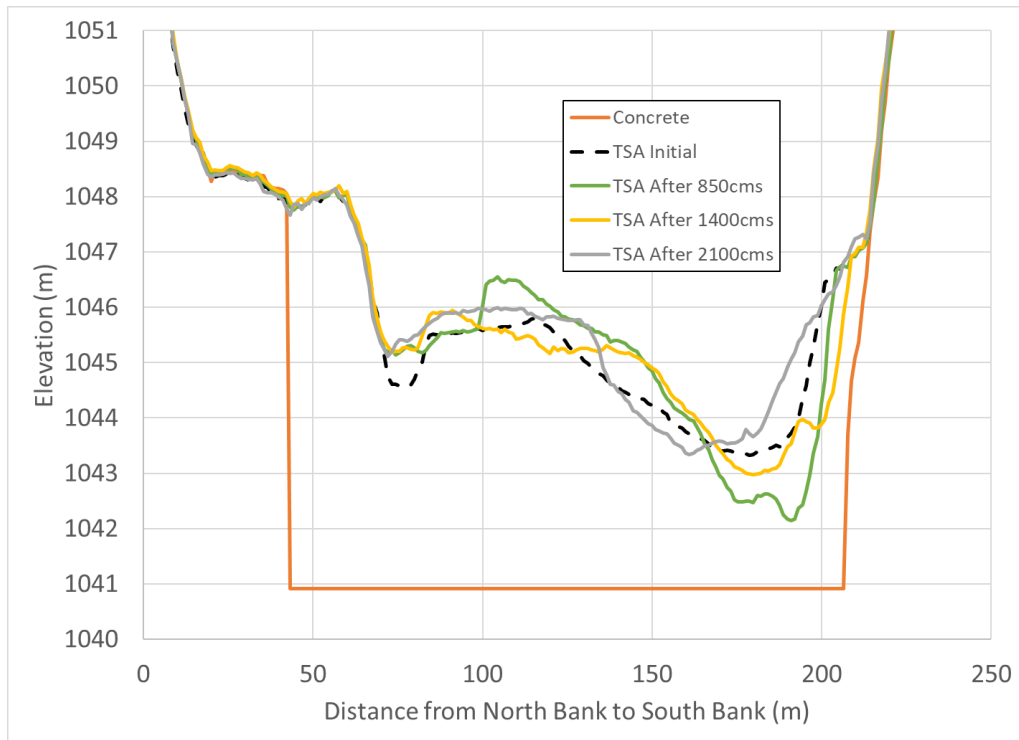


Figure 74. Bathymetry profiles for TSA at Cross-Section 1.

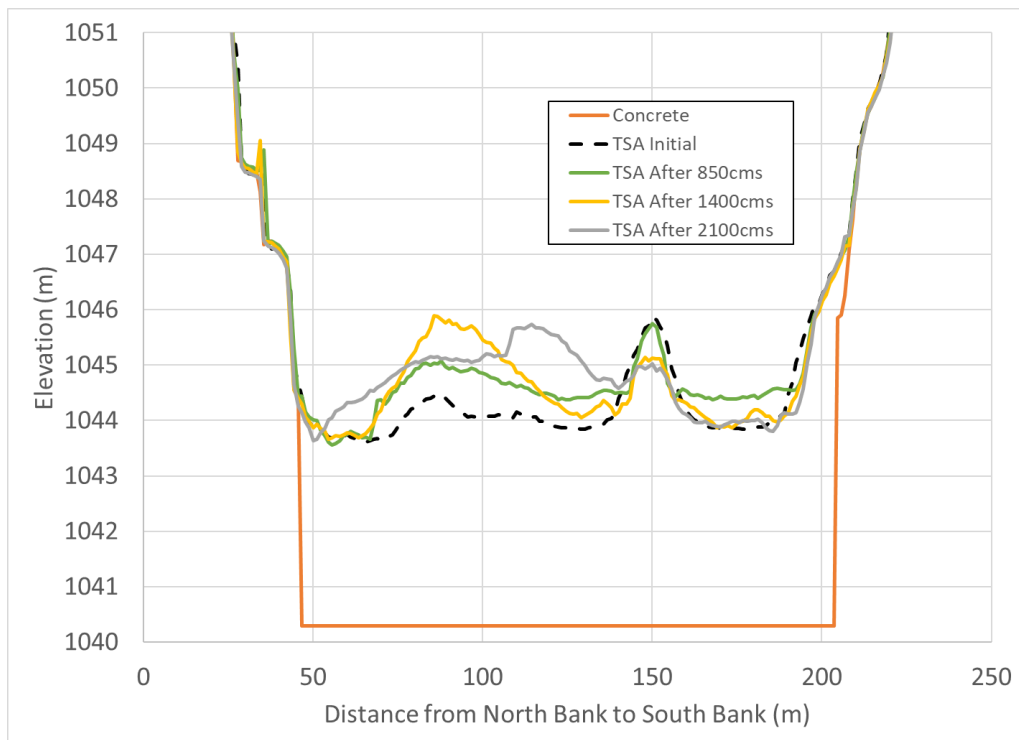


Figure 75. Bathymetry profiles for TSA at Cross-Section 2.

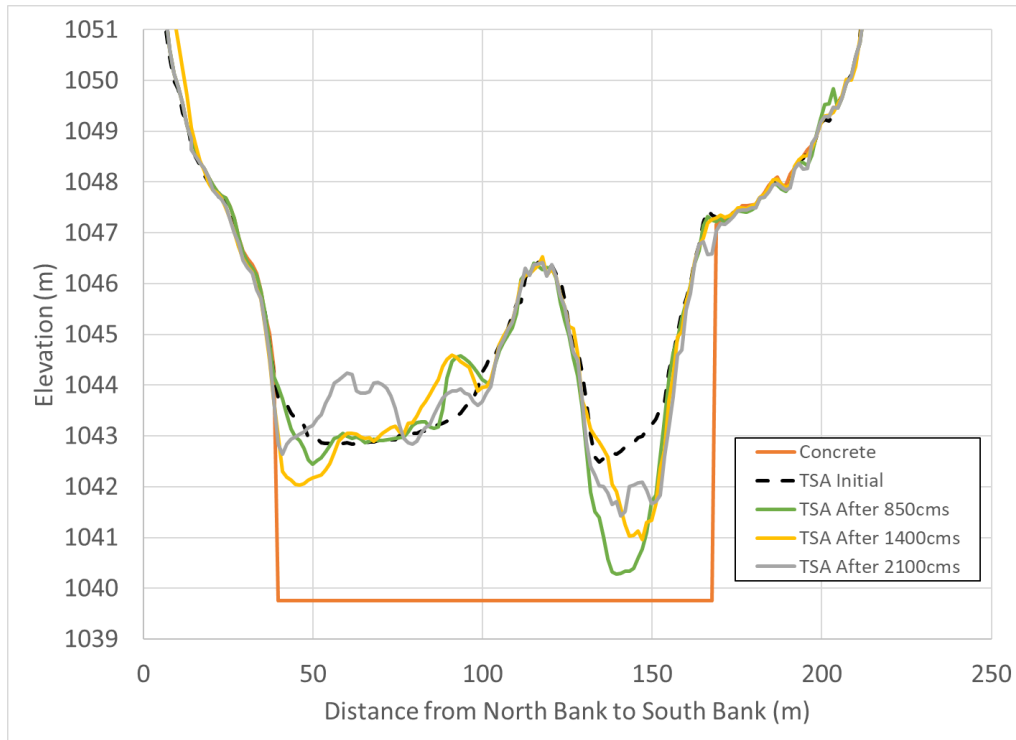


Figure 76. Bathymetry profiles for TSA at Cross-Section 3.

5.5 Test Series B – Design #2

Test Series B (TSB) was focused on evaluating Conceptual Design #2 under the 10-, 35-, and 100-year flow condition. The following subsections summarize observed water levels, flow velocities, and geomorphic change. As described in Section 3.5.4, the model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition. Figure 77 displays the as-built initial bathymetry for TSB and includes labels for each zone. Figure 78 displays the difference between the as-built initial bathymetry for TSB and the design bathymetry prescribed by Matrix Solutions Inc (see Appendix A).

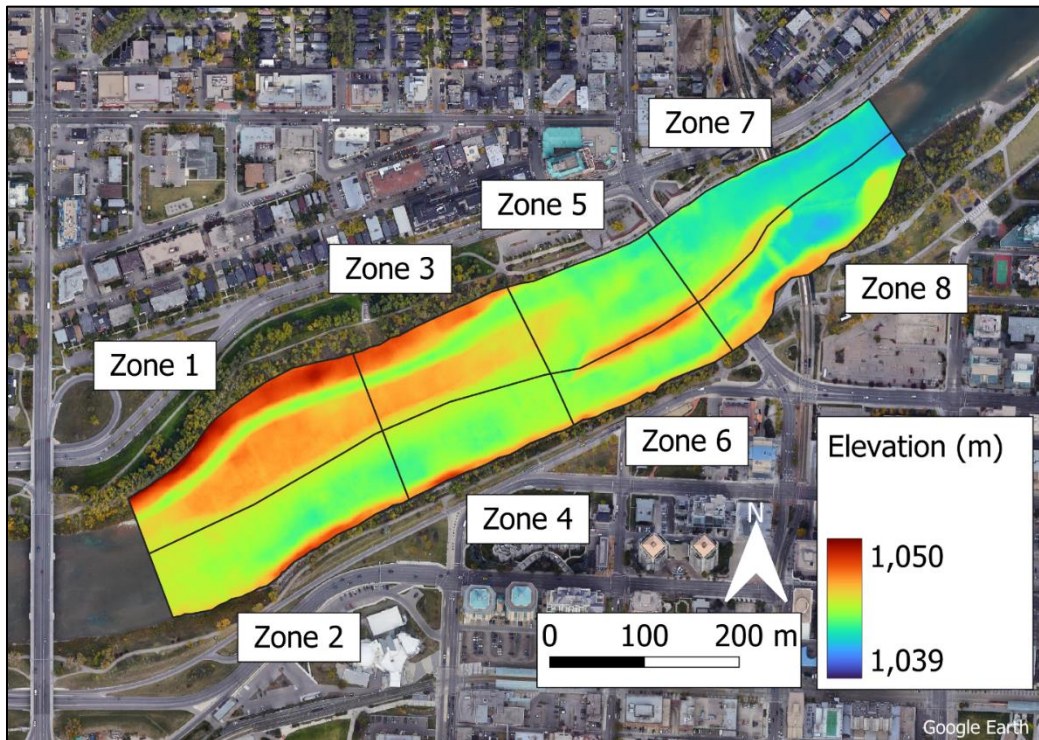


Figure 77. As-built initial bathymetry for TSB.

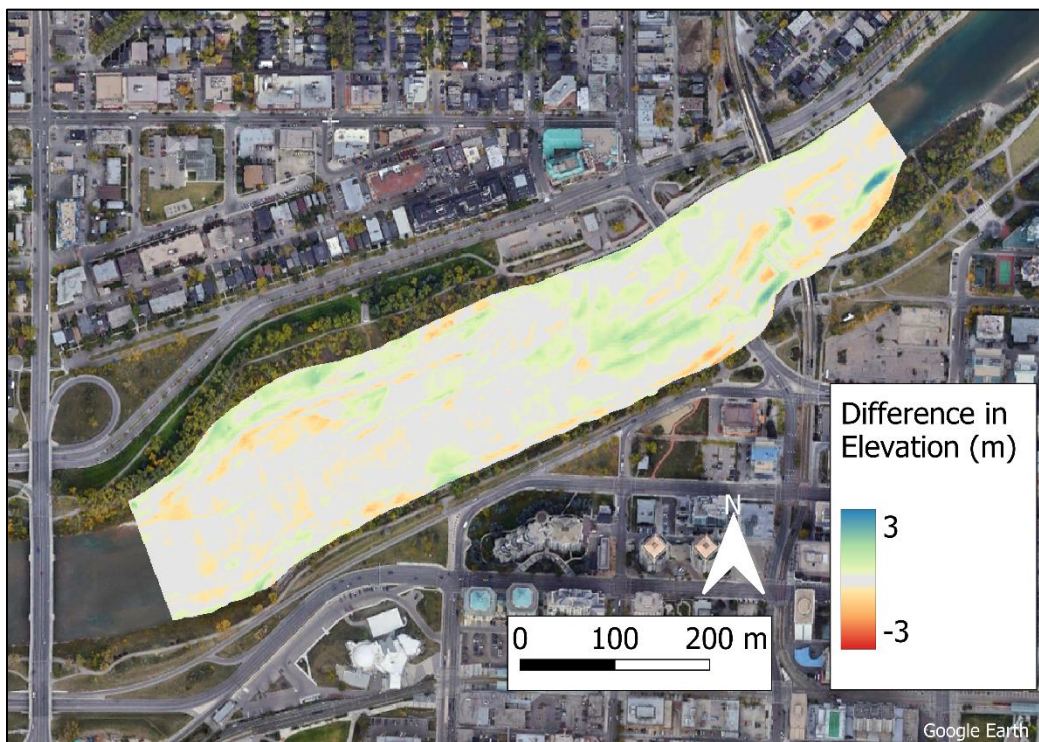


Figure 78. As-built initial bathymetry for TSB minus the design bathymetry prescribed by Matrix Solutions Inc. Differences in elevation between -0.15 m and 0.15 m are shown in grey.

5.5.1 10-Year Flow Condition (850 m³/s)

Conceptual Design #2 was first subjected to a 10-year flow condition with a flow equal to 850 m³/s for a duration of 30 hours. Table 24 and Table 25 summarize the average water levels and flow velocities observed during the 10-year flow condition, respectively.

Table 24. Average water levels for TSB_850.

