PLFP Hydrodynamic & Sediment Transport Modeling Report

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Environment | Engineers

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1 Introduction

LimnoTech is supporting the design for the Port Land Flood Protection (PLFP) project in the Lower Don Lands by using numerical modeling tools that predict the hydrodynamics and sediment transport conditions under a variety of design conditions.

The hydrodynamic performance of the PLFP project is critical to ensure that the project can provide flood protection for the Lower Don Lands under the Regulatory Flood event to allow for beneficial redevelopment of areas that are currently at risk of flooding. In addition, the design of the naturalized river system needs to account for the hydrodynamics to ensure the proposed infrastructure and environmental remediation measures are protected from damage due to high flows and shear stresses.

The location of the PLFP site at the mouth of the Don River means that the sediment that is delivered from the watershed will impact the PLFP site. The PLFP project design includes several elements whose long-term maintenance costs, and hydraulic and ecological services, are influenced by sediment dynamics: the sediment management area, Keating Channel, naturalized channels and wetlands, and critical infrastructure locations including the Lake Shore Boulevard bridge crossing. Sedimentation in the sediment management area and Keating Channel will require maintenance dredging to enhance regulatory flood conveyance and focus sedimentation in the sediment management area.

The primary purpose of this report is to document the technical approach used in developing and applying the hydrodynamic and sediment transport models for the project. As the design of the project is continuing to develop and be refined, any results presented within this report are only representative of the design up to the date of this report. Upon finalization of the PLFP project design, it is anticipated that this report will be updated to include the results of the final design modeling scenarios.

2 Hydrodynamic Modeling

2.1 Introduction

Two of the major purposes identified in the 2014 Don Mouth Naturalization and Port Lands Flood Protection Project Amended Environmental Assessment (DMNP EA) documents for undertaking the PLFP project are to achieve naturalization of the mouth of the Don River, and to provide flood protection to allow for the redevelopment of the surrounding lands. Understanding the hydrology and hydrodynamics of the Don River are critical to achieving both of those purposes through the project design. The goals of the hydrology and hydraulic design are to:

- Ensure that the PLFP project provides flood protection up to the regulatory flood levels from historic rainfall experienced in the region
- Create a sustainable system that can support a diverse aquatic ecology, in particular in the naturalized portions of the project.

A 2-dimensional (2D) depth-average model framework was selected to analyze the hydrodynamics and performance of the flood protection measures of the PLFP project due to the complex geometries on the project (varying channel dimensions, floodplain flows, bridge piers, flow splitting between multiple outlets, etc.) that would not be well represented in a 1-dimensional (1D) model framework, which averages across both the depth and the width of channels. The complex geometry on the PLFP project results in variable (non-uniform) flow distributions across sections of channels, resulting in variable water surface elevations, concentrating flow velocities, and bed shear stress depending on the location and bathymetry. 1D models are less reliable in hydraulically complex areas due to poor ability to address spatial variability in water velocity. In model application, 1D models often rely on more conservative representations of surface roughness to represent topographic variability, whereas 2D models can more precisely represent topographic variation.

2.2 Previous Reports & Models

The PLFP project modeling effort is a continuation of a previous modeling study called "Don Mouth MIKE FLOOD Modeling and Analysis Project" (DHI 2017). For the purpose of this modeling report, the DHI model and report will be referred to as the "Analysis Model" and the "Analysis Report."

Many of the model development details remain unchanged from the Analysis Model. For this information we will refer the reader to the appropriate section of the Analysis Report.

The Analysis Report Section 2 contains a comprehensive discussion of the available data including:

- Background Reports
- Existing Models
- GIS Files
- Hydrology Data
- Hydraulic Structures
- Existing and Previously Proposed Topography

2.3 Model Development

The Analysis Model served as the starting point for the design modeling effort; as such, many of the modeling details remain the same as those reported in the Analysis Report. However, the PLFP hydrodynamic model has been modified to meet the specific needs of the project and to reflect new information, and the design iterations as the project progresses. Primarily the updated model features will be discussed in this section.

2.4 Model Domain

The model domain covers an area of 483 ha, with the upstream portion of the Don River located upstream of the Dundas Street East Bridge, and extending down through the Keating Channel, and into the Inner Harbour. The southern portion of the model domain stretches to the Ship Channel. The full model domain with the bathymetry is shown in Figures 2.1.

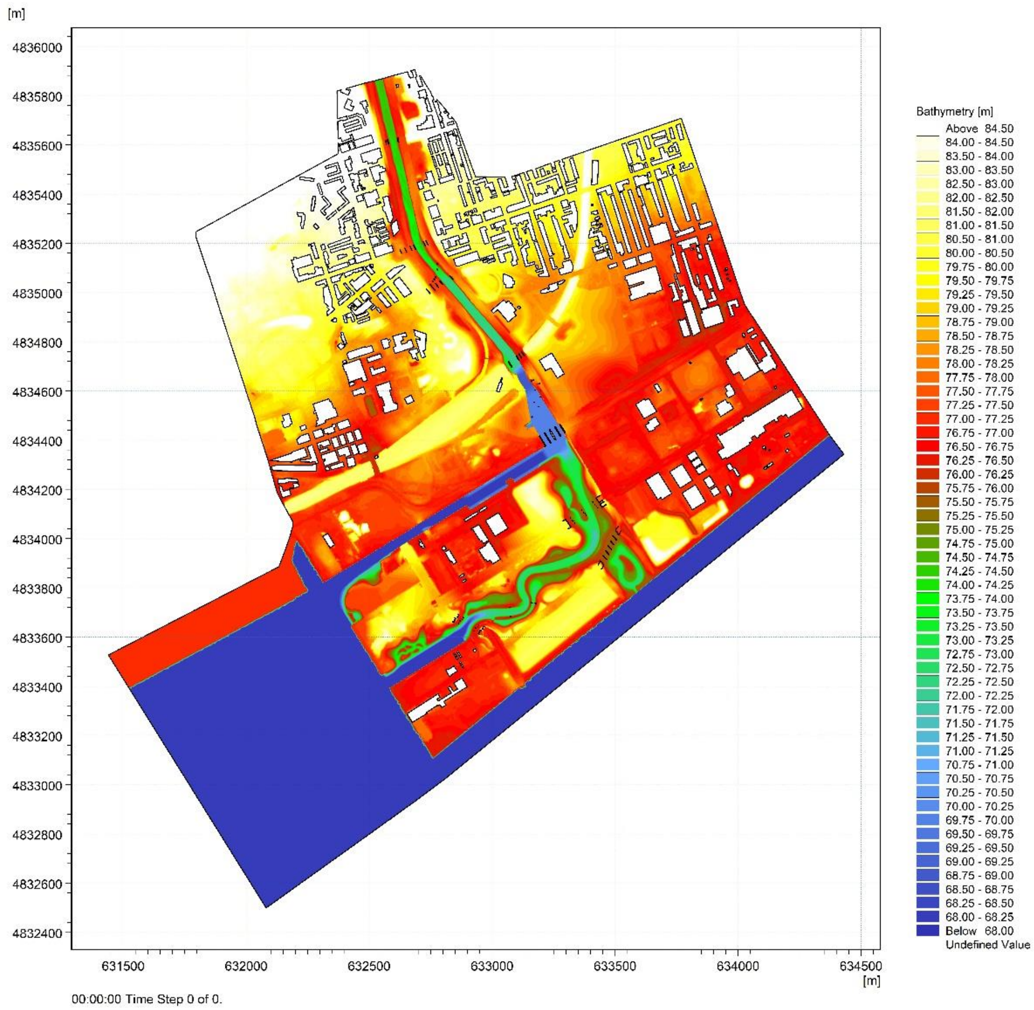


Figure 2-1: Model Domain and Topography

2.5 Model Mesh

The model mesh used for the Analysis Model was well resolved, with the flexible mesh features being used to appropriately increase the model resolution in critical areas of the project. The mesh developed for the Analysis Model was used as the starting point for the development of the PLFP mesh. Modifications to the model mesh were made for the following reasons:

1. To reflect design updates
2. Improve the representation of existing infrastructure (such as dock walls)
3. Improve model stability and/or run efficiency

2.5.1 Mesh Configuration

The Analysis Report, Section 3.2.1 discusses the general mesh configuration, specifically the relationship between the triangular elements, rectangular elements, and the inactive areas. The revisions that have been made to improve the model mesh are discussed below, and illustrated in in Figure 2-2 through Figure 2-11

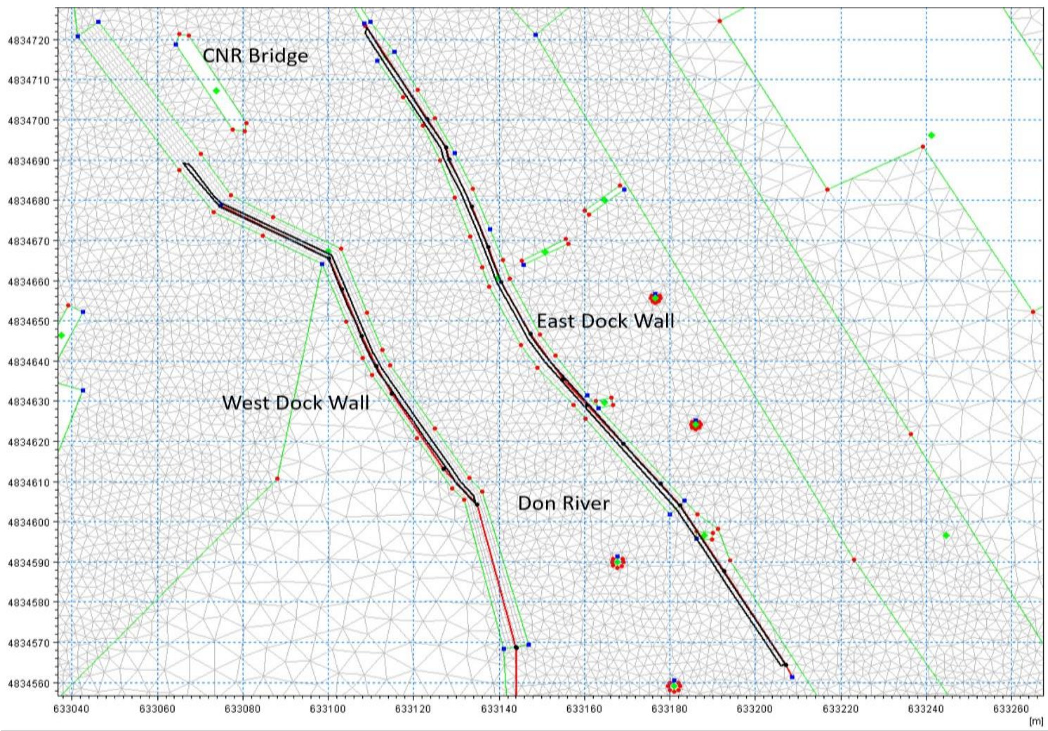


Figure 2-2: Dockwall between the CNR Bridge and the SDMA

In the area between the CNR bridge and the upstream end of the SDMA, the triangular mesh has been converted to a rectangular mesh along the existing and proposed dockwalls. (The dockwall between the CNR Bridge and the Sediment Management Area will be moved in between 0.5 m and 1 m on both sides of the channel, depending on the dockwall reinforcement details proposed at each section.)

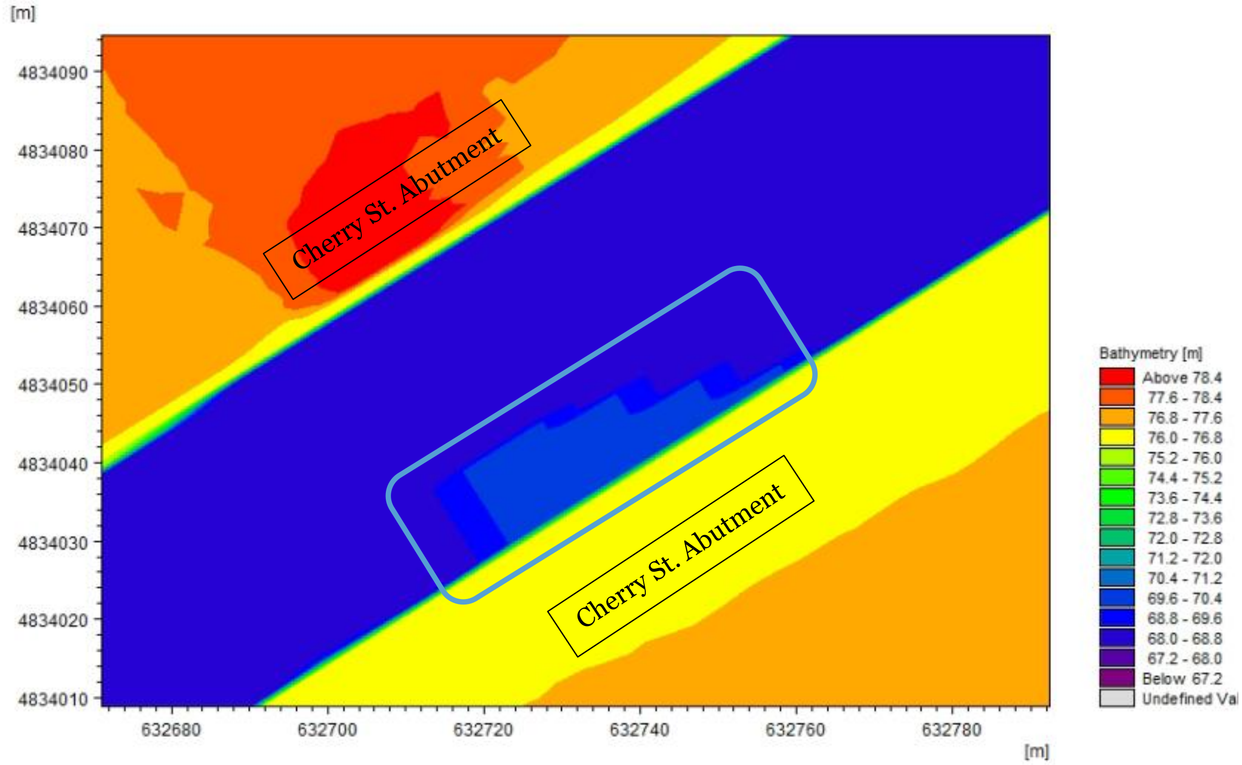


Figure 2-3: Pit Slab Location

The pit slab is an existing concrete slab located under the existing Cherry St Bridge. The pit slab has an elevation of 69.8. The channel elevation in the immediate area surrounding the slab is 69.6 within 10 metres of the existing dock walls, and 68.4 metres in the center of the channel. Under the Regulatory Event, the slab reduces the flow in the Keating by approximately 2 cms, and has negligible impacts on the water surface elevations and wet extents. Based on the minimal impact to flood conveyance on the project, the pit slab will be left in place, and has been included in the model surface for the proposed conditions.

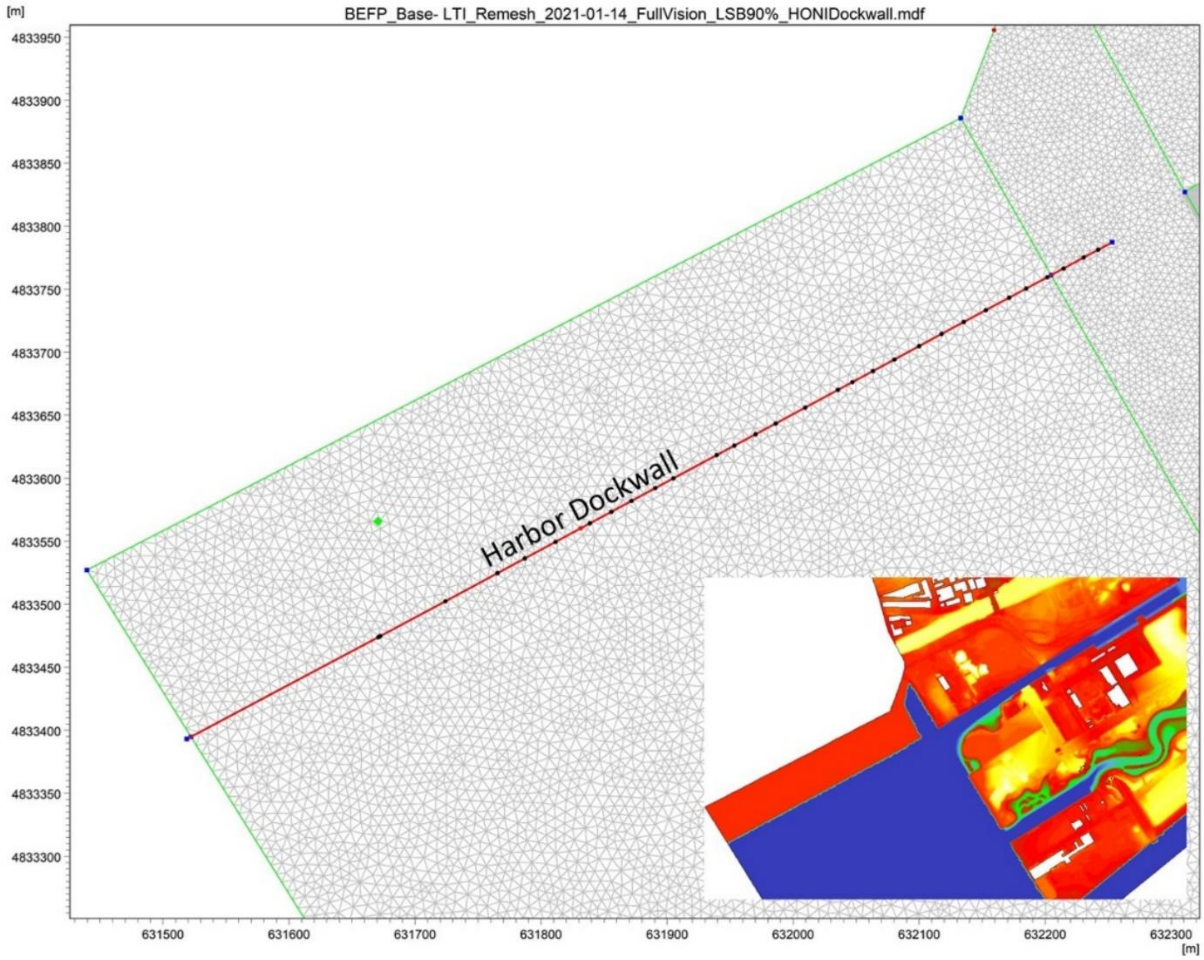


Figure 2-4: Harbour Dockwall Grid Modifications

The dockwall in the lake was modified and a breakline added to better represent the alignment and aid in cleaner interpolation. In several earlier grid modifications, the grid representing the harbour dockwall was not interpolating consistently with each mesh update. These inconsistencies would sometimes result in a model becoming unstable and crashing. A mesh arc line and breakline were added to the mesh development files better represent the straight dockwall configuration.

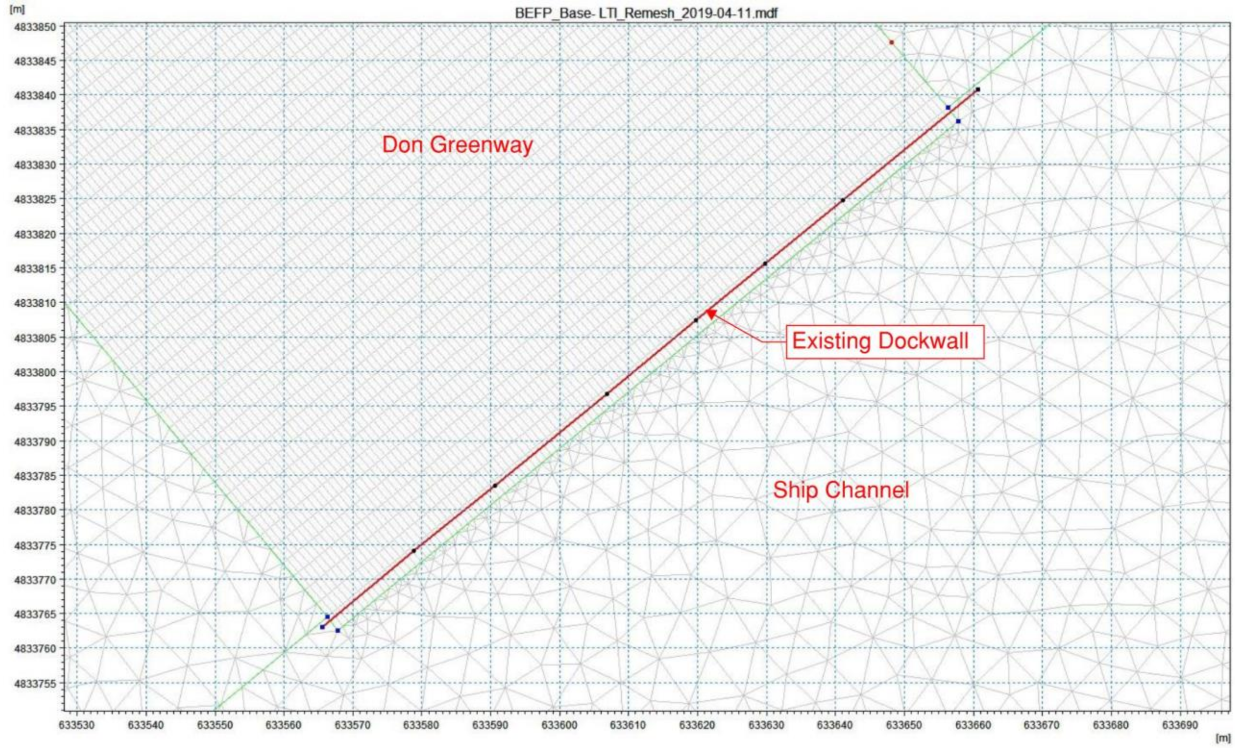


Figure 2-5: Don Greenway Weir Model Grid Refinement

The rectangular grid in the Don Greenway was extended one additional row beyond the outfall in order reduce the risk of erroneous interpolations between the Don Greenway spillway elevations and the deep Ship Channel bathymetry. A breakline was also added along the dockwall to aid the interpolation. This also allows for representation of the spillway openings into the Ship Channel

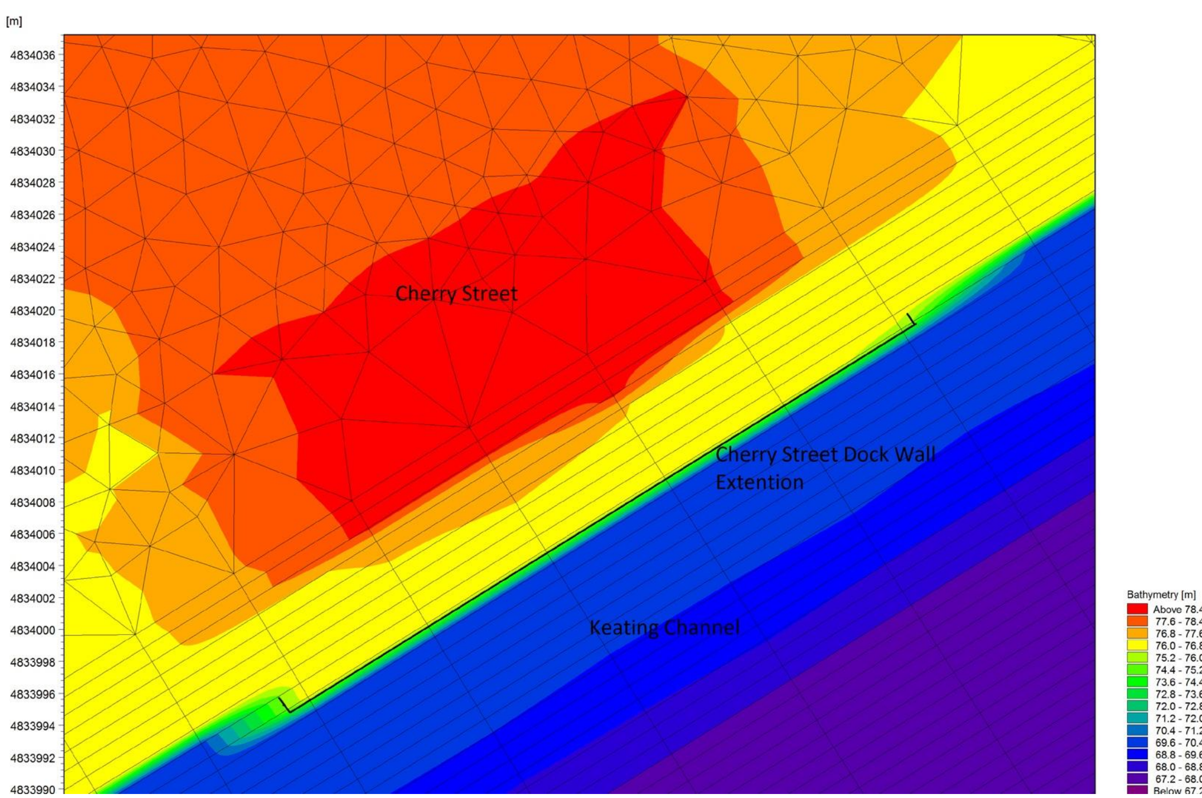


Figure 2-6: Keating Dockwall Revision at the Existing Cherry Street Bridge Abutments

The dockwall in the narrow section of the Keating Channel at the location of the existing Cherry Street Bridge abutment was moved 2 m into the channel to represent proposed sheeting to be installed at location to reinforce the existing dockwall after removal of the bridge.

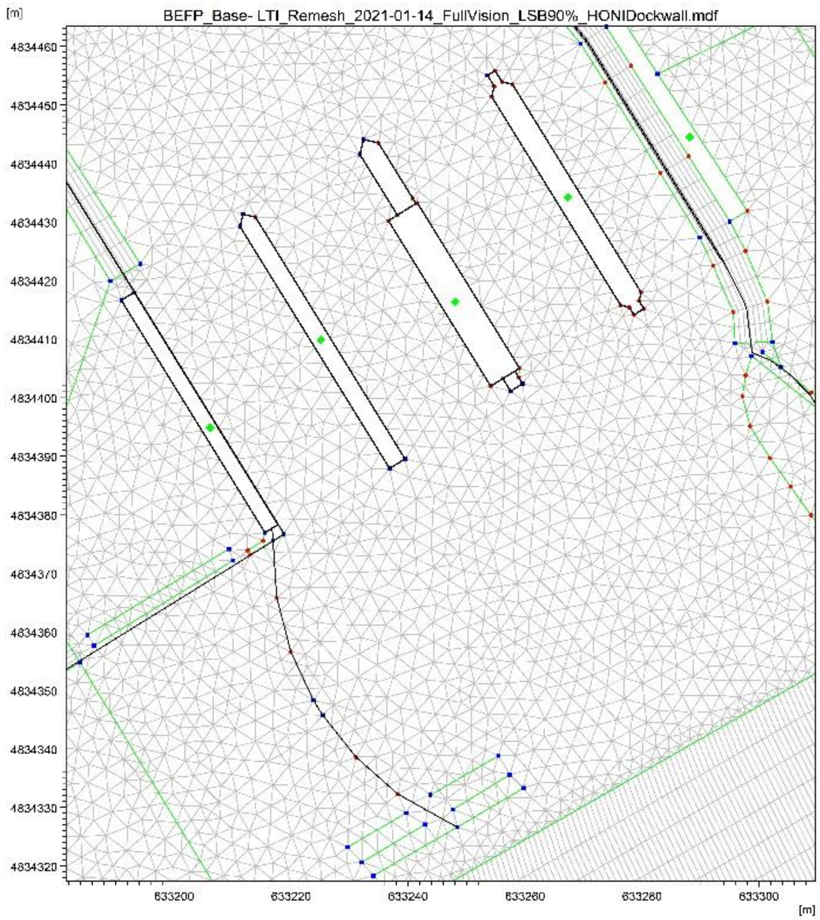


Figure 2-7: Grid Modifications for Lake Shore Bridge Piers, Dockwall, and Flow Curtain

The existing dockwall in the eastern bay of the Lake Shore Bridge has been represented by a rectangular grid to prevent interpolation across the top of the dockwall with irregular triangular grid elements.

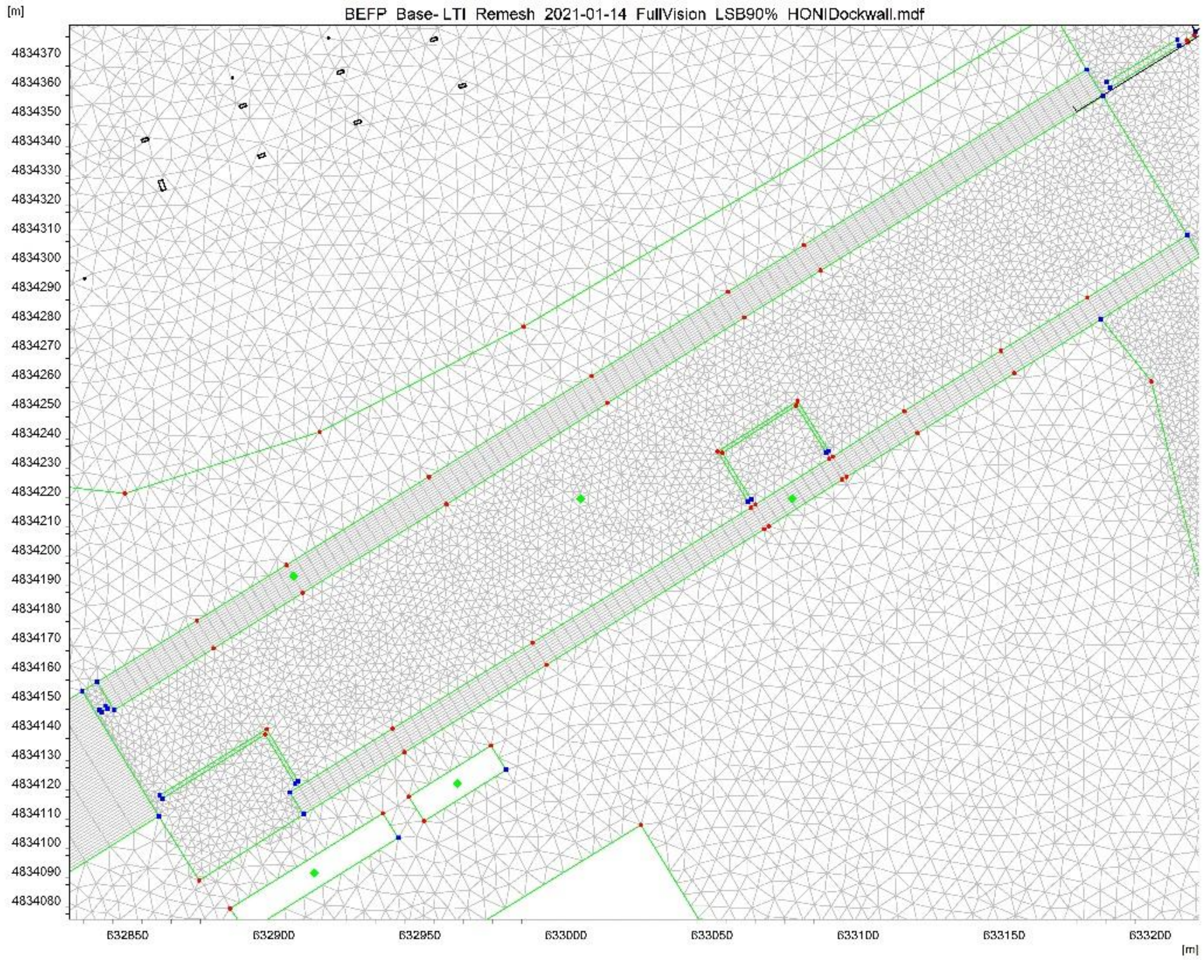


Figure 2-8: Keating Channel Rectangular Grid Alignment

The Keating Channel mesh was modified to align with the dockwall survey. (To the extent possible, the mesh was adjusted such that the mesh nodes do not sit directly on top of the dockwall, which improves model representation accuracy of available conveyance area along the wall face.)

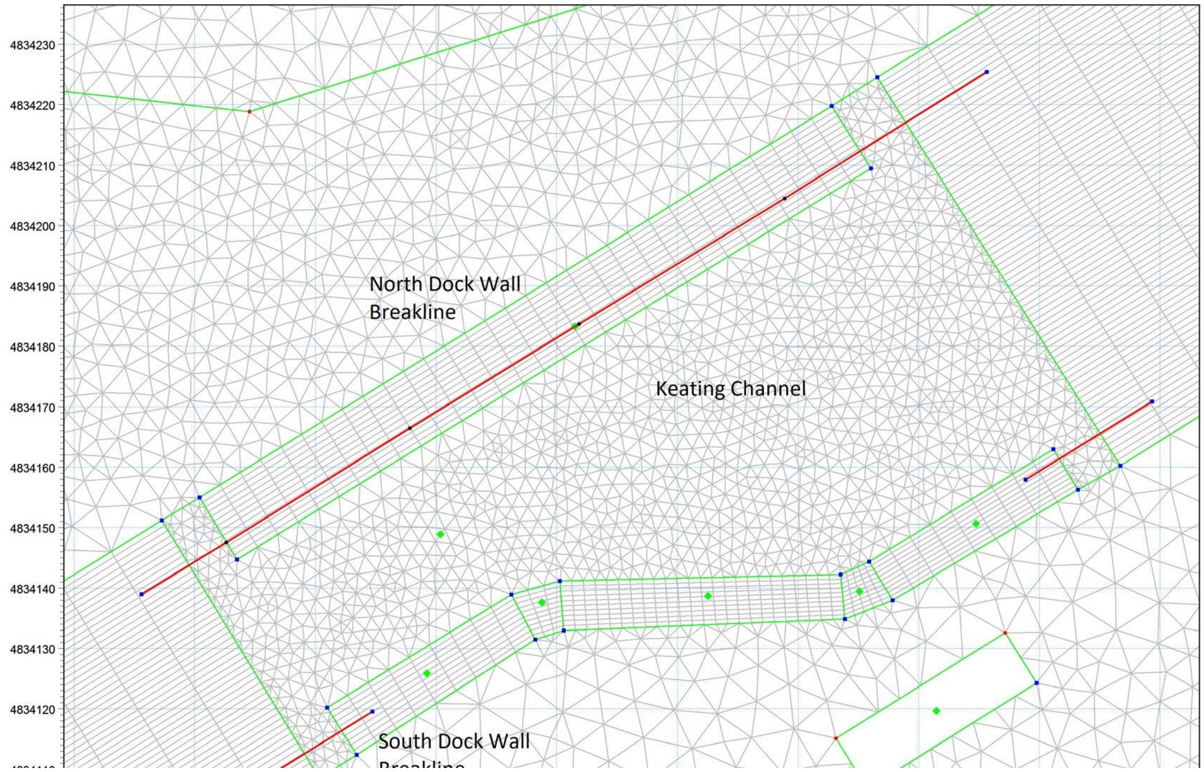


Figure 2-9: Keating Constriction Mesh Revision

The model mesh at the constriction in the Keating channel was modified to a rectangular grid that follows the proposed dockwall alignment.

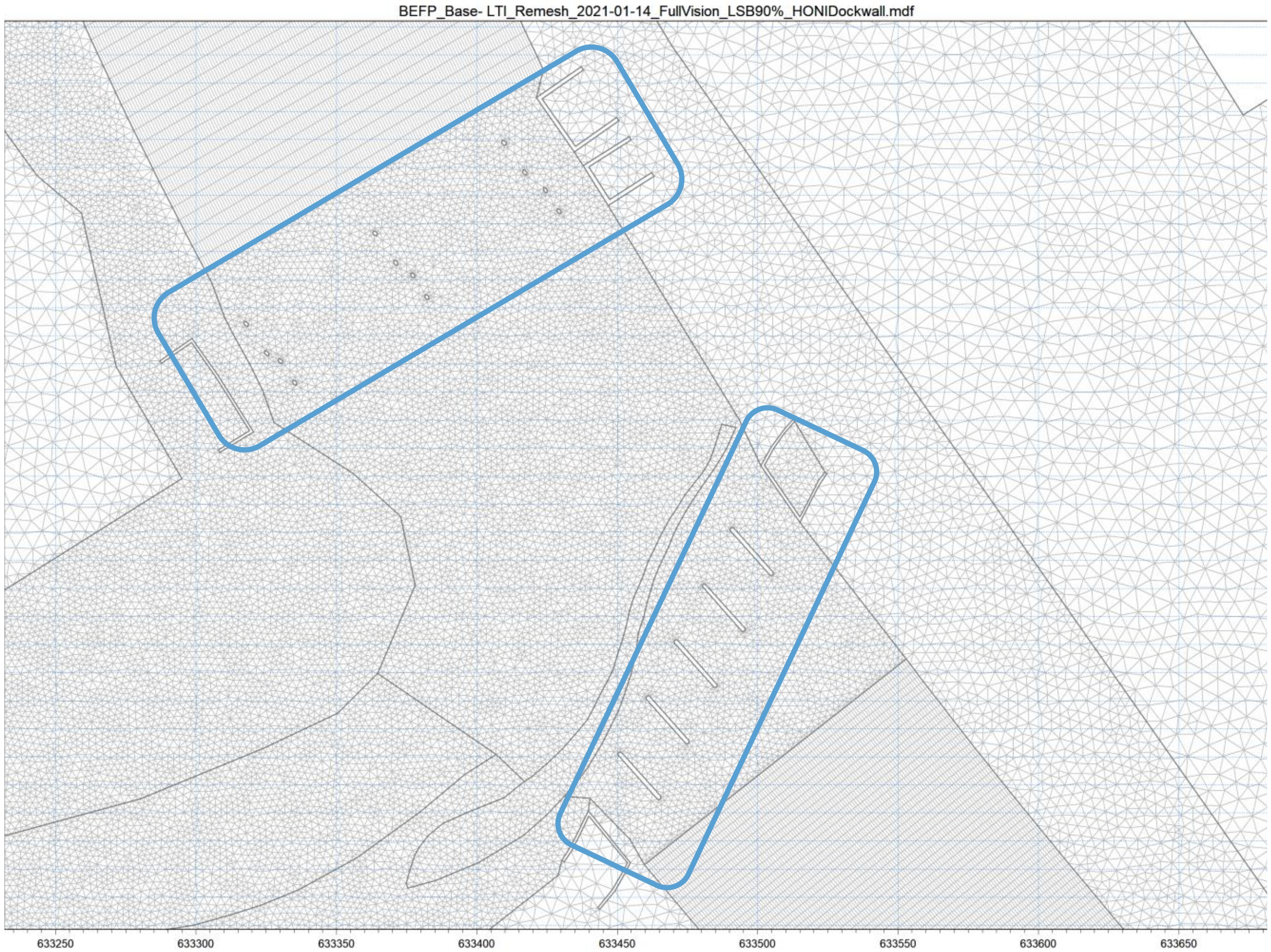


Figure 2-10: Model Grid at the Commissioners Street and Basin Street Bridges

The Commissioners St and Don Greenway Bridge piers and abutments have been updated to reflect the most recent proposed design. These piers and abutments are modified within the model grid any time the linework for updated bridge designs is provided.