Water Level Gauge	Average Water Level (m)
WL1	1048.02
WL2	-
WL3	1046.87
WL4	1047.07
WL5	1046.72
WL6	1046.60
WL7	1047.01
WL8	1046.59
WL9	-
WL10	1046.78
WL11	1046.54
WL12	-
WL13	1045.94
WL14	1046.13

Table 25. Average horizontal velocities for TSB_850.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.83
V2	1.87
V3	3.20
V4	-
V5	-
V6	3.30
V7	2.58

The average water level immediately upstream of Louise Bridge (WL8 and WL10) was equal to 1046.69 m. Therefore, the average water level was approximately 3.81 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north side of the main bar approaching Louise Bridge was equal to 3.20 m/s. The velocity south of the main bar approaching Louise Bridge could not be measured owing to shallow water conditions. The average velocity near the south embankment was equal to 3.83 m/s.

Following the 10-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 79 displays the resulting bathymetry and Figure 80 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 26 (positive values indicate deposition and negative values indicate erosion).

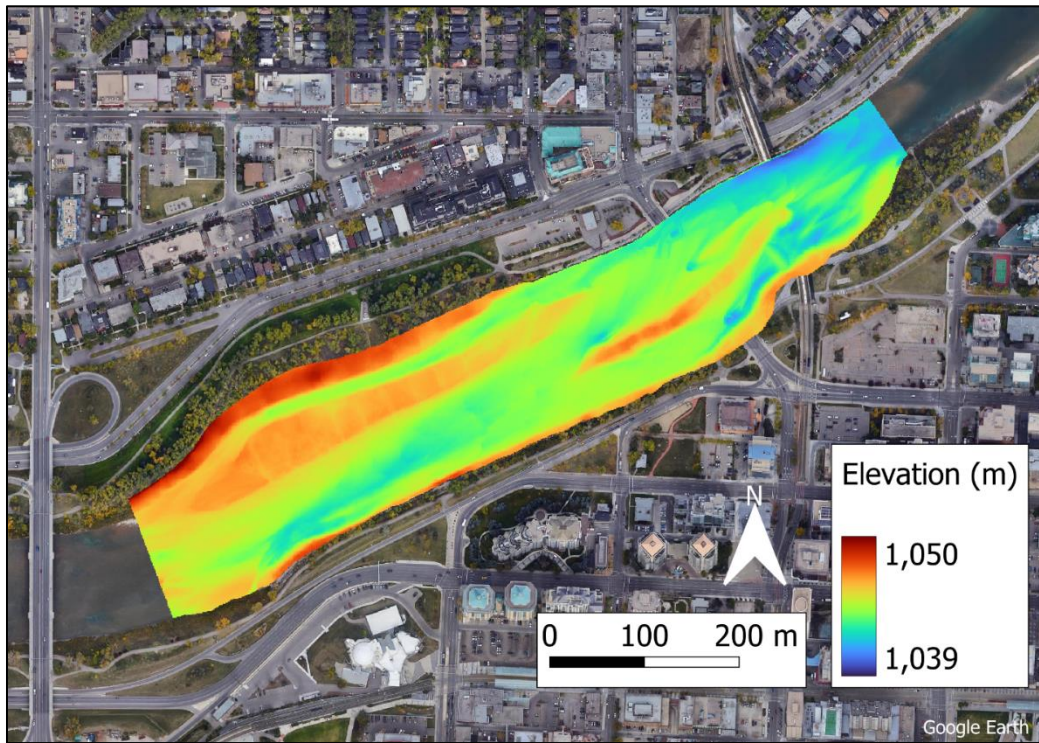


Figure 79. Bathymetry after TSB_850.

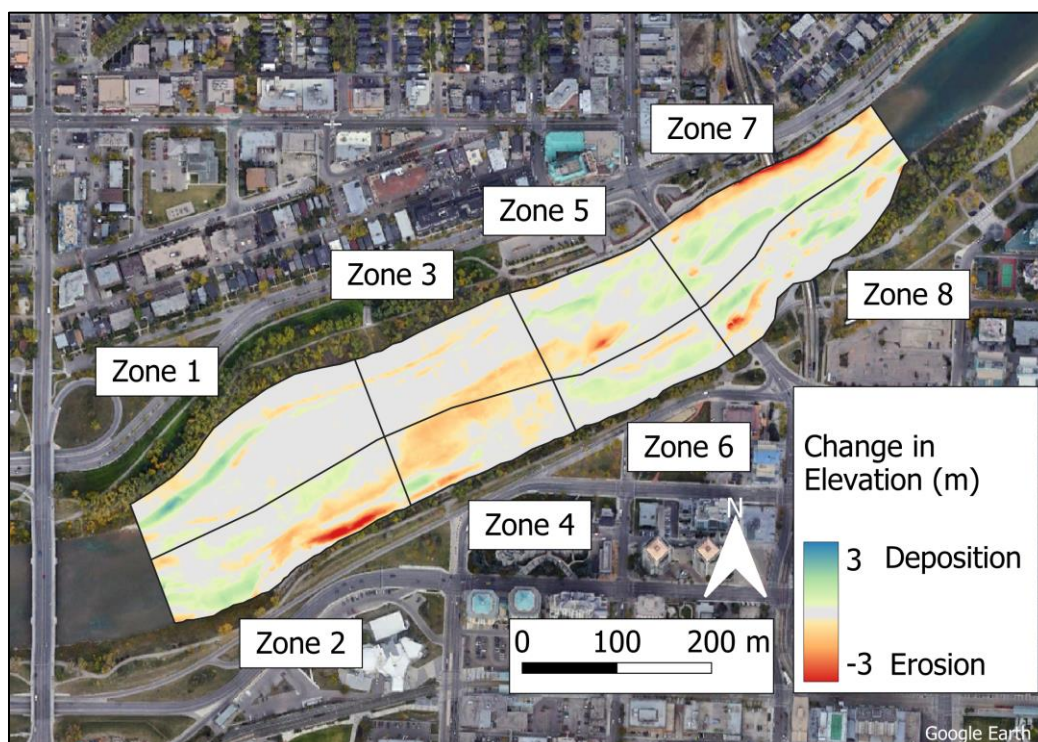


Figure 80. Change in bathymetry after TSB_850. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 26.

Table 26. Deposition and erosion by zone after TSB_850.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25022	-0.96	2.35	0.05	1133
2	21287	-3.83	1.79	-0.10	-2099
3	16639	-1.22	0.45	-0.20	-3251
4	13373	-2.01	1.51	-0.35	-4695
5	16306	-2.62	1.50	0.04	610
6	10137	-1.02	1.04	0.15	1532
7	16865	-3.71	1.63	-0.09	-1556
8	15769	-3.08	1.79	0.08	1313
Overall	135398	-3.83	2.35	-0.05	-7014

The scan data indicated localized sediment deposition in the habitat channel. Localized erosion of the south embankment was observed, immediately downstream of the armoured embankment nose; repelling groyne #1 is situated at the downstream end of the armoured embankment nose. Localized deposition was observed at the tip of repelling groyne #2. However, the thin layer of river sediment that initially covered the crests of the repelling groynes (repelling groynes #1, #2, and #3) was eroded during TSB_850, particularly

where no additional protection was provided by the bioengineering stone. Net erosion was observed on the north and south side of the channel leading up to the main bar (zone 3 and 4). Except for some localized erosion immediately downstream of the main bar nose, deposition was generally observed along the north edge of the main bar. Net deposition was observed in the upstream portion of the recreation channel (zone 6), although some localized erosion was observed along the south edge of the main bar. Localized erosion was observed underneath Louise Bridge, immediately adjacent to the landing. Figure 81 shows a photo of the model during TSB_850 and Figure 82 shows select photos of the model after TSB_850.



Figure 81. Photo of the model during TSB_850.



Figure 82. Photos of the model after TSB_850; a) standing at Poppy Plaza, looking across the channel and downstream at the main bar, b) standing at the south side of the downstream boundary, looking upstream, c) standing at Poppy Plaza, looking upstream, d) standing at the north side of the upstream boundary, looking at the south embankment.

5.5.2 35-Year Flow Condition (1400 m³/s)

Following the 10-year flow condition, Conceptual Design #2 was subjected to a 35-year flow condition with a flow equal to 1400 m³/s for a duration of 30 hours. Table 27 and Table 28 summarize the average water levels and flow velocities observed during the 35-year flow condition, respectively.

Table 27. Average water levels for TSB_1400.

Water Level Gauge	Average Water Level (m)
WL1	1049.14
WL2	-
WL3	1047.96
WL4	1048.12
WL5	1047.84
WL6	1047.77
WL7	1048.04
WL8	1047.69
WL9	1047.81
WL10	1047.87
WL11	1047.69
WL12	1047.45
WL13	1047.14
WL14	1047.28

Table 28. Average horizontal velocities for TSB_1400.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.79
V2	2.72
V3	3.07
V4	-
V5	3.11
V6	3.39
V7	3.64

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1047.79 m. Therefore, the average water level was approximately 2.71 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 3.07 m/s and 3.11 m/s, respectively. The average velocity near the south embankment was equal to 3.79 m/s.

Following the 35-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 83 displays the resulting bathymetry and Figure 84 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 29 (positive values indicate deposition and negative values indicate erosion).

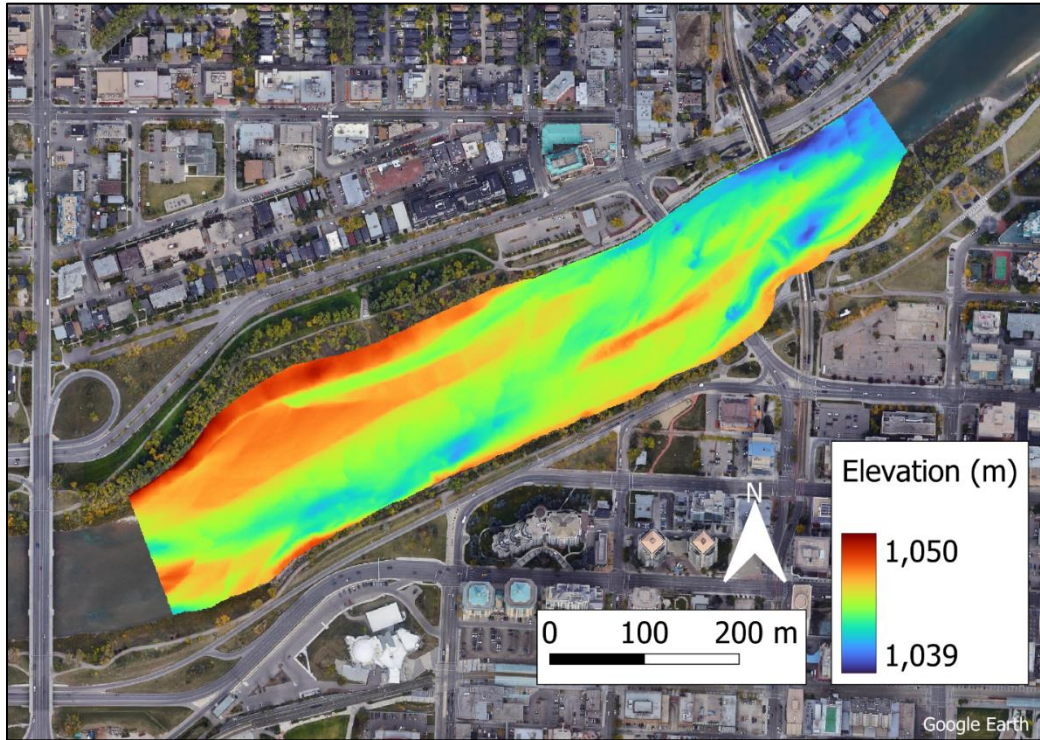


Figure 83. Bathymetry after TSB_1400.

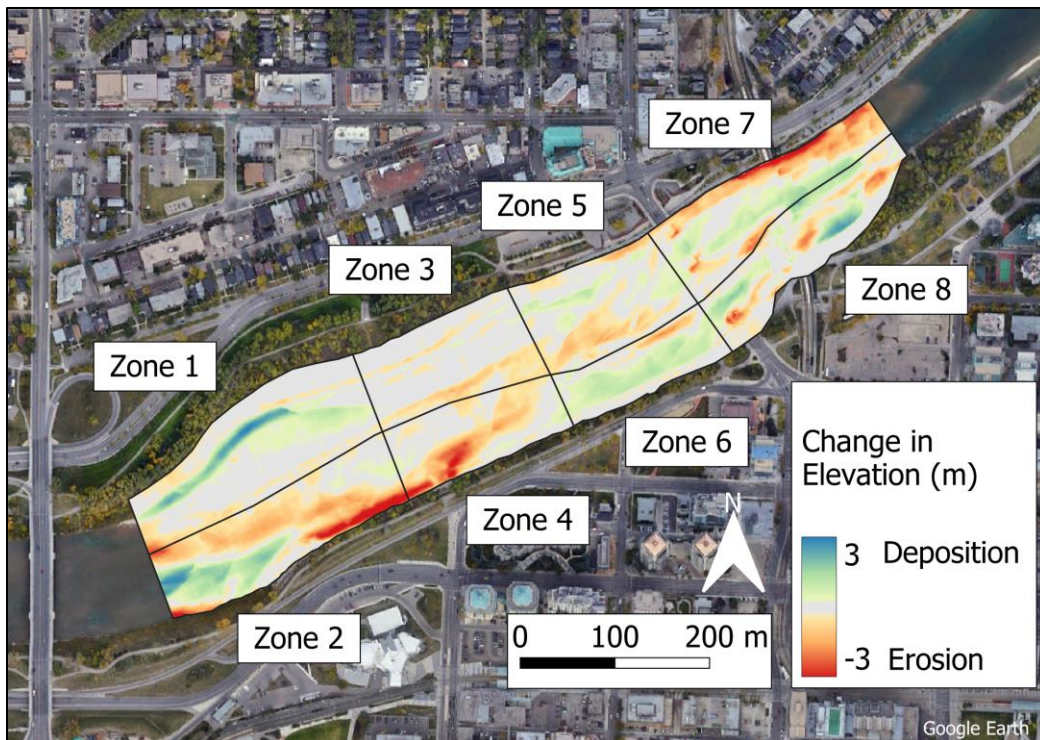


Figure 84. Change in bathymetry after TSB_1400 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 29.

Table 29. Deposition and erosion by zone after TSB_1400.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25022	-2.57	3.03	0.16	4078
2	21300	-4.37	2.92	-0.30	-6418
3	16639	-1.46	0.59	-0.18	-2964
4	13383	-3.39	0.86	-0.51	-6777
5	16306	-1.64	1.23	-0.13	-2133
6	10152	-1.92	1.42	0.29	2935
7	16862	-4.54	2.11	-0.33	-5532
8	15777	-2.84	2.69	0.07	1088
Overall	135441	-4.54	3.03	-0.12	-15722

The general patterns of erosion and deposition observed in TSB_1400 were generally similar to the patterns observed in TSB_850. Additional deposition was observed in the habitat channel. Additional erosion was observed downstream of the south embankment nose and in the vicinity of repelling groyne #2. Similar to TSB_850, localized erosion was observed on the north edge of the main bar, immediately downstream of the main bar nose. The upstream portion of the recreation channel, leading up to Louise Bridge, continued to be an area of net deposition (zone 6). However, similar to TSB_850, net erosion was observed along the south edge of the main bar. Localized erosion was observed at the crest of the main bar between Louise Bridge and the LRT bridge. Figure 85 shows a photo of the model during TSB_1400 and Figure 86 shows select photos of the model after TSB_1400.

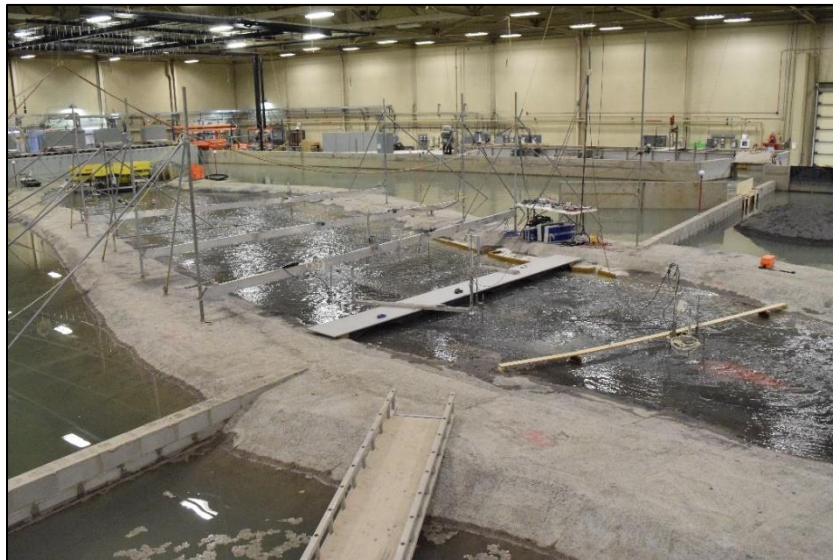


Figure 85. Photo of the model during TSB_1400.



Figure 86. Photos of the model after TSB_1400; a) standing at the south embankment, looking downstream, b) standing at Poppy Plaza, looking across the channel and downstream at the main bar, c) standing near the north abutment of the LRT bridge, looking upstream, and d) standing at Poppy Plaza, looking upstream.

5.5.3 100-Year flow Condition (2100 m³/s)

Following the 35-year flow condition, Conceptual Design #2 was subjected to a 100-year flow condition with a flow equal to 2100 m³/s for a duration of 30 hours. Table 30 and Table 31 summarize the average water levels and flow velocities observed during the 100-year flow condition, respectively.

Table 30. Average water levels for TSB_2100.

Water Level Gauge	Average Water Level (m)
WL1	1049.96
WL2	1049.52
WL3	1048.90
WL4	1049.06
WL5	1048.74
WL6	1048.70
WL7	1049.00
WL8	1048.68
WL9	1048.84
WL10	1048.76
WL11	1048.63
WL12	1048.42
WL13	1048.21
WL14	1048.28

Table 31. Average horizontal velocities for TSB_2100.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.65
V2	2.77
V3	3.28
V4	-
V5	-
V6	3.07
V7	3.43

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1048.76 m. Therefore, the average water level was approximately 1.74 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north side of the main bar approaching Louise Bridge was equal to 3.28 m/s. The average velocity near the south embankment was equal to 3.65 m/s.

Following the 100-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 87 displays the resulting bathymetry and Figure 88 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 32 (positive values indicate deposition and negative values indicate erosion).

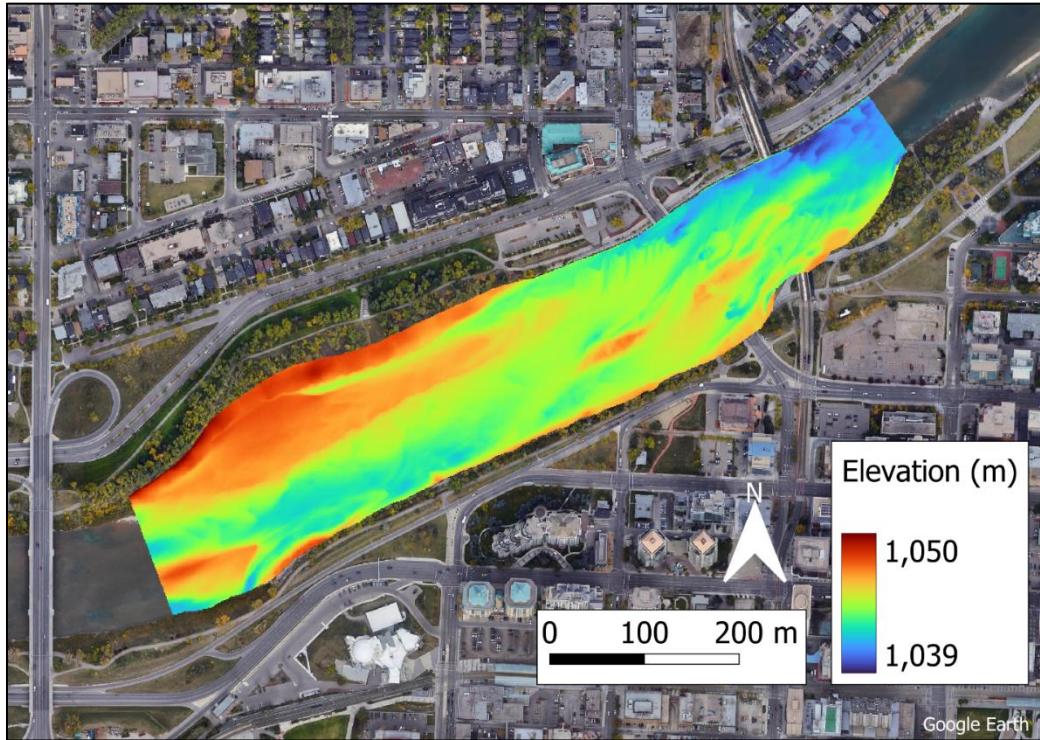


Figure 87. Bathymetry after TSB_2100.

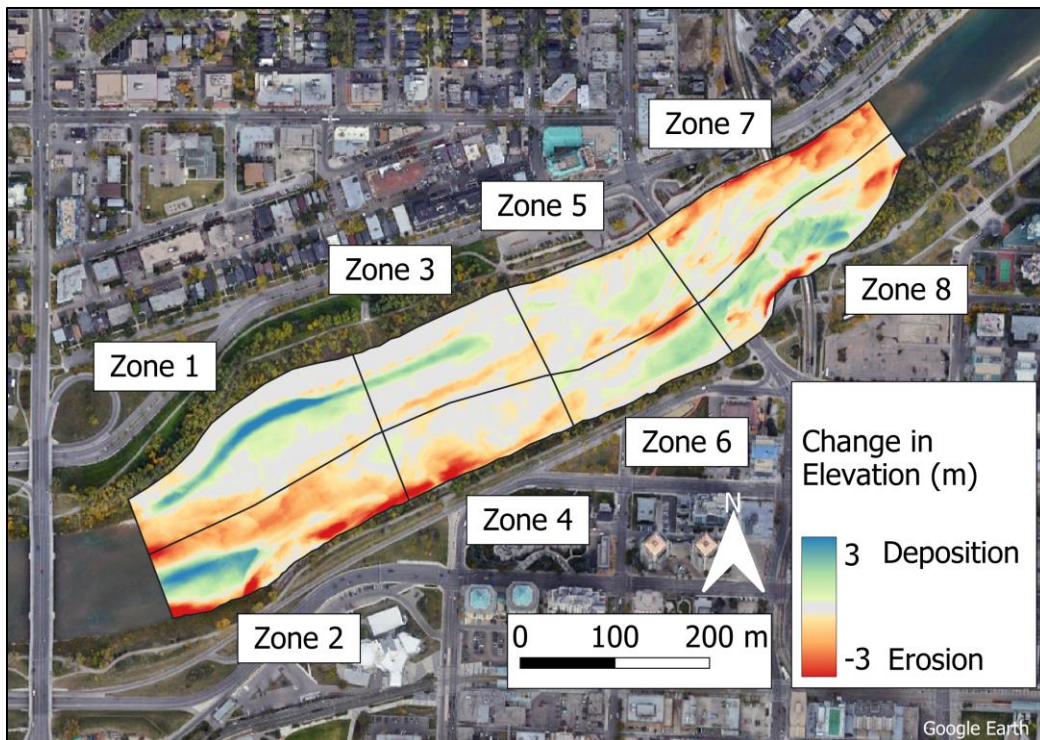


Figure 88. Change in bathymetry after TSB_2100 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 32.

Table 32. Deposition and erosion by zone after TSB_2100.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25009	-3.35	3.59	0.22	5380
2	21384	-4.32	3.12	-0.55	-11813
3	16639	-1.50	1.95	0.01	210
4	13445	-3.50	0.53	-0.52	-6962
5	16306	-2.70	1.21	-0.01	-125
6	10204	-2.75	1.79	0.14	1453
7	16871	-4.39	1.64	-0.63	-10576
8	15916	-4.62	2.64	0.16	2544
Overall	135775	-4.62	3.59	-0.15	-19890

Similar to the previous TSB tests, sediment deposition was observed in the habitat channel, erosion was observed along the south embankment, and net deposition was observed in the recreation channel upstream of Louise Bridge (zone 6). Localized erosion was observed immediately upstream of the armoured nose of the south embankment. A section of the main bar upstream of Louise Bridge failed during the 100-year flow, leading to localized erosion in the vicinity of the failure. Similar to TSB_1400 localized, net erosion was observed at the crest of the main bar between Louise Bridge and the LRT bridge. Deposition was observed in the recreation channel downstream of Louise Bridge, both upstream and downstream of the concrete hump (zone 8). Erosion was observed on the crest of the landing resulting in a failure of the feature. Figure 89 shows a photo of the model during TSB_2100 and Figure 90 shows select photos of the model after TSB_2100.



Figure 89. Photo of the model during TSB_2100.

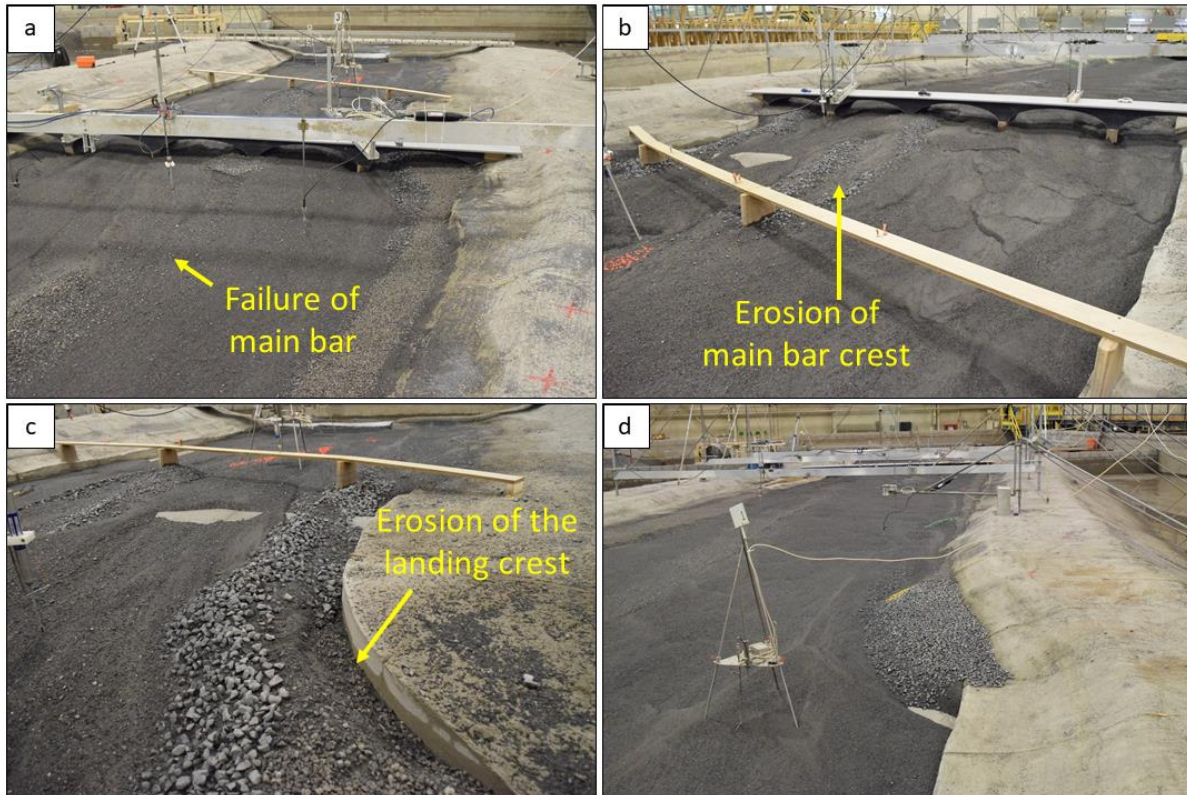


Figure 90. Photos of the model after TSB_2100; a) standing at the south embankment, looking downstream, b) standing near the north abutment of the LRT bridge, looking upstream, c) standing near the south abutment of Louise Bridge, looking downstream at the landing, and d) standing at the south side of the upstream boundary, looking downstream.