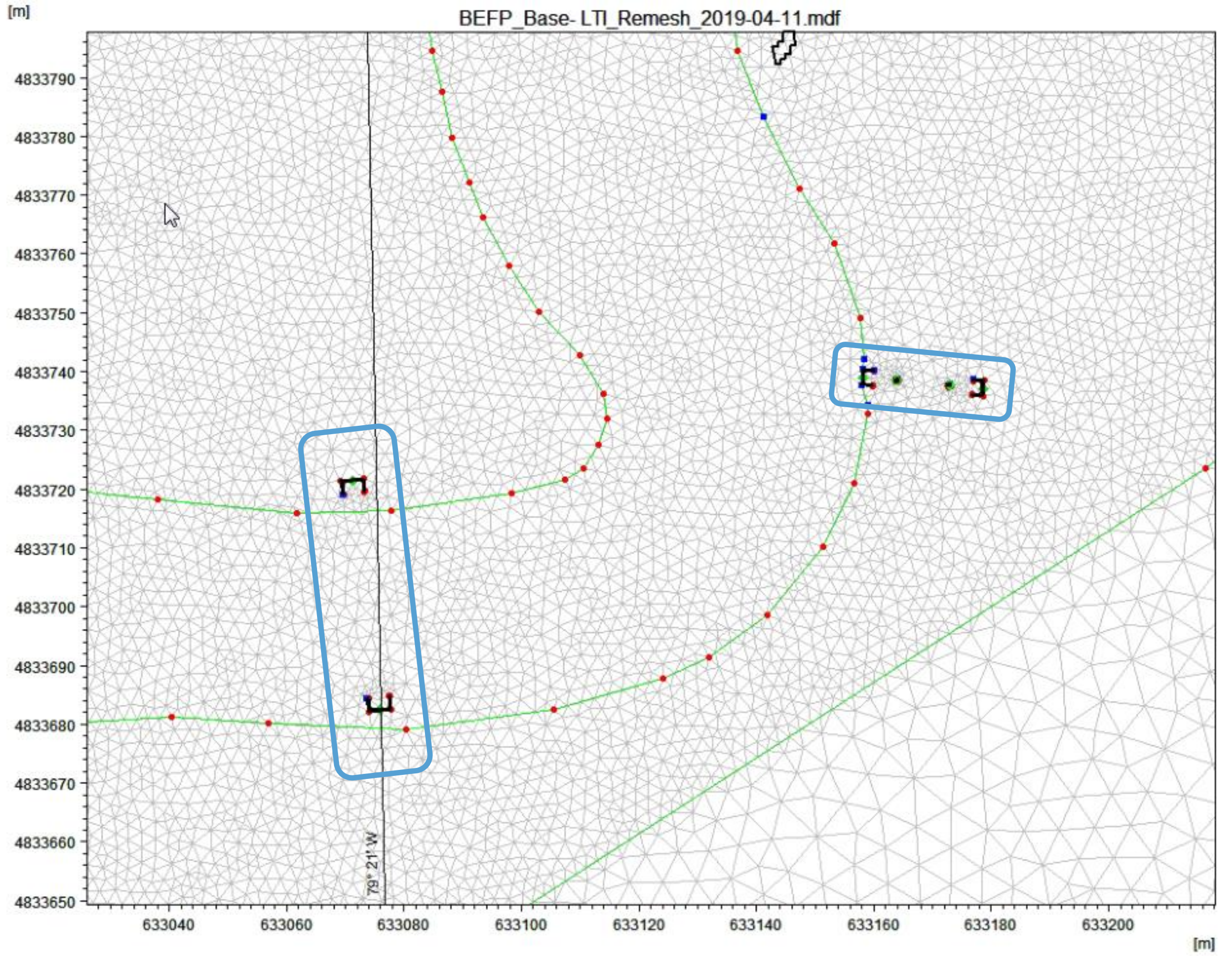


Figure 2-11: Model grid at Pedestrian Bridges in the River Valley

The pedestrian bridge piers and abutments have been included in the model mesh, and are updated to reflect the most recent proposed design.

2.5.2 Mesh Topography

The design elevations in the plans and specifications for the PLFP project are all referenced to the Canadian Geodetic Vertical Datum of 1928, without subsequent adjustments ("G.S.C. 1928", or CGVD28:PRE78). Given the importance of the hydrodynamic modelling and references to the adjacent modeled water surfaces, all the elevation references in the hydrodynamic model shall be in reference to this datum.

The primary data sources related to the topography and bathymetry in the hydrodynamic model are shown in Table 2.1, along with the original datum of the data sources.

Table 2-1: Elevation Data Sources Used in Hydrodynamic and Sediment Transport Models

Description	Notes	Source	Original Datum
WP#12 Grading Plan (Rev. Date: 01/14/2021)	MVVA Design Surface	MVVA	CGVD28:PRE78
Broadview Phase 1 Grading Solution (Received 08/07/2020)	FPL grading at Broadview Ave and around BMW dealership to protect properties east of the Don River;	TRCA	CGVD28:78
Portlands Flood Protection Survey (2018)	Existing survey of ground surface within and immediately adjacent to the PLFP Project Limits	Callon Dietz	CGVD28:PRE78
LiDAR - Underpass Corrected	Existing ground surfaces and bathymetry outside of survey limits	TRCA	CGVD28:78
2008 Survey	Existing survey of bathymetry ground surface within and immediately adjacent to the PLFP Project Limits. Includes existing bridge soffit elevations.	Barnes	CGVD28:PRE78
Water Levels	Historical water levels in Toronto Harbour, and the long-term modeling of Lake Ontario water levels based on Plan 2014.	Water Survey of Canada, International Joint Commission	IGLD85

The reported elevation differences between the datums for benchmark 60UT153 (located on the CNR bridge over the DVP) were used to transform data from sources that are not on the project datum (CGVD28:PRE78). The elevations used for this benchmark were based on station reports from Natural Resources Canada and the Ontario MNRF COSINE online application. Those station reports give the benchmark elevation in the various datums, and are summarized in Table 2-2.

Table 2-2: Summary of Elevations for Benchmark 60UT153 from Natural Resources Canada and Ontario MNRF COSINE reports

Data Source:	Natural Resources Canada Station Report	Ontario MNRF COSINE Report
Benchmark ID:	60UT153	0011960UT153
Datum	**Elevation (m)**	**Elevation (m)**
CGVD2013	82.076	82.076
CGVD28:78	82.484	82.483
CGVD28:PRE78	--	82.604
IGLD85	82.568	--

Based on these elevation differences, converting to the project datum (CGVD28:PRE78) from elevations referenced in the IGLD85 datum, add 0.036 metres. For the CGVD28:78 datum, there is a discrepancy of 0.001 metres in the elevations reported between the two station reports. To calculate the adjustment from that datum, the average of the two elevations was used to calculate the datum conversion. Therefore, to convert to the project datum (CGVD28:PRE78) from elevations referenced in the CGVD28: 78 datum, add 0.1205 metres.

The resulting mesh topology from the merged data sources is presented in Figure 2.1 with more detailed images provided in Figure 2-12 and Figure 2-13.

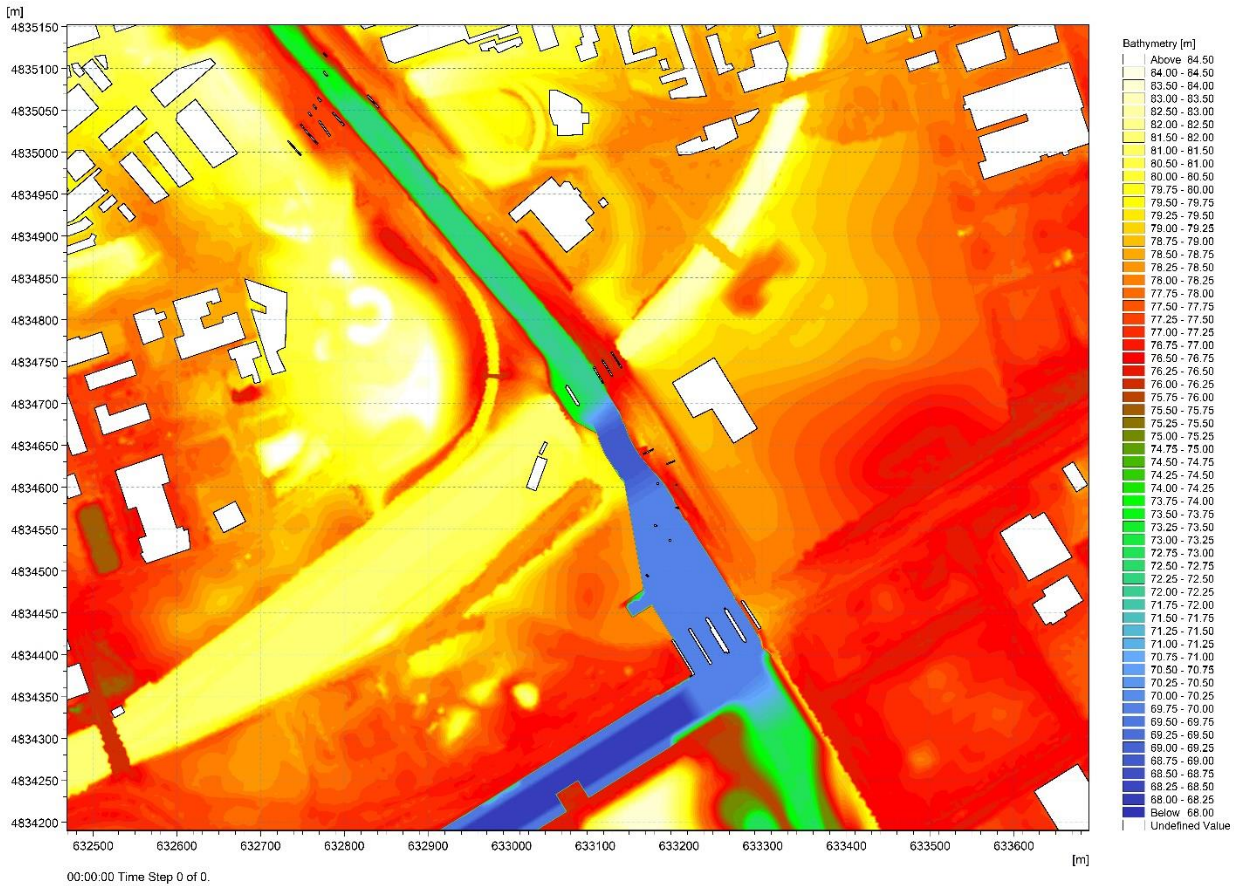


Figure 2-12: Northern Model Domain Topography

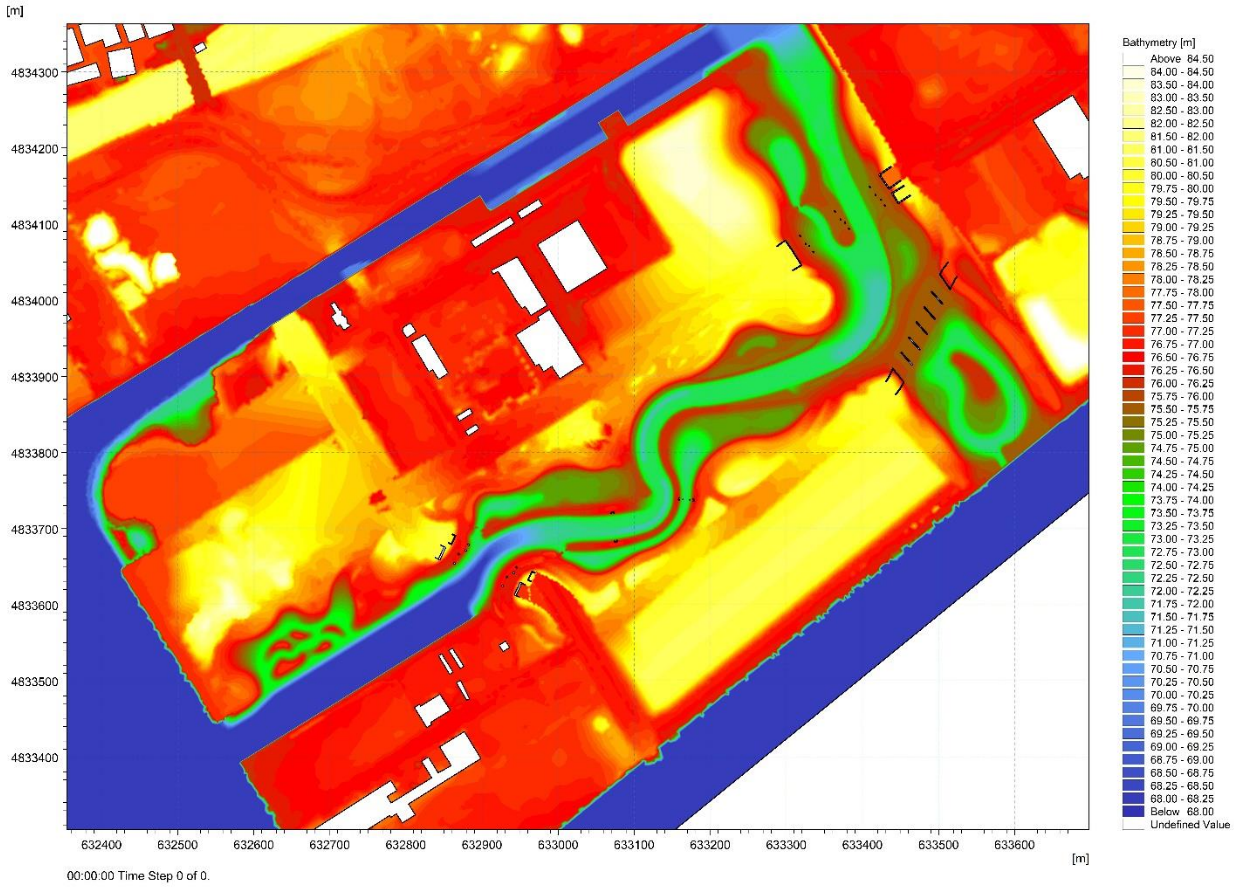


Figure 2-13: Southern Model Domain Topography

2.6 Design Updates

Since the last update to this Basis of Design Report, the model has been updated to incorporate several design changes that have occurred as the design has progressed. Most of these design changes are reflected in the modeled bathymetry, which are summarized in Figure 6.14, with additional detail provided in Figures 6.15 – 6.21.

The design changes that have occurred since the WP12 60% design update include the following:

- The Lake Shore Blvd. Bridge Pier Layout has been updated to reflect this Issued for Tender design from the PLFPEI Roads and Bridges Design Team.
- Updated SDMA layout in the Full Vision design, with the western dock wall aligned with the western Lake Shore Blvd. Bridge abutment. This is illustrated in Figure 2-15.
- Refinements to the SDMA layout in the Interim Conditions period, with realigned dock walls to reflect the future Full Vision, and an area where the existing sediment will remain in a sloped configuration to support the existing dockwalls under the westbound Gardiner Ramp. This is illustrated in Figure 2-16.
- Through coordination with the Broadview-Eastern Flood Protection (BEFP) project modeling being performed by TRCA, we have update the PLFP hydrodynamic model to include the BEFP Phase 1 grading around the Eastern Avenue exit ramp to the Don Valley Parkway and the BMW dealership site. This grading is shown in Figure 2-17.
- During the detailed design of the dock wall reinforcements required in the area between the HONI utility crossing and the SDMA, it was identified that foundation support piles for the HONI electrical tower adjacent to the west bank of the Don River channel project into the area where the

channel will be dredged out to provide flood conveyance capacity. The areas where the footings project into the channel will be boxed out with sheet piling, and filled with concrete to protect the piles. The design for this is illustrated in Figure 2-18, with the top elevation of the concrete and sheet piles set at elevation 73.0. The model grid in this area is shown in Figure 2-19 to show how the model bathymetry was set to simulate the impact of these protrusions.

- The fixed sideflow weir south of the Lake Shore Blvd. bridge has been removed from the project design, so the flow split between the Keating Channel and the River Valley is controlled by the constriction in the Keating Channel, and the impact of the longer flow path and friction losses in the naturalized river channel.
- The tie in to the existing dockwall and rock revetment slope on the eastern bank south of the Lake Shore Blvd. bridge has been modified to provide additional support the Don Roadway FPL that is being constructed above that bank. This is illustrated in Figure 2-20, along with grading modifications in the Ice Management Area that better align the naturalized channel with the Lake Shore Bridge bays, and the removal of the fixed sideflow weir.
- The existing dockwall projection ("bump out") from the south side of the Keating Channel is proposed to remain in place, and the angled flow transition from the narrow side has been removed from the design. This is illustrated in Figure 2-21.
- The grading in the Don Greenway has been adjusted to shift the channel to the west, and to eliminate one of the outflow channels. The model bathymetry for this area is illustrated in Figure 2-22.

These design changes were tested iteratively with interim modeling studies throughout the design period, and have all been incorporated into the model updates for this design milestone.