5.5.4 Test Series B Cross-Sections

Bathymetry profiles were extracted at three cross-sections as described in Section 3.5.4. Figure 91, Figure 92, and Figure 93 show the bathymetry profiles for all tests in TSB for Cross-Section 1, 2, and 3, respectively.

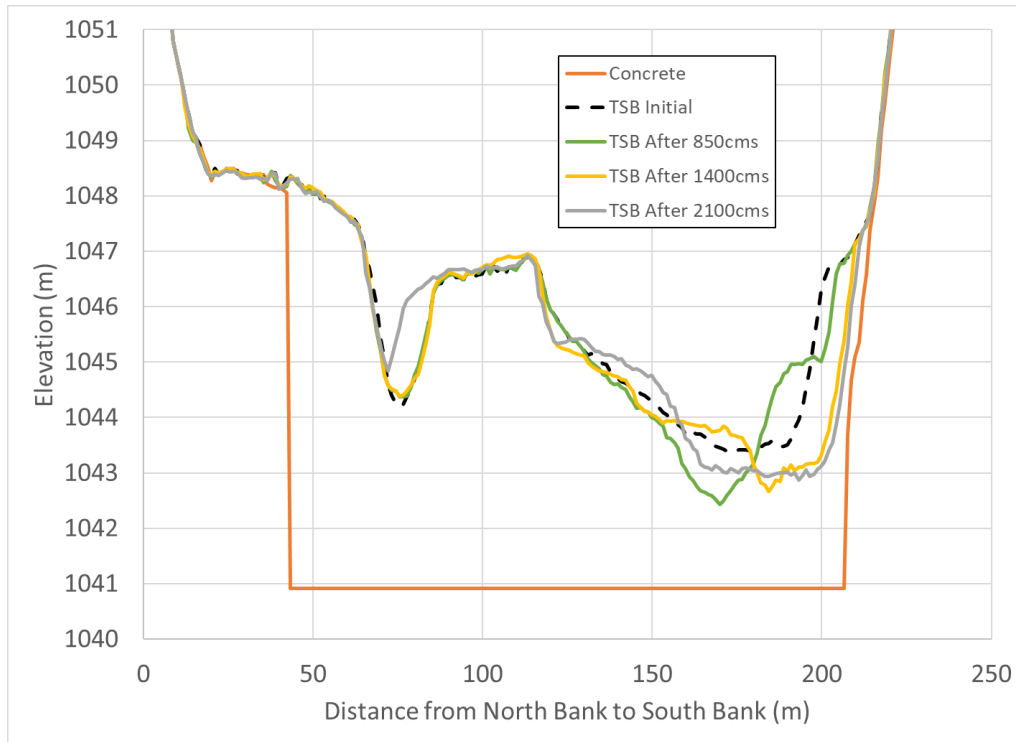


Figure 91. Bathymetry profiles for TSB at Cross-Section 1.

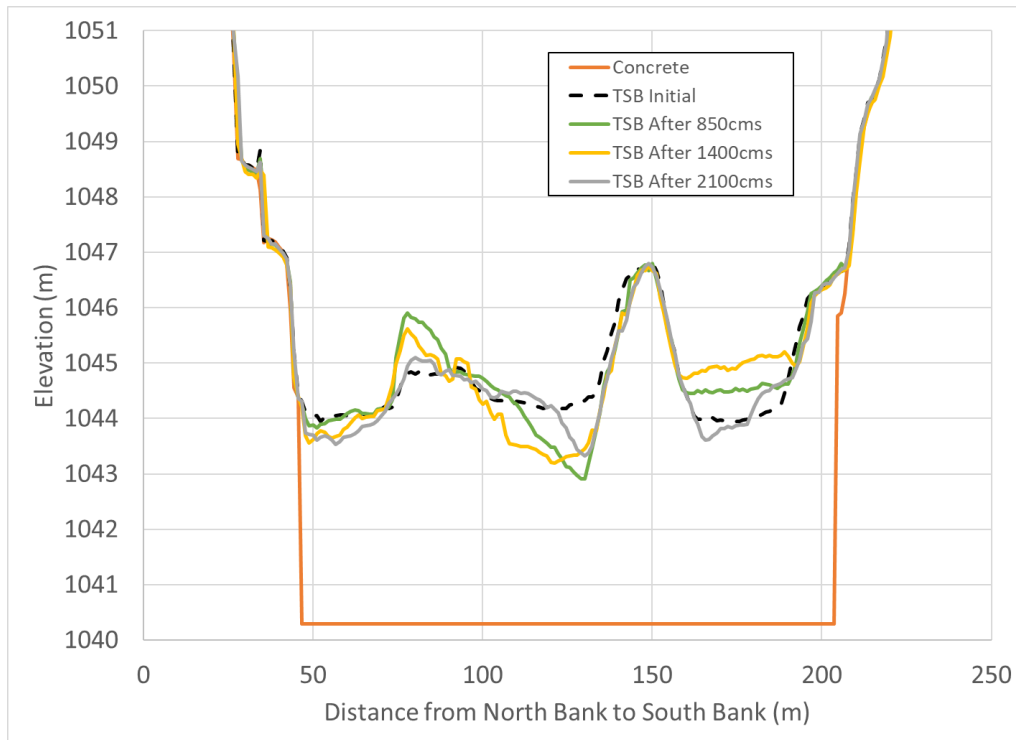


Figure 92. Bathymetry profiles for TSB at Cross-Section 2.

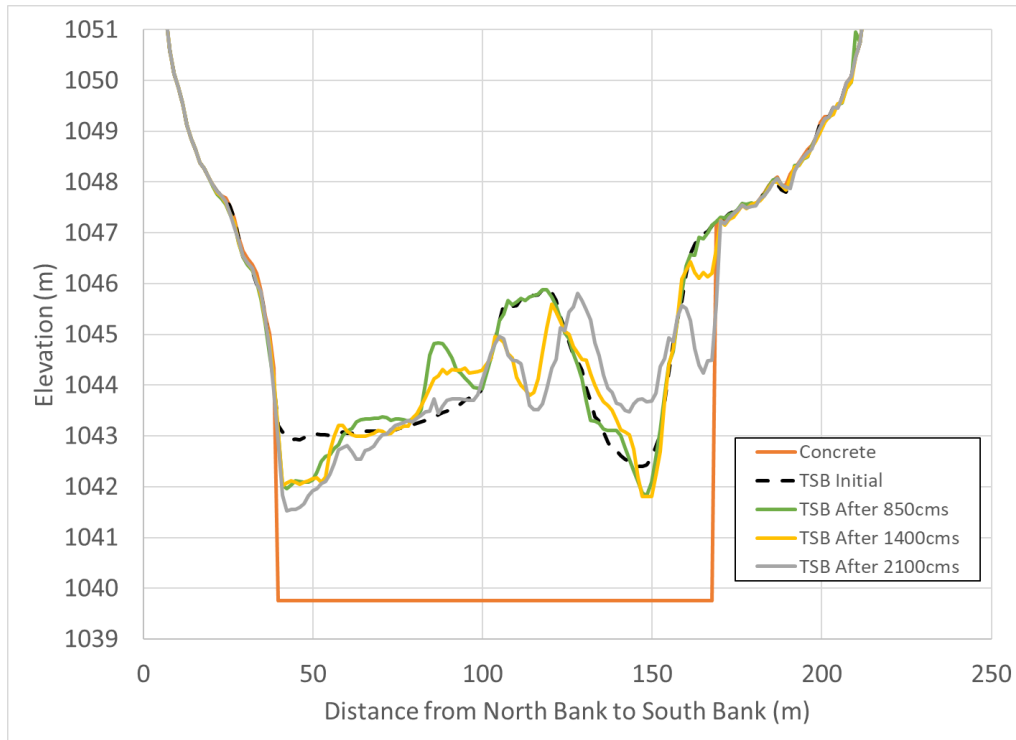


Figure 93. Bathymetry profiles for TSB at Cross-Section 3.

5.6 Test Series C – Design #3

Test Series C (TSC) was focused on evaluating Conceptual Design #3 under the 10-, 35-, and 100-year flow condition. The following subsections summarize observed water levels, flow velocities, and geomorphic change. As described in Section 3.5.4, the model domain was divided into eight zones to assist with the interpretation of the spatial distribution of erosion and deposition. Figure 94 displays the as-built initial bathymetry for TSC and includes labels for each zone. Figure 95 displays the difference between the as-built initial bathymetry for TSC and the design bathymetry prescribed by Matrix Solutions Inc (see Appendix A). The discrepancy visible in Figure 95 in the vicinity of the concrete hump, situated within the recreation channel, does not accurately reflect the discrepancy of the as-built initial bathymetry. For Conceptual Design #3, NRC-OCRE was instructed to lower the crest elevation of the concrete hump to be flush with the surrounding bathymetry (see design note on Conceptual Design #3 in Appendix A). However, this modification was not reflected in the design DEM. Therefore, the crest elevation of the concrete hump shown in the design DEM (Appendix A) is greater than the prescribed crest elevation, and this explains the discrepancy.

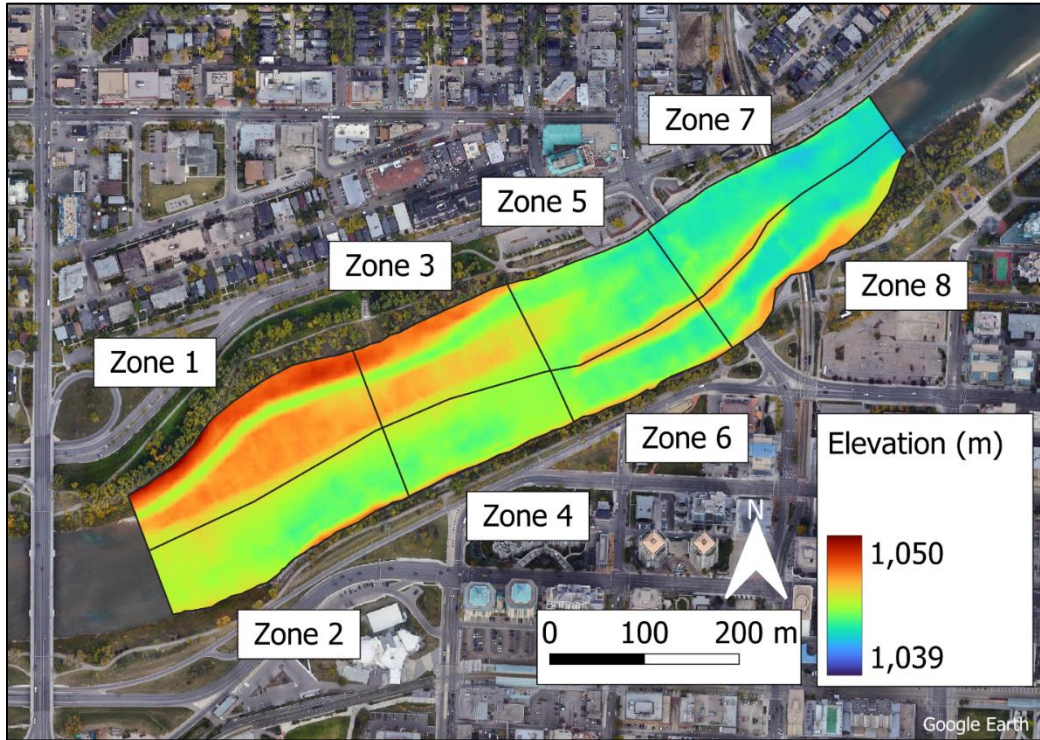


Figure 94. As-built initial bathymetry for TSC.

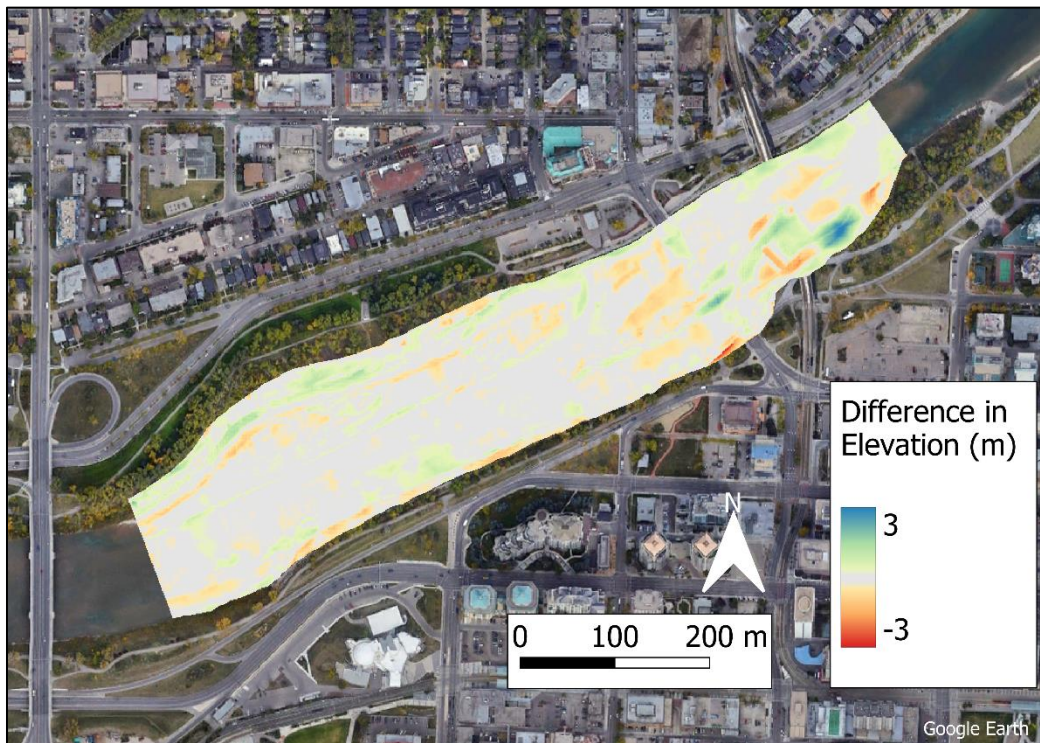


Figure 95. As-built initial bathymetry for TSC minus the design bathymetry prescribed by Matrix Solutions Inc. Differences in elevation between -0.15 m and 0.15 m are shown in grey. The discrepancy visible in the vicinity of the concrete hump, situated within the recreation channel, does not accurately reflect the discrepancy of the as-built initial bathymetry (see Section 5.6).