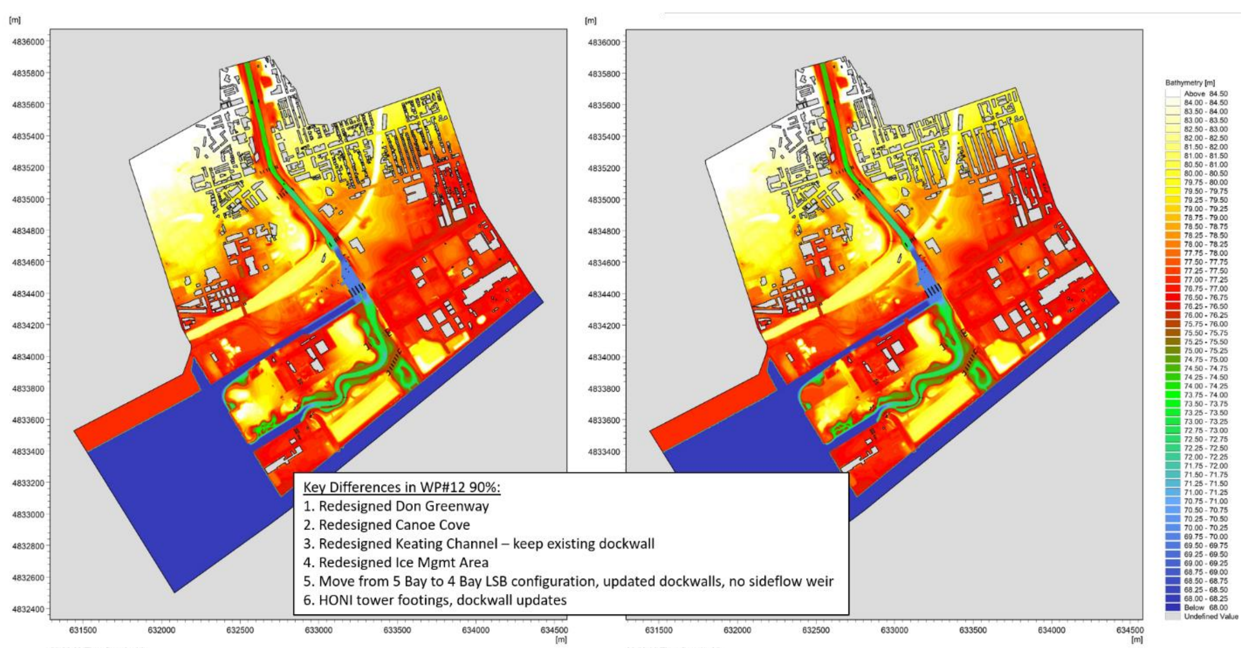


Figure 2-14. Model Bathymetry - Design Changes from WP12 60% to WP12 90% Design

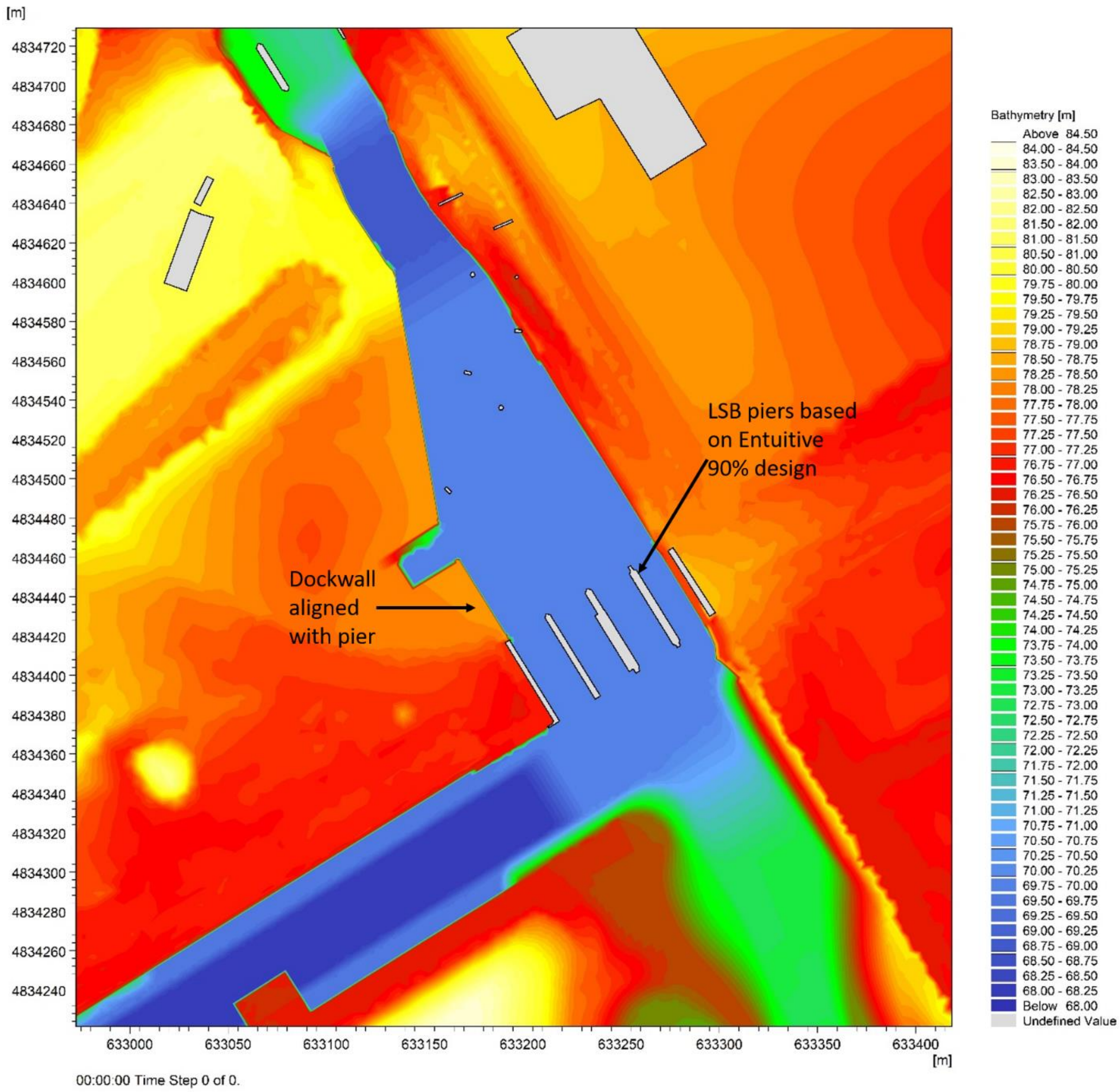


Figure 2-15. Model bathymetry in the SDMA - Full Vision Design

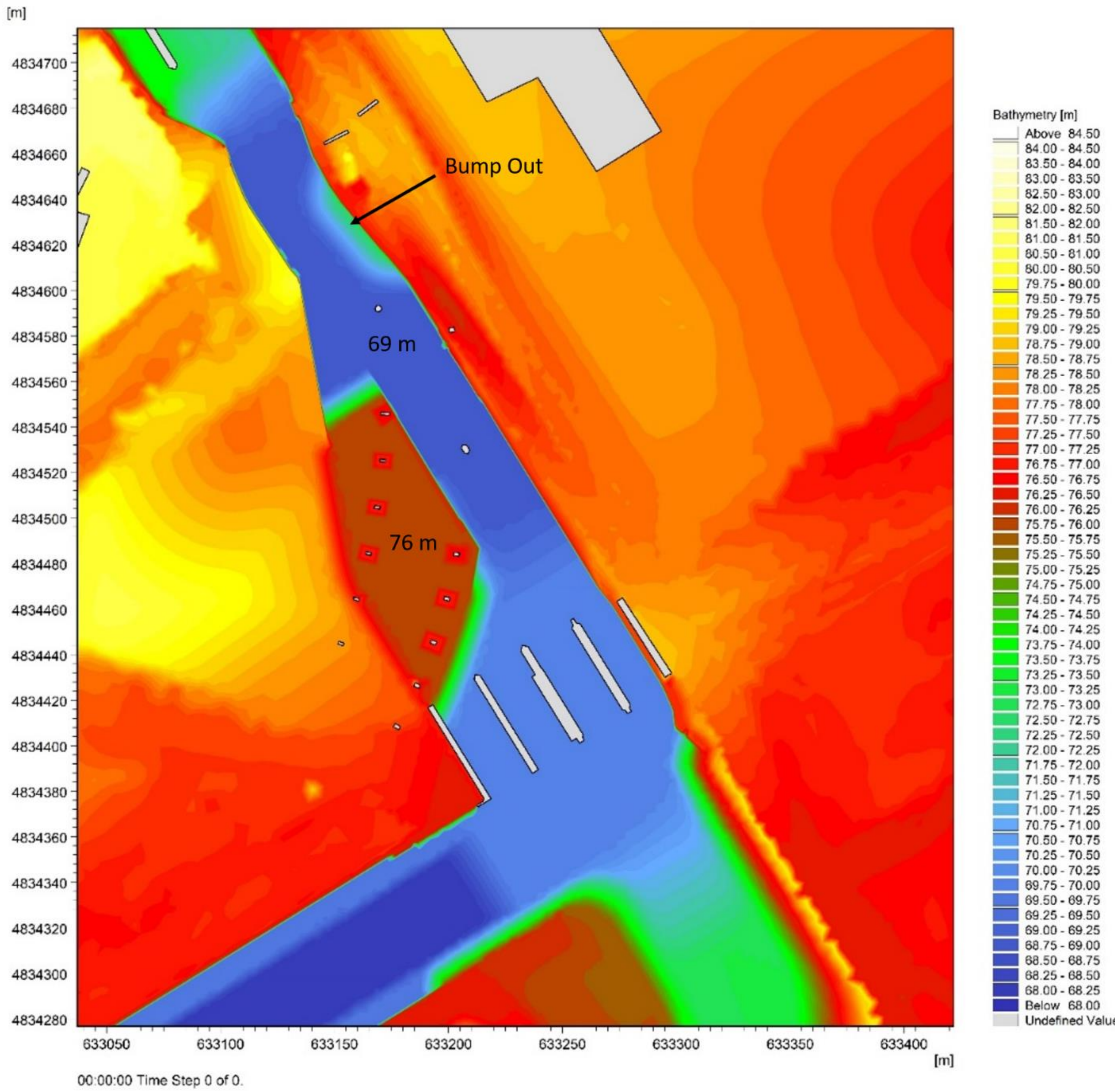


Figure 2-16. Model bathymetry in the SDMA - Interim Condition

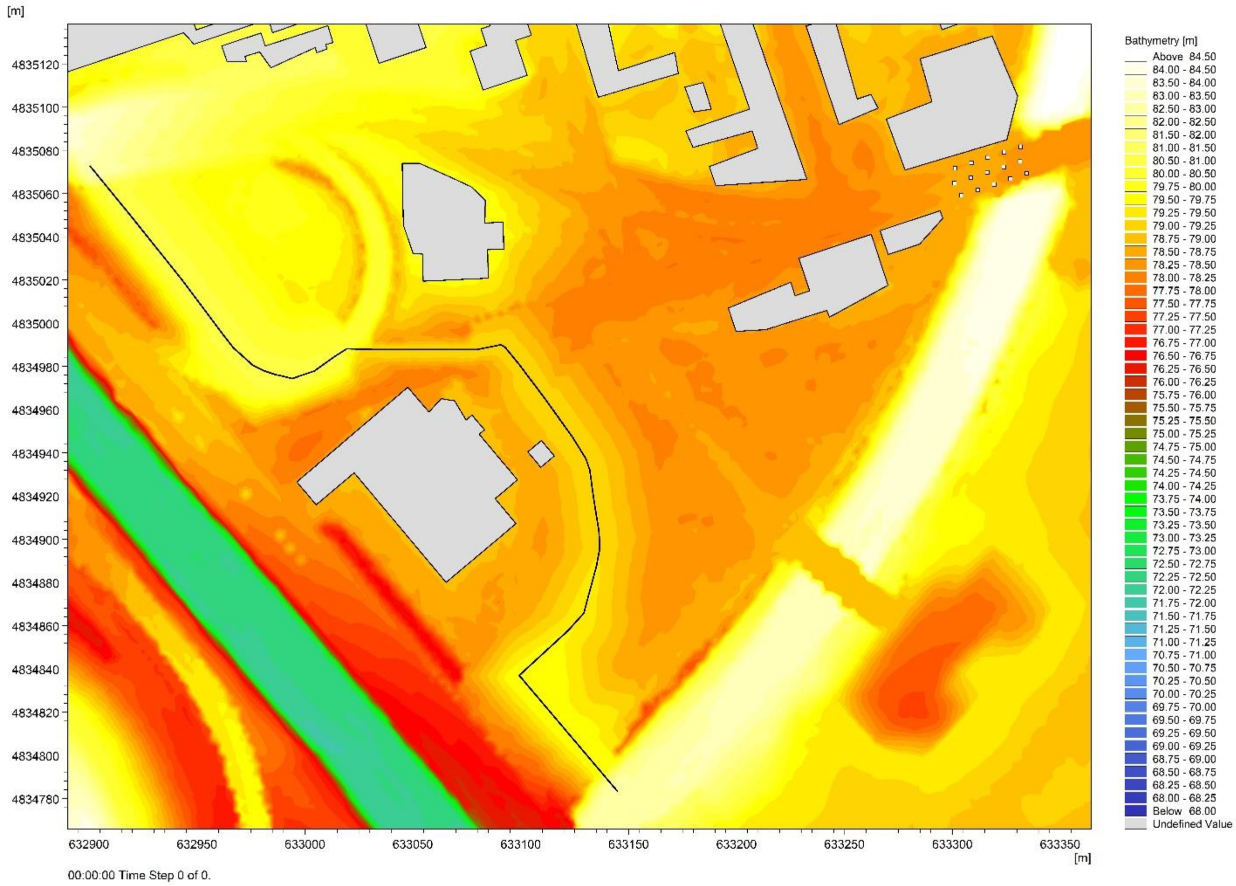


Figure 2-17. Model bathymetry - BEFP Phase 1 Grading

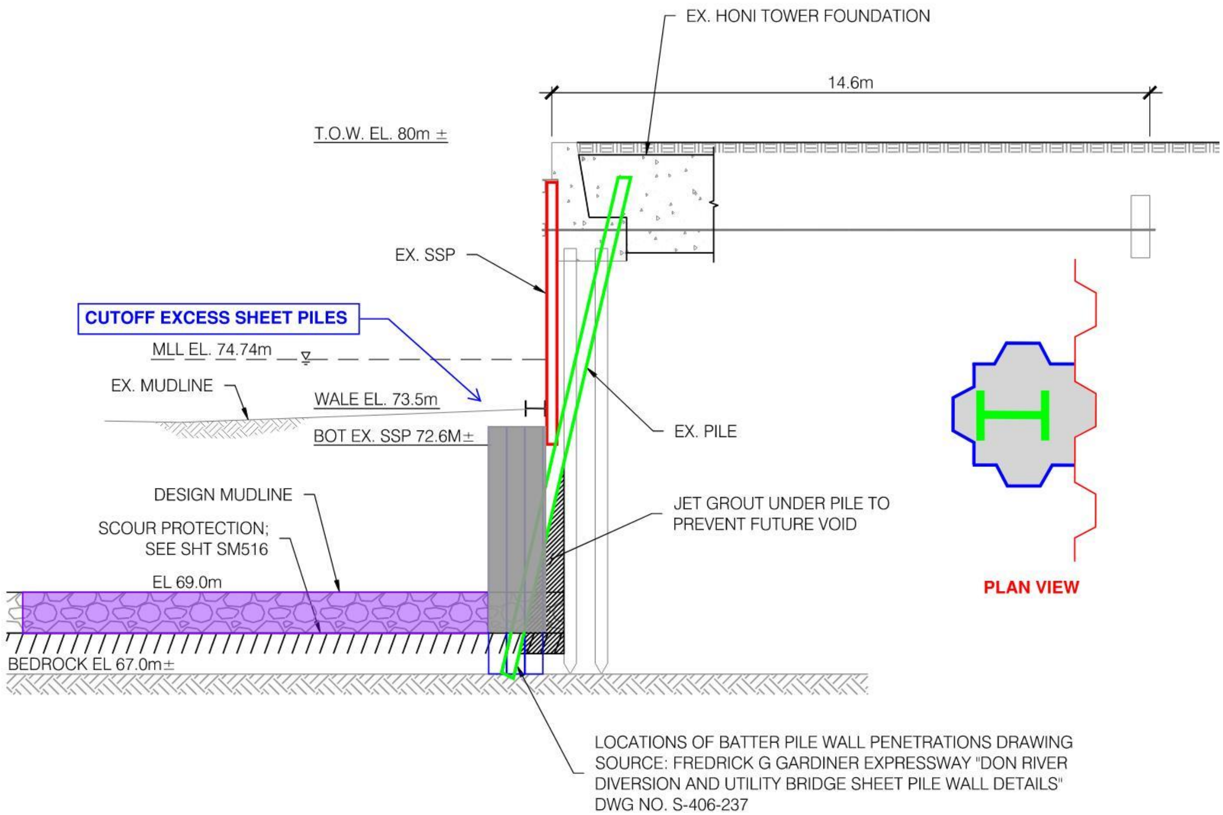


Figure 2-18. HONI Tower Footing Projection Design

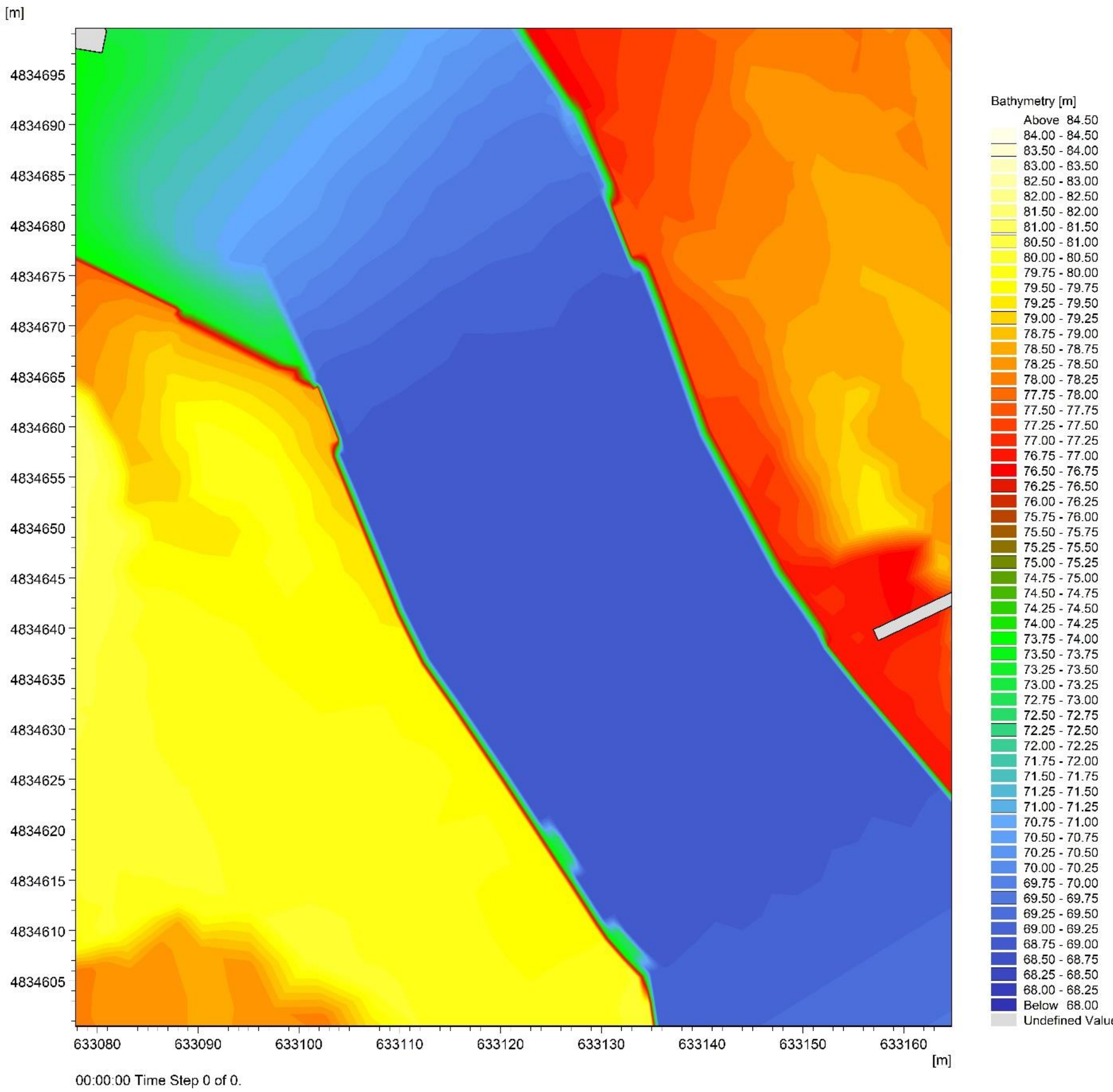


Figure 2-19. Model Bathymetry - HONI Tower Footing Projections

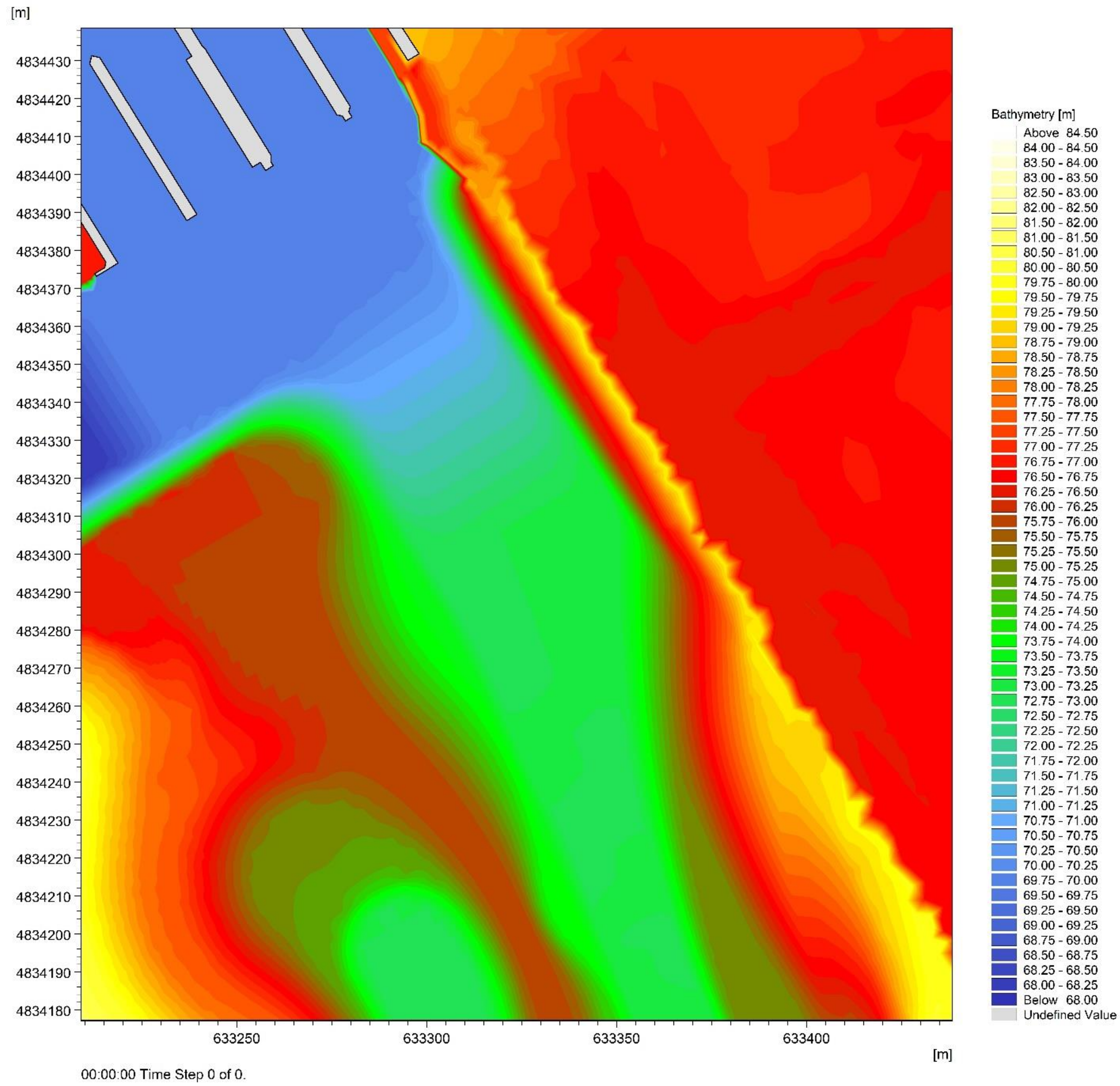


Figure 2-20. Model Bathymetry - Ice Management Area

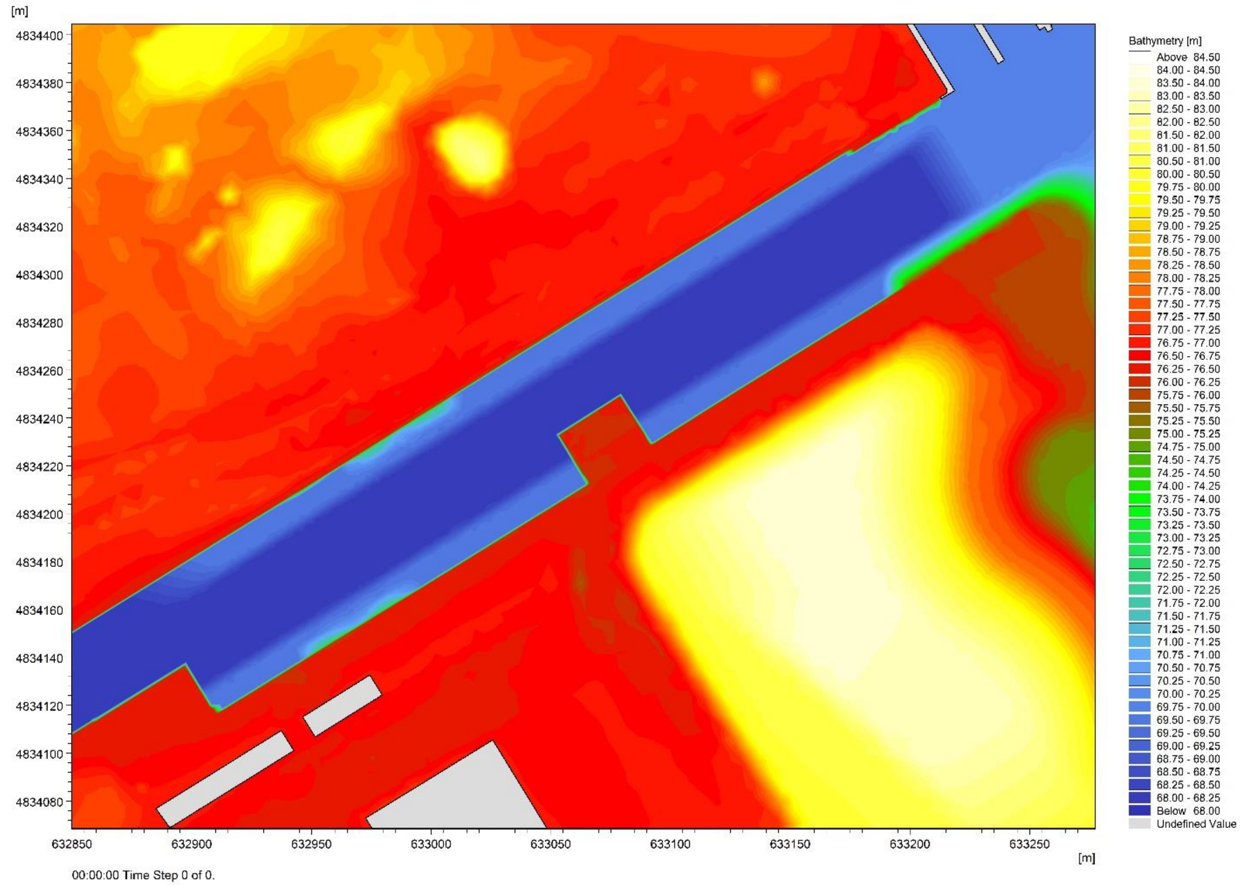


Figure 2-21. Model Bathymetry - Keating Channel

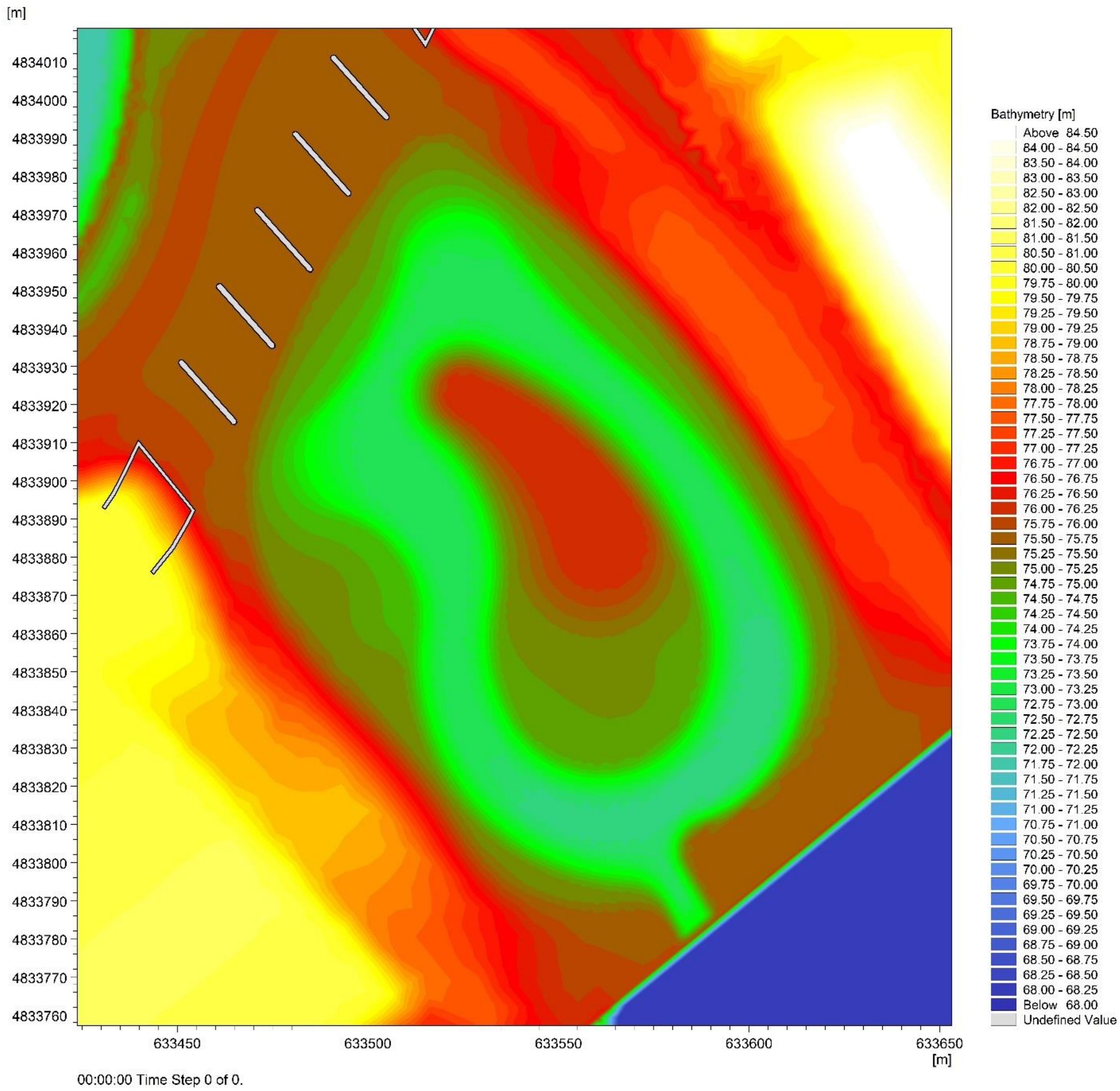


Figure 2-22. Model Bathymetry - Don Greenway

2.7 Boundary Conditions

The land/building boundary conditions utilized in the Analysis Model remained unchanged. The incoming flows from the Don River have been modified and are discussed in the "Flood Event Inflows" and the "Low-Flow Inflows" sections. The roughness inputs have also been modified to reflect the design, but are consistent with TRAC guidance.

2.7.1 Flood Event Inflows

TRCA and their consultant have updated the Don River watershed hydrology model, which was used to determine the design flows for the PLFP site. The updated report was provided to the PLFP project design team to utilize in the design process. The hydrology used two methods to estimate the return period for various flows in the watershed. The first was using a Flood Frequency Analysis (FFA) based on the annual peak flows measured at the flow gage near Todmorden.

The second was using a return period rainfall design storm in the watershed hydrology model. For the lower flows (5 year return period and below), the FFA tended to predict higher flows than the hydrology model results. Above that, the hydrology model predicted higher flows for each of the return frequencies analyzed. The peak flows used in the Design Development phase of the PLFP were the higher of the flows predicted by the two methods. The draft design storm flow rates from the watershed are identified in the table below (Table 2-3).

These flows were used to define the inflow boundary conditions for the upstream end of the PLFP hydrodynamic model for the various flow conditions.

Table 2-3: PLFP Project Design Flows

Return Frequency / Description	Design Storm Peak Flows (AES 12-Hour Storm) Node 48.2	Scaled Flows from Flood Frequency Analysis [m3/s]	Peak Flows to use in PLFP Design [m3/s]
1-Year	--	--	91.1[1]
2-Year	85.71	138	138[2]
5-Year	160.94	181	181[2]
10-Year	215.79	208	216[3]
25-Year	283.08	232	283[3]
50-Year	349.00	257	349[3]
100-Year	422.15	276	422[3]
350-Year	588.40	--	588[3]
Regulatory Flood	1,560	--	1,560[3]

[1] Calculated from the 90-Percent Annual Exceedance Probability at the Don River at Todmorden, plus 15% to account for downstream drainage areas.

[2] Calculated from the FFA for the Don River at Todmorden, multiplied by the ratio of modeled flows between Todmorden and the PLFP site location to account for downstream drainage areas.

[3] From AECOM Don River Hydrology Update SWMM modeling report (Node 48.2)

In addition to the peak flows, the hydrographs of the predicted flows from the design storms were provided by TRCA based on the model output. The flow time series from the model output were scaled to match the peak flows in the table above for all the design storms. The design storm hydrographs are shown below in Figure 2-23, and the storm hydrograph for the Regional Flood is shown in Figure 2-24.

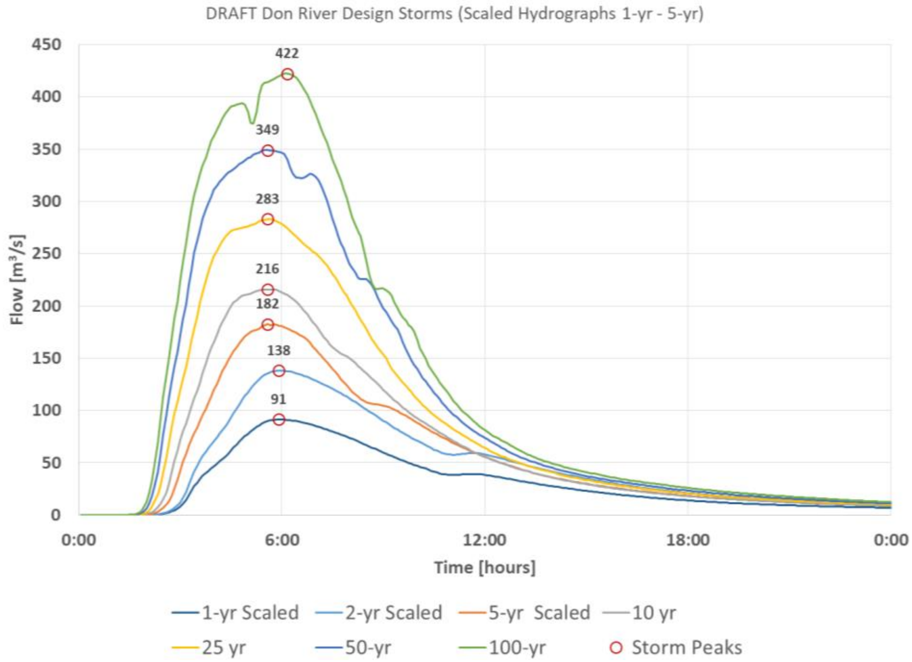


Figure 2-23: PLFP Don River Design Storm Hydrographs

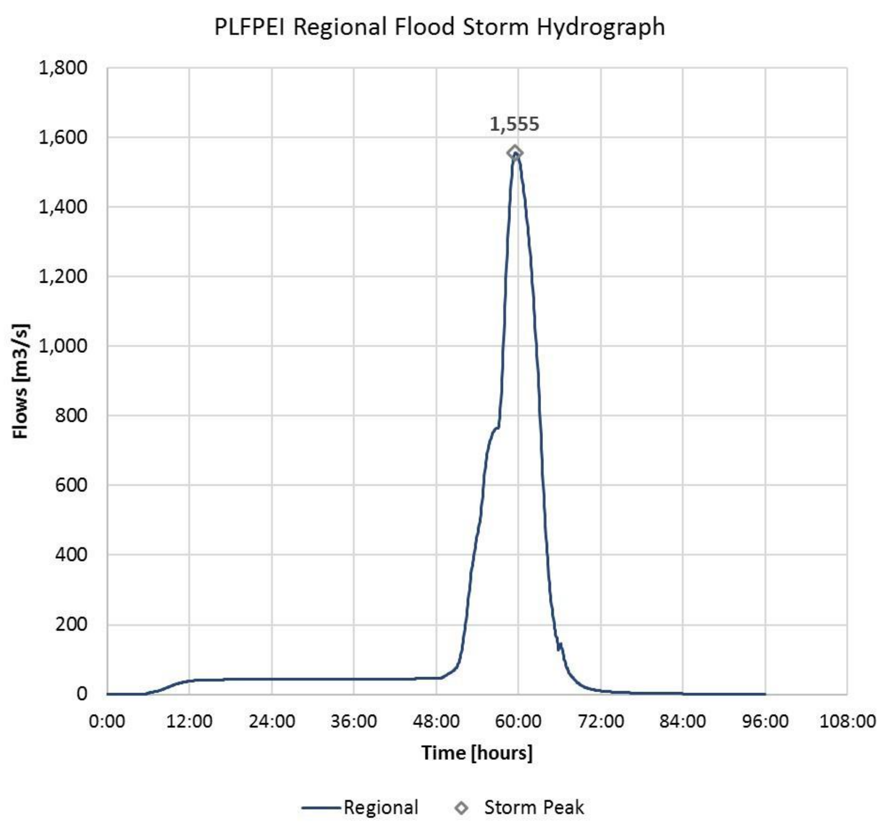


Figure 2-24: PLFP Regional Flood Storm Hydrograph

2.7.2 Low Flow Inflows

The closest flow gauge on the Don River is located at Todmorden, approximately 4.5 km upstream of the PLFP site. That gauge has a watershed area of 318.5 square km, which is 89% of the total watershed area upstream of the PLFP project site (356.8 square km). The tributary watershed areas downstream of the Todmorden gauge are primarily developed urban areas, with some open spaces and parks along the ravines. The hourly average flows from the Todmorden gage from 2000-2017 were analyzed to develop an understanding of the typical base flows to the system. Boxplots of the flow data (Figure 2-25) were produced following the Tuckey method, showing the 25th percentile, median, and 75th percentile flows, and the skew of the outlying data.

In general, the base flow to the PLFP site was considered to be represented by the inner quartile range (between the 25th and 75th percentiles), as represented by the box in the box plots. That represents the range of flows that occur 50% of the time. The box plots show that the distribution of the data skews to the higher range due to the influence of wet weather flows. The top of the whisker for each month represents approximately the 90th percentile flow.

The seasonal variability in the base flow (Figure 2-25 & Table 2-4) shows that the highest base flows occur during the early spring (March – April), with the flow decreasing over the summer to the lowest period during July through September.

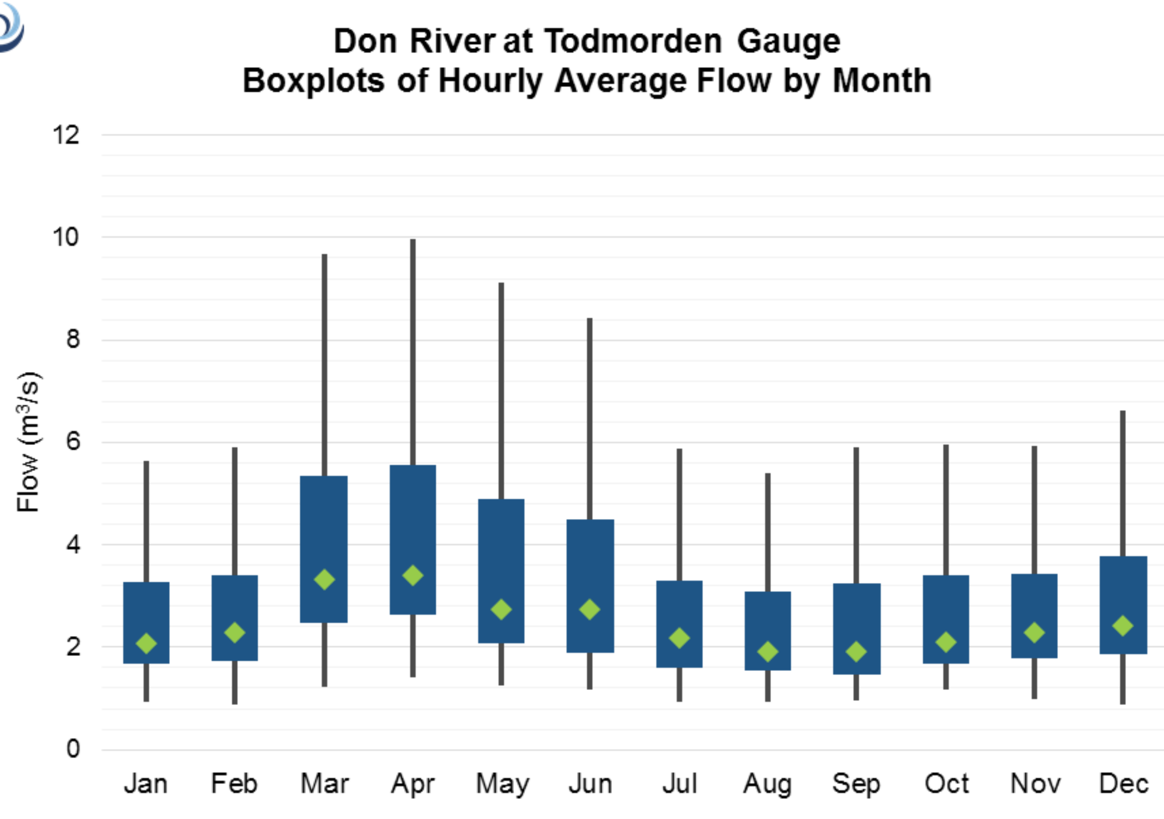


Figure 2-25: Monthly boxplots of Don River flows at Todmorden Gauge

Table 2-4: Don River Flows at the Todmorden Gauge by Month

	Flows (m3/s)											
Month:	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
75th Percentile	3.26	3.39	5.35	5.57	4.90	4.51	3.31	3.09	3.24	3.39	3.44	3.78
Median	2.08	2.28	3.33	3.41	2.75	2.75	2.17	1.92	1.93	2.11	2.29	2.43
25th Percentile	1.68	1.72	2.46	2.63	2.09	1.90	1.60	1.56	1.46	1.68	1.78	1.87

2.7.3 Lake Levels

The lake level within the Inner Harbour has a significant impact on the design of the naturalized river mouth, particularly for the ecological design of the system. In the hydrodynamic model, the lake level is represented as a downstream boundary condition. During high flow events, the lake level does not have a significant impact on the water surface elevations, particularly upstream of the constriction in the Keating Channel and the riverine wetlands in the River Valley. This has been confirmed by sensitivity analyses by both LimnoTech and TRCA for lake levels during Regulatory Flood simulations by testing different lake levels, including lake levels up to 75.84 metres.

A statistical lake level analysis for this project was performed by LimnoTech based on the historical water surface elevations measured in the Inner Harbour between 1958 – when the Lake Ontario water surface regulations began – and 2008. Since that analysis was performed, two events have occurred to necessitate the need to update it. First, the International Joint Commission has adopted a new lake level scheme (Plan 2014) for Lake Ontario, which is expected to result in a wider variation of the lake levels, primarily on the lower end. Second, during the spring and summer of 2017, Lake Ontario water levels were significantly higher than normal. Utilizing the most recent data to incorporate the new measured high lake levels will generally increase the probability of higher lake levels. A synthetic model-generated time series projecting the lakeside average water surface elevation based on the historical hydrology will be used along with the observed data at the Toronto Harbour gage to analyze the variation in the water surface at this site, since the locally observed data considers the impact of seches, wind set up, or other local variations in the water level.

The updated analysis shows that the Plan 2014 regulatory scheme will likely result in a wider variation in lake levels on both the higher, and the lower lake level elevations. The results of the updated statistical analysis are shown in Figure 2.17.

The analysis relied on a synthetic historic quarter monthly data set of the Lake Ontario WSE scheme modeled for the Plan 2014 regulation scheme and the historical water supplies to the basin. In addition, it incorporates daily WSE records from the Toronto Harbour station (02HC048) to analyze local variability (due to wind and seiche conditions), and the observed data in 2017 with the high lake levels observed.

Toronto Harbour daily WSE records were obtained through the Water Survey of Canada website for station 02HC048, Lake Ontario at Toronto, located at 43°38'38"N, 79°22'50" W. Data available consisted of seasonal daily WSE records from 1906 to 1909 and continuous daily WSE records from 1910 through the end of 2017, the latest values available at the time of analysis. Over the 100 years of daily record were analyzed, including 39,712 total daily WSE measurements were available. These daily Toronto Harbour WSE records were use in this current analysis to provide local variability to the synthetic historic quarter monthly data set representing Plan 2014, which represents a lake-wide average of Lake Ontario which was available from 1906 through the end of 2015. Quarter-monthly average lake water levels calculated for Plan 2014 under the historical net basin supply scenario were obtained from the Great Lakes Adaptive Management (GLAM) committee and used to support the analysis described in this memorandum (IJC 2014).

Monthly box plots of the data (Figure 2-26) were created to show the seasonally variable WSE, and the conditions that are normally encountered that will impact the hydrology and hydrodynamics of the PLFP project site. The water surface elevations in the boxplots have been adjusted from the IGLD85 datum to the CGVD28:PRE78 datum to correspond with the project datum.

The data analysis (Table 2.5) shows that the over the spring, WSEs tend to rise, with the highest WSEs typically occurring during the months of May through July. The WSEs then recede to the winter low levels over the months of August through October. There is approximately 0.5 metres of variation in the median monthly WSEs over the year.

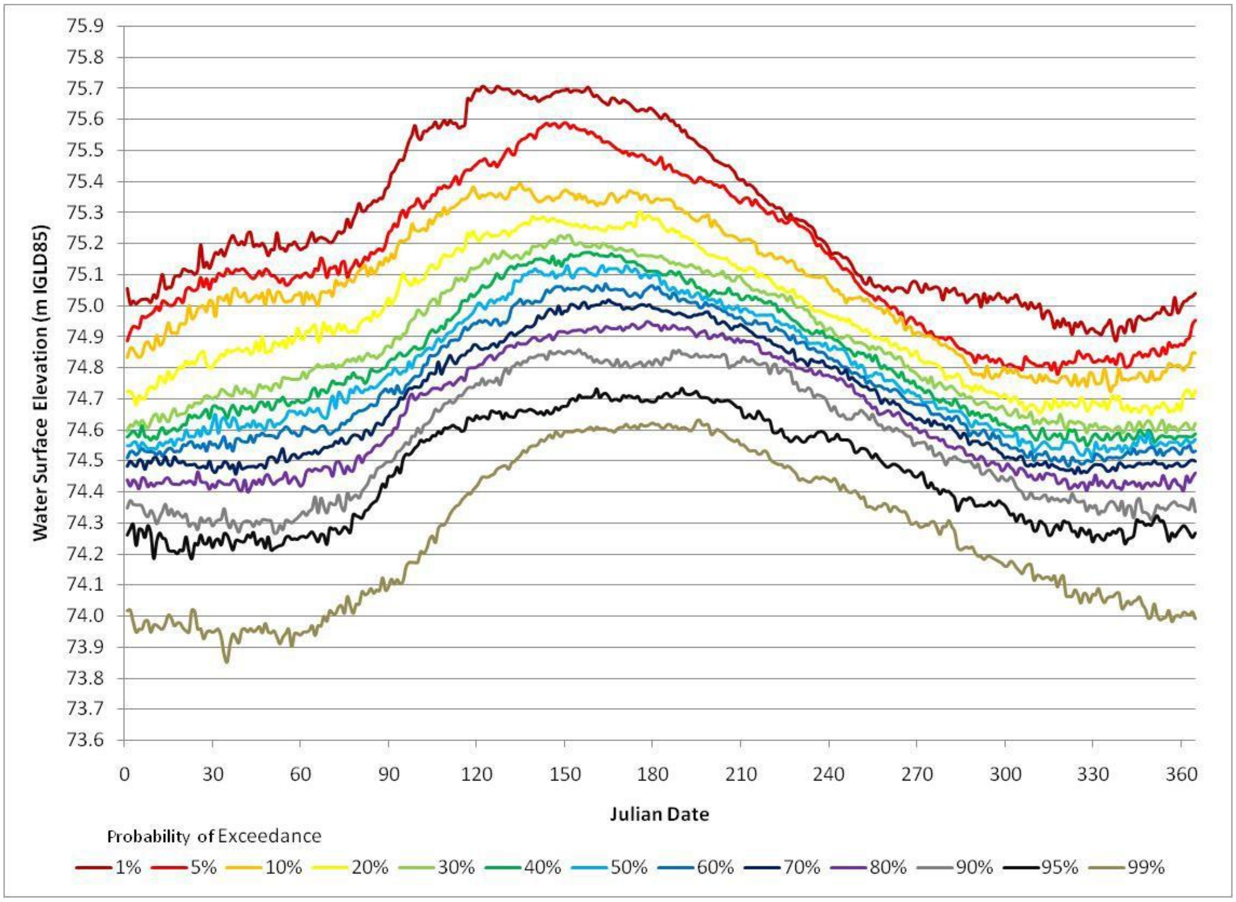


Figure 2-26: Daily Toronto Harbour WSE Post Regulation, Probability of a WSE at or Above an Elevation as a Function of Julian Date

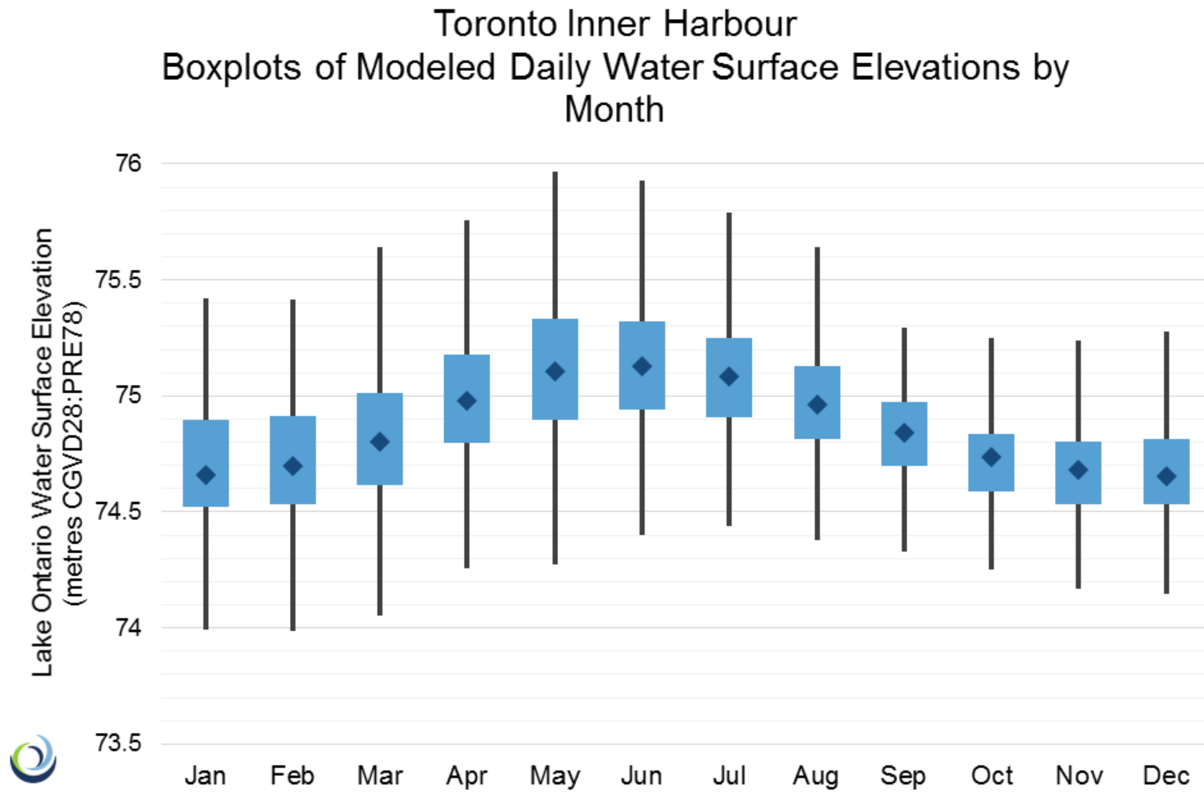


Figure 2-27: Monthly boxplots of modeled Lake Ontario Toronto Harbour WSEs

Table 2-5: Summary of monthly Lake Ontario Toronto Harbour WSEs

Toronto Harbour Water Surface Elevations (metres CGVD28:PRE78)												
Month:	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
75th Percentile	74.9	74.9	75.0	75.1	75.3	75.3	75.2	75.1	74.9	74.8	74.8	74.8
Median	74.6	74.7	74.8	74.9	75.1	75.1	75.0	74.9	74.8	74.7	74.6	74.6
25th Percentile	74.5	74.5	74.6	74.8	74.8	74.9	74.9	74.8	74.7	74.5	74.5	74.5

2.8 Roughness

The modeled roughness has been selected to be consistent with the March 2017 "Technical Guidelines for Flood Hazard Mapping" (Doherty et al, 2017). The surface materials utilized for the Naturalized Channel design are presented in Figure 2-28. The corresponding roughness (from Table 4.1.2 of Doherty et al 2017), are provided in Table 2-6, and the consequential MIKE Flood roughness map is provided in Figure 2-29. It should be noted that Doherty et al does not provide a standard roughness for Root Cribs, so a conservative value of n = 0.10 was selected.

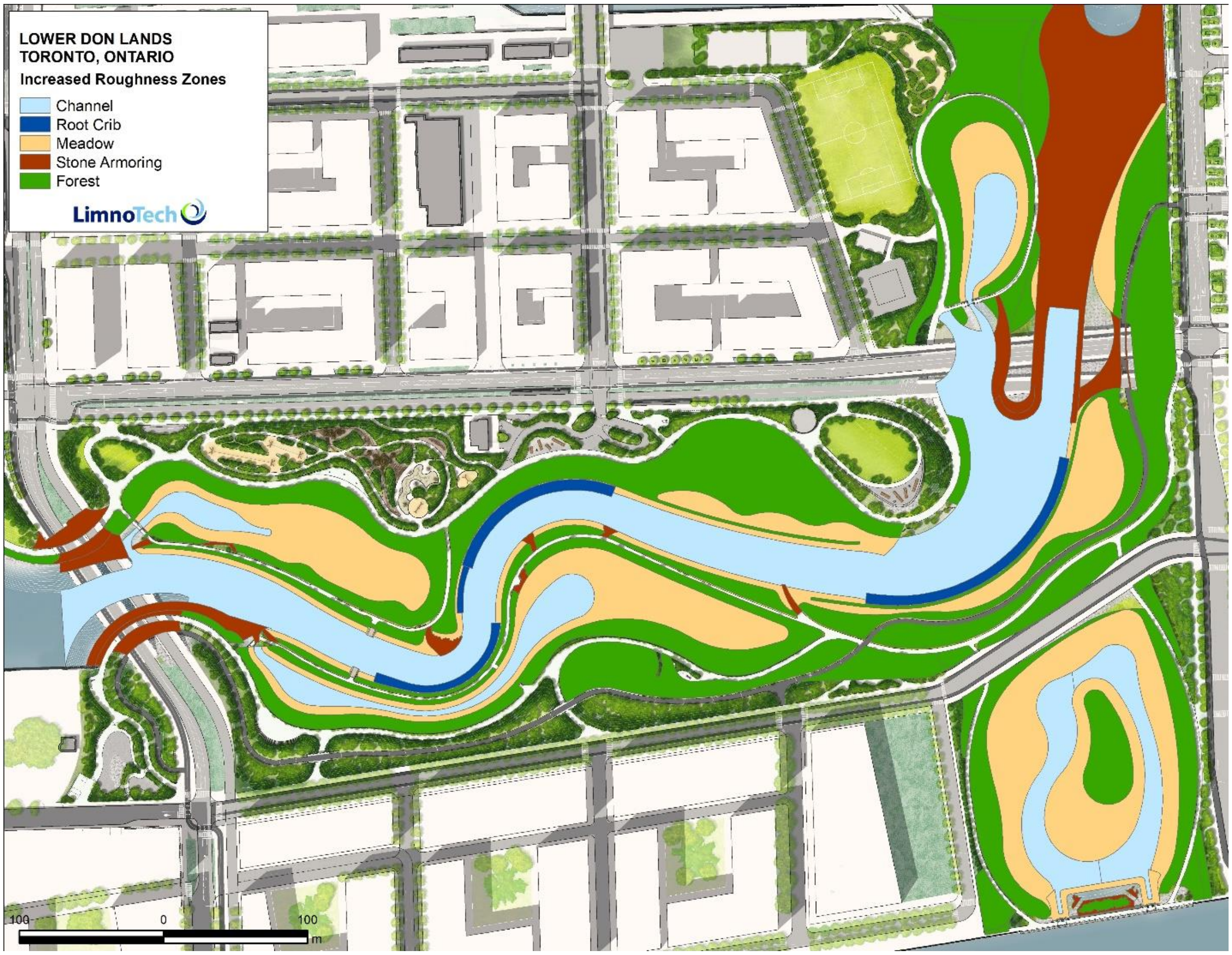


Figure 2-28: Naturalized Channel materials types

Table 2-6: Model roughness coefficients

Material Type	Manning's n	Manning's m
Armor Stone	0.025	40.0
Natural Channel	0.035	28.6
Meadow	0.055	18.18
Forest (Overbank Woods)	0.08	12.5
Root Crib	0.10	10.0

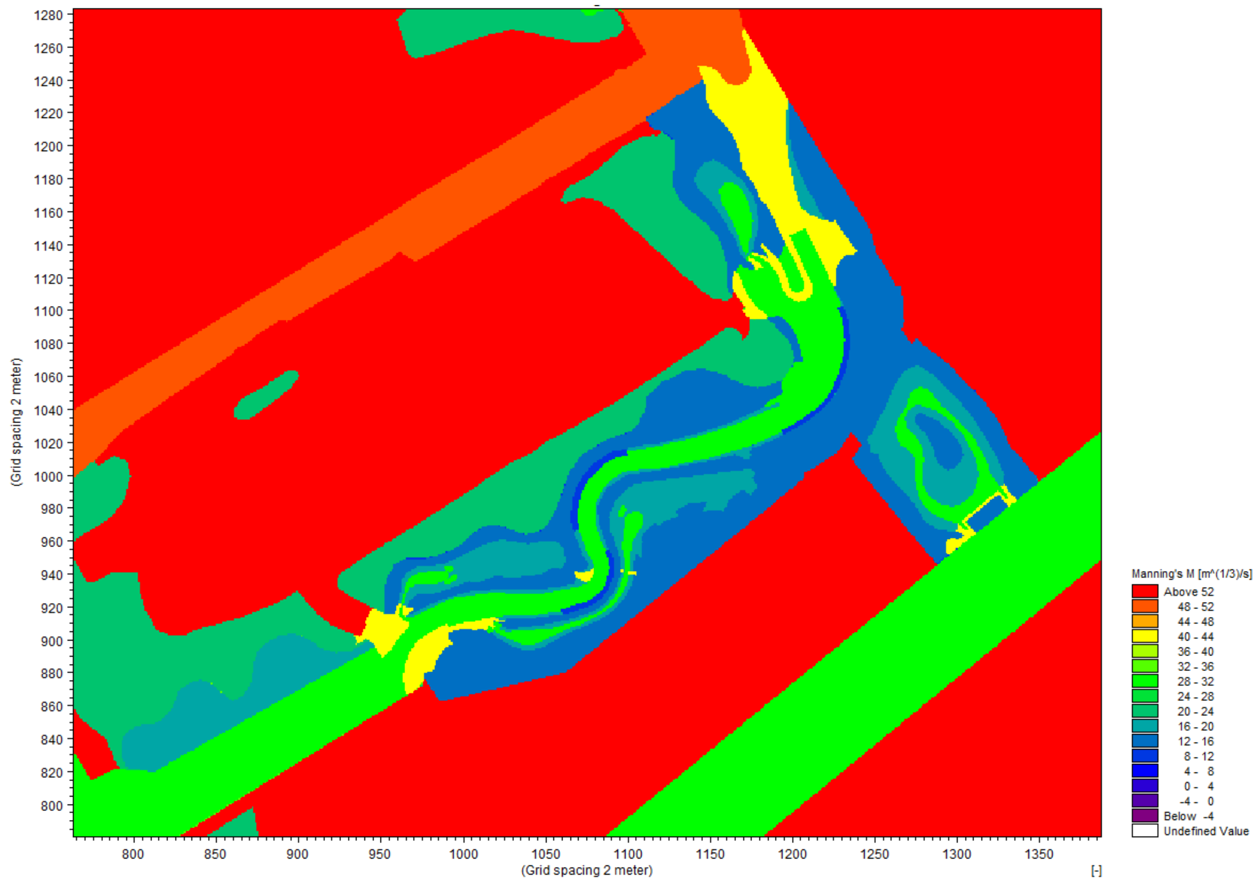


Figure 2-29: MIKE Flood Manning's m roughness map

2.9 Hydraulic Structures

2.9.1 Bridges

In the Analysis Report, four bridges were determined to be impacted by the Regional Flood. These bridges were represented hydraulically as a culvert (conveyance under the low chord) and a weir (water overtopping the bridge deck) and include the following:

- Old Eastern Ave Bridge
- HONI Bridge
- CNR Bridge
- Lake Shore Blvd Bridge

In the PLFP hydrodynamic model, the Old Eastern Ave Bridge was removed as it will be decommissioned. The Lake Shore Blvd Bridge and HONI Bridge were also removed because simulation runs showed that the water surface elevation do not strike the low chord during the Regional Flood event. At the CNR Bridge, only the center bay is represented in the Structures module and only as a culvert since the model does not predict flow over the bridge deck. Additionally, the CNR Bridge is only represented as a culvert in the Regional Flood event model. It should also be noted that when the CNR Bridge is represented without any culvert, the Regional Flood event's peak water surface elevation is slightly below the low chord of the CNR Bridge. This suggest that the culvert routine itself causes a slight increase in the water surface elevation.