5.6.1 10-Year Flow Condition (850 m³/s)

Conceptual Design #3 was first subjected to a 10-year flow condition with a flow equal to 850 m³/s for a duration of 30 hours. Table 33 and Table 34 summarize the average water levels and flow velocities observed during the 10-year flow condition, respectively.

Table 33. Average water levels for TSC_850.

Water Level Gauge	Average Water Level (m)
WL1	1047.92
WL2	1047.33
WL3	1046.72
WL4	1046.79
WL5	1046.47
WL6	1046.57
WL7	1046.84
WL8	1046.29
WL9	1046.35
WL10	1046.39
WL11	1046.16
WL12	1046.27
WL13	1046.33
WL14	-

Table 34. Average horizontal velocities for TSC_850.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	3.99
V2	2.36
V3	-
V4	-
V5	-
V6	3.50
V7	3.10

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1046.34 m. Therefore, the average water level was approximately 4.16 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity north and south of the main bar could not be measured owing to shallow water conditions. The average velocity near the south embankment was equal to 3.99 m/s.

Following the 10-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 96 displays the resulting bathymetry and Figure 97 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 35 (positive values indicate deposition and negative values indicate erosion).

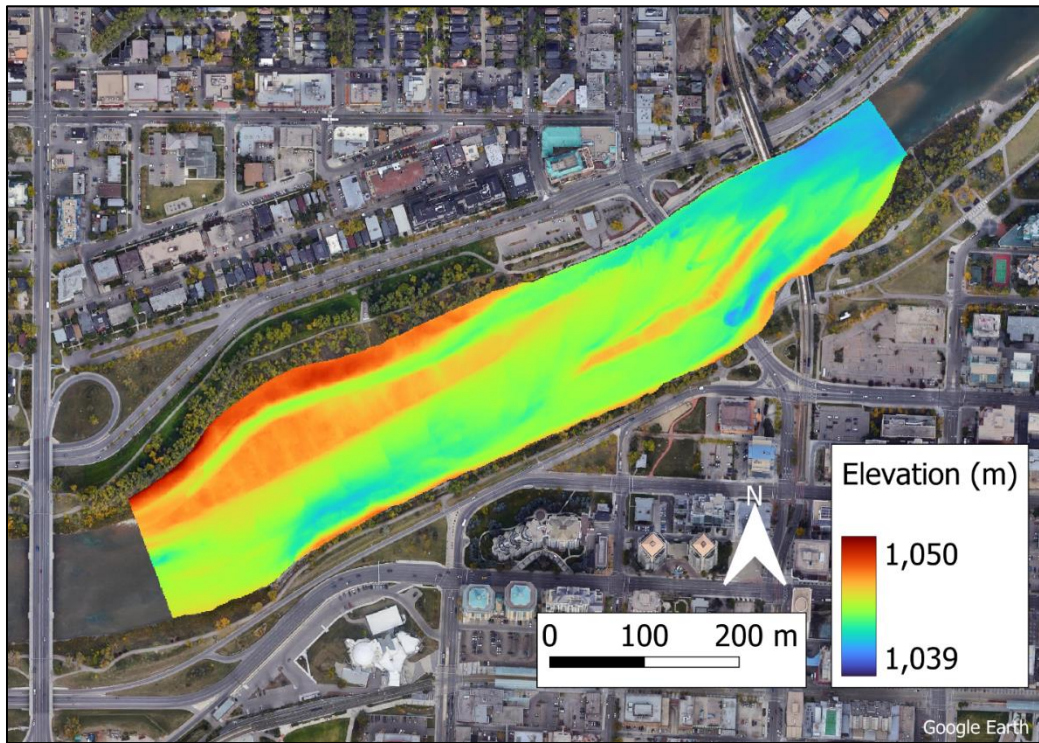


Figure 96. Bathymetry after TSC_850.

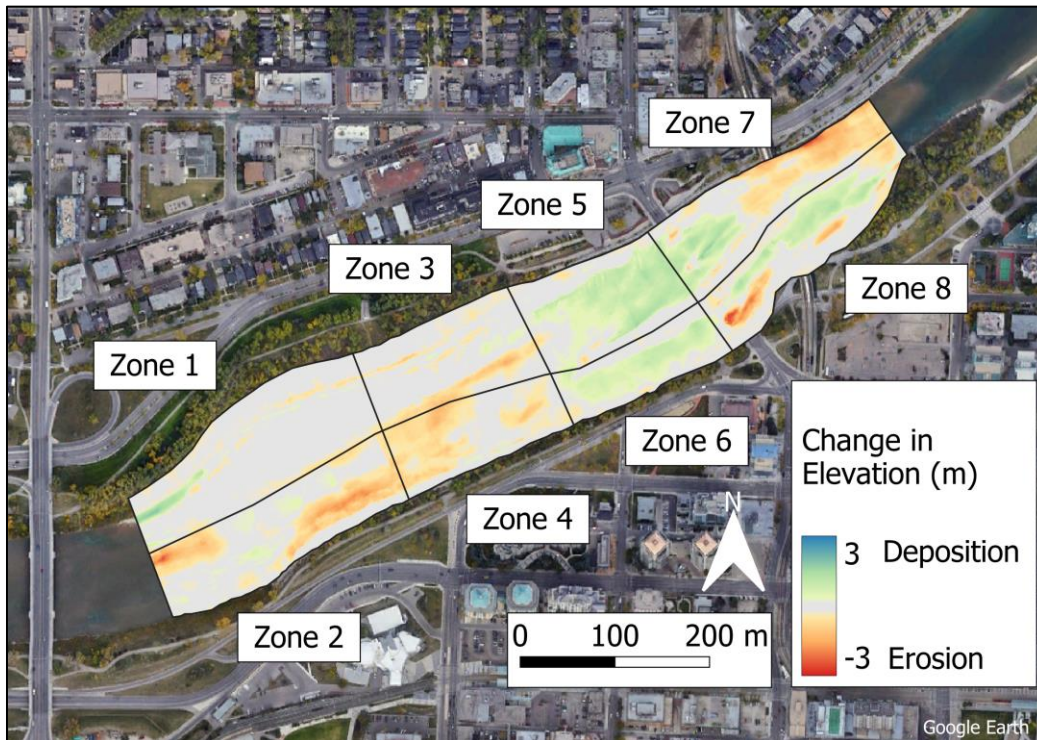


Figure 97. Change in bathymetry after TSC_850. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 35.

Table 35. Deposition and erosion by zone after TSC_850.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25027	-0.79	1.92	-0.03	-856
2	21337	-2.44	0.68	-0.27	-5798
3	16642	-1.51	0.63	-0.17	-2883
4	13408	-1.36	0.81	-0.33	-4430
5	16305	-0.82	1.70	0.43	7083
6	10171	-0.59	1.35	0.30	3064
7	16843	-3.00	1.84	-0.13	-2195
8	15807	-2.61	1.47	0.02	363
Overall	135540	-3.00	1.92	-0.04	-5651

The scan data indicated some localized sediment deposition in the habitat channel, although the deposition was not as extensive as the deposition observed for the 10-year flow tests for TSA (TSA_850) and TSB (TSB_850). Erosion was observed along the south embankment upstream of the main bar (zones 2 and 4) and deposition was observed north and south of the main bar upstream of Louise Bridge (zones 5 and 6). Deposition was also observed north of the main bar underneath, and immediately downstream of, Louise

Bridge. Deposition was generally observed along the south edge of the main bar downstream of Louise Bridge. Localized erosion was observed adjacent to the landing. Figure 98 shows a photo of the model during TSC_850 and Figure 99 shows select photos of the model after TSC_850.



Figure 98. Photo of the model during TSC_850.



Figure 99. Photos of the model after TSC_850; a) standing at the south embankment, looking downstream, b) standing at Poppy Plaza, looking downstream, c) standing at the downstream boundary, looking upstream, and d) standing at the north side of the upstream boundary, looking downstream.

5.6.2 35-Year Flow Condition (1400 m³/s)

Following the 10-year flow condition, Conceptual Design #3 was subjected to a 35-year flow condition with a flow equal to 1400 m³/s for a duration of 30 hours. Table 36 and Table 37 summarize the average water levels and flow velocities observed during the 35-year flow condition, respectively.

Table 36. Average water levels for TSC_1400.

Water Level Gauge	Average Water Level (m)
WL1	1049.09
WL2	1048.52
WL3	1047.94
WL4	1048.02
WL5	1047.75
WL6	1047.86
WL7	1048.10
WL8	1047.52
WL9	1047.63
WL10	1047.63
WL11	1047.52
WL12	1047.35
WL13	1047.20
WL14	-

Table 37. Average horizontal velocities for TSC_1400.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	4.02
V2	2.65
V3	2.71
V4	-
V5	3.18
V6	3.29
V7	3.73

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1047.59 m. Therefore, the average water level was approximately 2.91 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 2.71 m/s and 3.18 m/s, respectively. The average velocity near the south embankment was equal to 4.02 m/s.

Following the 35-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 100 displays the resulting bathymetry and Figure 101 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 38 (positive values indicate deposition and negative values indicate erosion).

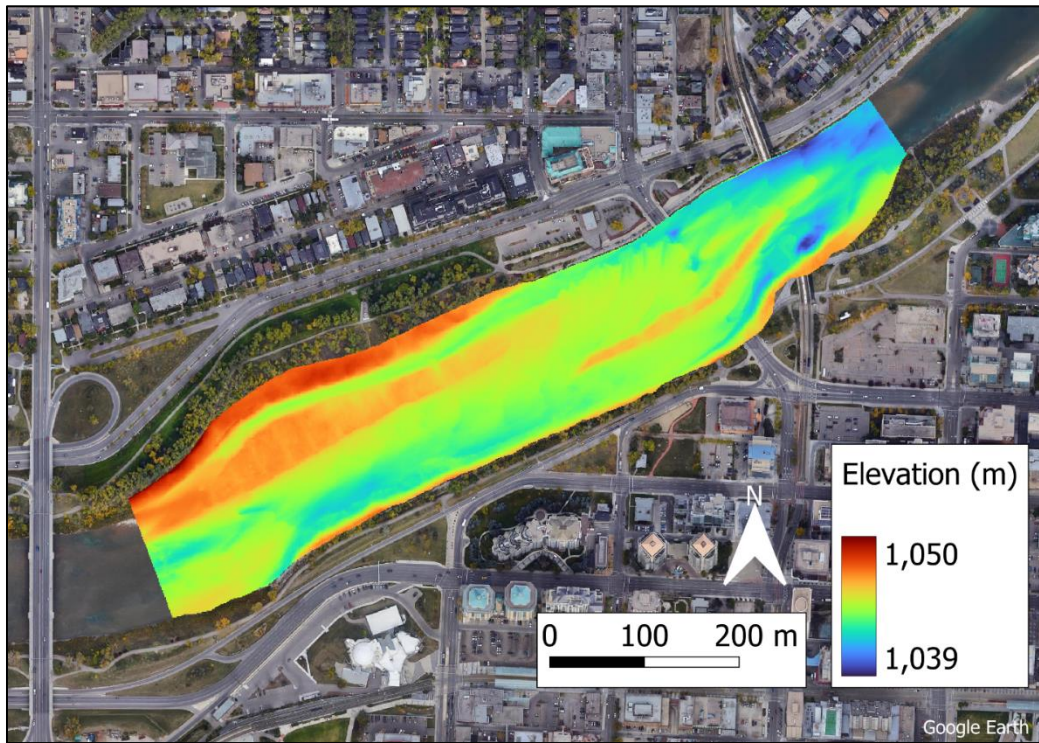


Figure 100. Bathymetry after TSC_1400.

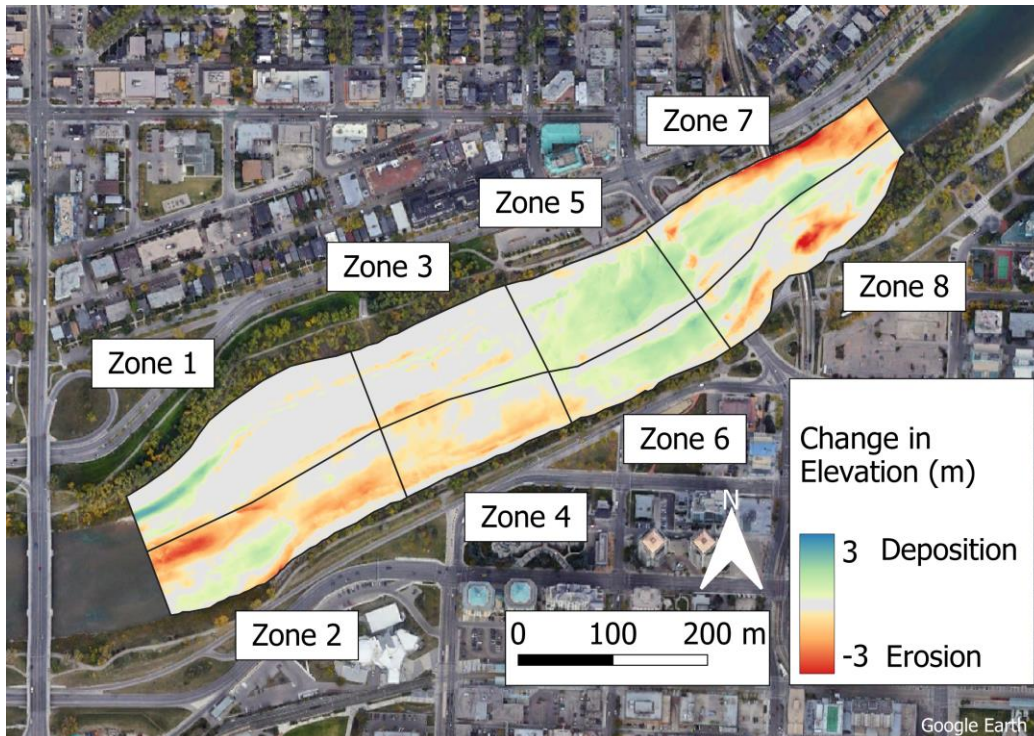


Figure 101. Change in bathymetry after TSC_1400 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 38.

Table 38. Deposition and erosion by zone after TSC_1400.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	24985	-1.59	2.48	0.00	117
2	21394	-2.65	1.28	-0.46	-9786
3	16622	-1.62	0.90	-0.05	-800
4	13442	-1.84	0.58	-0.37	-4931
5	16288	-0.97	1.82	0.60	9732
6	10209	-1.10	1.94	0.38	3887
7	16826	-4.23	1.92	-0.21	-3558
8	15917	-3.39	1.98	-0.15	-2368
Overall	135683	-4.23	2.48	-0.06	-7707

Similar to TSC_850, localized net deposition was observed in the upstream portion of the habitat channel and net erosion was generally observed along the edge of the south embankment. Localized net erosion was observed immediately adjacent to the nose of the south embankment. However, localized deposition was observed northward and upstream of the nose of the south embankment adjacent to the localized erosion. Similar to TSC_850, net deposition was observed north and south of the main bar, upstream of Louise Bridge (zones 5 and 6). Except for localized pier scour, net deposition was generally observed north

of the main bar underneath, and immediately downstream of, Louise Bridge. Deposition was generally observed along the south edge of the main bar underneath of Louise Bridge. Localized erosion was observed adjacent to the landing. Figure 102 shows a photo of the model during TSC_1400 and Figure 103 shows select photos of the model after TSC_1400.



Figure 102. Photo of the model during TSC_1400.

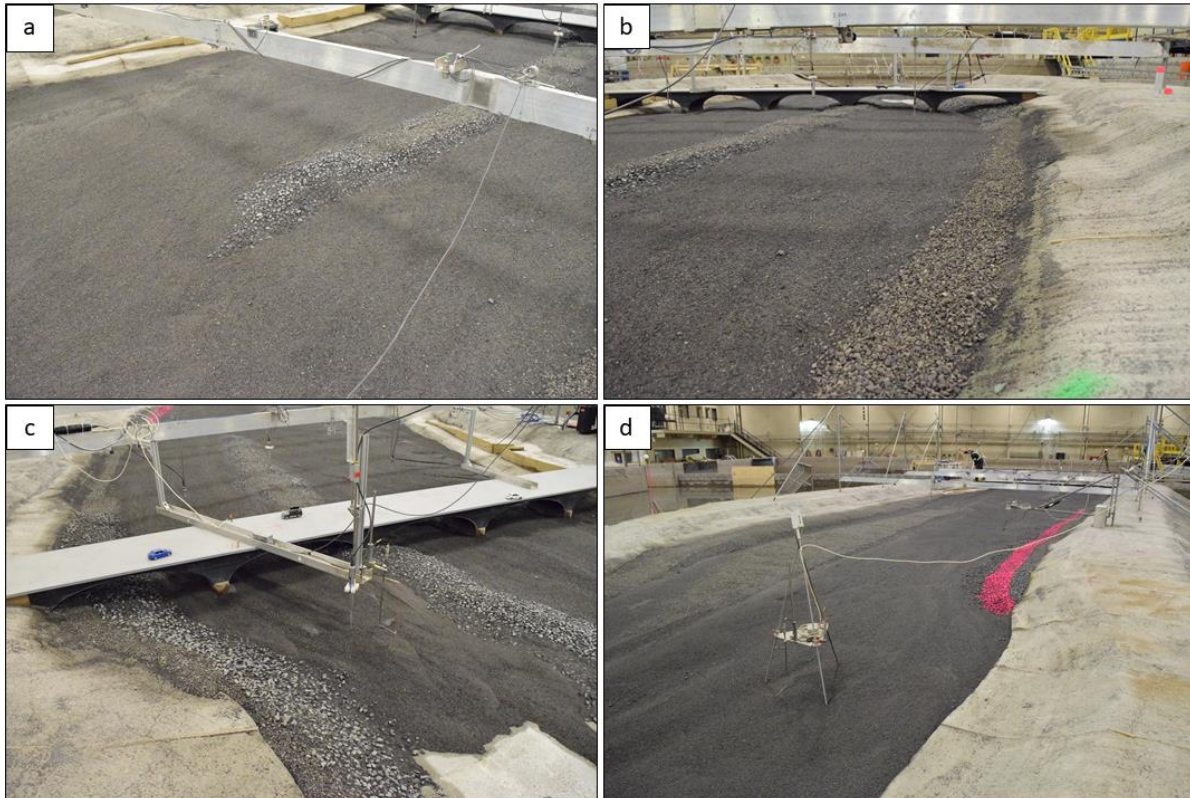


Figure 103. Photos of the model after TSC_1400; a) standing at the south embankment, looking downstream at the nose of the main bar, b) standing at the south embankment, looking downstream, c) standing near the south abutment of the LRT bridge, looking upstream, and d) standing near the south side of the upstream boundary, looking downstream.

5.6.3 100-Year Flow Condition (2100 m³/s)

Following the 35-year flow condition, Conceptual Design #3 was subjected to a 100-year flow condition with a flow equal to 2100 m³/s for a duration of 30 hours. Table 39 and Table 40 summarize the average water levels and flow velocities observed during the 100-year flow condition, respectively.

Table 39. Average water levels for TSC_2100.

Water Level Gauge	Average Water Level (m)
WL1	1050.13
WL2	1049.43
WL3	1049.04
WL4	1049.07
WL5	1048.81
WL6	1048.97
WL7	1049.17
WL8	1048.61
WL9	1048.72
WL10	1048.69
WL11	1048.65
WL12	1048.54
WL13	1048.12
WL14	-

Table 40. Average horizontal velocities for TSC_2100.

Velocity Sensor	Average Horizontal Velocity (m/s)
V1	4.10
V2	2.85
V3	2.67
V4	3.64
V5	3.28
V6	3.02
V7	3.94

The average water level immediately upstream of Louise Bridge (WL8, WL9, and WL10) was equal to 1048.67 m. Therefore, the average water level was approximately 1.83 m lower than the peak elevation of the Louise Bridge arch barrels, which have a peak elevation of 1050.5 m. The average velocity on the north and south side of the main bar approaching Louise Bridge was equal to 2.67 m/s and 3.28 m/s, respectively. The average velocity near the south embankment was equal to 4.10 m/s.

Following the 100-year flow condition, the model was drained and scans were conducted to measure the resulting bathymetry and interpret geomorphic change. Figure 104 displays the resulting bathymetry and Figure 105 displays the change in bathymetry compared to the initial bathymetry. Deposition and erosion volumes are summarized in Table 41 (positive values indicate deposition and negative values indicate erosion).

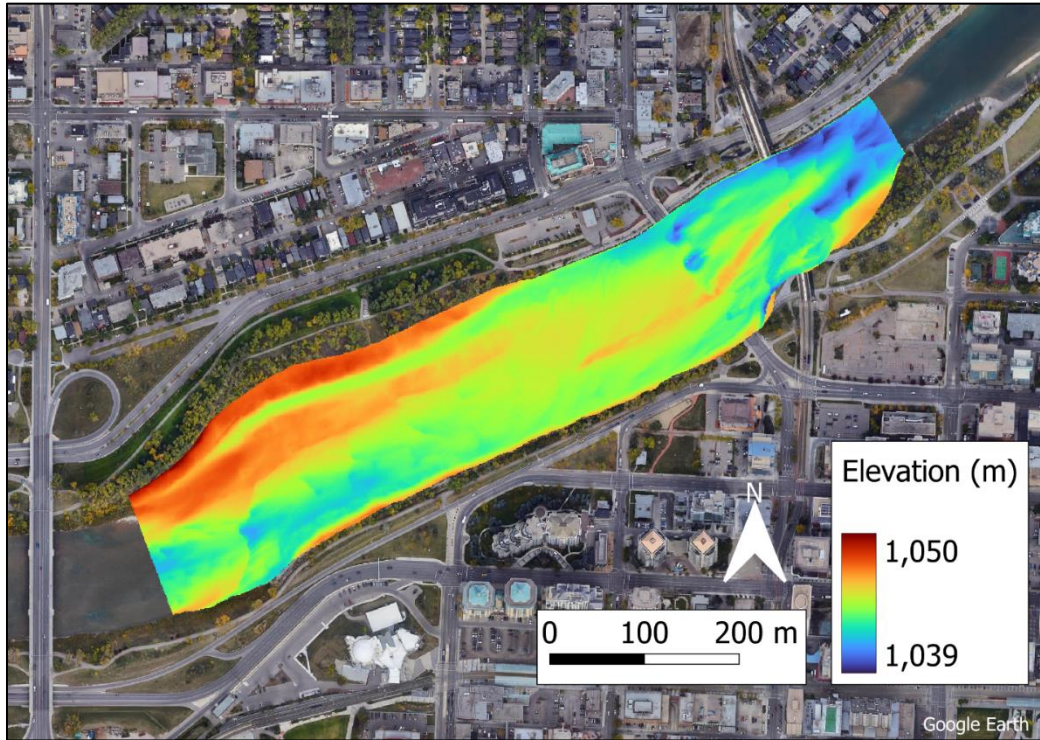


Figure 104. Bathymetry after TSC_2100.

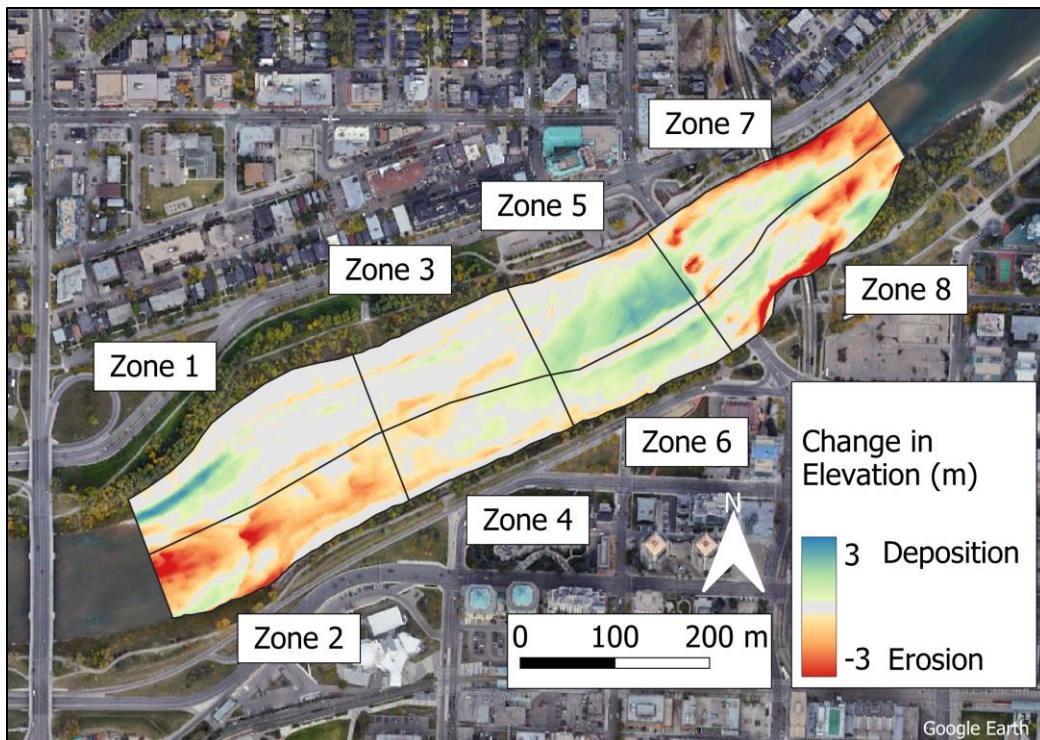


Figure 105. Change in bathymetry after TSC_2100 compared to the initial bathymetry. Changes in elevation between -0.15 m and 0.15 m are shown in grey. Zone names conform to Table 41.

Table 41. Deposition and erosion by zone after TSC_2100.

Zone	Zone Area (m ²)	Maximum Erosion (m)	Maximum Deposition (m)	Average Bed Elevation Change (m)	Volumetric Change (m ³)
1	25027	-2.05	2.89	0.11	2663
2	21295	-3.45	1.36	-0.92	-19688
3	16642	-1.41	1.08	-0.08	-1342
4	13383	-1.64	1.05	-0.08	-1059
5	16305	-1.73	2.43	0.70	11480
6	10148	-1.00	2.20	0.41	4159
7	16838	-4.19	4.80	-0.33	-5546
8	15734	-7.12	4.67	-0.51	-8005
Overall	135372	-7.12	4.80	-0.13	-17338

Similar to the previous TSC tests, localized net deposition was observed in the upstream portion of the habitat channel. Some localized deposition was also observed in the downstream portion of the habitat channel. Net erosion was observed along the south embankment and upstream of the embankment nose. Net deposition was observed north and south of the main bar, upstream of Louise Bridge (zones 5 and 6). Except for localized pier scour, net deposition was generally observed north of the main bar underneath, and immediately downstream of, Louise Bridge. Net deposition was generally observed along the south edge of the main bar. A large portion of the landing was eroded, including erosion of the crest, resulting in a failure of the feature. Figure 106 shows a photo of the model during TSC_2100 and Figure 107 shows select photos of the model after TSC_2100.