All events equal to and smaller than the 100-yr (422 cms) have no MIKE 21 structures included; however, the CNR Bridge and Lake Shore Bridge abutments and piers are built into the mesh and bathymetry. The

3D Analyst Tool in ArcMap 10.6 was used to extract the bathymetry profile (station and elevation) under the CNR Bridge based on the MVVA Design Surface. This information was then used to populate the Level/Width dimension table in MIKE 21.

The design model includes significant changes made to how the CNR Bridge is represented in MIKE 21 compared to the Analysis Model. The first was its location, the line feature representing the culvert was moved from the center of the piers to the upstream end. The second change was updating the upstream and downstream invert elevations to reflect slope through the culvert (based on US and DS soffit elevations). The final update was using the 2008 Barnes survey data to update the soffit elevations in the Level/Width dimension table. Figure 2-30 shows the location of the bridge structure in MIKE 21 and Table 2-7 summarized the culvert parameters for the center bay at CNR and how certain parameters were updated between the Analysis model and the PLFP model.

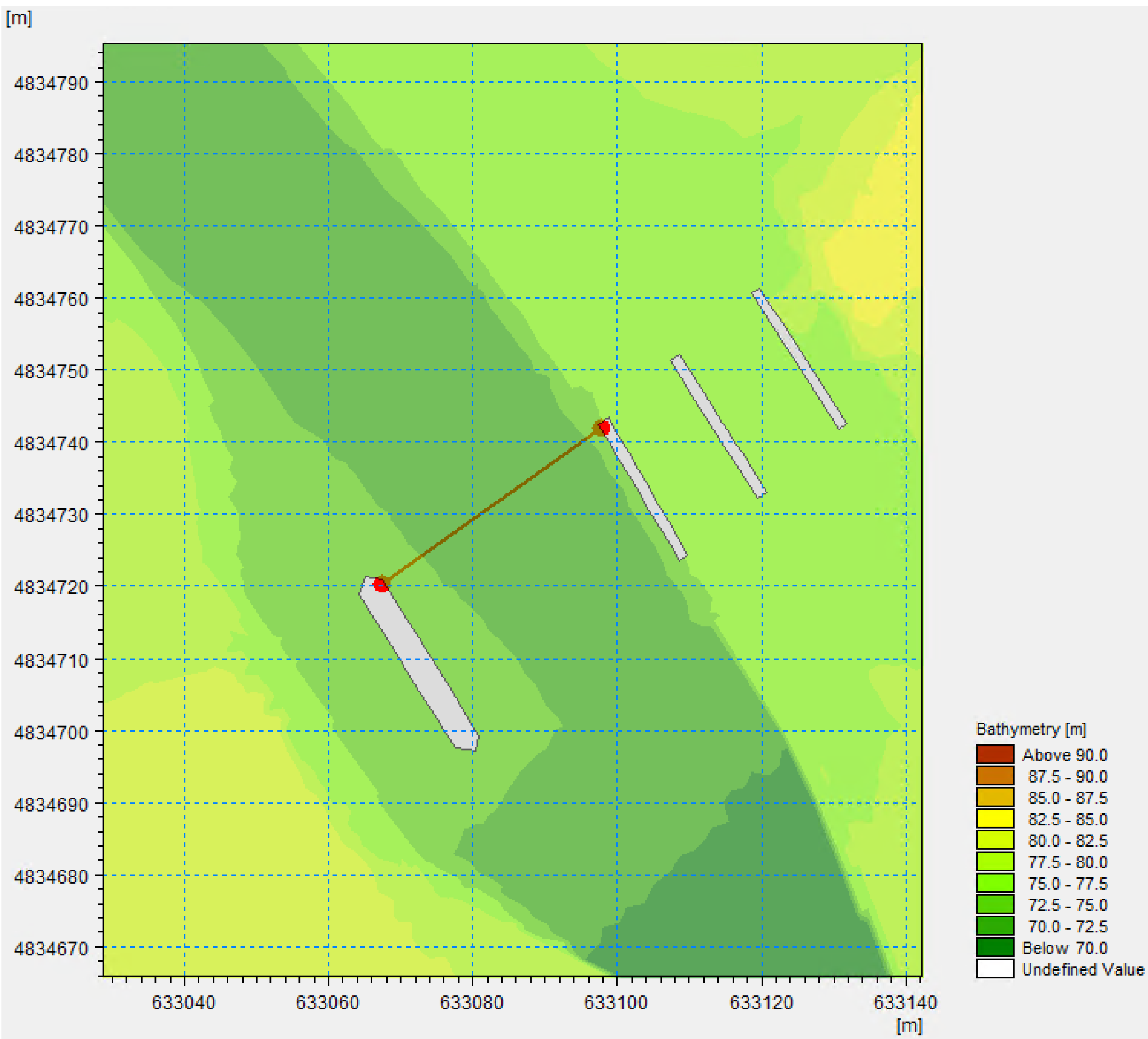


Figure 2-30: Location of CNR Center bay bridge structure in the PLFP Model.

Table 2-7: Comparison of summary culvert parameters between the Analysis Model and PLFP Model for the CNR Bridge.

Parameter	CNR Bridge (Center Bay)	
	Analysis Model	PLFP Model
Upstream Invert Elevation (m)	70	72.059
Downstream Invert Elevation (m)	70	72.049
Soffit Elevation (m)[1]	78.3	78.32
Culvert Length (m)	22.8	22.8
Manning's n	0.02	0.02
Head Loss Factor (Inflow)[2]	0.01	0.01
Head Loss Factor (Outflow)[2]	0.01	0.01
Head Loss Factor (Free)[2]	1	1
Head Loss Factor (Bends)	0	0
Non-uniform Flow	Yes	Yes
Free Flow Elevation (m)	77.89	78.195
[1] Lowest elevation where soffit is initially encountered		
[2] Head Loss factors are the same for positive and negative flows		

- The Upstream invert elevation corresponds to the existing bathymetry of the channel with a 1.5 m scour assumption. The Downstream invert elevation corresponds to the slope of the soffit, which was obtained from the 2008 Barnes survey.
- The Soffit elevations on each side of the Center bay were set using the 2008 Barnes survey. This soffit configuration was tested as a sensitivity run after the submission of Work Package #8, which used a soffit elevation of 78.3 m. All subsequent models will be run with the 2008 Barnes Survey soffit elevations.
- From the Analysis Report: "The Head Loss factors were all set to a very low number such that the calculated contraction and expansions head losses would be minimized since the piers are explicitly represented in the model mesh as flow barriers."
- From the Analysis Report: "The Non-uniform flow setting indicates that flow across the structure will be distributed according to the depth of water in each mesh element (i.e. elements with more depth get more flow)."
- From the Analysis Report: "The Free flow elevation indicates the water level below which the culvert equation is ignored and flow passes freely across the structure."

The CNR Bridge soffit elevations for the main bay over the Don River reflected in the hydrodynamic model was set at 78.32 metres, which is the lowest bridge soffit elevation noted in the Barnes survey. In addition, modifications were made to the free flow elevation threshold for this structure, which is the point the model transitions from a fully 2-dimension flow equations to a 1-dimensional representation of a culvert, which would be the condition if the flow contacts the bridge girders. A series of additional model tests were performed to verify the model sensitivity to the free flow elevation. The WSEs in all 4 runs with the varying free flow threshold options were very similar or identical. Based on those results, the free flow threshold elevation for the CNR Bridge in the hydrodynamic model was set to 78.195 metres, which is 98% of depth between the lowest invert elevation and the lowest bridge soffit elevation. Raising the free flow threshold in the model allowed for the free water surface calculations to show that the WSE is not impacting the soffit under the full vision design runs.

See the Analysis Report, Section 3.4.1 for additional discussion on the Head Loss factors and Free Flow elevation parameters.

2.9.2 Flow Curtain

Previous design milestones have included a sideflow fixed weir to split the flows between the Keating Channel and River Valley on the downstream side of Lake Shore Blvd Bridge. The sideflow fixed weir has been removed from the design, which allows for greater access for maintenance activities, including dredging, on the downstream side of the Lake Shore Blvd. bridge.

A floating flow curtain is included in the current design, spanning the eastern end of the Keating Channel from the west abutment of the Lake Shore Blvd. bridge to the dockwall on the southern bank. The purpose of this flow curtain is to promote favorable flow splits during low-flow events resulting in more water sent down the River Valley for wetland ecological function and sediment transport. The flow curtain is designed to freely rotate as flow rates change, rotating between 3° and 23°, which corresponds to an approximate flow range of 5.5 cms to 91.1 cms (1-yr event), respectively. The system is designed to break free once the curtain has rotated 30°, or approximately 126 cms. Figure 2-31 shows the location and orientation of the flow curtain as represented in the PLFP model.

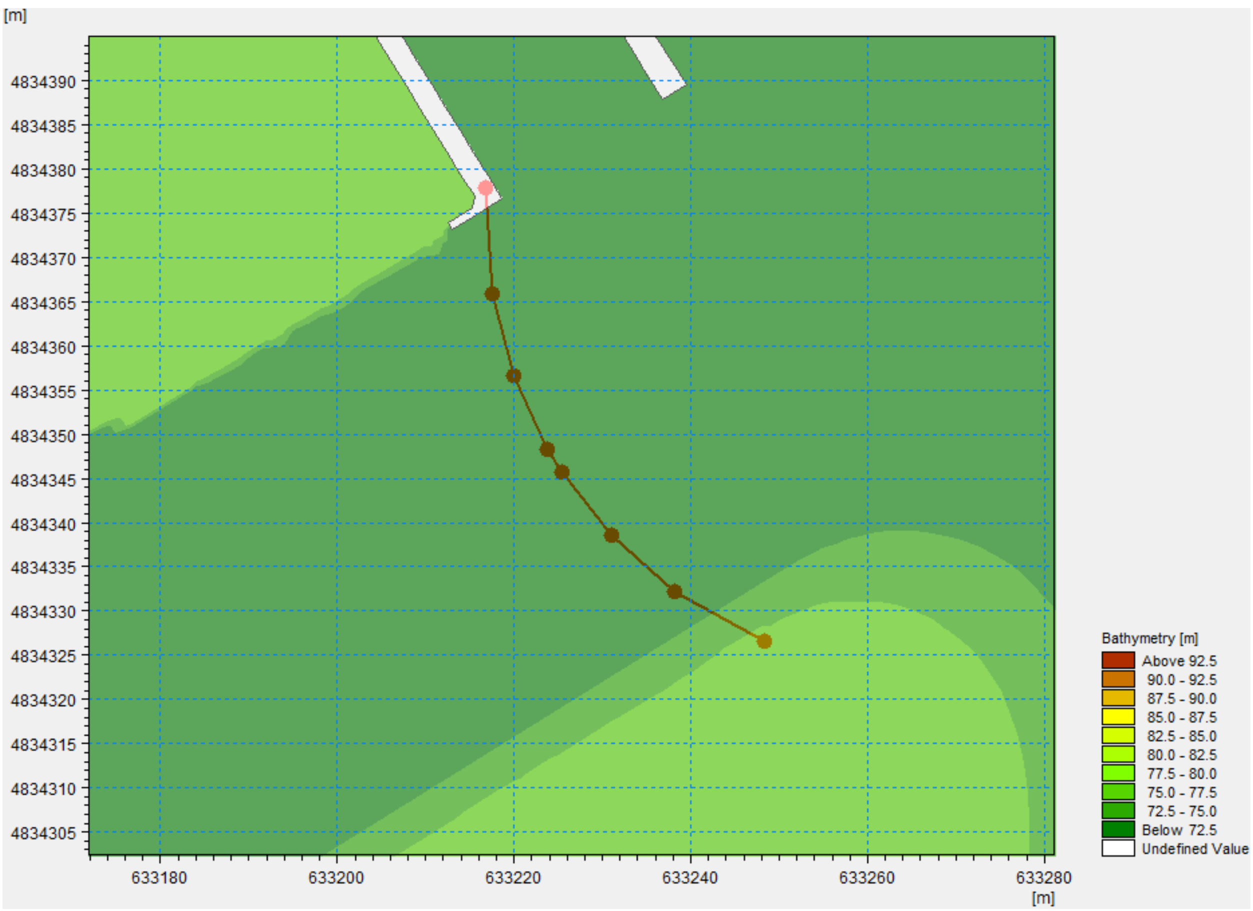


Figure 2-31: Location and orientation of the flow curtain across the eastern end of the Keating Channel

Within MIKE 21, the flow curtain was modeled as a Sluice Gate structure (Figure 2-32). The curtain is 3 m deep relative to the water surface. Since at low flows, the water surface within the Keating Channel is near flat, the top of the gate was set at an elevation equal to the tailwater boundary elevation and the bottom elevation of the gate was adjusted to compensate for the rotation of the curtain under varying flow rates.

Under Low Flow conditions up to 5.5 cms, the resulting elevation difference from a 3° rotation is negligible. For the 1-yr event, the 23° rotation results yields an effective gate depth of 2.76 m (0.24 m difference) relative to the water surface.

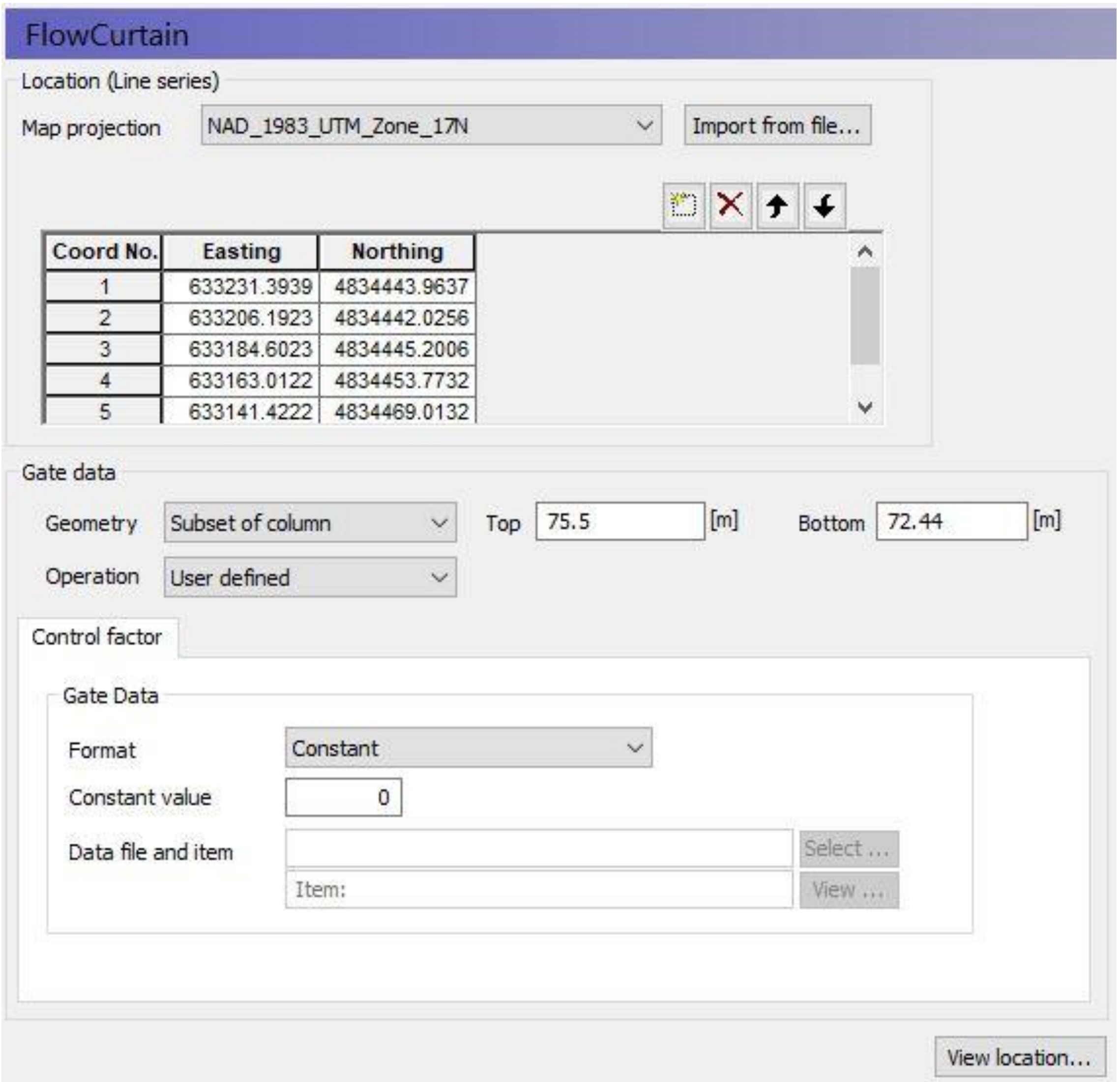


Figure 2-32: Example of the Gate structure options in MIKE 21. This specific example is for a Low Flow run with a tailwater of 75.2 m.

2.9.3 Flood Protection Landforms (FPLs) and Valley Wall Features (VWFs)

The flood protection landforms are represented in MIKE as bathymetric features (West Don, Eastern Ave, & Broadview) and as dike structures (First Gulf). These are the same methods as utilized by the Analysis Model (See Analysis Report section 3.4.3).

One of the design criteria states that there must be at least 0.5 metres of freeboard provided for each FPL/VWF. In order to test this requirement, water surface and FPL crest profiles were created for each FPL/VWF.

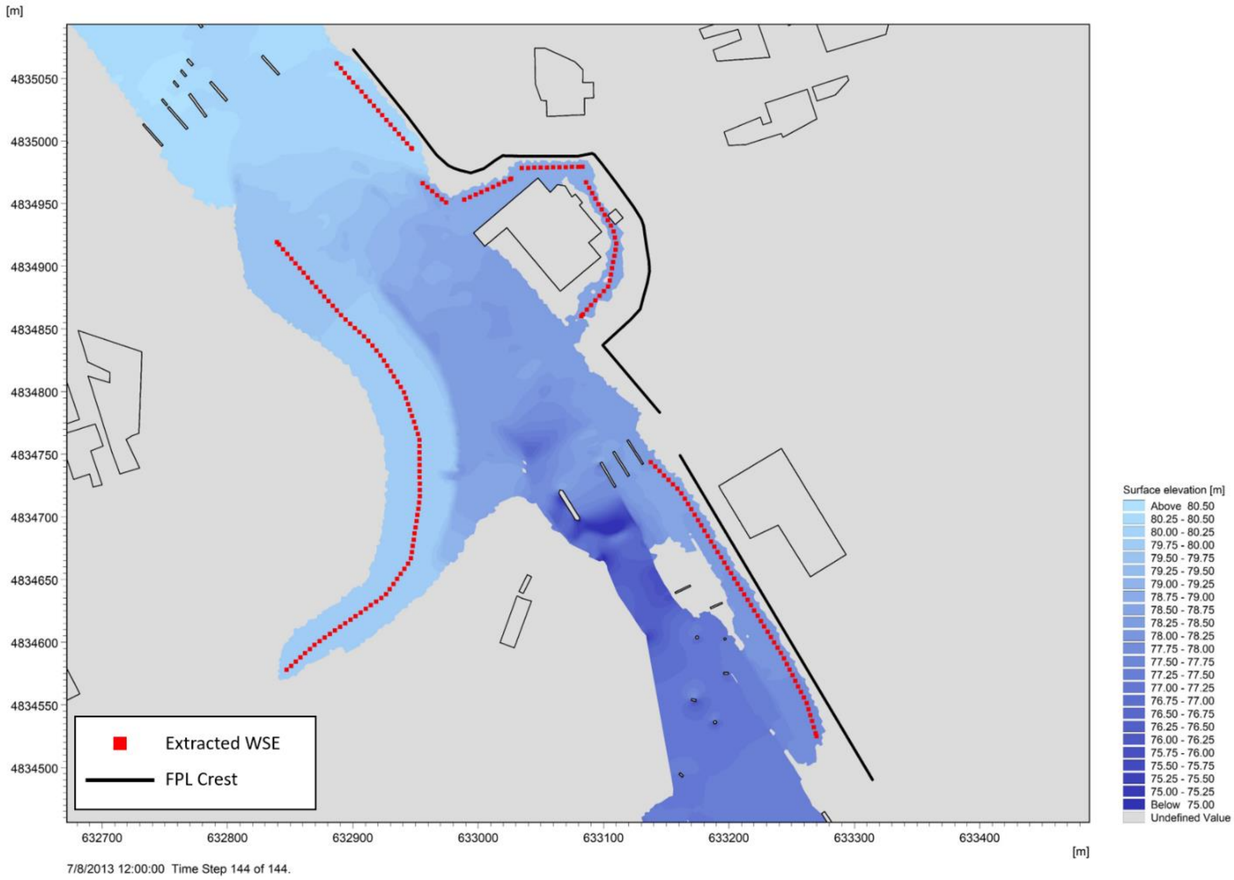


Figure 2-33: Flood Protection Landform Profile Locations

2.10 Hydrodynamic Model Settings

The model simulations have focused on steady flows as opposed to modeling the full inflow hydrograph for each of the proposed design storms. This is viewed as a conservative decision, since it reduces the peak flow mitigation impacts of storage in the channel and floodplain areas. In addition, it was made to increase design efficiency, and more efficient use of computer resources.

The Regulatory Event model begins with a zero inflow and ramps up to the peak over the course of 1.5 hours and then runs at a constant inflow for 2 hours. The smaller events also begin with zero inflow, but ramp up to the peak flow over the course of 20 minutes and then run as a constant flow for 2 hours. Longer constant flow run times where tested, and the simulations where found to have achieved steady-state within the 2-hour run time.

The remaining model settings are provided below. With the exception of the initial condition elevation and the additional output files, these model settings are consistent with the Analysis Model.

Solution Period

- Time step: 0.2 s
- Number of time steps: variable
- Start time: variable
- End time: variable

Solution Technique

- Modules Selected: Hydrodynamic & Inland Flooding
- Time and Space Integration: Higher order
- Min time step: 0.01 s
- Max time step: 0.2 s
- CFL number: 0.8

Flood and Dry

- Method: Advanced Flood and Dry
- Drying depth: 0.005 m
- Flooding depth: 0.01 m
- Wetting depth: 0.02 m

Eddy Viscosity: Constant = 1 m^2/s

Initial Conditions: Surface Water Elevation = Tail-Water Elevation (74.5 m or 72.5 m)

Output:

- Items:
    - Surface Elevation
    - Total Water Depth
    - U Velocity
    - V Velocity
    - Current Speed
    - Current Direction
    - Bed Shear Stress
    - CFL Number
- Cross-Sectional Flow Measurement Locations
    - River Valley Mouth
    - Don Greenway Spillway
    - Keating Channel
    - Commissioners St. Bridge (Upper River Valley)
    - Sideflow Weir
    - CNR Bridge
- Point or Profile Water Surface Elevation
    - Lake Shore Bridge (each bay, upstream and downstream)
    - Eastern Ave FPL
    - Broadview FPL
    - First Gulf FPL
    - West Don FPL

3 Hydrodynamic Model Application

3.1 Regulatory Flood Hydrodynamic Model Results

The current design surface to represent the model bathymetry was modeled to evaluate the hydrodynamics, including the flood extents, water velocities, and bed shear stresses from the flows to design the PLFP infrastructure improvements and natural channel design.

At the Lake Shore Bridge, the water surface elevations for the Regulatory Event with a Lake level of 75.2 m Bridge are shown in Figure 6.28.

The model results for the Regulatory Event can be found in Figures 6.38 – 6.40. Results for all the simulated design events can be found in Appendix G, Model Results Maps.

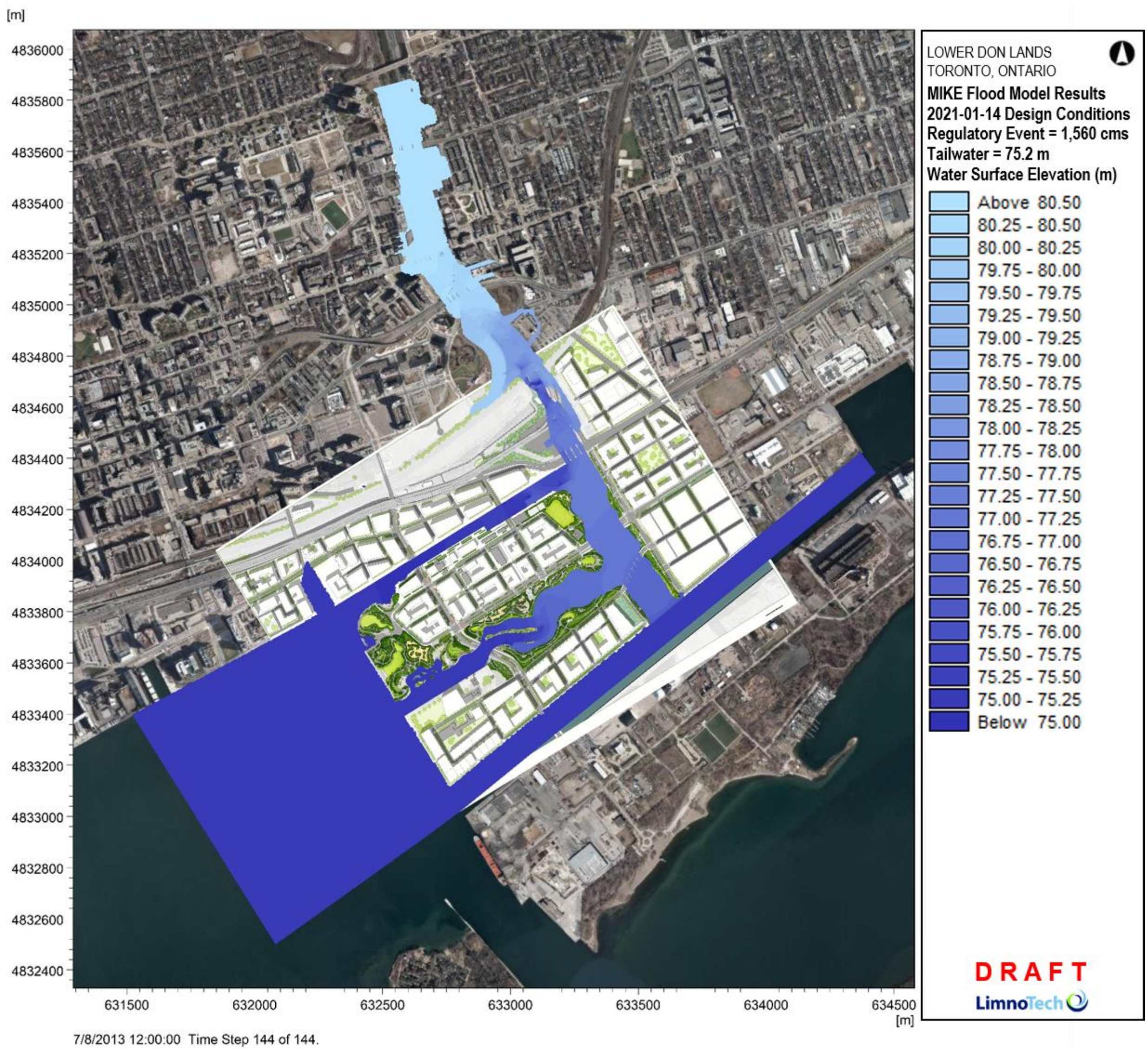


Figure 3-1. Flood extents and water surface elevations during the Regulatory Event for the Full Vision design (Lake level boundary condition 75.2 m)

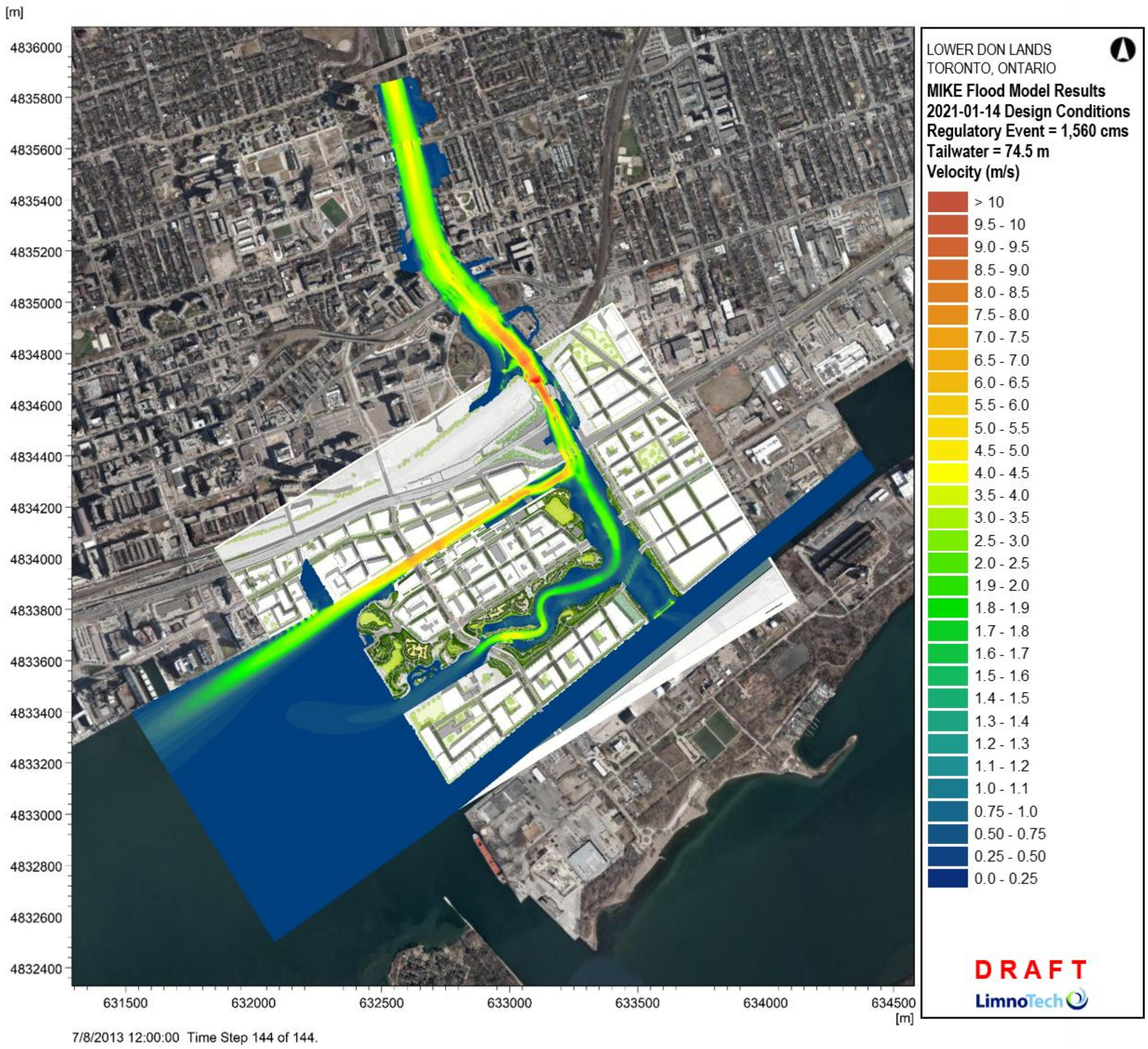


Figure 3-2. Velocities during the Regulatory Event for the Full Vision design (Lake level boundary condition 74.5 m)

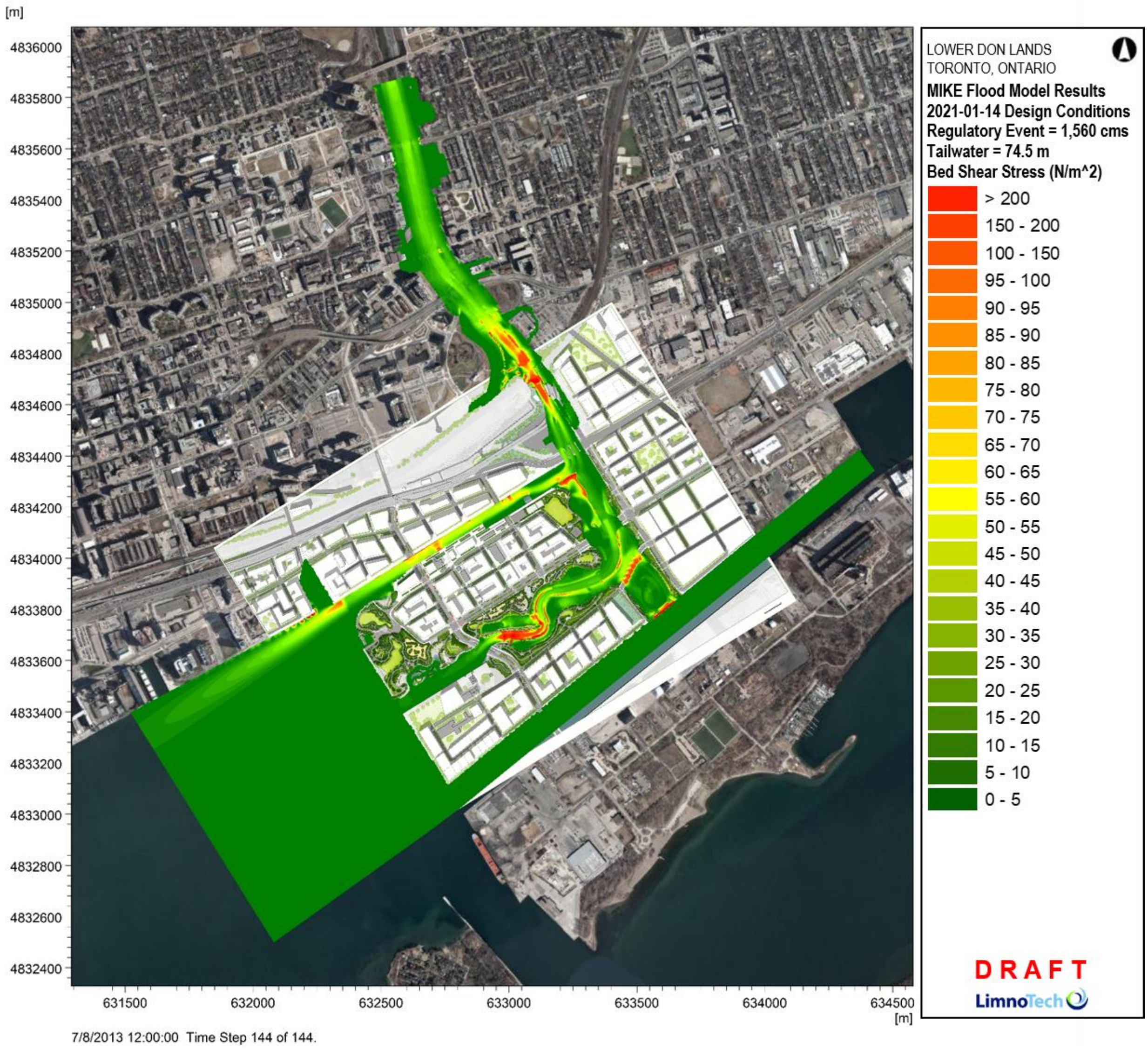


Figure 3-3. Bed shear stress during the Regulatory Event for the Full Vision design (Lake level boundary condition 74.5 m).

The design changes in the Keating Channel have changed the water surface elevation profile along the length of the Keating Channel, with the upstream dockwall projection being the hydraulic constriction that now controls the higher water surfaces that were previously controlled by the constriction where the channel transitions from the wider cross section of 60 metres to 35 metres in the narrow section. A profile of the water surface elevations along the length of the Keating Channel for the 90% design compared with the 60% design is shown in Figure 6.41.

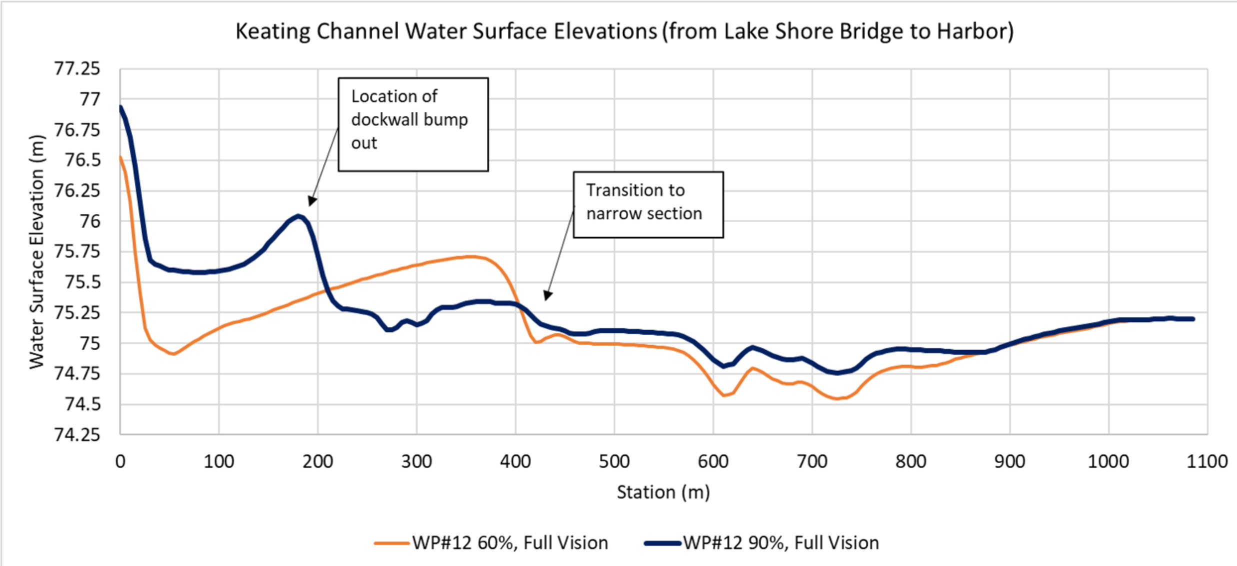


Figure 3-4. Model Results - WSE Profile in Keating Channel (Comparison of 60% Design and 90% Design)

The hydrodynamic modeling also includes a scenarios with the interim conditions in place before the Gardiner Ramps are relocated. The critical design components required to convey the Regulatory flood are:

- Set the grading in the interim area around the existing Gardiner ramp support piers at elevation 76.0 metres, with higher areas above the pier footings.
- Construct a dock wall along the majority of the east bank of the future SDMA to stabilize the bank and support the adjacent roadway, and minimize restriction of the channel.
- Leave a sloped area of the existing sediments to support the existing dock walls on the east bank under the ramp from the Don Valley Parkway to the Westbound Gardiner highway. This area has low overhead clearances, and would require substantial underwater work to perform the dock wall repairs.

The model bathymetry for the interim conditions is illustrated in Figure 2-16. Results for the Interim Condition Regulatory model run can be found in the Basis of Design Report Appendix G, Model Results Maps.

This option minimizes the encroachment on the Lake Shore Bridge soffit without excessively over excavating the Naturalized Channel. The additional capacity within the Keating Channel also provides sufficient flexibility in the naturalized channel that an ecologically functional channel design is possible without generating excessive velocities and shear stresses during the Regulatory Event.

Under the current design the elevation of the spillway into the Don Greenway is set at elevation 75.8 so that it will only activate at flood events larger than the 100-year event using a lake boundary condition of 75.2 m.

The current flood model runs demonstrate that the design of the PLFP will provide adequate conveyance of the regulatory flood event, with sufficient freeboard at the FPLs south of the CN Rail Bridge. The freeboard at the West Don Lands FPL is less than the 0.5 metres, but the design of the PLFP improvements are not what controls the critical water surface elevation at that tie off point. The flood modeling indicates that flow that escapes the Don River channel immediately downstream of the Eastern Avenue bridge builds up behind the railway embankment, and the outflow from that area is primarily controlled by the size of the Bala Underpass culvert, and the top of the railway embankment overflow.

Those are primarily controlled by the scour assumptions in the Don River channel north of the CN rail bridge.

Given that the FPL elevations are variable, profile plots along each of the FPLs have been generated. Figure 3-5provides a map of the profile locations and Figures 3-5 through 3-8 provide the water surface profiles along each FPL.

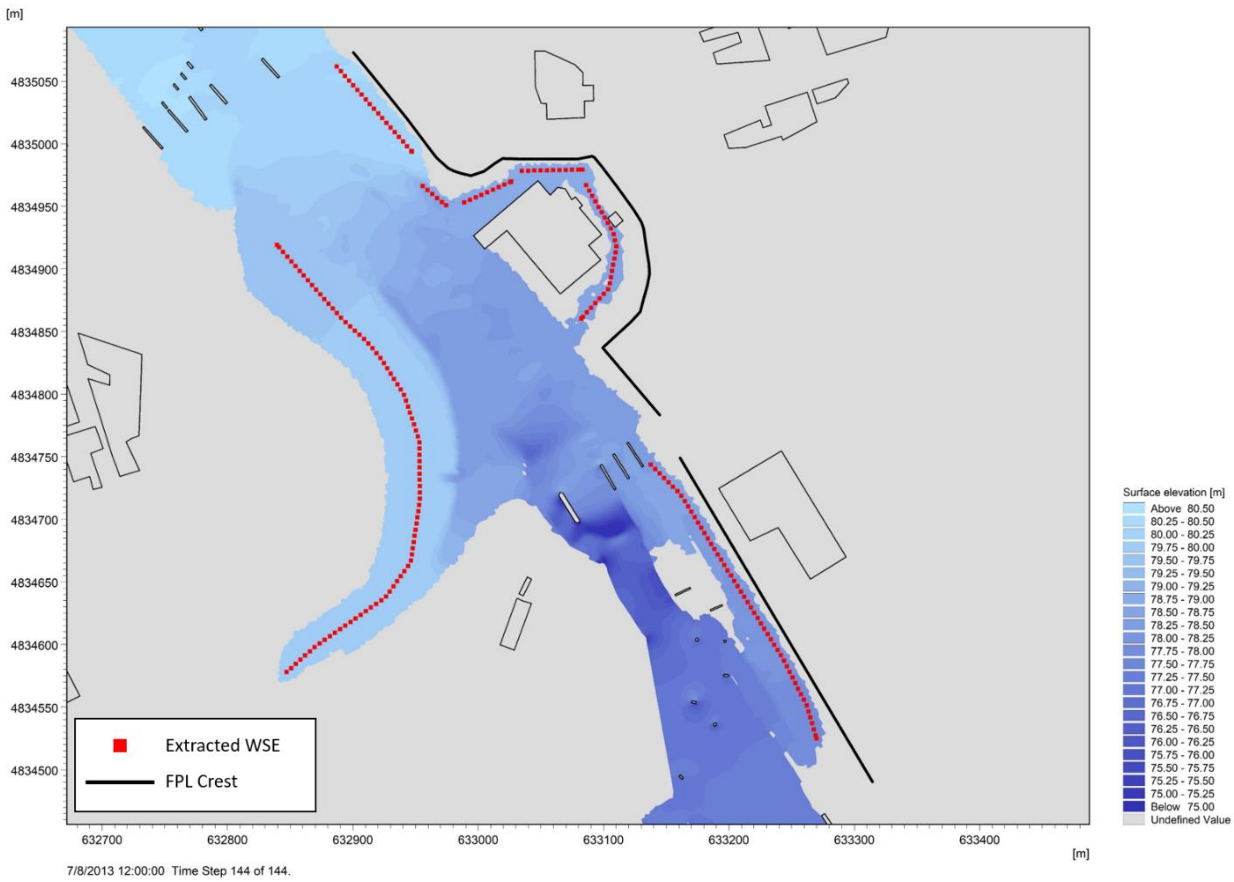


Figure 3-5. FPL Map

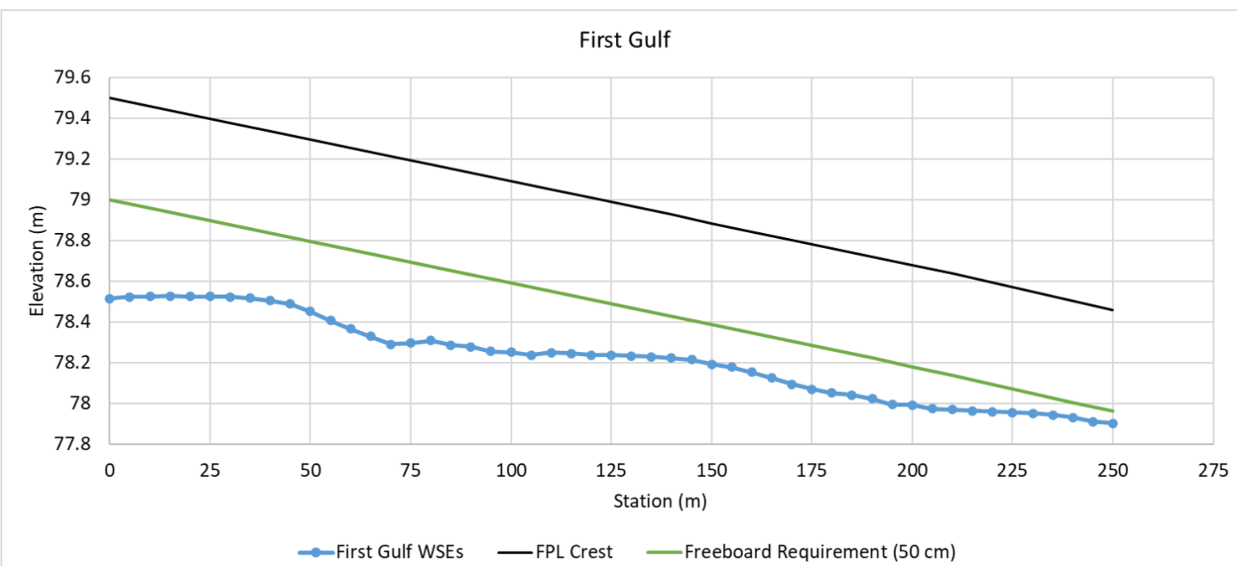


Figure 3-6. Profile along the First Gulf FPL

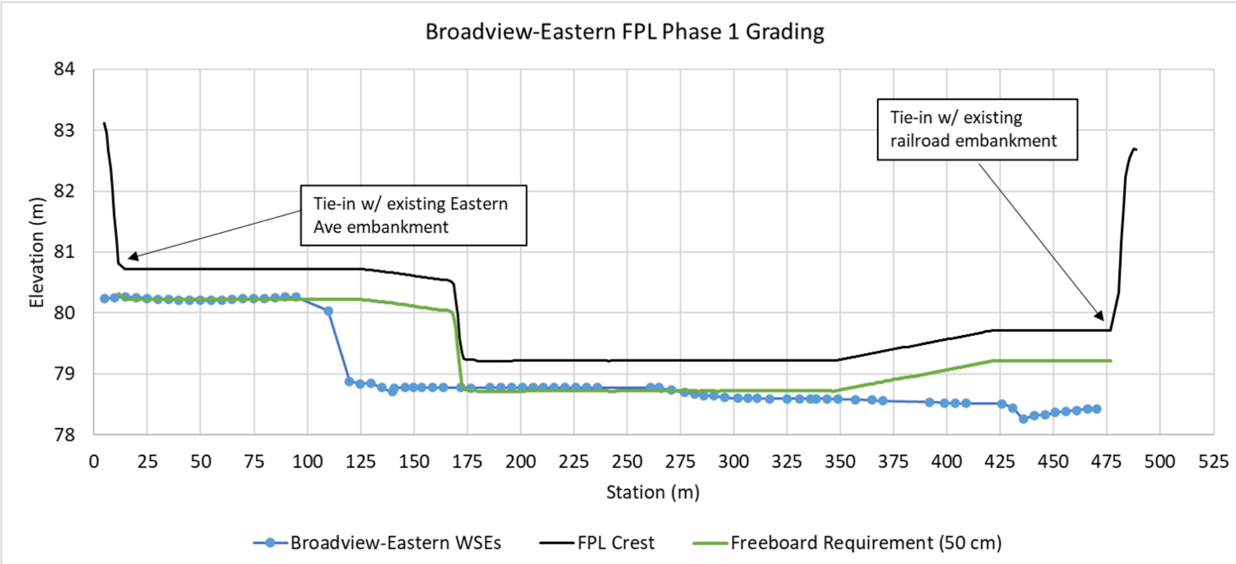


Figure 3-7. Profile along the Broadview-Eastern Flood Protection Phase 1 Grading

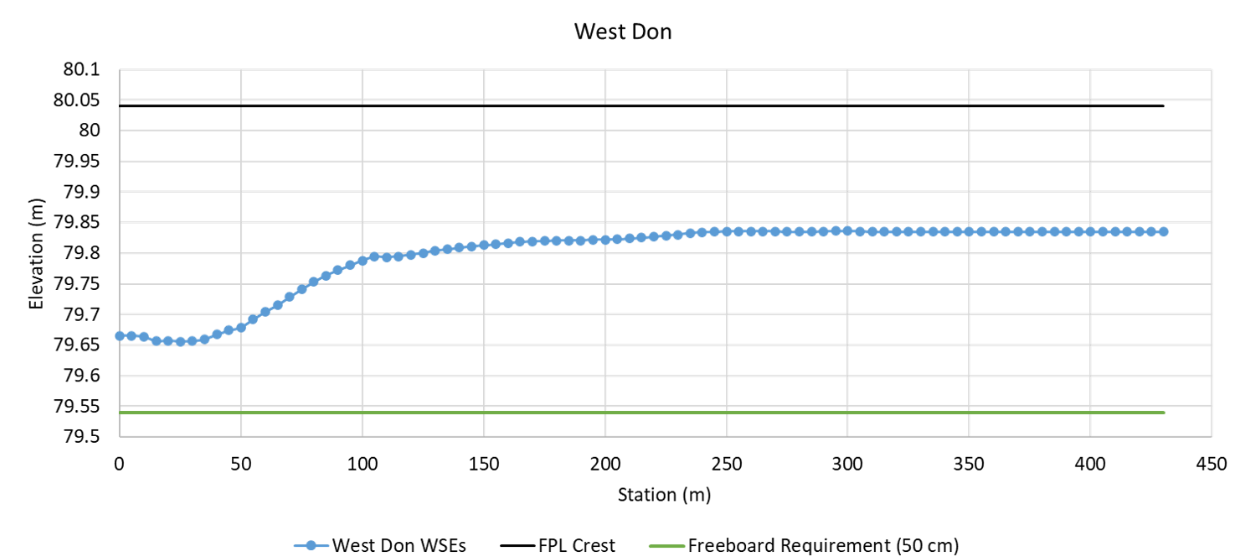


Figure 3-8. Profile along the West Don FPL

3.2 Inundation Frequency Analyses

The hydrodynamic models were used to analyze the frequency of activation of the Don Greenway and the elevated area of the SDMA that is being left in place to support the existing Gardiner ramp piers as a part of the Interim Conditions ("Interim Conditions SDMA"). These analyses were performed using combinations of static water level and Don River flows for various recurrency periods. The static water elevations were used in this analysis since the peak storm surge recurrence typically represents the peak instantaneous still water level, and there is a low probability of that occurring simultaneously with the peak of a short duration storm event. The elevated Lake Ontario water levels in 2017 and 2017, however, show that the static water elevations can remain high for several months in a row, increasing the probability that they will coincide with the peak of a storm event in the Don River watershed. The estimated combined return period for the lake level and storm event combination is based on the assumption that lake levels and storm events are independent events (e.g. higher lake levels do not necessarily increase the likelihood of a peak storm event).

3.2.1 Don Greenway Wetland

The DMNP EA contains specific language about the frequency of storm overtopping of the levee separating the River Valley from the Don Greenway. For the purposes of testing the PLFP design for compliance with the DMNP EA, the modeling has been performed using a lake level of 75.2 metres, which corresponds to the 2-year return period lake level without surge as noted in Appendix N of the DMNP EA. To supplement the above analysis and provide additional information that can be used to inform the ongoing management and maintenance of the PLFP, an full inundation frequency analysis of the Don Greenway was performed based on combinations of the static lake level return periods based on the updated lake level analysis performed by Baird (2019), and the design storms from the TRCA hydrology model.

The hydrodynamic model was run with varying lake levels and peak storm flows to develop a more complete recurrence estimate of the activation of the Don Greenway. The combination of static lake levels and storm flows used for this analysis are shown in Table 3-1.

Table 3-1. Don Greenway Inundation Frequency Analysis Based on Plan 2014 Static Water Levels and Don River Flows

Static Lake Level Return Period	Storm Flow Return Period	Estimated Combined Return Period	Peak WSE adjacent to the spillway to the Don Greenway	Don Greenway Spillway Overtopping?
2-Year	1-Year	2-Year	75.247	No
2-Year	2-Year	4-Year	75.265	No
2-Year	5-Year	10-Year	75.287	No
2-Year	10-Year	20-Year	75.308	No
2-Year	25-Year	50-Year	75.358	No
5-Year	1-Year	5-Year	75.535	No
5-Year	2-Year	10-Year	75.550	No
5-Year	5-Year	25-Year	75.571	No
5-Year	10-Year	50-Year	75.588	No
10-Year	1-Year	10-Year	75.704	No
10-Year	2-Year	20-Year	75.717	No
10-Year	5-Year	50-Year	75.738	No
25-Year	1-Year	25-Year	75.862	Yes
25-Year	2-Year	50-Year	75.875	Yes
50-Year	1-Year	50-Year	75.952	Yes

The result of this analysis shows that the Don Greenway is inundated under three of the lake level and storm flow combinations, and all of them are where the lake levels are above the spillway elevation of 75.8 metres. There is relatively little water surface elevation difference between this location and the Inner Harbour for the range of flows analyzed, so the activation frequency is primarily controlled by the lake level.

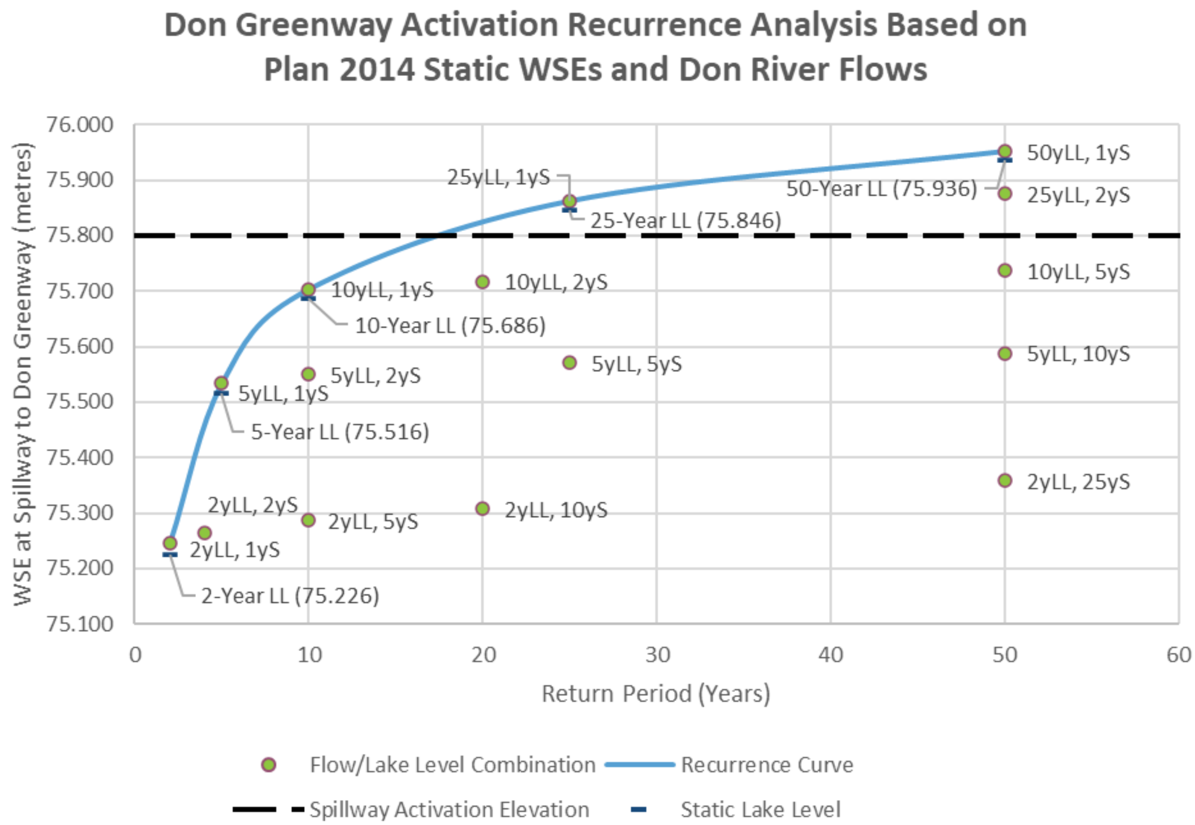


Figure 3-9. Don Greenway Activation Recurrence Frequency Analysis Results

For the events modeled in this analysis that had flows through the Don Greenway, the actual flows into the Ship Channel and velocities in the Don Greenway were both relatively low. The peak flow rate in the Don Greenway under the 50-Year recurrence static lake level and 1-Year storm was 1.3 cubic metres per second.

3.2.2 Interim Conditions SDMA

The elevated area of the SDMA that is being left in place to support the existing Gardiner ramp piers as a part of the Interim Conditions can be flooded periodically, based on a combination of the elevated water levels and storm flows from the Don River. To support the analysis of the necessity of risk management measures in in this area, the inundation a frequency was analyzed based on combinations of the static lake level return periods based on the updated Baird Analysis, and the design storms from the TRCA hydrology model.

The following static lake levels and storm flows were used to analyze the conditions under which the elevated area around the Gardiner Piers in the Interim Conditions SDMA is inundated (Table 3-2).

Table 3-2. Interim Conditions SDMA Inundation Frequency Analysis Based on Plan 2014 Static Water Levels and Don River Flows

Static Lake Level Return Period	Storm Flow Return Period	Estimated Combined Return Period	Peak WSE adjacent to the upstream end of the Interim Conditions SDMA	Interim Conditions SDMA Overtopping?
2-Year	1-Year	2-Year	75.276	No
2-Year	2-Year	4-Year	75.321	No
2-Year	5-Year	10-Year	75.363	No
2-Year	10-Year	20-Year	75.401	No
2-Year	25-Year	50-Year	75.482	No
5-Year	1-Year	5-Year	75.566	No
5-Year	2-Year	10-Year	75.611	No
5-Year	5-Year	25-Year	75.653	No
5-Year	10-Year	50-Year	75.691	No
10-Year	1-Year	10-Year	75.736	No
10-Year	2-Year	20-Year	75.781	No
10-Year	5-Year	50-Year	75.823	No
25-Year	1-Year	25-Year	75.896	No
25-Year	2-Year	50-Year	75.941	No
50-Year	1-Year	50-Year	75.986	No

These results were plotted in Figure 3-10, which shows that up to the 50-Year recurrence period for the combination of still water levels and storm flows, the inundation frequency of the Interim Conditions SDMA is greater than 50 years. Similar to the Don Greenway frequency analysis, the lake levels dominate the potential for inundation of this area, as even up to the 50-year storm there is relatively little difference in water surface elevation between the upstream end of the Interim Conditions SDMA and the Inner Harbour.

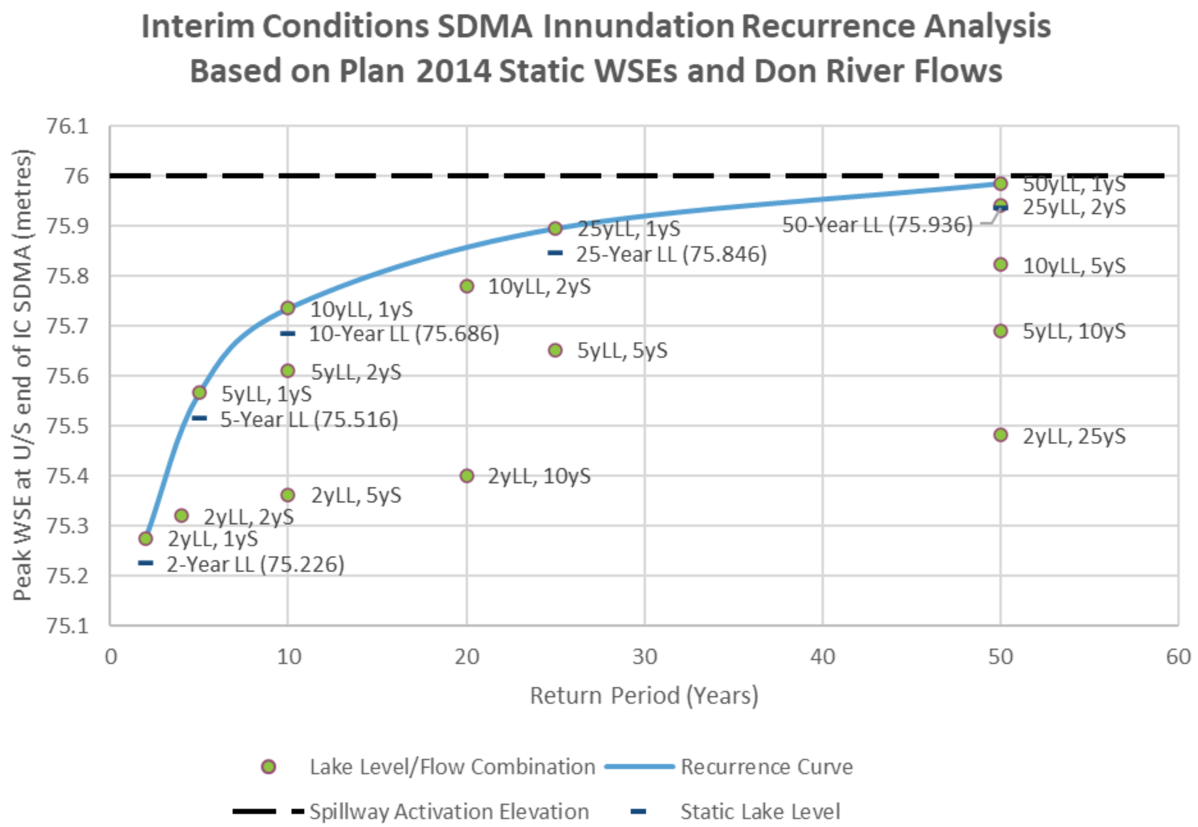


Figure 3-10. Interim Conditions SDMA Inundation Recurrence Frequency Analysis Results

4 Sediment Transport

A sediment transport model was developed to predict rates of sedimentation for the design condition, and to identify sediment maintenance thresholds that would permit acceptable passage of the regulatory flood event. This model was grounded in observed data so that it would represent site-specific conditions. Primary datasets for constraining model inputs and assumptions were (1) Keating Channel bathymetric data which were used to compute actual contemporary sedimentation rates, and (2) Keating Channel grain size data which describe the change in sediment type at depositional locations in the Keating Channel. By comparing model predictions with these observed data, base sediment transport model inputs were selected. Additionally through this process, ranges in plausible sediment transport model inputs were identified. These input ranges were then carried forward to modeling of the regulatory flood event modeling to describe the ranges in potential sediment transport and flooding outcomes.

After the model was found to adequately predict contemporary rates and patterns of Keating Channel sedimentation, it was applied to predict sedimentation for the design condition. This was done by simulating individual storm events, and scaling and combining those results to represent an 18-year hydrologic period. The design condition would increase sedimentation above the Lake Shore Boulevard (LSB) bridge crossing (i.e. in the Sediment Management Area). Increasing sedimentation farther upstream would help to maintain flood conveyance in downstream areas, reduce dredge frequency in the Keating Channel, and improve water quality in the Keating Channel relative to a condition without a sediment management area upstream of LSB. Maintenance frequency in the Keating Channel would be reduced from once per year to once per five years, on average, and maintenance frequency in the SDMA would need to occur annually to maintain flood conveyance.

Simulation of the regulatory flood event helped to identify these thresholds for sediment maintenance. The sediment transport model was configured to represent various degrees of sedimentation in the project area. The model predicted change in sediment bed morphology (i.e. sediment bed elevations) due to deposition and scour prior to the peak of the flood. These changes in morphology influenced flood conveyance and water levels. Thresholds for sediment maintenance were identified by determining which sediment maintenance thresholds would permit acceptable passage of the regulatory event. Based on this analysis, the SDMA should be dredged after it has reached a mean bed elevation of 72 metres near the upstream end.

The location where predicted regulatory event flood levels are most sensitive to maintenance condition and morphologic change is at the Lake Shore Boulevard crossing. This is in part because flood levels are very near the bridge soffit, and increases in flood levels above the soffit exacerbate flooding. It is also because this location is one of complex flow and sedimentation patterns. As described above, a range of plausible morphological outcomes were simulated by considering the full range of plausible input parameters defined during the model calibration process.

This report documents the development, calibration, and application of the sediment transport model, and is intended to support a review of the modeling process and findings. It is organized into the following sections:

- **Model Development and Calibration.** A description of how the model was developed based on observed data, then compared with observed data to calibrate model inputs and identify a range of plausible model inputs.
- **Model Application: Regulatory Event Flood Modeling.** A description of how the model was applied to predict changes in regulatory even flood levels associated with changes in sediment bed morphology (i.e. deposition and scour) during the event.

- **Model Application: Sediment Maintenance.** A description of how the model was applied to simulate long-term sedimentation patterns and identify maintenance frequencies in sub-areas of the project area.

5 Sediment Transport Model Development and Calibration

Model development and calibration are the processes of configuring a model to represent site-specific conditions, then adjusting model inputs for consistency with observed site data. Through these processes, emphasis was placed on two particular outcomes that relate directly to the reliable use of the sediment transport model.

First, the model was configured to predict hydraulics consistently with the more detailed MIKE hydraulic model. Emphasis was placed here because the MIKE model provides the best available representation of hydraulic conditions because of its level of detail in representing infrastructure and its calibration to Don River hydraulic data.

Second, the model was configured for consistency with observed sedimentation patterns in the Keating Channel. Observed sedimentation data are especially useful for constraining sediment transport model inputs because they represent the net result of both sediment accumulation and sediment erosion, so these data help to jointly constrain these two modeled processes.

5.1 Model Framework Selection

The sediment transport model used for this project is based on the SEDZLJ model algorithms developed by Craig Jones and Wilbert Lick at the University of California – Santa Barbara. Sandia National Laboratory modified a version of the Environmental Fluid Dynamics Code (EFDC) model to incorporate the SEDZLJ algorithms into a model known as SNL-EFDC. The SNL-EFDC hydrodynamic and sediment transport model code were used to simulate sediment transport behavior of the Don River.

SEDZLJ is capable of simulating the resuspension, deposition, and transport of cohesive and non-cohesive sediments. The model predicts temporal and spatial variations in suspended sediment concentration, sediment bed elevation, and bed composition (relative fractions of different particle size classes). The SEDZLJ model simulates bedload transport of non-cohesive sediment. Predicted changes in bed elevation associated with sediment deposition and scour dynamically effect predictions of water depth and velocity. The use of SEDZLJ algorithms for the sediment transport model provides the capability to represent sediment transport using the latest and most widely accepted sediment transport algorithms.

5.2 Model Inputs

A model grid was developed at a spatial scale fine enough to represent bridge piers in the project area, including at the Gardiner Ramps, CNR crossing, and LSB crossing. Simulations were conducted to identify a sufficiently fine spatial scale so that further increases in resolution would not appreciably change model predictions. There are 22,220 cells in the model domain, and cell sizes range in dimension from 1 to 20 metres. Figure 5-1 illustrates the model grid in the vicinity of the Lake Shore Boulevard bridge crossing.

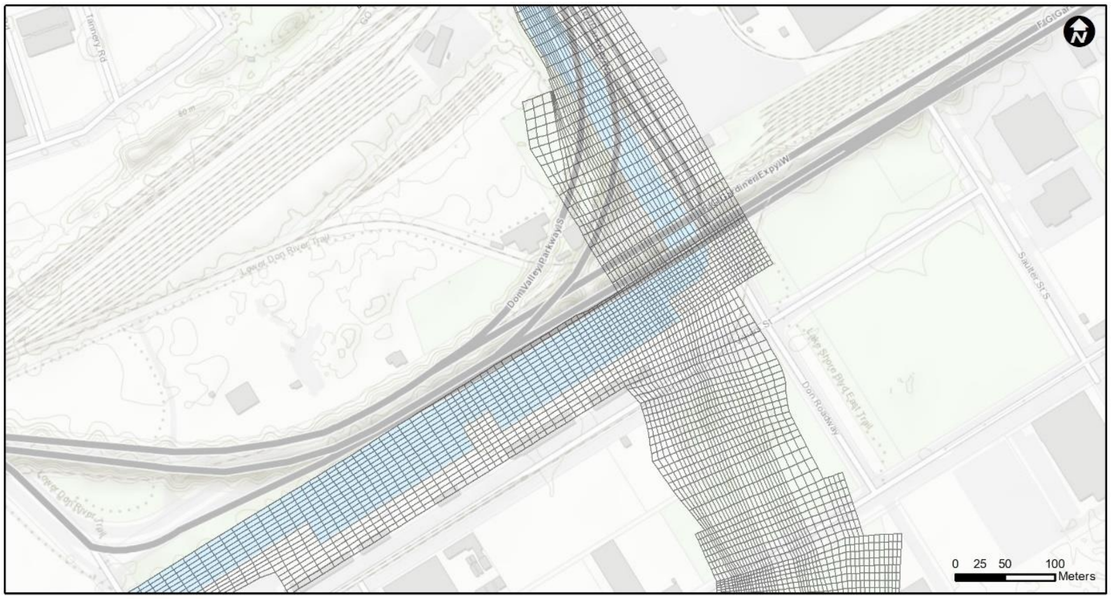


Figure 5-1: Sediment Transport Model Grid Detail

5.2.1 EFDC Model Hydraulic Inputs

Model hydraulics inputs in the EFDC sediment transport model were set similarly to the MIKE hydraulic model to achieve consistency in model predictions. Bed roughness inputs for the sediment transport model were computed from the MIKE Manning's M input values to EFDC roughness height inputs using the Strickler equation:

$$k\ (feet) = \left(\frac{n}{0.0342}\right)^{1/6}$$

where:
k: physical roughness height (=30*Z_o)
Z_o: EFDC roughness height input value (height of zero velocity)

These two types of roughness inputs are conceptually different, which causes some of the EFDC roughness inputs to be higher than is typical. This is due in part to the fact that Manning's M is a composite roughness input that represents multiple forms of fluid resistance, including sediment grain roughness and vegetative resistance, but the EFDC roughness height inputs are generally used to merely represent sediment grain roughness. As a result, some of the converted roughness heights are higher than typically used in EFDC models. This outcome of the input conversion was considered acceptable because the hydraulic model results (e.g. flow split between Keating Channel and Naturalized Channel) agree closely in EFDC and MIKE. Alternatives to this approach were evaluated, but they required more assumptions to be made in converting the composite roughness values in MIKE to multiple types of resistance inputs in EFDC, and these alternatives produced less comparable flow splits between the two models.

Model diffusivity inputs were set similarly in the two models, with eddy diffusivity set to a constant of 1 square meters per second in MIKE, and a constant of 1 square metres per second in EFDC. This diffusivity input was computed based on the typical cell dimensions and timestep for both models.

The flow curtain at the upstream entry point to the Keating Channel was also represented similarly in the two models. This flow curtain is expected to break free at a Don River flow rate of 126 cms (GEI, 2019). For flows lower than this threshold, the flow curtain was modeled in EFDC using a rating curve computed from the MIKE hydraulic model results, expressed as the relationship between discharge into the Keating Channel as a function of the upstream water level.

Simulations were conducted in EFDC and MIKE to compare modeled hydraulics. The MIKE hydrodynamic model includes much greater spatial detail than the EFDC model and is considered to be the more accurate model. The purpose of this comparison between models was to confirm that the EFDC model produces sufficiently similar results and is therefore reliable for describing morphological change during the regulatory event. Comparisons were prepared for the regulatory flow event for the 4-bay Lake Shore Bridge Condition under the full vision scenario.

The models produce very similar flow splits between the Keating Channel and the Naturalized Channel. MIKE predicts that 78% of the regulatory event peak flow will discharge through the Keating Channel while EFDC predicts that 76% will discharge through the Keating Channel.

EFDC model and MIKE model comparison plots for modeled current velocities and modeled water surface elevations of the 1-yr, 2-yr, 5-yr, 10-yr, 25-yr, 50-yr, 100-yr, 350-yr, 1000 cms event, and the regulatory flow event are shown below. Water surface elevations and velocities compare favorably through most of the project area, though with some identifiable differences that we consider to be minor in the context of model findings related to regulatory flood levels and sediment maintenance. A major difference between the two models is the transition in current velocities from the broader upstream end to the narrower section of the Keating Channel. The MIKE model tends to produce higher velocities and lower surface elevations than the EFDC model. The impact of this is on predicted sediment transport is that the EFDC model produces lower shear stresses than the MIKE model, so the EFDC model is less erosional than it would otherwise be. In the context of the regulatory event flood modeling, this result is slightly conservative in that it would tend to marginally understate erosion potential in the Keating Channel and overstate flood potential upstream. In other locations, the models compare favorably in terms of modeled velocities and water surface elevations, and our conclusion is that EFDC is sufficiently consistent with MIKE and provides the best available estimate of morphologic change.