Figure 106. Photo of the model during TSC_2100.

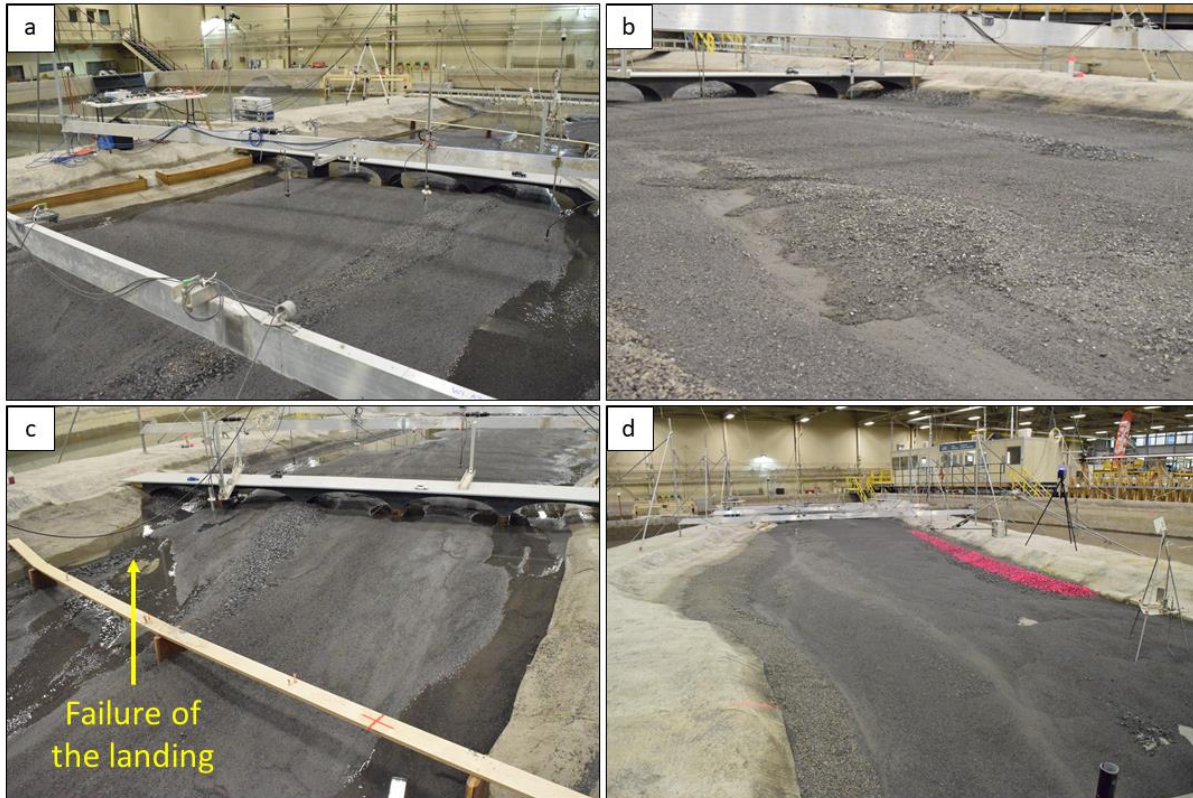


Figure 107. Photos of the model after TSC_2100; a) standing at the south embankment, looking downstream, b) standing at Poppy Plaza, looking across the channel and downstream at the main bar, c) standing near the north abutment of the LRT ridge, looking upstream, and d) standing near the north side of the upstream boundary, looking downstream.

5.6.4 Test Series C Cross-Sections

Bathymetry profiles were extracted at three cross-sections as described in Section 3.5.4. Figure 108, Figure 109, and Figure 110 show the bathymetry profiles for all tests in TSC for Cross-Section 1, 2, and 3, respectively.

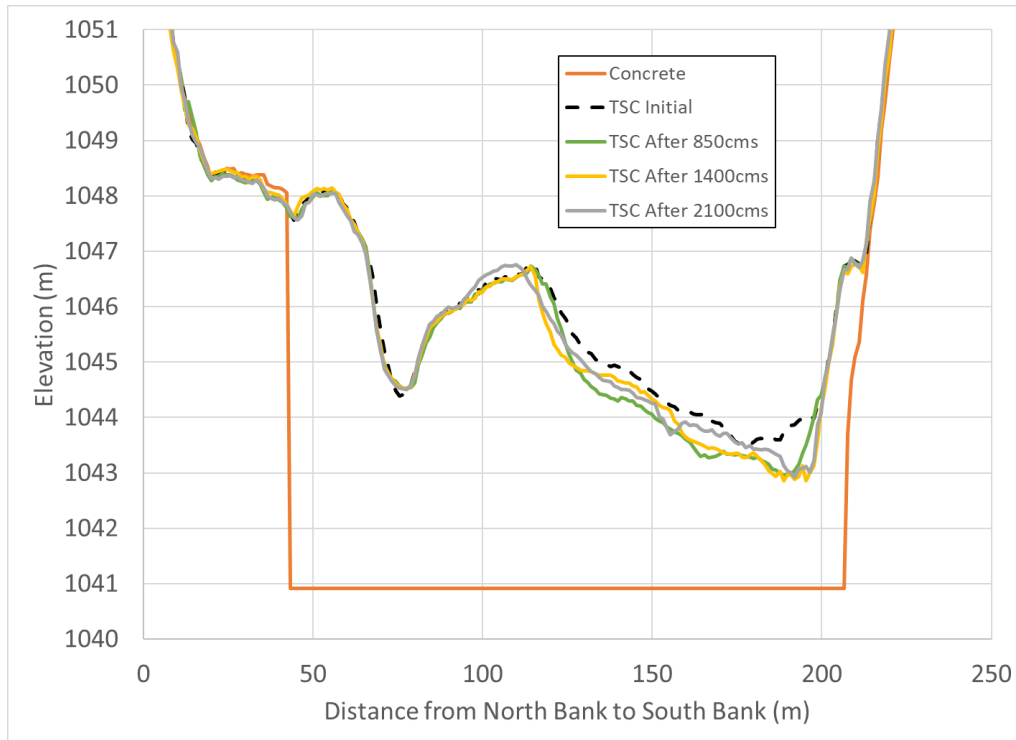


Figure 108. Bathymetry profiles for TSC at Cross-Section 1.

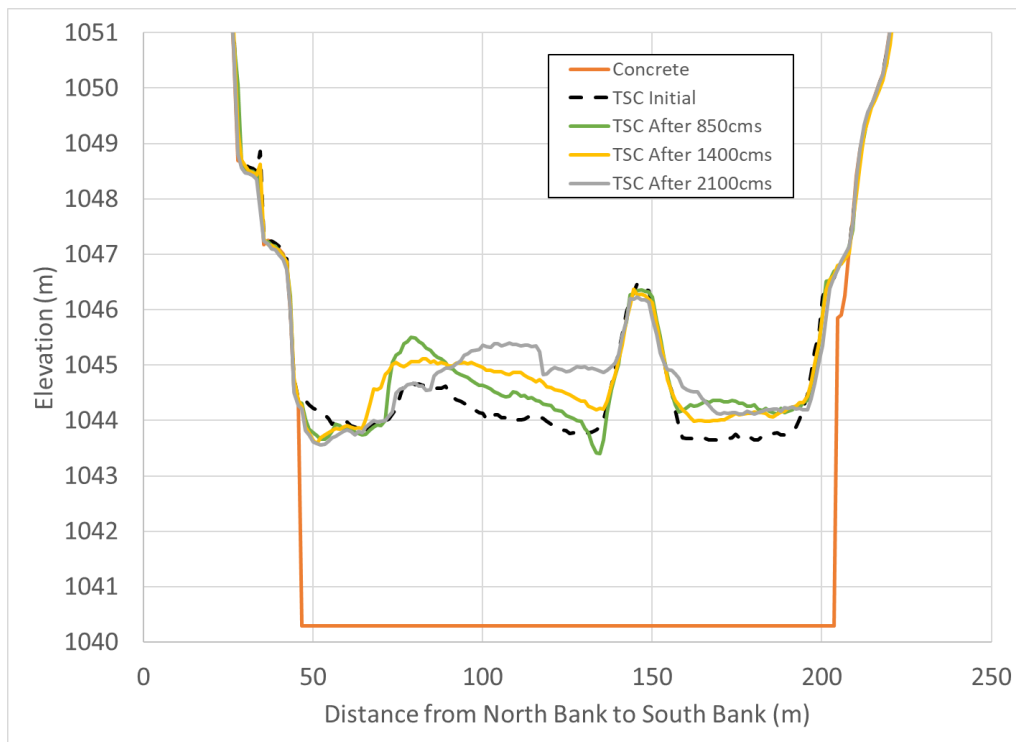


Figure 109. Bathymetry profiles for TSC at Cross-Section 2.

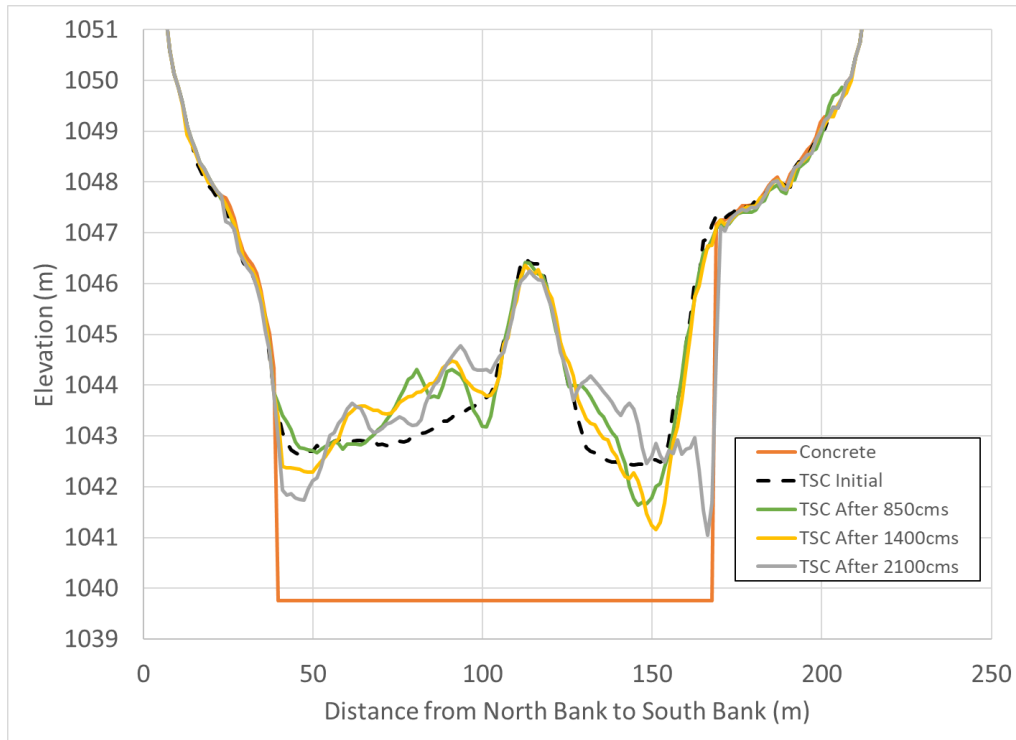


Figure 110. Bathymetry profiles for TSC at Cross-Section 3.

6 Synthesis

The two calibration tests (CAL1 and CAL2) demonstrated that the physical model was generally successful at recreating the geomorphic change caused by the 2013 flood event. Both CAL1 and CAL2 successfully recreated the development of the main bar in the vicinity of Louise Bridge as well as the patterns of erosion and deposition observed along the south embankment and in the vicinity of Poppy Plaza.

Except for TSA_850 all of the conceptual design tests resulted in net erosion in the study area (i.e. a net decrease in sediment volume). Although each test produced unique patterns of erosion and deposition in the study area, some common, general themes were apparent in the test results. Sediment deposition in the proposed habitat channel was observed in all of the conceptual design tests, particularly near the upstream end of the habitat channel. Erosion was generally observed in the vicinity of the south embankment, although the magnitude and distribution of the erosion was influenced by the presence and the composition (i.e. the amount of armoring) of the prescribed design features. Deposition was generally observed upstream of Louise Bridge in the vicinity of the main bar (zones 5 and 6 in the preceding sections), although the magnitude and distribution of the deposition differed amongst Test Series A, B, and C. Net deposition was observed in zones 5 and 6 in all of the conceptual design tests except for TSB_1400 and TSB_2100 where net erosion was observed in zone 5 (north of the main bar). A net deposition was also observed in zones 5 and 6 for the do-nothing test. However, the do-nothing test was comprised of only a single 100-year flow condition for a duration of 30 hours, and does not reflect the full range of flow conditions evaluated in the conceptual design tests (i.e. TSA, TSB, and TSC).

Table 42 provides a summary of observed water levels coinciding with the 100-year flow for each conceptual design test.

Table 42. Summary of water level observations for 100-year flow events for each conceptual design.

Water Level Gauge	Water Level (m)		
	TSA_2100	TSB_2100	TSC_2100
WL1	1050.03	1049.96	1050.13
WL2	1049.52	1049.52	1049.43
WL3	1049.29	1048.90	1049.04
WL4	1049.28	1049.06	1049.07
WL5	1048.99	1048.74	1048.81
WL6	1049.16	1048.70	1048.97
WL7	1049.29	1049.00	1049.17
WL8	1048.97	1048.68	1048.61
WL9	1049.11	1048.84	1048.72
WL10	1049.09	1048.76	1048.69
WL11	1048.88	1048.63	1048.65
WL12	1048.84	1048.42	1048.54
WL13	1048.59	1048.21	1048.12
WL14	1048.63	1048.28	-

Of the three conceptual designs evaluated, TSA produced the highest water levels immediately upstream of Louise Bridge, followed by TSB and TSC for the 100-year flow condition. The average water levels observed immediately upstream of Louise Bridge (based on WL8, WL9, and WL10) for TSA_2100, TSB_2100, and TSC_2100 were equal to 1049.06 m, 1048.76 m, and 1048.67 m, respectively. It is important to note that although TSA_2100, TSB_2100, and TSC_2100 shared a common target tailwater elevation (i.e. downstream water level condition), the actual tailwater elevation differed amongst the three tests. The average actual tailwater elevations for TSA_2100, TSB_2100, and TSC_2100 were equal to 1048.61 m, 1048.25 m, and 1048.12 m, respectively. As described in section 7, these differences in actual tailwater elevation should be considered when interpreting the water level observations.