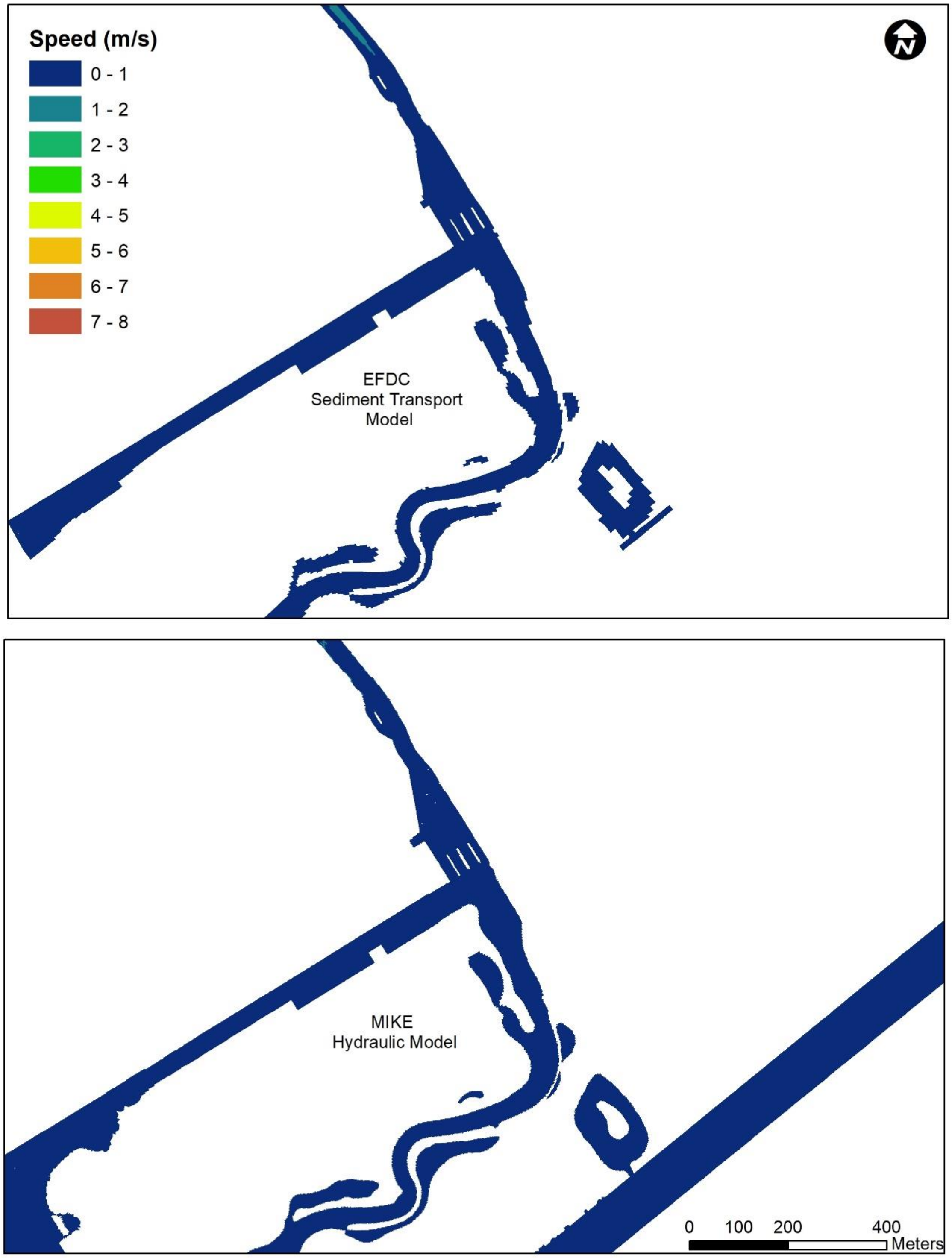


Figure 5-2: Comparison of EFDC and MIKE Predicted Current Speeds, 1-yr Event

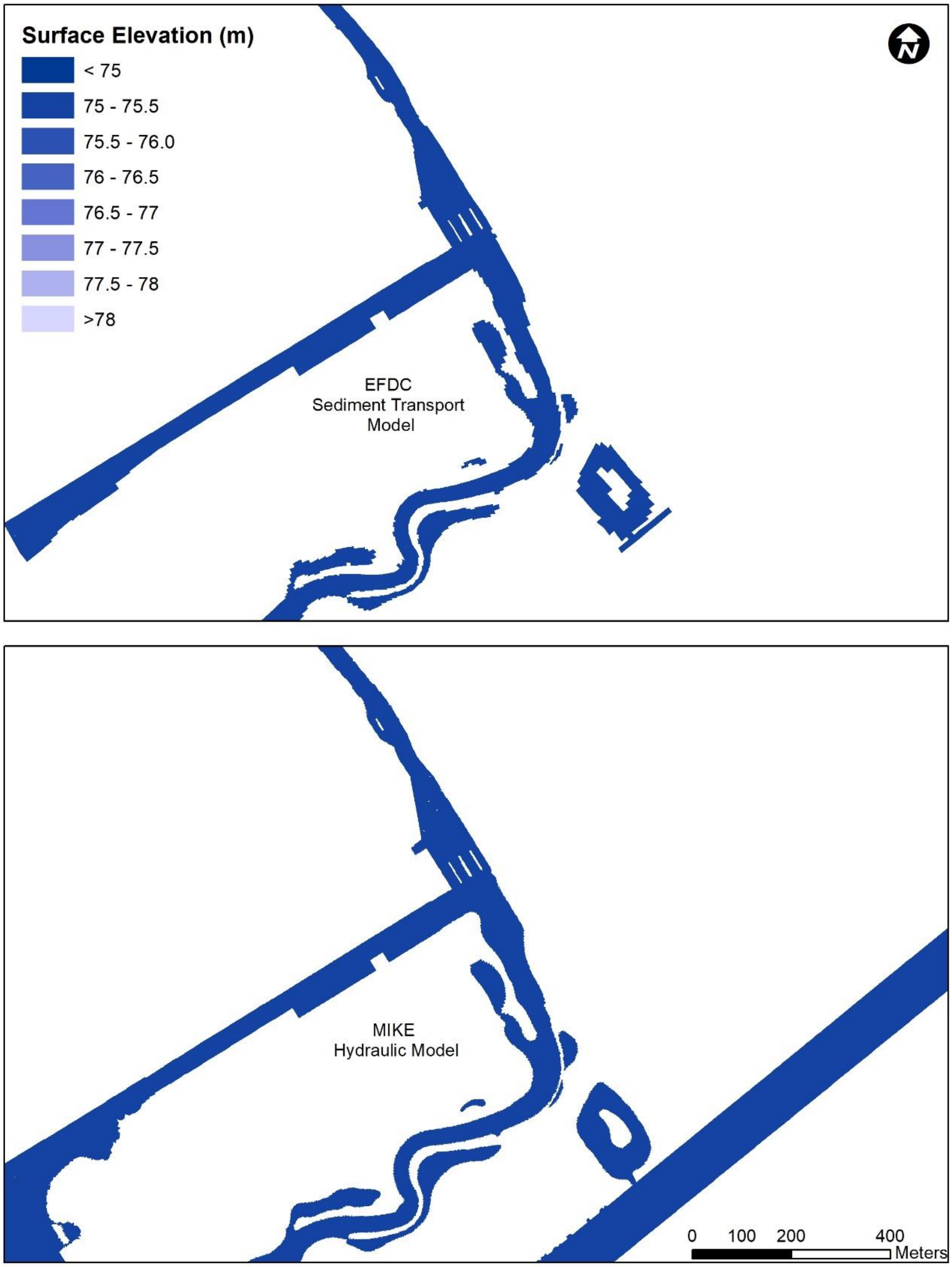


Figure 5-3: Comparison of EFDC and MIKE Predicted Water Levels, 1-yr Event

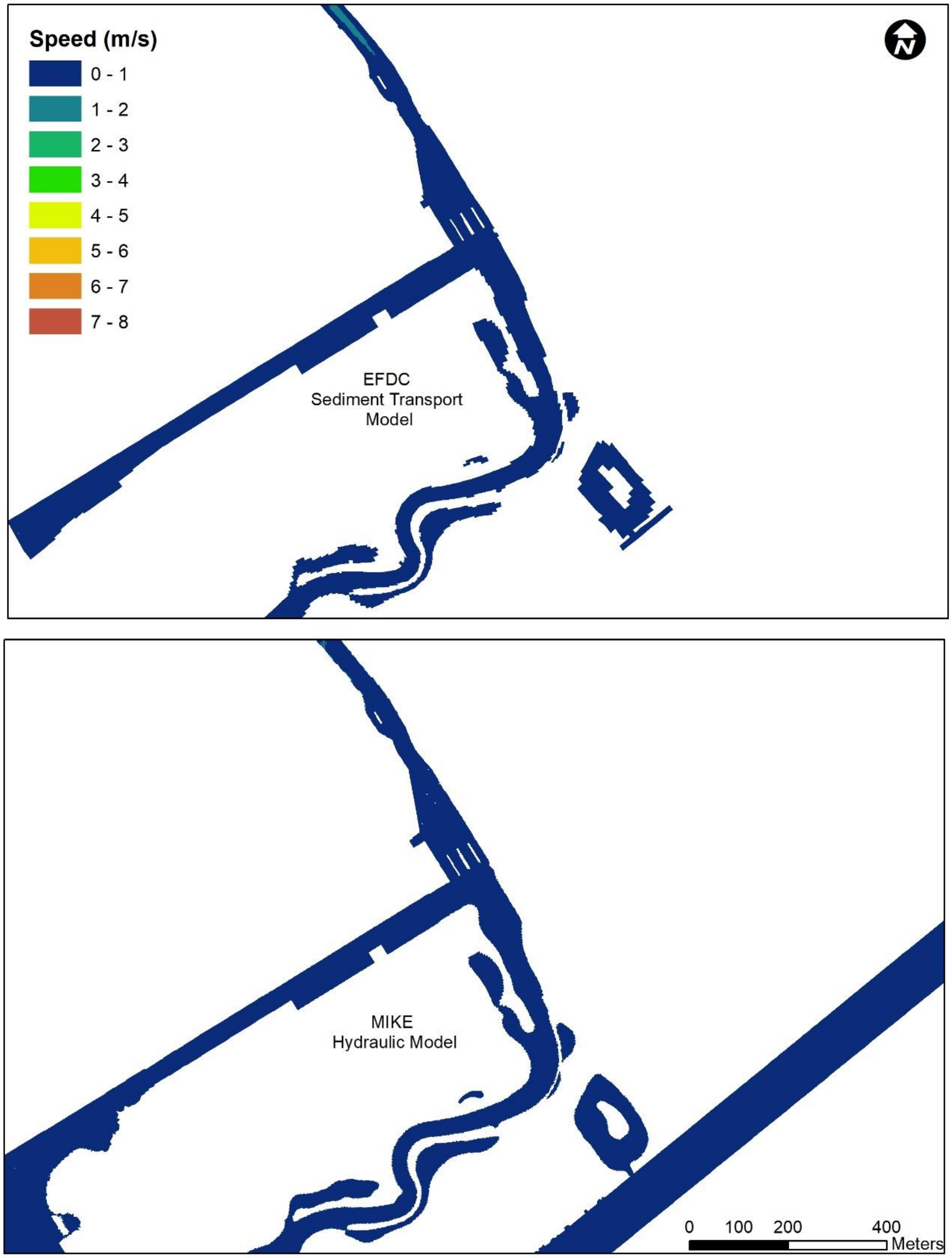


Figure 5-4: Comparison of EFDC and MIKE Predicted Current Speeds, 2-yr Event

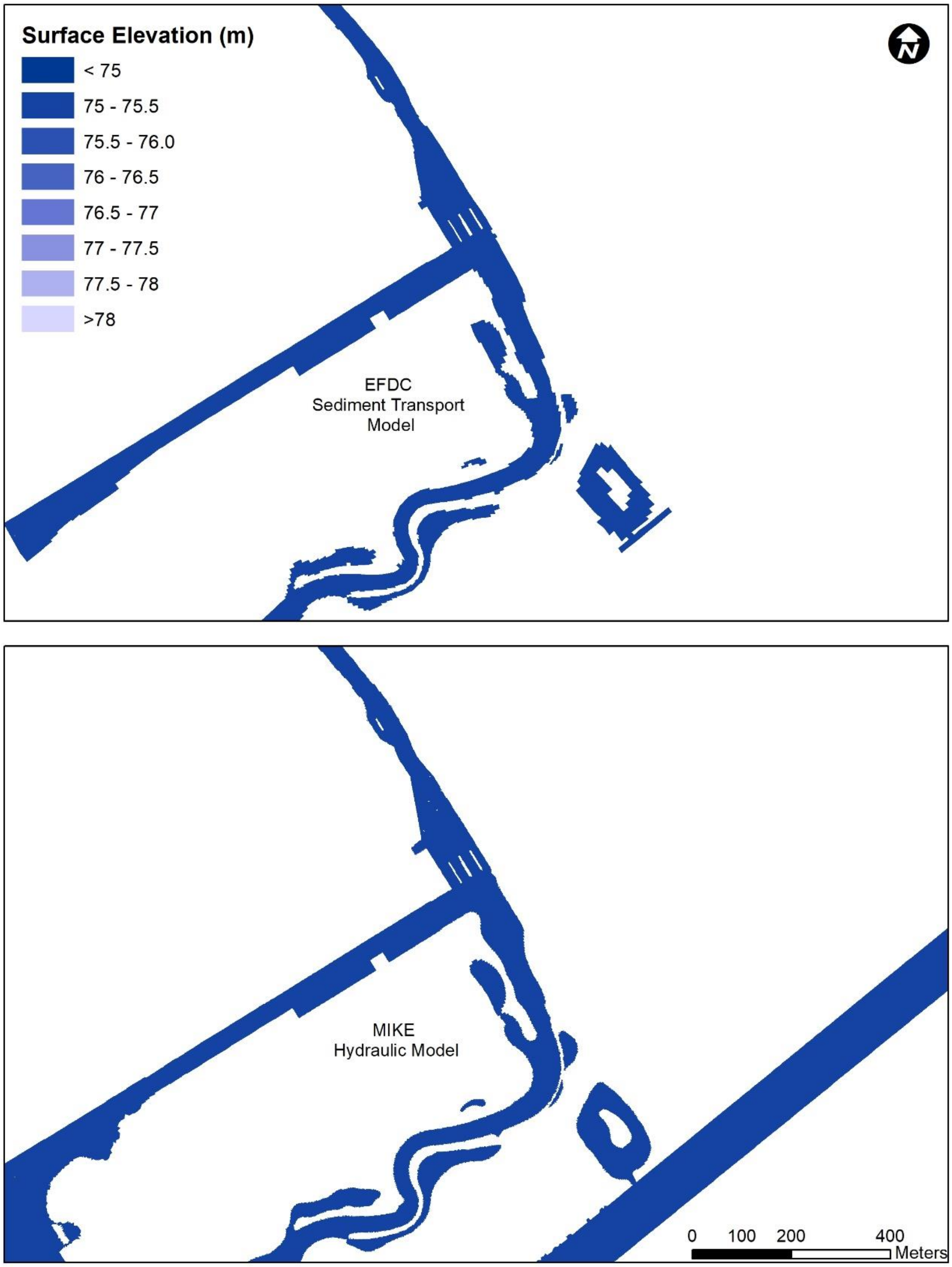


Figure 5-5: Comparison of EFDC and MIKE Predicted Water Levels, 2-yr Event

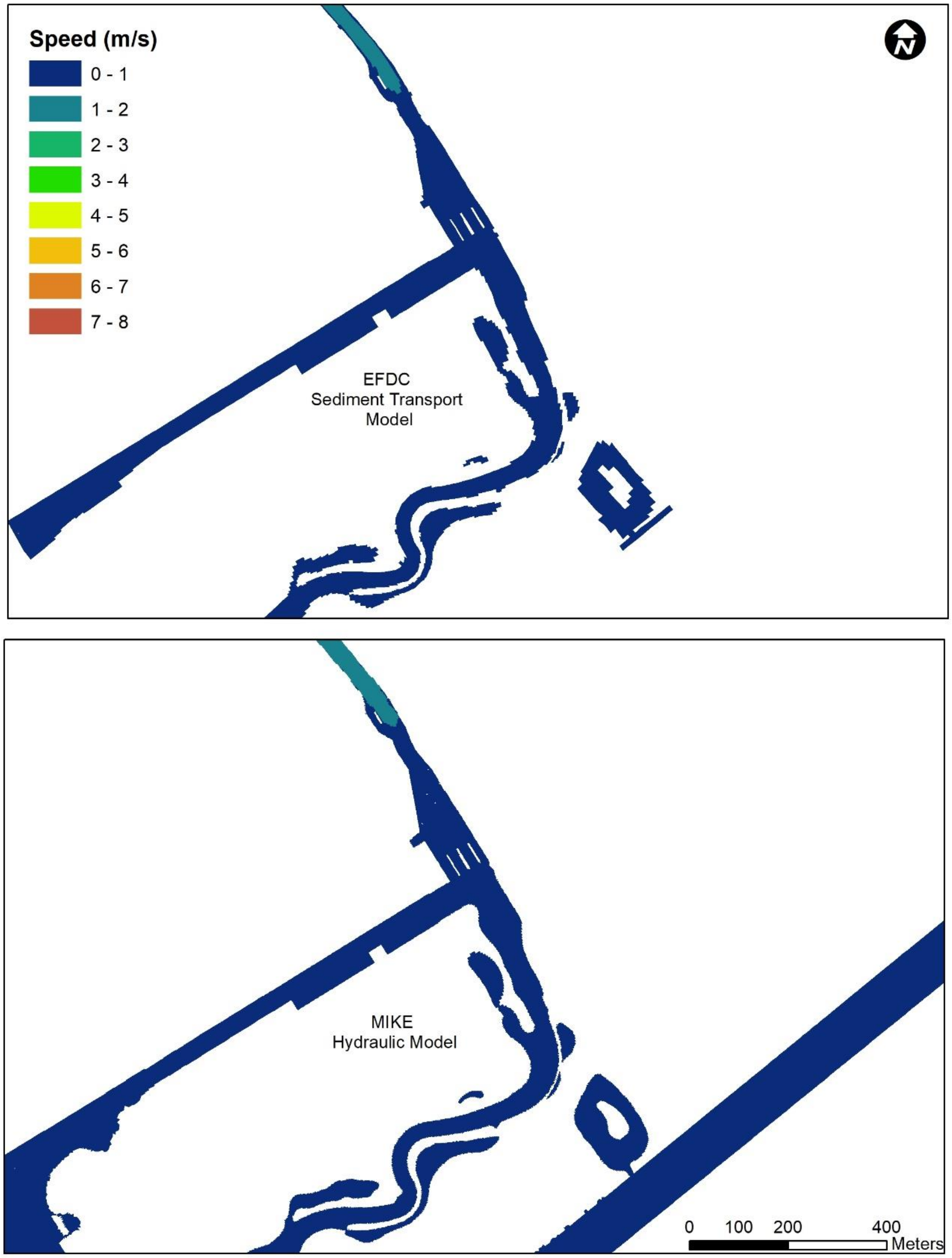


Figure 5-6: Comparison of EFDC and MIKE Predicted Current Speeds, 5-yr Event

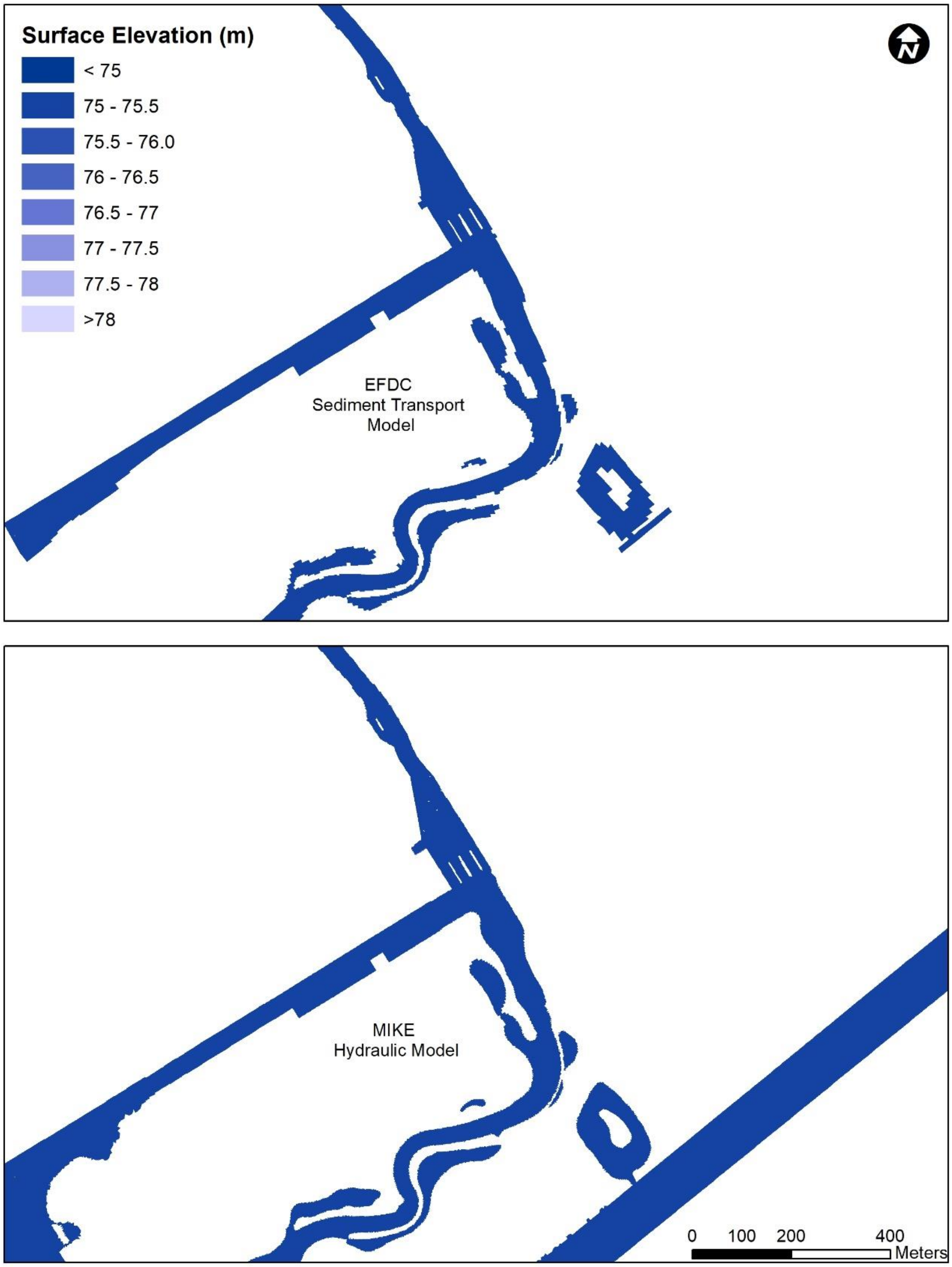


Figure 5-7: Comparison of EFDC and MIKE Predicted Water Levels, 5-yr Event

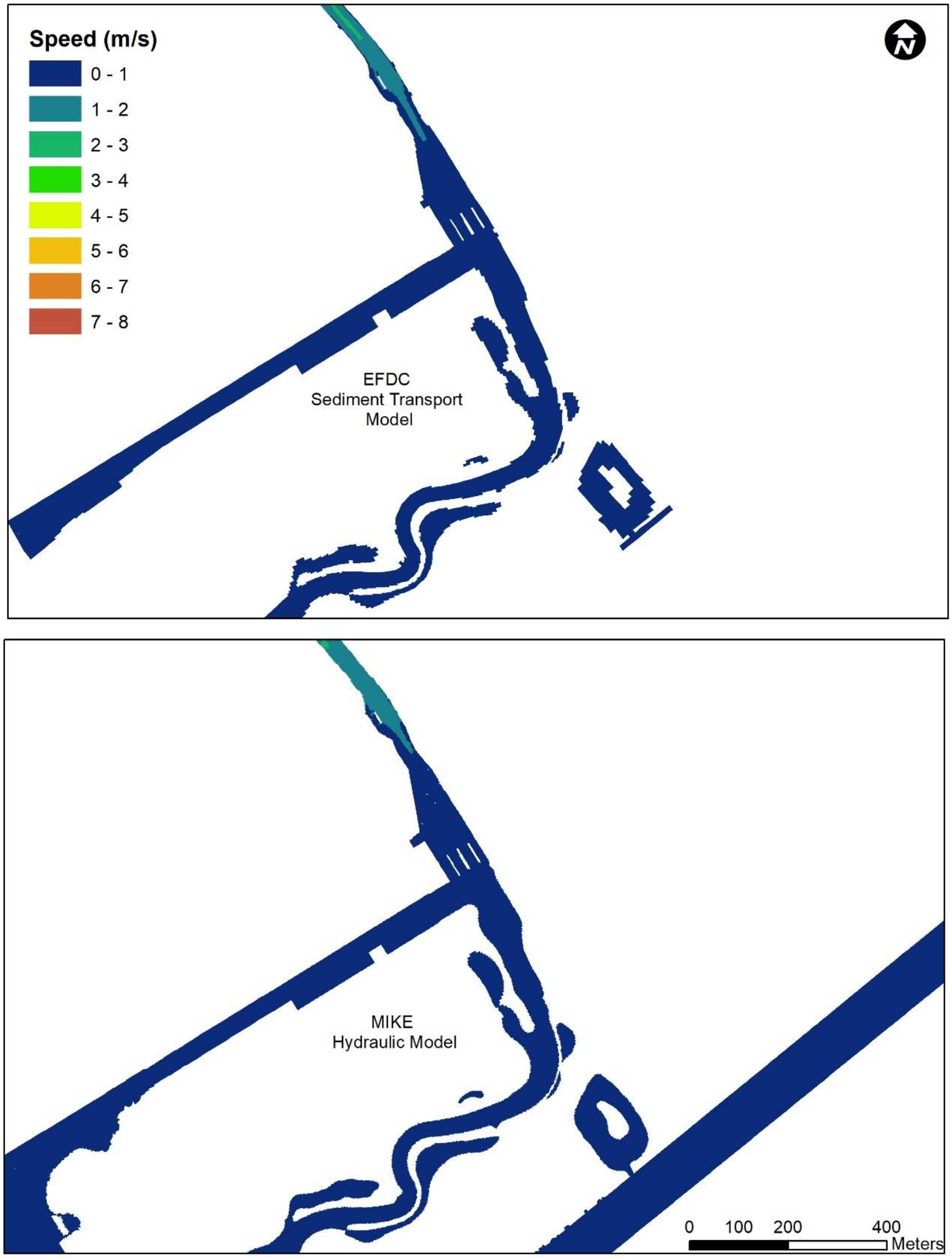


Figure 5-8: Comparison of EFDC and MIKE Predicted Current Speeds, 10-yr Event

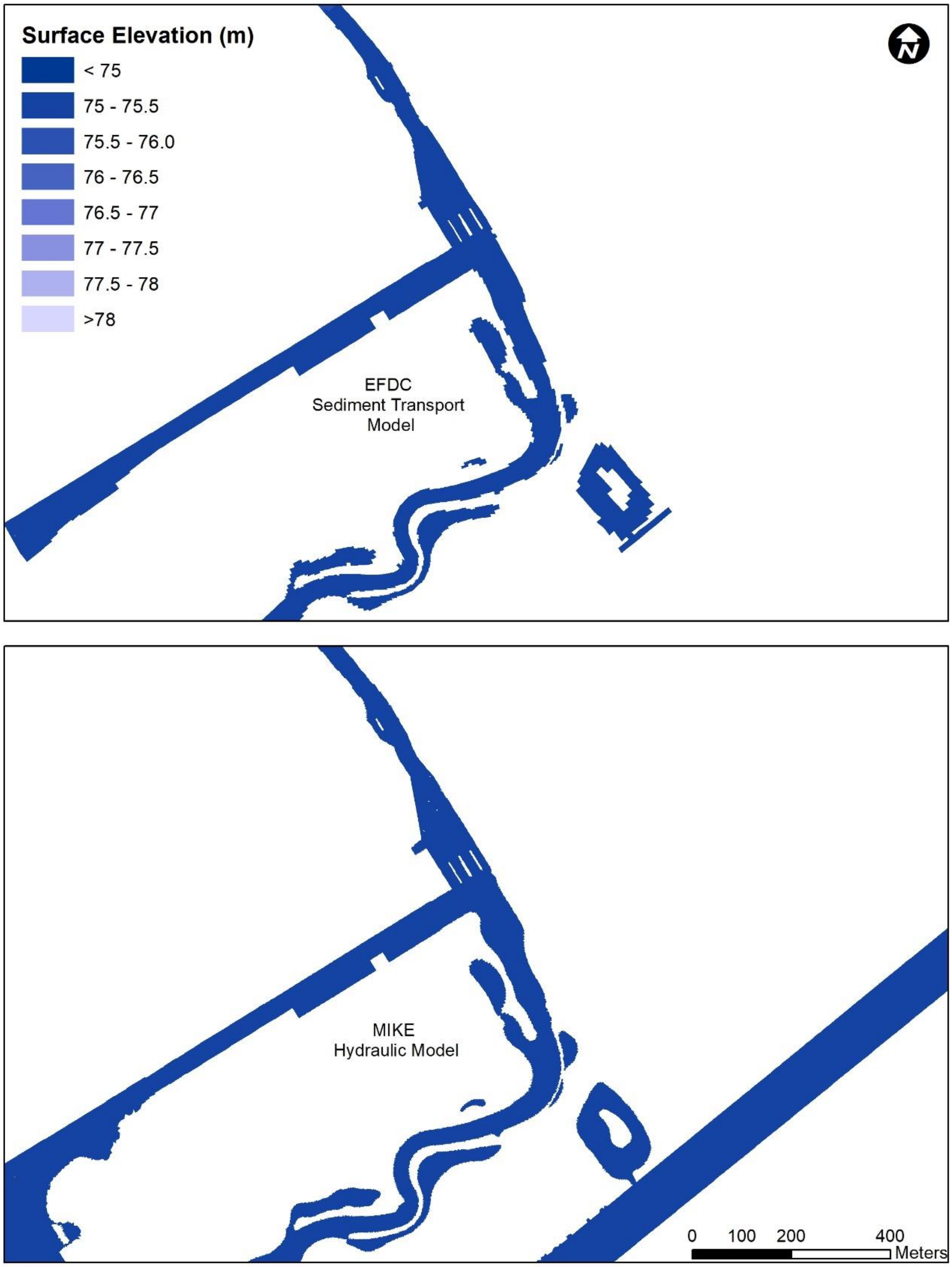


Figure 5-9: Comparison of EFDC and MIKE Predicted Water Levels, 10-yr Event

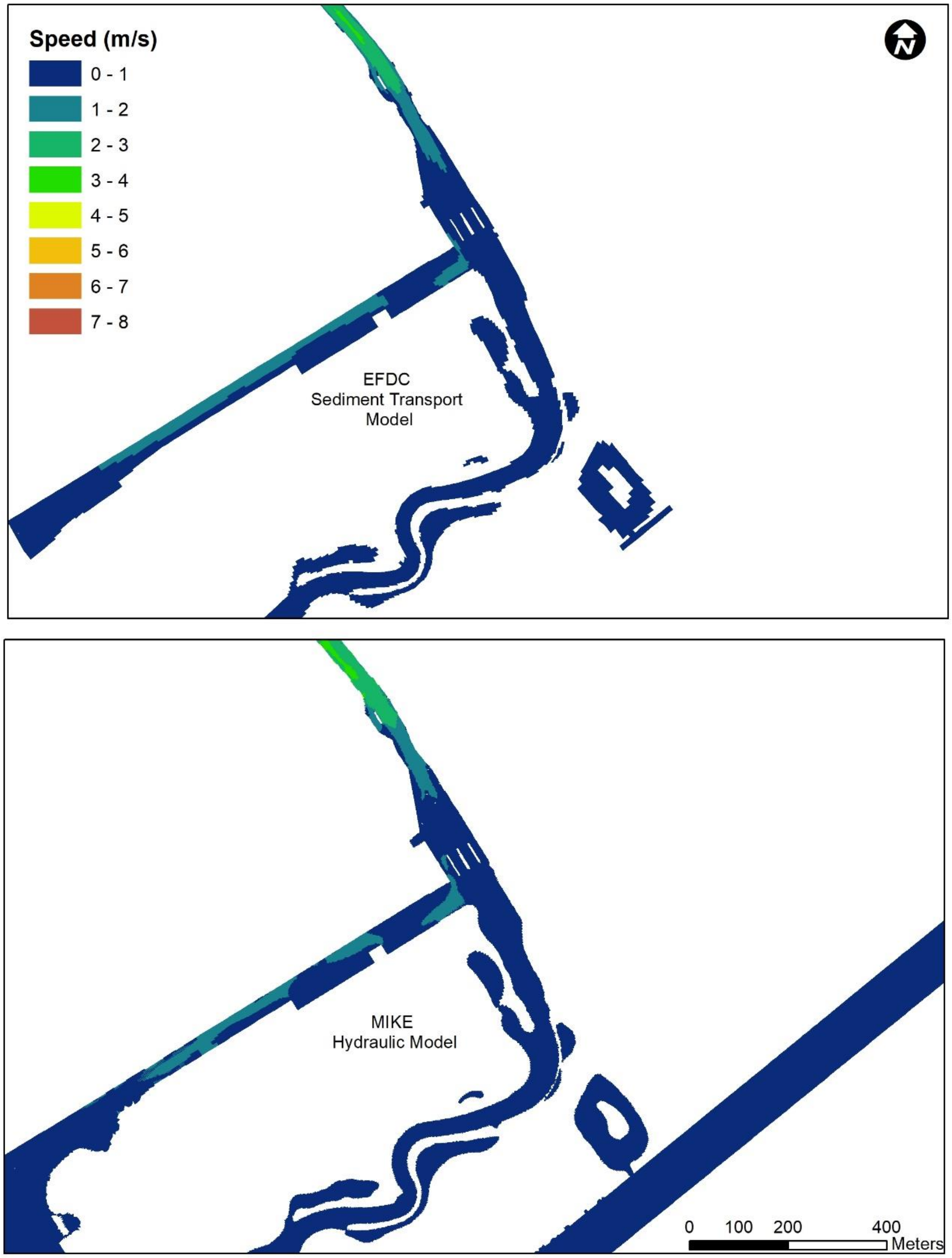


Figure 5-10: Comparison of EFDC and MIKE Predicted Current Speeds, 25-yr Event

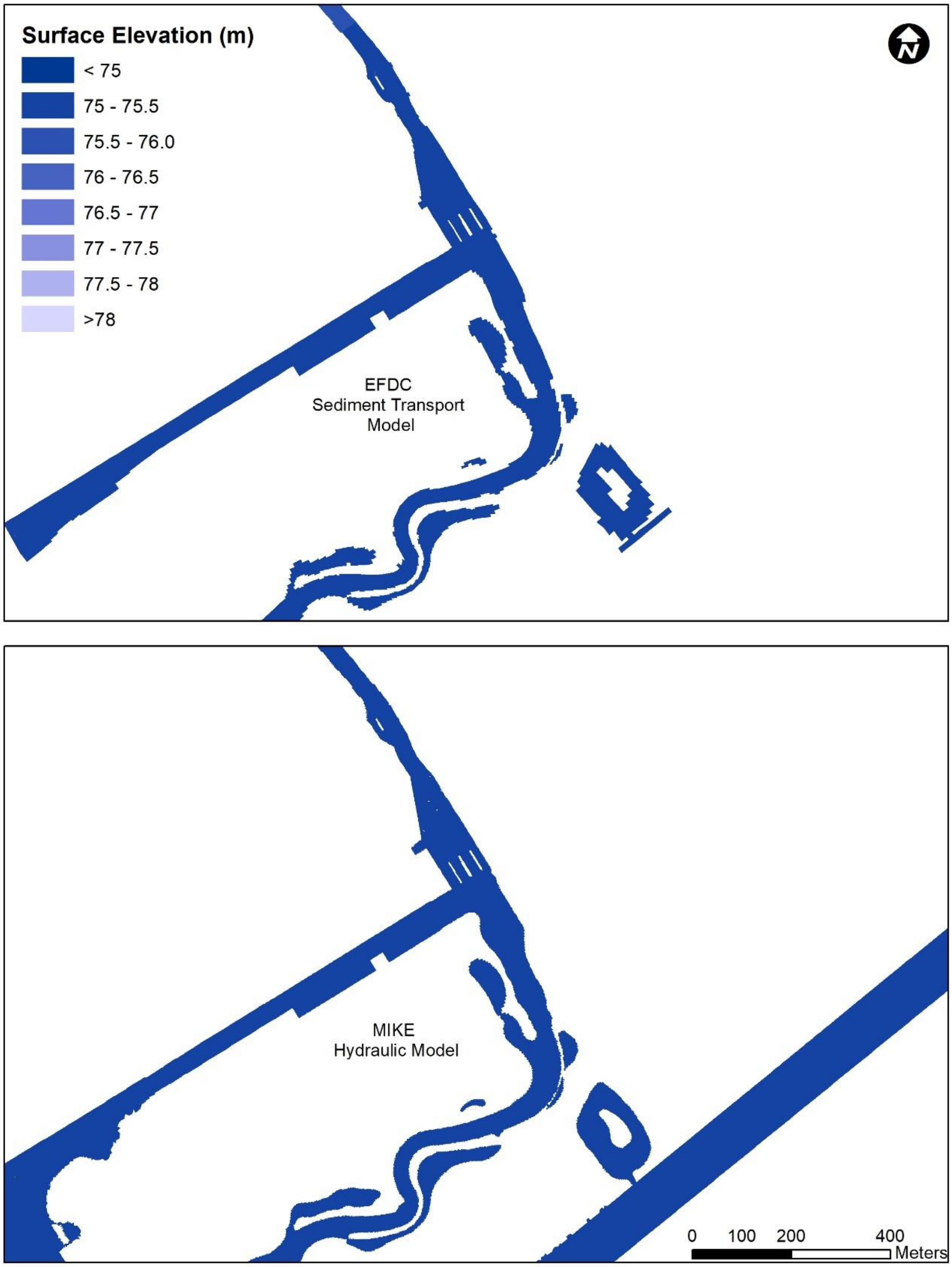


Figure 5-11: Comparison of EFDC and MIKE Predicted Water Levels, 25-yr Event

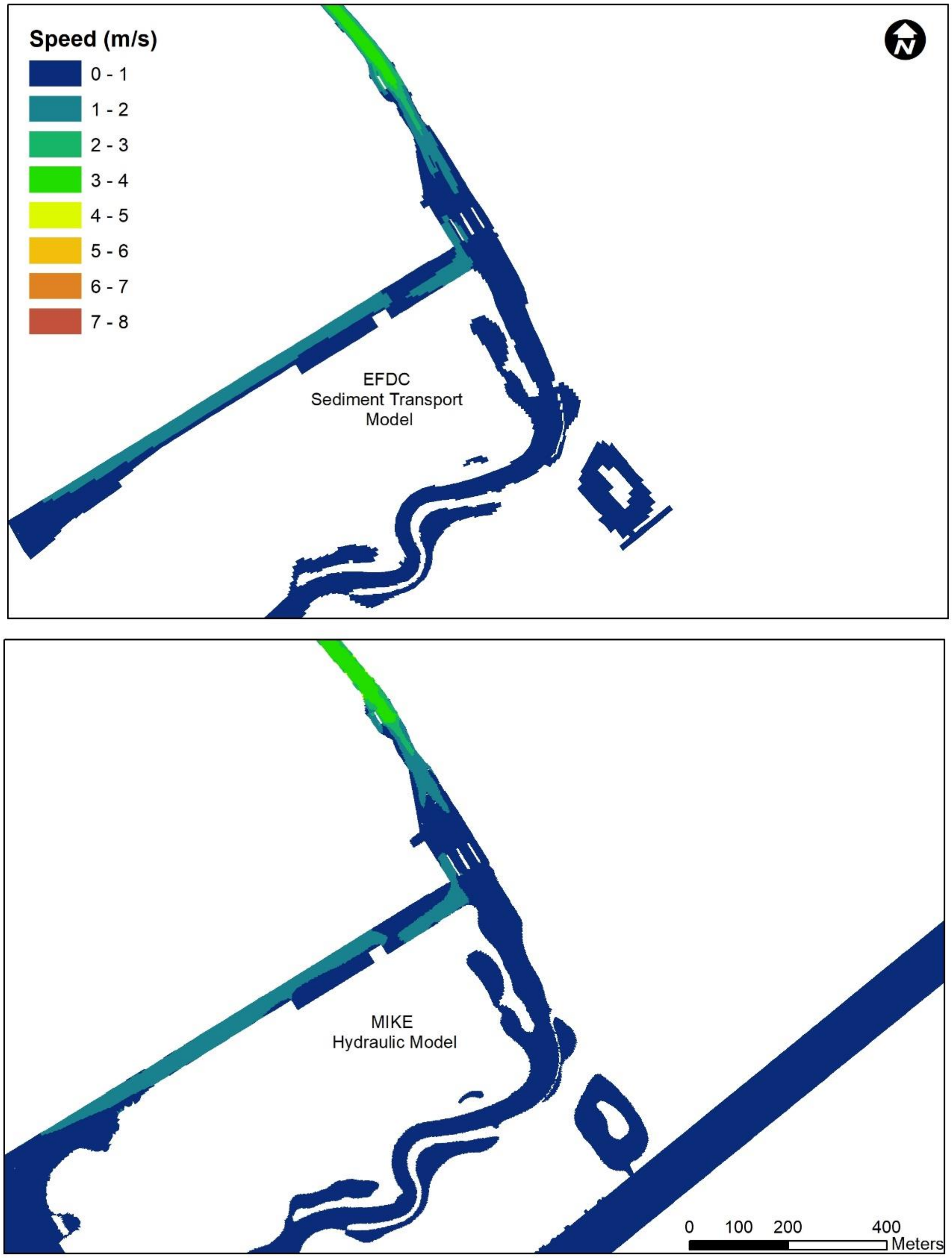


Figure 5-12: Comparison of EFDC and MIKE Predicted Current Speeds, 50-yr Event

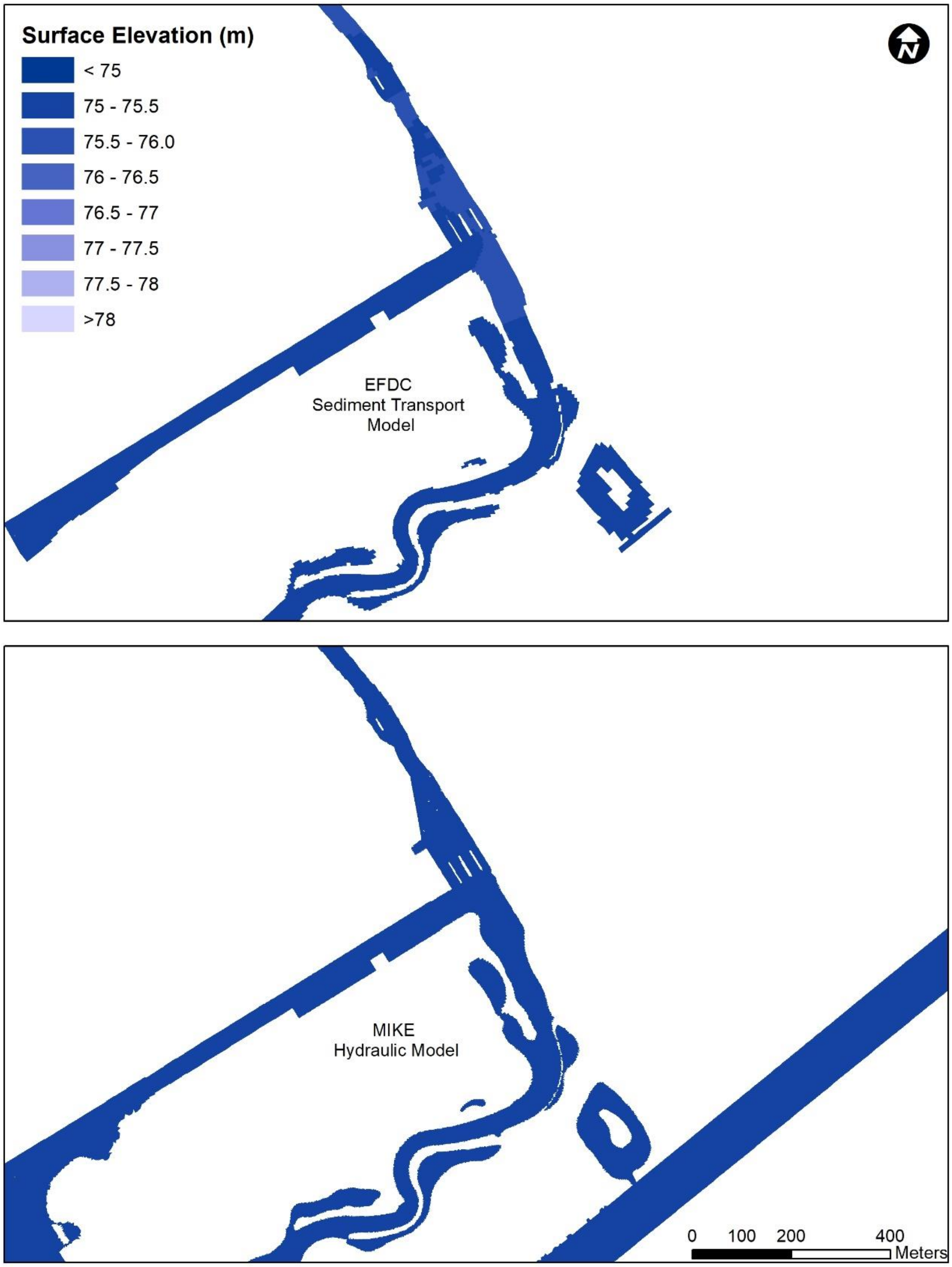


Figure 5-13: Comparison of EFDC and MIKE Predicted Water Levels, 50-yr Event

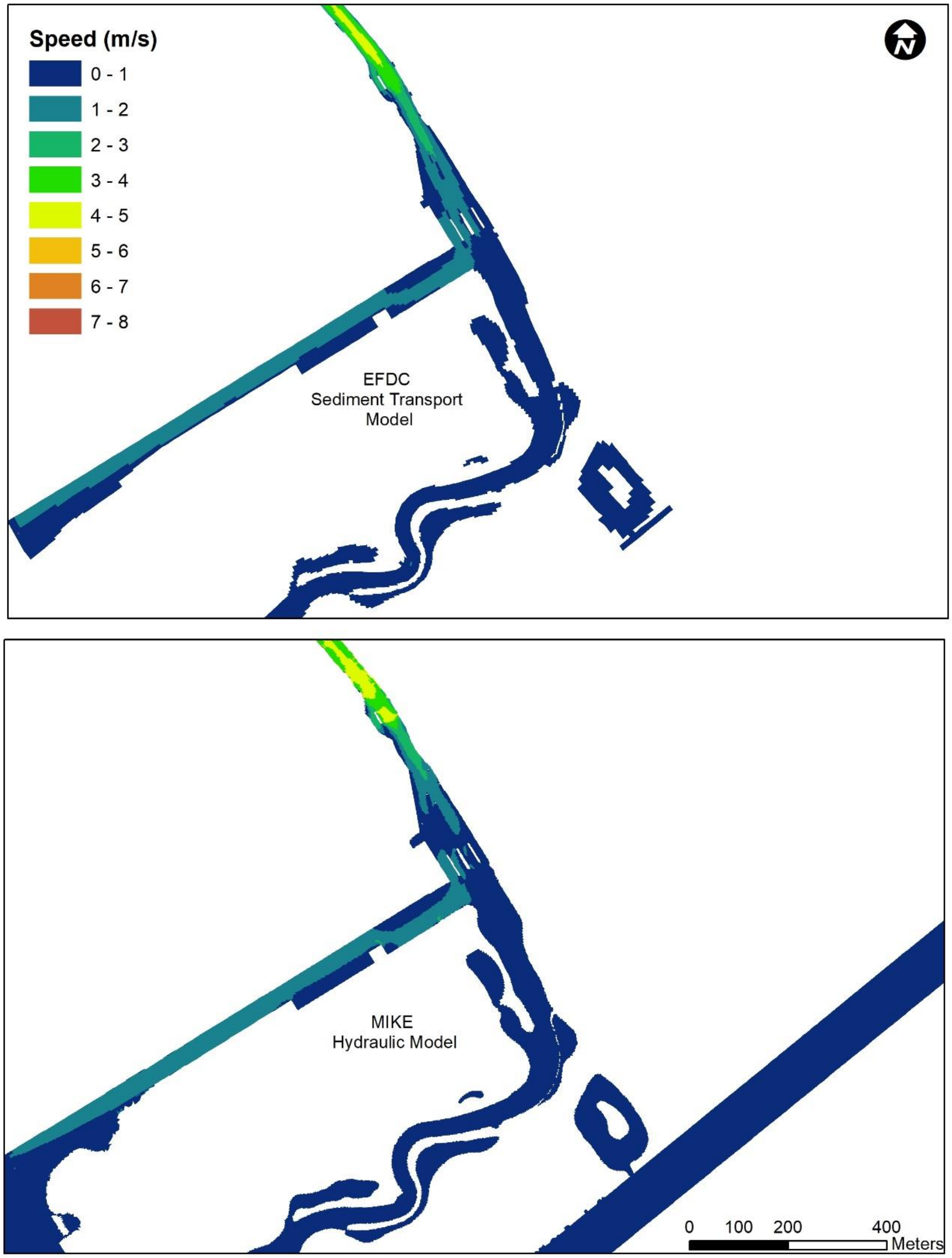


Figure 5-14: Comparison of EFDC and MIKE Predicted Current Speeds, 100-yr Event

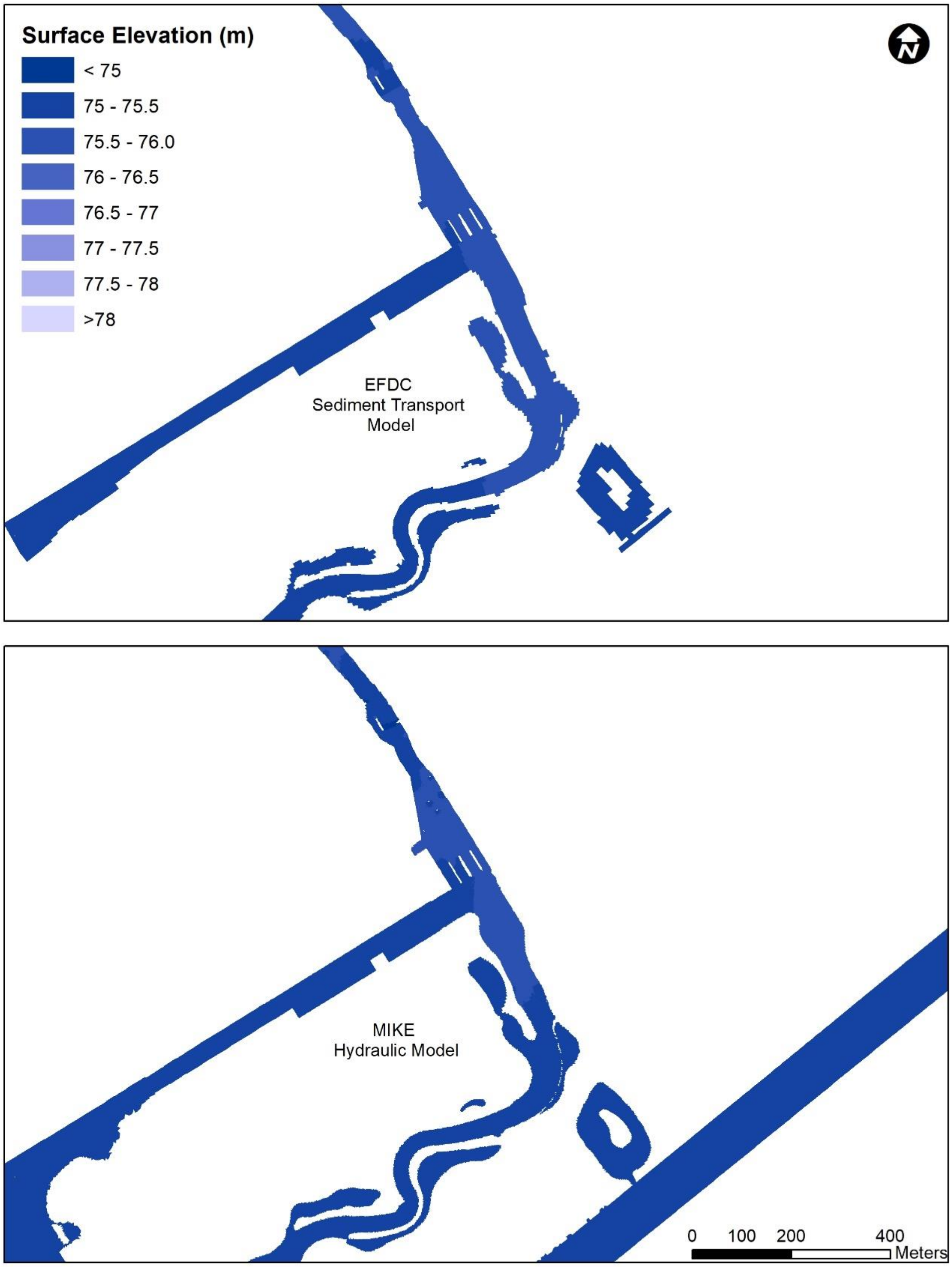


Figure 5-15: Comparison of EFDC and MIKE Predicted Water Levels, 100-yr Event

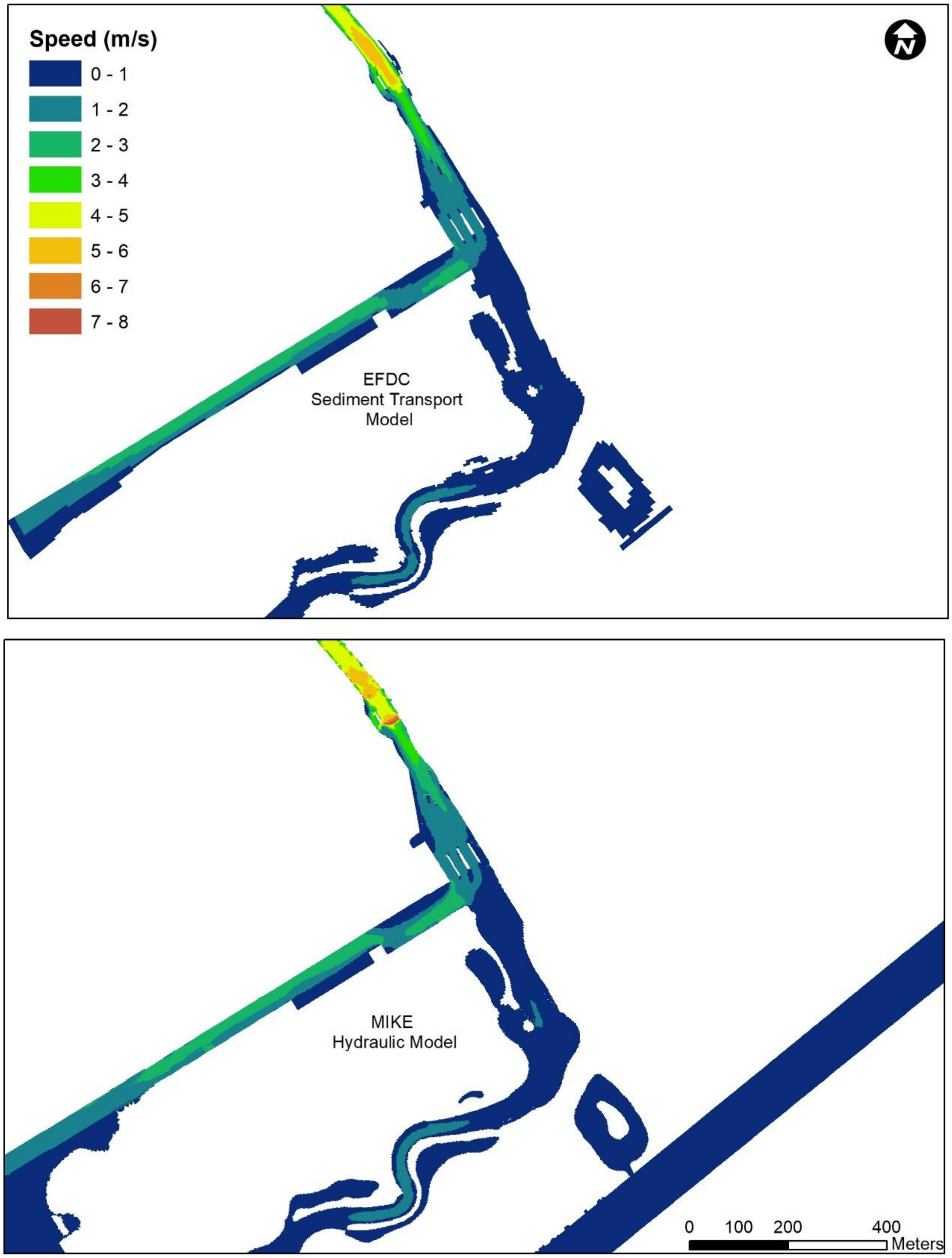


Figure 5-16: Comparison of EFDC and MIKE Predicted Current Speeds, 350-yr Event

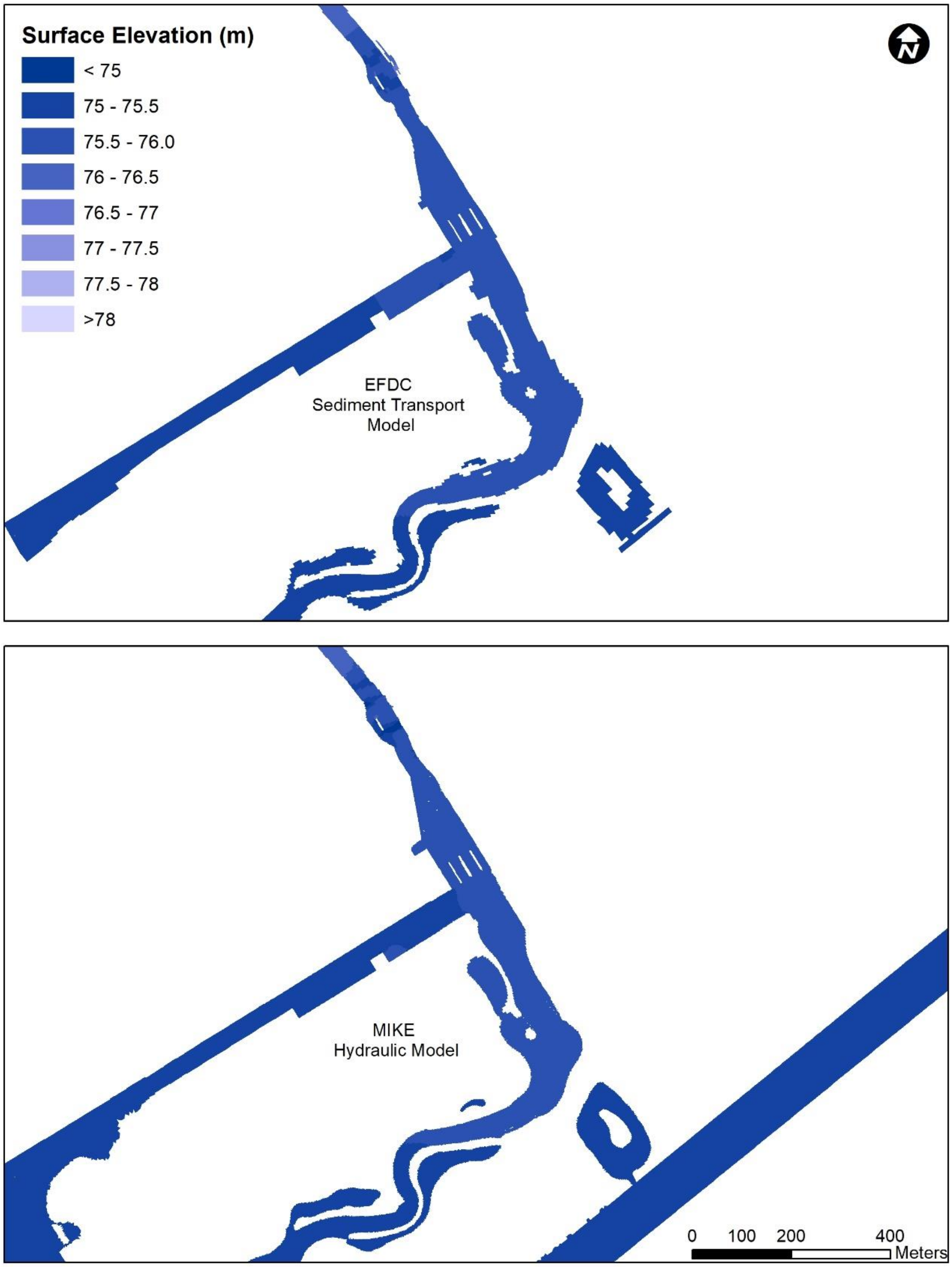


Figure 5-17: Comparison of EFDC and MIKE Predicted Water Levels, 350-yr Event

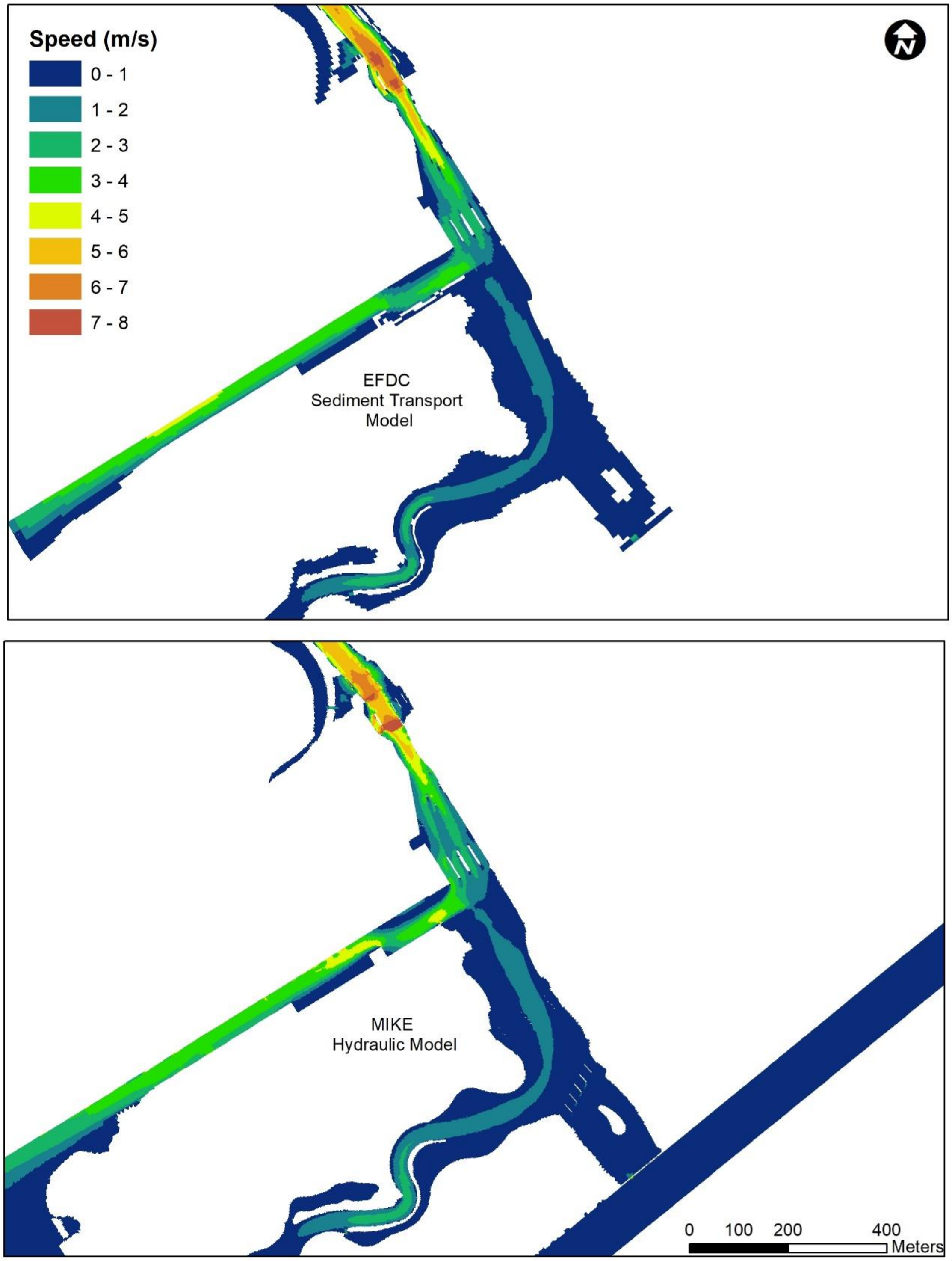


Figure 5-18: Comparison of EFDC and MIKE Predicted Current Speeds, 1000-cms Event

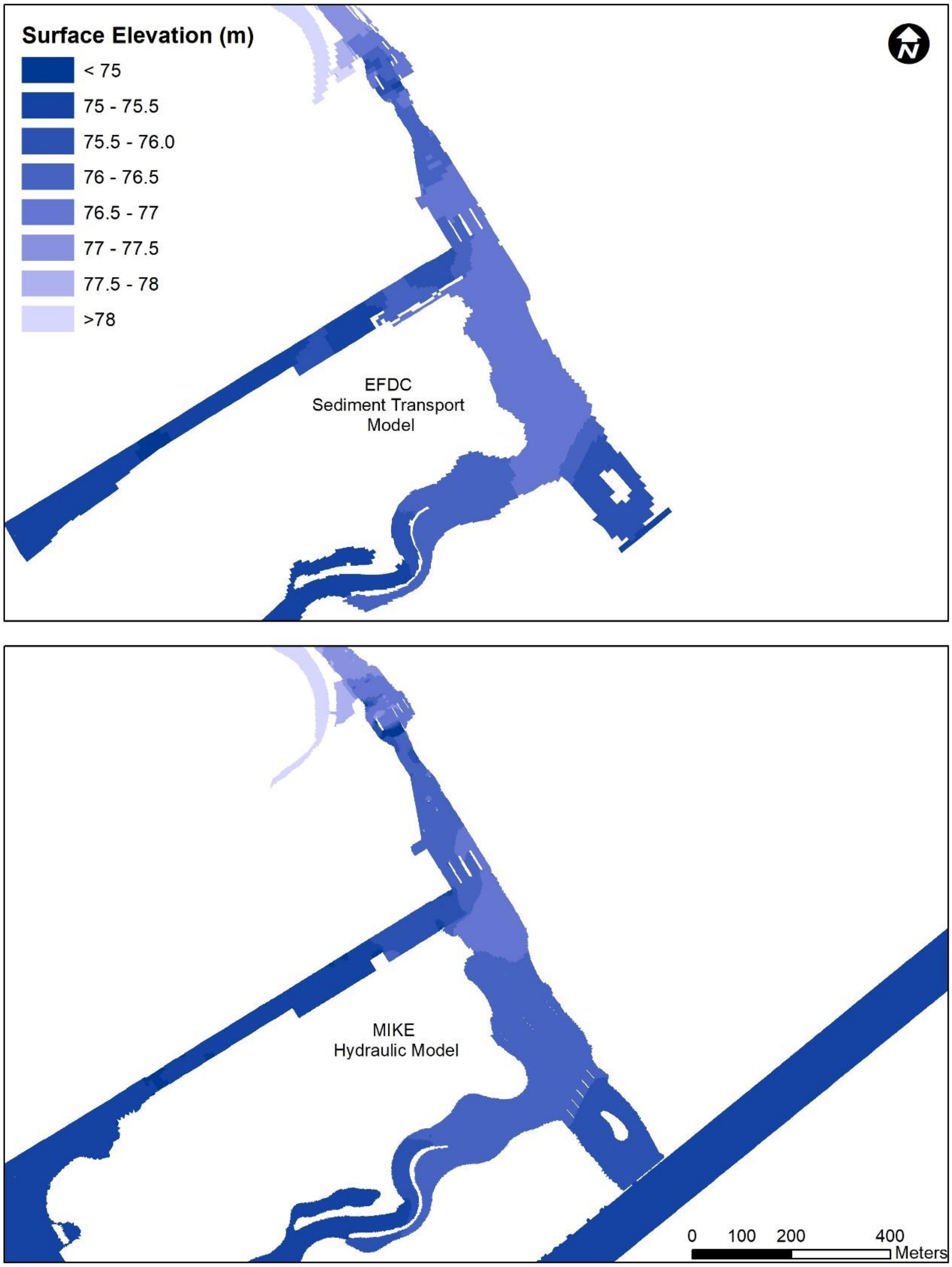


Figure 5-19: Comparison of EFDC and MIKE Predicted Water Levels, 1000-cms Event

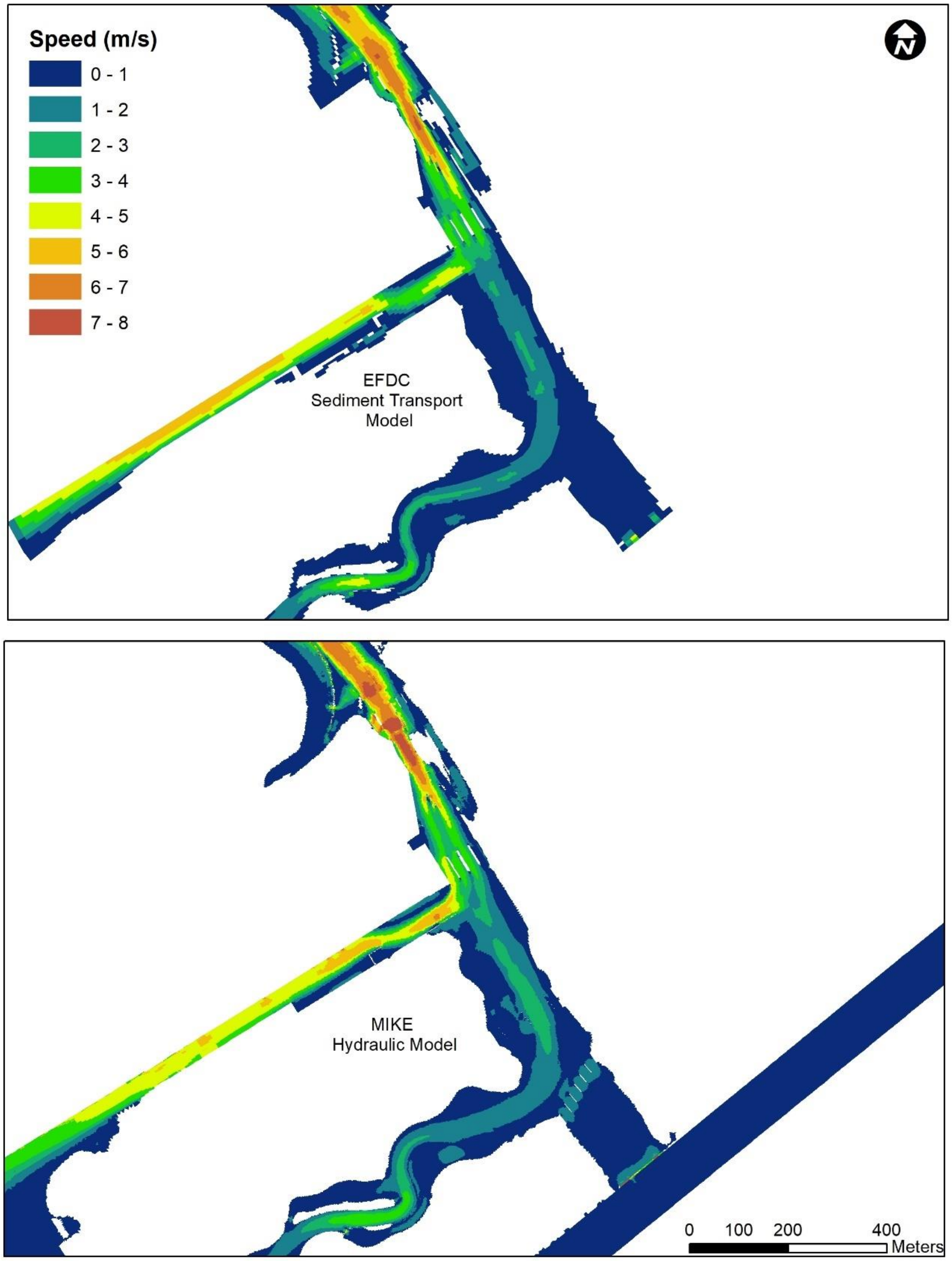


Figure 5-20: Comparison of EFDC and MIKE Predicted Current Speeds, Regulatory Event

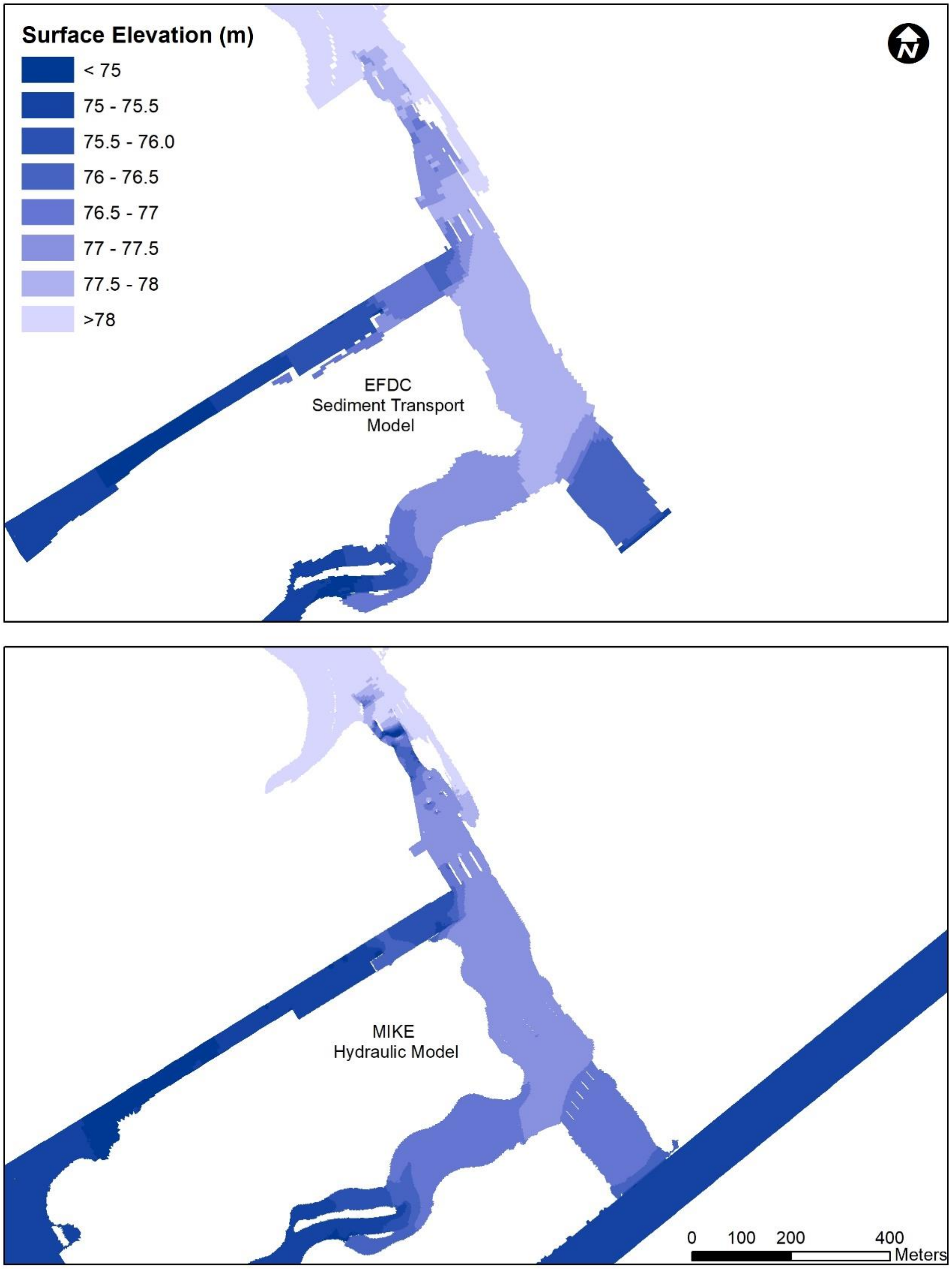


Figure 5-21: Comparison of EFDC and MIKE Predicted Water Levels, Regulatory Event

5.2.2 Sediment Transport Model Inputs

Sediment transport inputs describe characteristics of the incoming Don River solids load and the sediment bed in the project area. In combination with predictions of modeled hydraulics, these inputs largely determine predictions of modeled erosion and deposition. In the context of predicting long-term sediment maintenance on the site, model inputs related to solids loads are most impactful. This is because erosion of the sediment bed is minimal under most conditions, and the SDMA and Keating Channel are highly depositional. In the context of predicting morphologic change and flooding during the regulatory event, both upstream load inputs and bed inputs are impactful because some locations are highly depositional while others are highly erosional.

The fine-grained sediment loading rates were developed using historical water column suspended solids data measured at the Todmorden gage. Figure 5-22 illustrates the calibrated relationship between flow and fine grained suspended solids concentration. An expression of the form below was used to estimate fine-grained sediment concentrations.

$C(Q) = k * Q^m$

Where:
$C(Q)$ is fine-grained sediment concentration as a function of flow rate (mg/L);
k is a calibrated coefficient (calibrated to 18);
m is a calibrated exponent (calibrated to 1.1); and,
Q is the total Don River flow rate (cms);

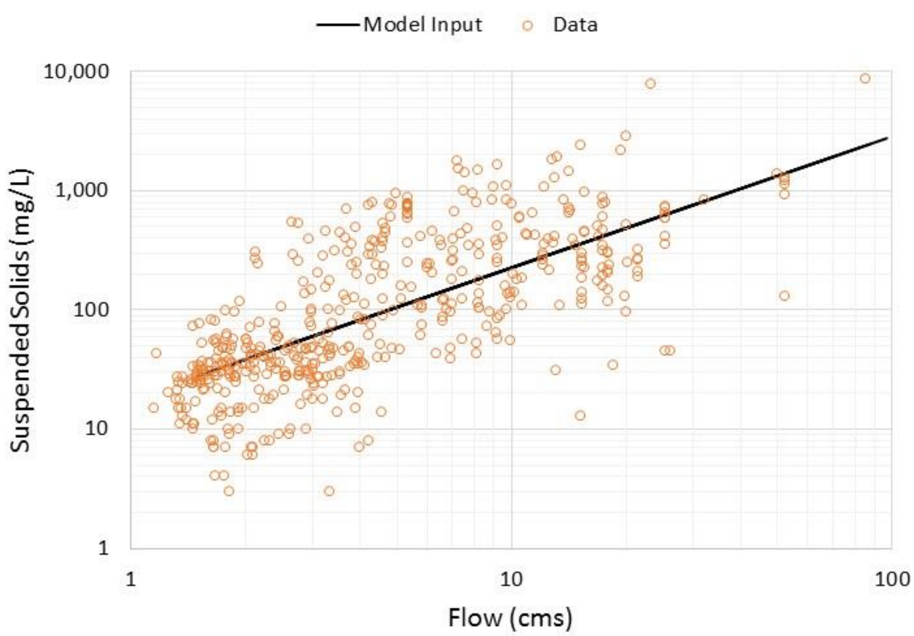


Figure 5-22: Modeled Relationship (R^2 = 0.3) between Flow and Fine-grained Sediment Concentrations

The coarse-grained (non-cohesive) sediment loading rates were calibrated for consistency with the inter-annual variability in Keating Channel sedimentation illustrated above in Figure 5-22. An expression of the form below was used to estimate non-cohesive sediment concentrations. The parameters k, m, C_{max}, and Q_{crit} were adjusted per particle class.