7 Model Limitations

The physical model was designed, constructed, and executed with careful attention to detail in order to minimize inaccuracies and enhance the quality of the results. However, the results of the test program should be interpreted with consideration of the limitations noted below.

As described in Section 3.3, the mobile bed included all spaces within the study area where geomorphic change during large flood events could reasonably be expected (taking into account existing armouring) as well as locations where topographic modifications were expected in accordance with the conceptual bar realignment designs. The elevation of the concrete bed underlying the mobile bed (and, therefore, the thickness of the mobile bed) was selected such that the maximum scour depths anticipated from numerical modelling by Yu et al. (2022) could be fully accommodated by the mobile bed, without scour down to the

concrete. However, during the test program, some portions of the concrete bed (and the edges of the concrete bed) became exposed owing to localized scour. This was particularly evident along the northern portion of the mobile bed, near the downstream boundary, where lateral and downward erosion was limited by the concrete (see Figure 111, for example, which shows the resulting bathymetry after TSC_2100 near the downstream boundary). However, in reality, lateral erosion at this location of the study site is also limited by the presence of armour stone. It is challenging to comment on the realism of model results where downward scour exposing the non-erodible concrete floor was observed. As described in Section 3.4.3, available borehole data suggest that there is a non-erodible clay layer underlying the alluvial channel sediment at the study site. However, the continuity and elevation of the clay layer outside of discrete borehole locations is not well understood. The non-erodible concrete floor of the model was not designed to represent the clay layer. However, it is reasonable to assume there is a vertical limit to downward scour at the study site owing to the presence of the clay layer.



Figure 111. Example of scour exposing the concrete bed and the edge of the concrete bed.

The bioengineering stone prescribed for the conceptual design tests was not present in either of the calibration tests. In CAL1 and CAL2 the entire mobile bed was represented using scaled river sediment except for portions of Poppy Plaza and the landing where dense, established vegetation was represented using rip-rap. Therefore, the accuracy with which the bioengineering stone faithfully represents vegetation could not be fully assessed in this study. AMEC (2012) suggests that the erosion resistance provided by the bioengineering stone is comparable to live willow stakes. However, it is unclear if the bioengineering stone faithfully represented the influence of vegetation post-mobilization. In situation where bioengineering stone was mobilized by the flow, the bioengineering stone often became intermixed with the river sediment.

Differences in actual tailwater elevation for common flow tests (e.g. TSA_1400 versus TSB_1400, or TSB_2100 versus TSC_2100) should be considered when interpreting water level observations, particularly for locations close to the downstream boundary of the model. With the exception of CAL1_600, the actual tailwater elevations observed in the physical model were all higher than the target tailwater elevations (see Table 9). The *RMSE* between the actual tailwater elevations and the target tailwater elevations for all tests shown in Table 9 was equal to 0.49m. The observations indicate that the actual tailwater elevations were more closely aligned with (albeit, still generally higher than) the tailwater estimations derived from the Yu

et al. (2022) numerical model ($RMSE=0.41m$). Considering only the conceptual design tests (i.e. TSA, TSB, and TSC), the $RMSE$ values compared to the target tailwater elevations and the tailwater elevations derived from the Yu et al. (2022) model were equal to 0.47 and 0.29 m, respectively. It should be noted that tailwater elevations derived from the Yu et al. (2022) model were adopted as the target tailwater elevations for the calibration tests, but not the do-nothing test or conceptual design tests where values were adopted from the Government of Alberta's Flood Awareness Map Application (Government of Alberta, n.d.). The discrepancies were not thoroughly investigated. However, a brief assessment revealed that the downstream boundary of the physical model was situated at a mild horizontal constriction, and inconsistencies between the resolution of the physical model and the numerical modelling that forms the basis of the map application may explain some of the discrepancies. In this context, it is to be expected that the tailwater elevations observed in the physical model would be more closely aligned with numerical modelling conducted by Yu et al. (2022) which was conducted at a high resolution (6.5 m), such that the horizontal constriction was captured in detail.

As described in Section 4.1.1 and Section 4.1.2 sediment input rates for all of the tests (i.e. the calibration tests, the do-nothing test, and the conceptual design tests) were informed by numerical modelling, and not by field observations. Without reliable field observations of sediment transport, it is not possible to assess the accuracy of the sediment input rates applied in the physical model. In addition, the scaled river sediment used in the physical model was applied, as received, from the local quarry source and no actions were taken to recreate the natural sediment layering (i.e. armouring) present at the study site. Estimates for surface and subsurface sediment gradations were available from KCBL (2016). The river sediment used in the physical model was selected based on subsurface sediment gradations, assuming that the subsurface material is more representative of the bulk of the material available for transport during high flow events. Despite uncertainties related to sediment supply and sediment layering, the calibration tests described in this report permitted an assessment of model skill at reproducing geomorphic change, which is directly linked to these parameters.

The do-nothing test and conceptual design tests were conducted using steady flow approximations of characteristic flood events with defined return periods. In reality, a river flooding event would be characterized by a continuous, dynamic hydrograph (see the actual 2013 flood hydrograph shown in Figure 33, for example). Owing to technical limitations, a continuous hydrograph could not be applied in the physical model. Therefore, the physical modelling results may not reflect the full range of hydraulic conditions and geomorphic adjustments associated with a natural flooding event. However, the results provide context to evaluate and compare conceptual design performance under extreme flow conditions.

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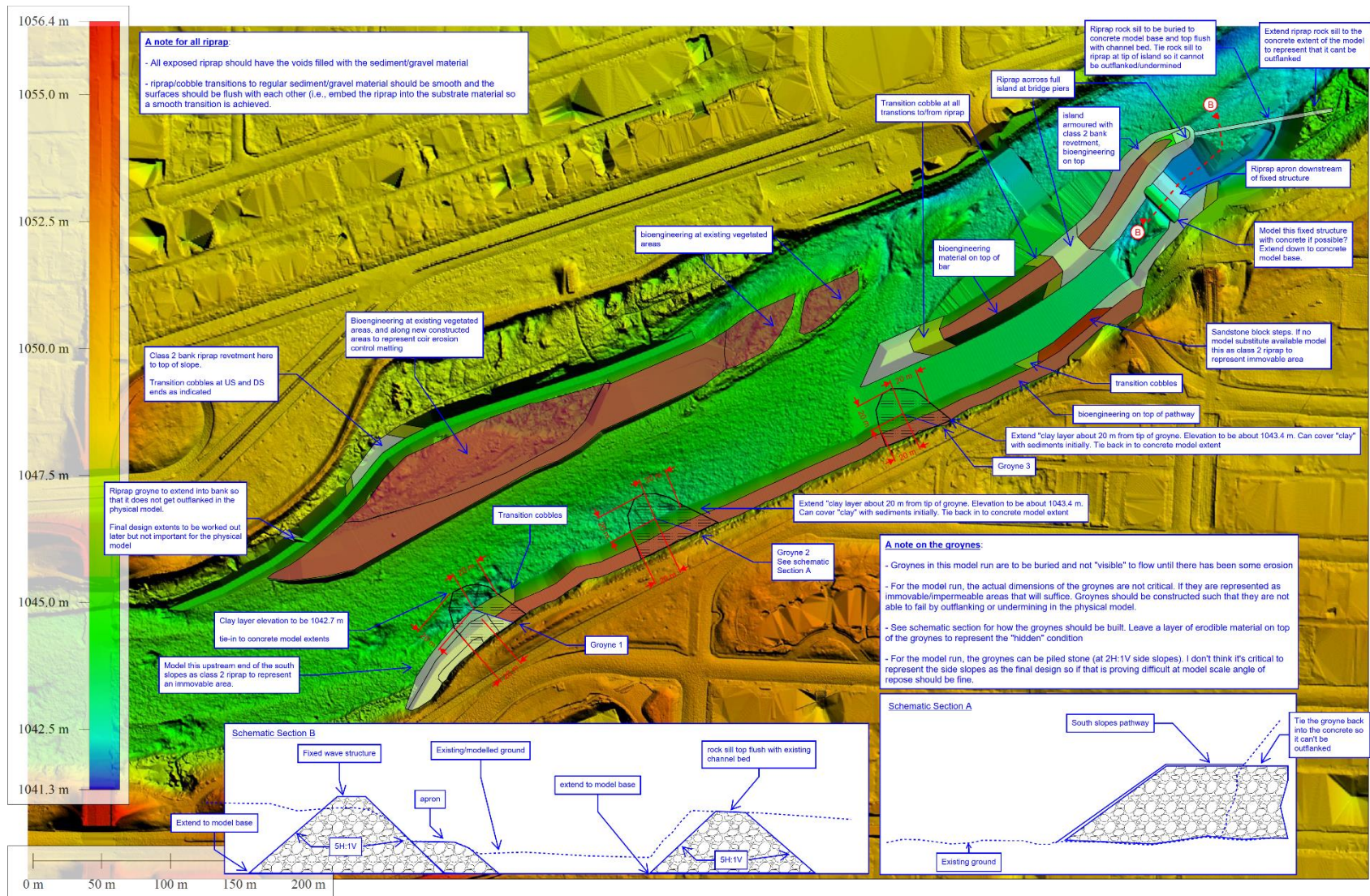
10 Appendix A – Design Drawings

The following design drawings were provided by Matrix Solutions Inc.

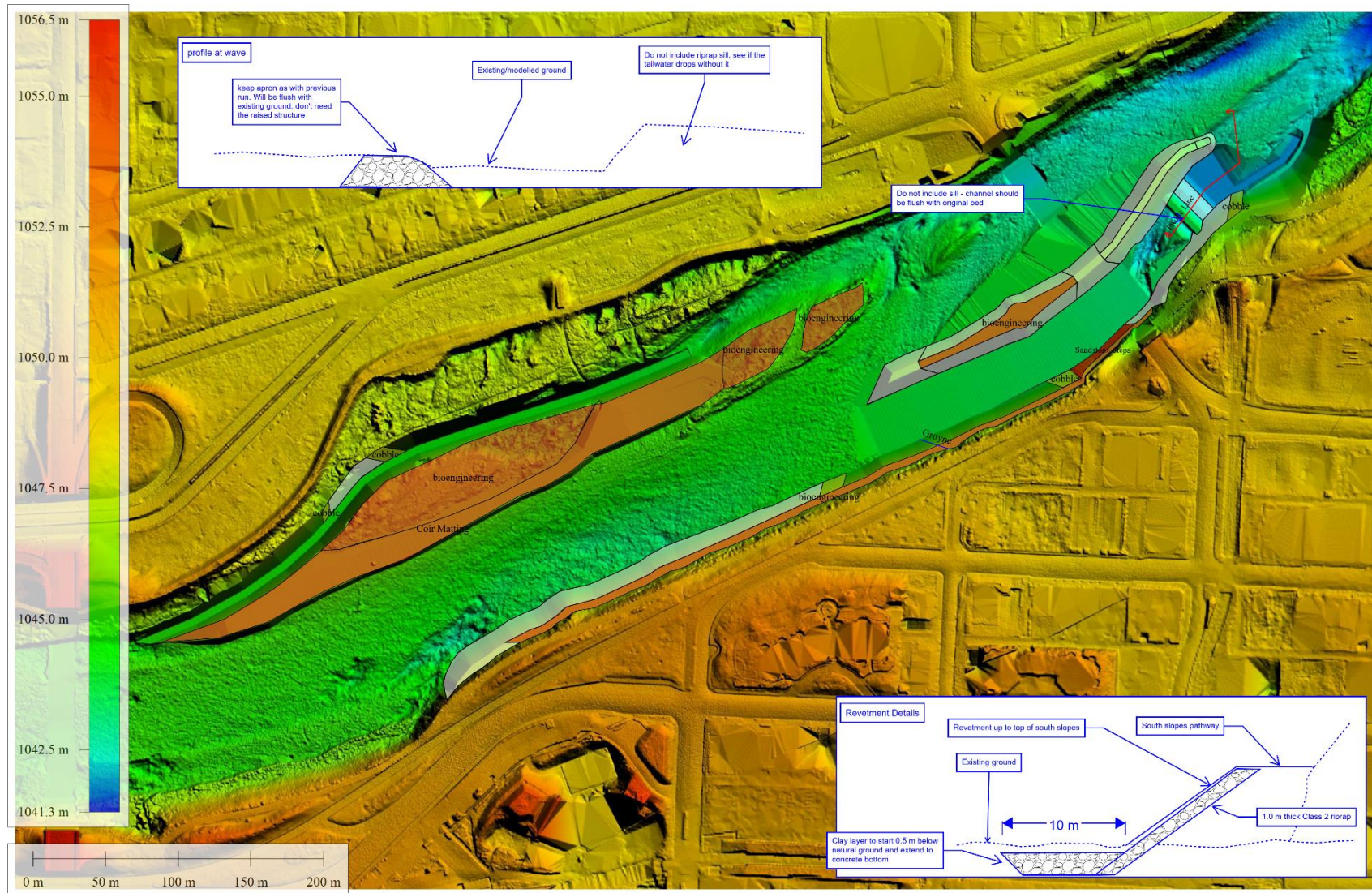
Conceptual Design #1 (courtesy of Matrix Solutions Inc.)



Conceptual Design #2 (courtesy of Matrix Solutions Inc.)



Conceptual Design #3 (courtesy of Matrix Solutions Inc.)



11 Appendix B – Electronic Archive

See electronic archive