$C(Q) = Max(k * Q^m, C_{max})$ for $Q >= Q_{crit}$
$C(Q) = 0$ for $Q < Q_{crit}$

Where:
$C(Q)$ is non-cohesive sediment concentration as a function of flow rate (mg/L);
k is a calibrated coefficient;
m is a calibrated exponent (calibrated to 4 for all particle classes);
Q is the total Don River flow rate (cms);
C_{max} is the maximum non-cohesive sediment concentration (mg/L, also calibrated); and,
Q_{crit} is the flow at which non-cohesive transport begins to occur (cms).

Table 5-1 summarizes the input parameters for each non-cohesive particle class. Figure 5-23 illustrates these expressions graphically.

Table 5-1: Summary of Non-cohesive Concentration Input Parameters

Particle Type	k	C_{max} (mg/L)	Q_{crit} (cms)
Fine Sand	1e-4	3,500	8
Medium Sand	5e-5	1,750	10
Coarse Sand	2e-5	1,050	12
Gravel	n/a	n/a	n/a

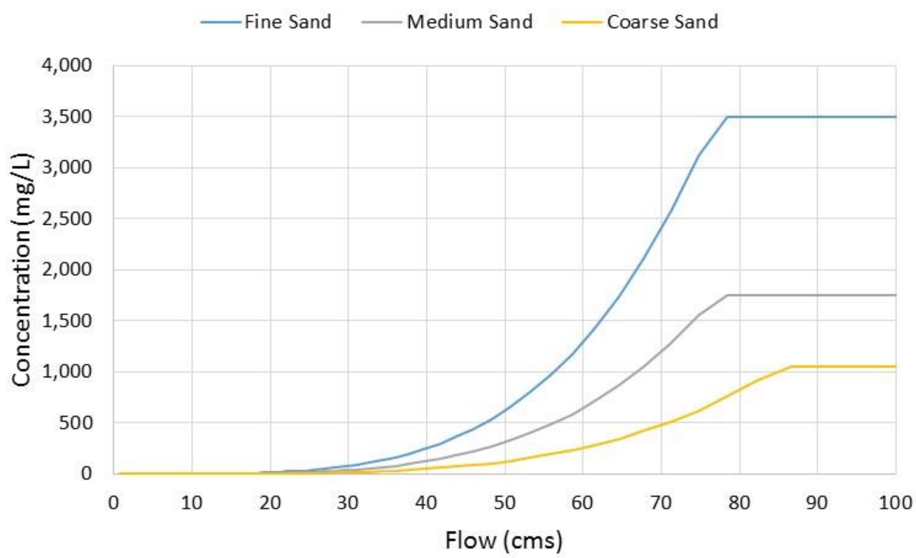


Figure 5-23: Modeled Relationship between Flow and Coarse-grained Sediment Concentrations

This variation in conditions helped to constrain assumptions relating flow condition to the Don River coarse grained solids load.

An exponent of four on the flow term was chosen for two primary reasons:

1. According to sediment transport theory, suspended solids concentrations should be proportional to stream flow to the fourth power because (a) sediment erosion rates are commonly observed in

flume studies to be proportional to the square of bed shear stress, (b) bed shear stresses are proportional to the square of fluid velocity, and (c) stream velocity and discharge are proportional.

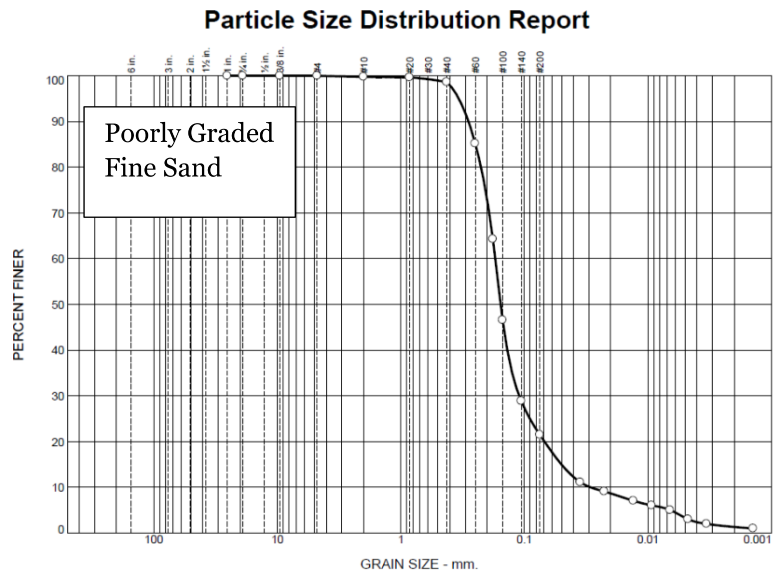
2. Tests using smaller exponents, e.g. 3 instead of 4, were not able to reproduce the variability in Keating Channel sediment deposition volumes for the six calibration periods. Increasing the exponent from three to four produced greater annual variability in the model predictions in a manner that was consistent with the observed data.

The coefficient and maximum suspended solids concentration inputs were constrained through the model calibration process, and based in part on the Todmorden suspended solids data. More discussion related to these inputs is included in the calibration section below.

Inputs related to the sediment bed effect erodibility, and the size and fractionation of sediments available for erosion. These also have some impact on deposition in the project area, as sediments eroded from areas upstream in the model may be deposited farther downstream. Five sediment classes were used to represent the distribution of Don River sediments: one fine-grained class (a medium silt), and four coarser-grained classes (fine, medium, and coarse sands, and gravel). Greater resolution was chosen for the coarse-grained sediment size distribution to account for armoring of the sediment bed during high flow conditions, i.e. preferential erosion of finer-grained sediments, leading to the coarsening of the sediment bed, and reduced erodibility. This greater degree of coarse-grained resolution was also useful for evaluating the model's ability to accurately represent sorting of particle sizes in depositional zones, which helped to constrain model inputs and assumptions and improve model certainty.

Modeled bed erodibility was initially parameterized using SedFlume erosion rates described in Roberts (1998) and Lopez-Soto and Robbins (2018), then modified during the calibration process based on observed depositional patterns. A lookup table is used as input to the model to describe the relationship between the median particle size in the sediment bed at a given location (D50), the computed bed shear stress, and the composite erosion rate of sediments. Bed erodibility was calibrated based on observed depositional patterns in the Keating Channel, and long-term sediment bed elevation predictions in the lower Don River, which are expected to be in equilibrium over long time periods. This process is further described below.

The grain size distribution within the project area was characterized based on five samples of the surficial sediment bed within the Don Narrows. These samples are generally characterized by two types of grain size distribution: a poorly-graded sediment bed comprised mostly of fine sand, and a well-graded sediment bed with much larger non-cohesive particles including appreciable fractions of gravel. Figure 5-24illustrates these two sediment types.



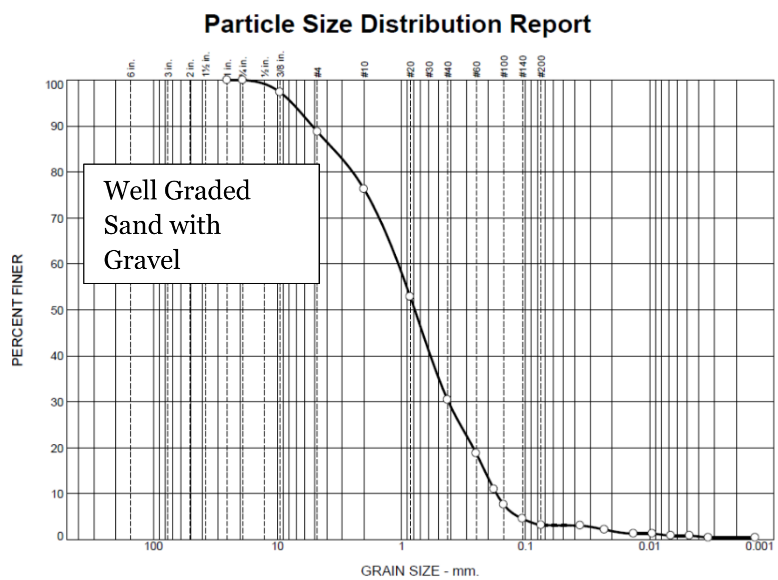


Figure 5-24: Representative Grain Size Distributions in the Lower Don River Surficial Sediments

Modeled sediments in the Lower Don River were characterized as an equal mixture of these two sediment types. As described further below, model results are relatively insensitive to these inputs because 1) under most conditions, current velocities are not high enough to scour the sediment bed significantly relative to the upstream sediment load, and 2) during the regulatory event, while significant scour does occur, the amount of scour in the project area is small relative to the estimated quantity delivered from the watershed.

Additional discussion of model inputs is included below in the context of model calibration and sensitivity testing.

5.3 Model Calibration and Sensitivity Testing

Model calibration is the process of fine-tuning model parameters to provide a suitably accurate representation of the actual system. This is accomplished by simulating conditions for comparison with observed data and iteratively refining model inputs until a single best set of inputs are identified. During this process, modelers also explore the range of potential sets of inputs that are still reasonably consistent with the observed data, but less fitting than the best set of inputs (i.e. sensitivity tests). By then applying these ranges of potential inputs, we describe the range of uncertainty associated with the model inputs. Included below is a summary of our process for conducting model calibration for the sediment transport model, final calibration results, and sensitivity testing related to those calibration results.

5.4 Model Calibration and Identification of Base Parameters

The model calibration included six periods for which there were data describing sedimentation from after dredging occurred in the summer or early fall to before dredging occurred the following year. Such data were available for each year from 2001 through 2007, and the peak flow during this period was 97 cubic metres per second. Figure 5-25 compares the measured sedimentation volumes along the Keating Channel for these periods. Most deposition occurs at the upstream end of the Keating Channel (between stations 0 and 400 metres), and sedimentation in this zone varies significantly by period depending on upstream flow conditions.

Data from the 2001-2002 period are indicative of a net sediment loss at, e.g., station 300m. These data are likely subject to some error because it is unlikely that erosion or dredging occurred during this period. Still, these data are indicative of lower sedimentation rates that occurred in 2001-2002, which is consistent with the relatively low flow conditions during this period, and were useful for informing the calibration.

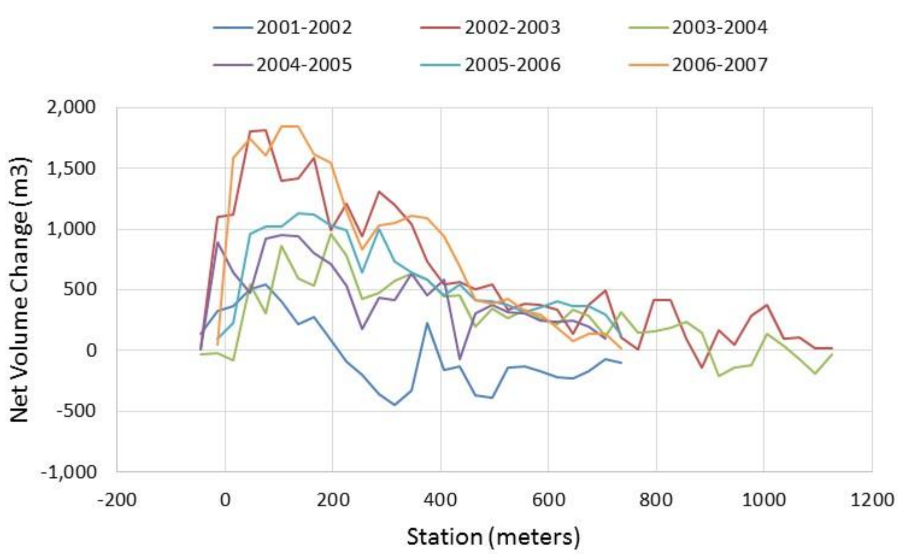


Figure 5-25: Measured Sedimentation Patterns along Keating Channel (Channel stationing measured from west side of Lake Shore Bridge)

Exact dates of the pre-dredge and post-dredge bathymetric surveys were not available for most surveys, and dates of these surveys were assumed to be consistent with the dates of surveys from other years for which dates were known. This information gap introduces some uncertainty in comparing modeled and observed sedimentation, especially during years when high flow events occurred near the assumed bathymetric survey dates. All post-dredge surveys were assumed to occur on October 15 and all pre-dredge surveys were assumed to occur on June 1. Model simulations were conducted for this period, but the measurements may have occurred on slightly different dates.

Keating Channel grain size data also guided the model calibration. These data describe how sediments are sorted from the upstream end of the Keating Channel toward the harbour, with reductions in grain size in the downstream direction. Moving from the upper 200 metres of the channel to farther downstream, sediment contents shifts from 56% coarse-grained to 36% coarse-grained, with nearly all medium to coarse sands settling out in the upper 200 metres of the channel.

Modeled and observed sedimentation in the Keating Channel agreed well, with model results being within 8% of total observed sedimentation for the six modeled periods. Figure 5-26 and Figure 5-27 compare modeled and observed sedimentation for the total of the six calibration periods (Figure 5-26) and for the individual periods (Figure 5-27).

The model tends to slightly over-predict sedimentation near the upstream end of the Keating Channel (station = 100 m) and underpredict sedimentation near the downstream end of the Keating Channel (stations 200 to 700). Numerous model tests were conducted to improve the modeled distribution of sediments, and the calibrated model was determined to provide a suitable balance between consistency with both the grain size data and sedimentation data, and a degree of conservatism in predicting flood conditions. Further, the sediment transport model and MIKE hydrodynamic model were applied for a wide range of sensitivity tests to evaluate the impacts of model uncertainty on flood predictions. The primary factor reducing agreement between modeled and observed sedimentation appears to be the distribution of fine sands. Fine sands are transported less far into the Keating Channel than the grain size data indicate that they actually are.

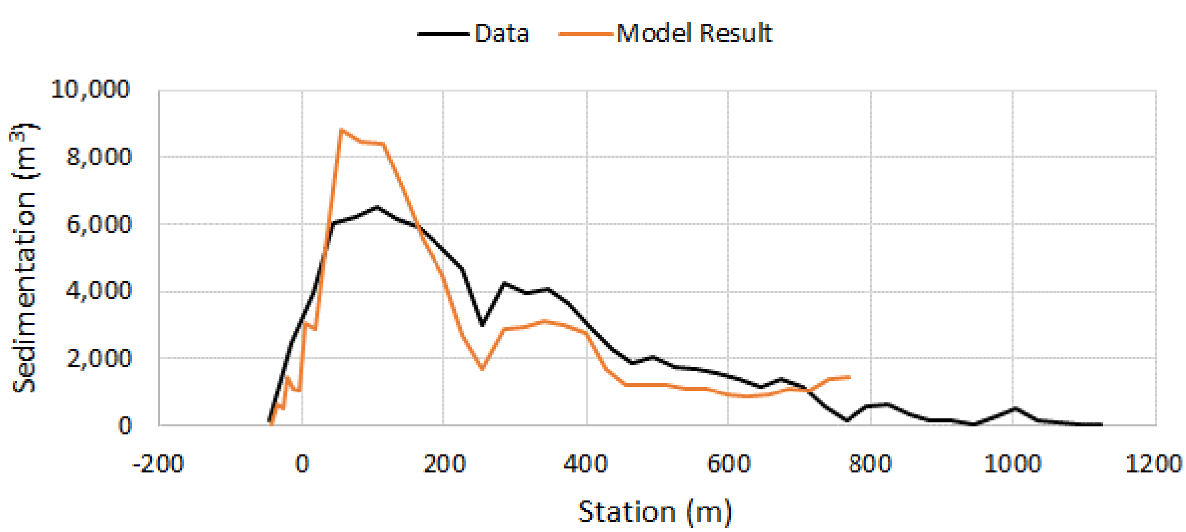


Figure 5-26: Comparison of Modeled and Observed Sedimentation along the Keating Channel (Total of Six Periods) (Channel stationing measured from west side of Lake Shore Bridge)

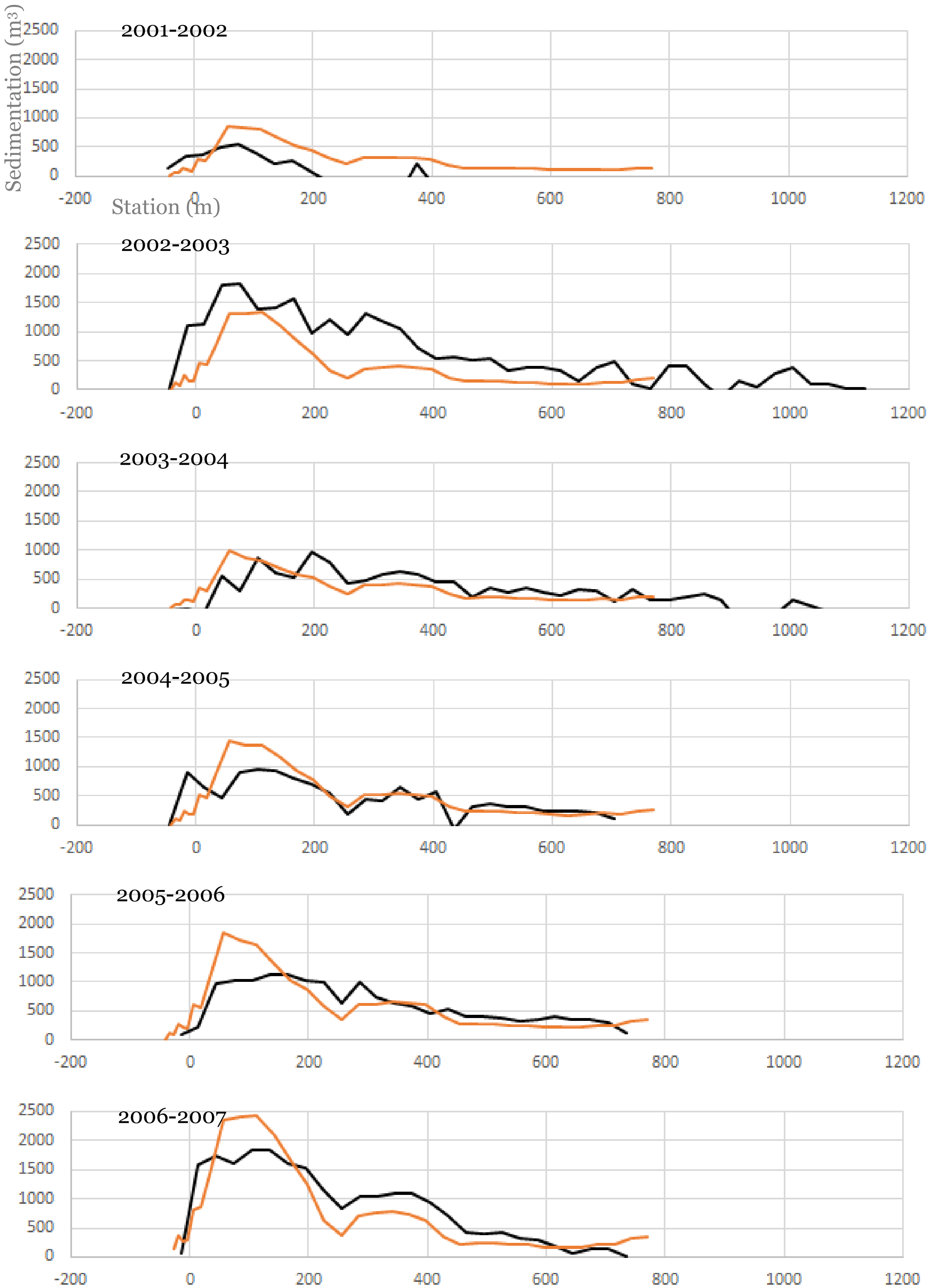


Figure 5-27: Comparison of Modeled and Observed Sedimentation along the Keating Channel (Each of Six Periods) (Channel stationing measured from west side of Lake Shore Bridge)

Data during two of the six periods were less reliable for model calibration. The 2001-2002 data indicate localized areas of sediment loss (i.e. bed scour or dredging), though it is unlikely that sediment loss occurred during this period of low flow conditions. Instead, it is likely that one or both of the bathymetric measurements were not corrected for the assumed datum, and so the calculations of sedimentation are in error but cannot be corrected and used as a quantitative target for calibration.

The 2002-2003 data are much higher than can be explained by river flow conditions that occurred during this period. It is likely that the assumed period between which bathymetric measurements were made (October 15, 2002 to June 1, 2003) is different than the period when measurements were actually made. Multiple high flow events occurred just before the start and just after the end of the assumed period, and if the data account for these events but the model does not, the model would tend to be biased significantly low, as it is.

Modeled and observed grain size distributions in the Keating Channel generally compare well, with consistency in modeled and observed sorting patterns along the channel. However, the model tends to overpredict the fraction of medium and coarse sand that accumulates in the Keating Channel relative to finer particle types. By extension, the model may tend to accumulate sediments more locally in depositional areas than distribute them more broadly. Figure 5-28 compares modeled and observed grain size distributions along the length of the Keating Channel.

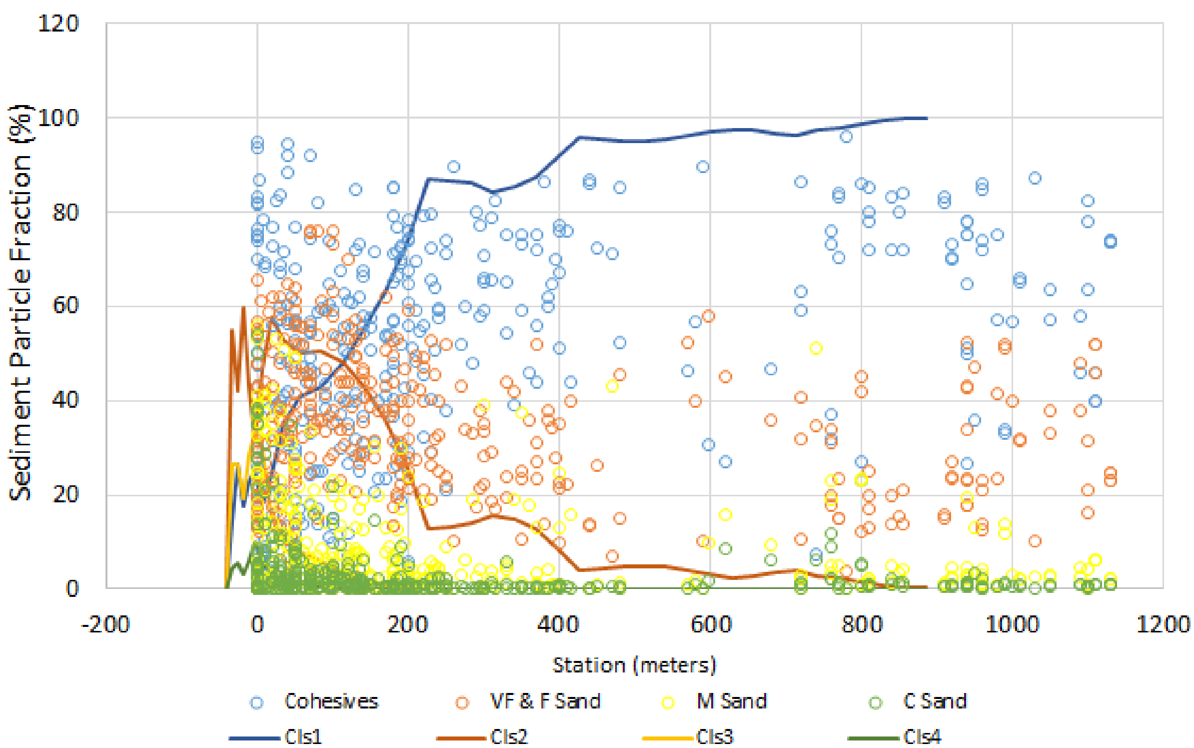


Figure 5-28: Comparison of Modeled and Observed Grain Size Distributions along the Keating Channel (Channel stationing measured from west side of Lake Shore Bridge). Lines represent model output and circles represent observed data.

Most calibration simulations were conducted to refine inputs and assumptions related to the watershed solids loading of non-cohesive sediments, and the erosion rates of deposited sediments. Numerous combinations of simulations were conducted, exploring plausible input assumptions that influence the long-term predicted impacts of these processes on sedimentation in the Keating Channel. Watershed solids load inputs were constrained by comparing modeled and observed total deposited sediment volumes among the six years. It was determined that a relatively narrow range of non-cohesive input assumptions were consistent with the wide inter-annual range of sedimentation volume computed from the six periods of survey data. Erosion rates were constrained in parallel with watershed solids loading rates. These rates were originally set to the lower end of observed erosion rates from flume studies. Increases to erosion rate inputs were explored, but it was determined that erosion rates more than approximately three times higher than the calibrated rates generated net scour at the upstream end of the Keating Channel at a highly depositional location. This helped to constrain the upper end of erosion rate inputs.

Calibrated erosion rate inputs are included in Table 5-2 below.

Table 5-2: Sediment Transport Model Erosion Rate Inputs

D50 of Layer (micron)	Erosion Rates (cm/s) at Applied Shear Stress (Pa)											
	0	0.1	0.2	0.4	0.8	1.6	3.2	6.4	12.8	25.6	51.2	102.4
1	1.0E-10	1.0E-10	1.0E-10	2.0E-06	1.1E-05	6.5E-05	3.7E-04	2.1E-03	1.2E-02	6.6E-02	3.8E-01	2.1E+00
75	1.0E-10	1.0E-10	1.0E-10	2.0E-06	1.1E-05	6.5E-05	3.7E-04	2.1E-03	1.2E-02	6.6E-02	3.8E-01	2.1E+00
125	1.0E-10	1.0E-10	1.0E-10	2.0E-06	1.1E-05	6.5E-05	3.7E-04	2.1E-03	1.2E-02	6.6E-02	3.8E-01	2.1E+00
222	1.0E-10	1.0E-10	1.0E-10	2.0E-06	1.1E-05	6.5E-05	3.7E-04	2.1E-03	1.2E-02	6.6E-02	3.8E-01	2.1E+00
432	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.4E-05	2.3E-04	1.4E-03	6.9E-03	3.0E-02	1.3E-01	5.1E-01	2.1E+00
1,020	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.4E-05	2.3E-04	1.4E-03	6.9E-03	3.0E-02	1.3E-01	5.1E-01	2.1E+00
1,350	1.0E-09	1.0E-09	1.0E-09	1.0E-09	2.5E-07	1.4E-04	1.2E-03	6.4E-03	2.9E-02	1.2E-01	5.1E-01	2.1E+00
2,000	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	2.2E-05	7.5E-04	5.3E-03	2.7E-02	1.2E-01	5.0E-01	2.1E+00
3,000	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	2.3E-04	3.6E-03	2.3E-02	1.1E-01	4.8E-01	2.0E+00
5,000	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-03	1.5E-02	9.2E-02	4.4E-01	1.9E+00
10,000	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	3.5E-03	5.8E-02	3.6E-01	1.8E+00
20,000	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.0E-09	1.4E-02	2.3E-01	1.5E+00

5.5 Model Sensitivity Testing and Identification of Parameter Ranges

Model sensitivity tests were conducted to explore the range of potential sets of inputs that are still reasonably consistent with the observed data, but less fitting than the best calibrated set of inputs. By then applying these ranges of potential inputs to the model application, we describe the range of uncertainty associated with the model inputs. Such input ranges were identified for three types of model inputs: sediment erosion rates, watershed solids loading rates, and sediment bed grain size distribution.

An upper bound on erosion rates was determined to be a multiplicative factor of three greater than calibrated erosion rates. For erosion rates significantly higher than three times higher than the calibrated rates, the model began to predict scour at locations known to be highly depositional at the upstream end of the Keating Channel. Figure 5-29 illustrates model sensitivity to erosion rates for the 2006-2007 calibration period.

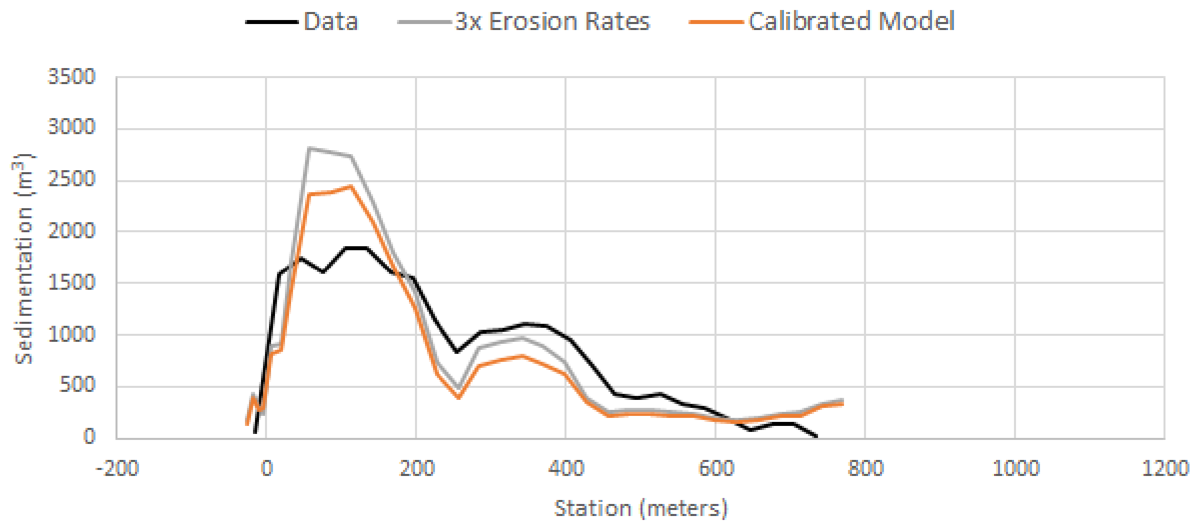


Figure 5-29: Model Sensitivity Test Results: Increased Erosion Rates

Watershed solids loads were increased and decreased by 50% to represent the range in plausible long-term watershed loads. This range was identified by conducting sensitivity tests for various ranges, and identifying the largest range that was still relatively consistent with observations. Figure 5-30 below illustrates model sensitivity to watershed solids loads for the 2006-2007 calibration period.

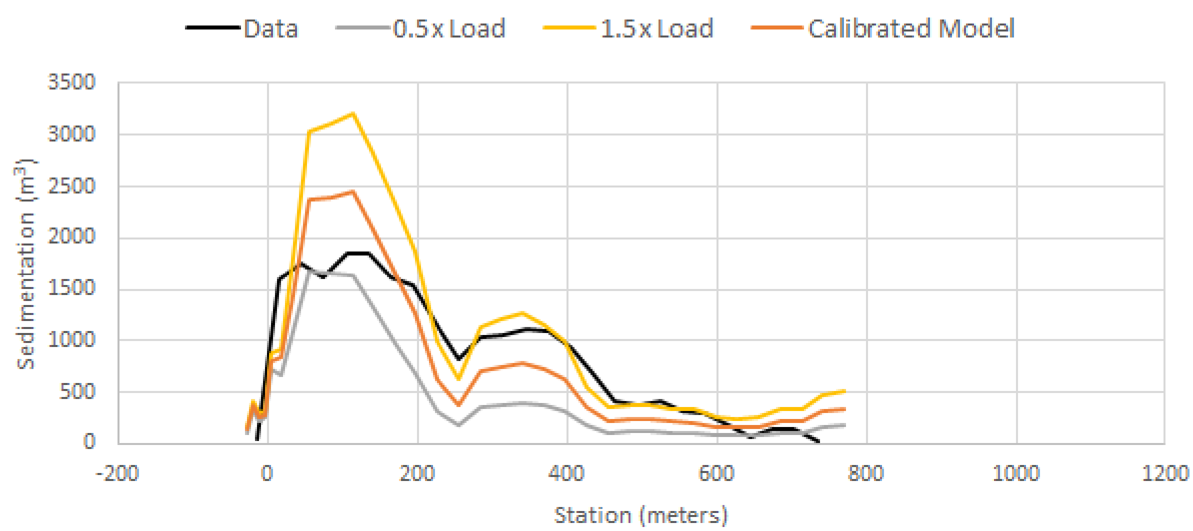


Figure 5-30: Model Sensitivity Test Results: Watershed Solids Load

Sensitivity tests for the sediment bed grain size distribution were based on the available surficial sediment data illustrated in Figure 5-31. The calibrated model assumes a sediment bed that is a uniform mixture of the two sampled sediment types, a poorly-graded fine sand bed, and a well-graded sand bed with gravel. Two sensitivity tests were evaluated: one with the bed comprised entirely of poorly-graded fine sand, and one with the bed comprised of entirely well-graded sand with gravel. Figure 5-31 below illustrates model sensitivity to the sediment bed grain size distribution for the 2006-2007 calibration period. Each of these sensitivity tests were carried forward to simulations of the regulatory event to describe the range of potential sediment transport outcomes during the regulatory event.

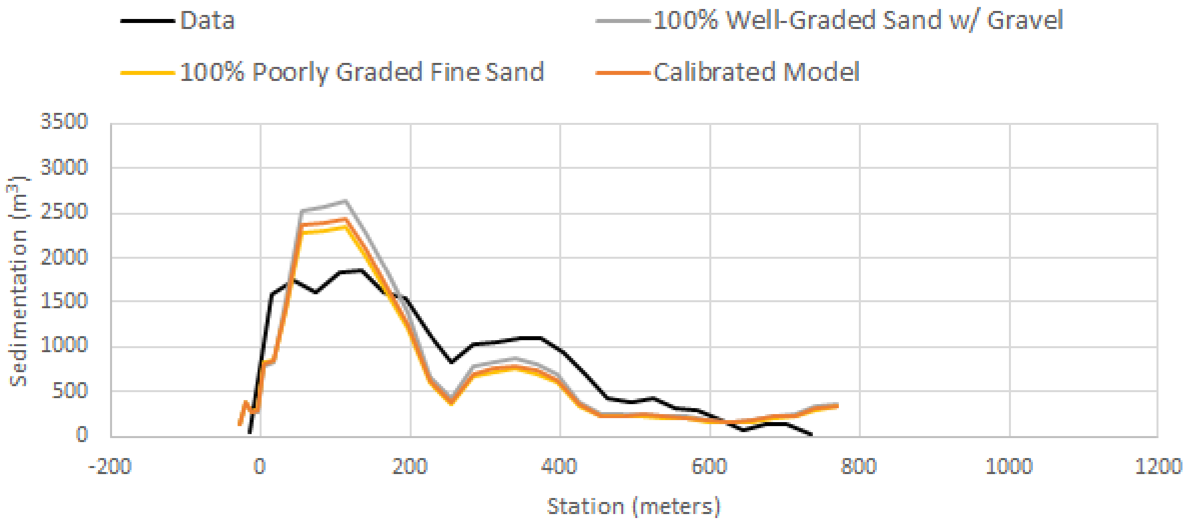


Figure 5-31: Model Sensitivity Test Results: Sediment Bed Grain Size Distribution

6 Model Application: Regulatory Event Flooding

The calibrated sediment transport model was applied to predict the degree of scour and sediment deposition (i.e. morphologic change) that would occur during the regulatory flood event, and the effects of those changes on flood risk. Our approach to conduct this modeling was as follows: first, simulate morphologic change during the event using the sediment transport model; second, transfer the simulated morphologic change to the MIKE model to specify starting sediment bed elevations in the hydraulic model; third, compute predicted flood levels in the MIKE model. Recognizing the uncertainty inherent in predictions of modeled morphologic change during an extreme event like the regulatory flood, we produced eighteen simulations that helped bound the degree of uncertainty in the model predictions. This modeling approach is further described below.

Predictions of morphologic change from the sediment transport model were sampled just prior to the peak of the flood event. This point in time was determined to produce the critical condition for flood risk by sampling morphologic change output at various times during the regulatory event hydrograph, transferring the predicted morphologic change to the MIKE model, and predicting the associated flood levels.

Sensitivity tests were conducted to describe how input uncertainty in the sediment transport model affects simulated regulatory flood levels. These tests quantified the range of uncertainty associated with the upstream solids load, the grain size distribution of the sediment bed, and modeled erosion rates. Ranges for these inputs were determined in part during model calibration and sensitivity testing and based in part on professional judgment.

The sediment transport model was run for base design conditions, along with a number of sensitivity runs to test the impact of increasing the upstream load, modifying the erosion rates, and the varying the composition of the upstream bed particle size distributions to test a range of outcomes of the bed morphology at the peak of the regulatory flood event. The sections below discuss the range of outcomes predicted by the model for each of the scenarios.

The sensitivity simulations were conducted with two different initial bed elevations. The first initial bed condition of “Full” represents a scenario when the sedimentation areas are filled to capacity, and the second initial bed condition represents the “Design” elevations that will be maintained by dredging. The SDMA Full initial conditions indicate that the SDMA was filled with sediment up to elevation 72.5 in Areas A and B, sloping down to elevation 72.0 in the Lake Shore Boulevard Bridge, the Ice Management Area, and the western portion of the Keating Channel. The SDMA Design initial conditions indicate that the SDMA was filled with sediment up to elevation 70.0, and all the other project areas were as designed in the PLFP. The initial bed elevations for each run are provided with the maps of the model run results for all the sensitivity test runs in Appendix A.

The 36 sensitivity tests were conducted for both the Full Vision design and the Interim Condition design for a total of 72 sensitivity tests. The Interim Condition design includes a section of the SDMA only partially excavated to allow protection of the existing Gardiner piers in the vicinity.

Additional plots of the model output from the sediment transport model runs have been included in Appendix A. The predicted bed morphology and bed delta (elevation change since the beginning of the simulation) are shown for all 72 sensitivity test for selected key locations in the model domain.

Table 6-1: Summary of Sediment Transport Model Runs – Full Vision Design

Run No.	SDMA Initial Conditions	Sediment Bed Composition Alternatives	Erosion Rates Alternatives	Upstream Load Alternatives
001	Full	Base Particle Size Distribution	Base	Base
002	Full	Base Particle Size Distribution	Base	+50%
003	Full	Base Particle Size Distribution	Base	-50%
004	Full	Base Particle Size Distribution	Increased 3X Base	Base
005	Full	Base Particle Size Distribution	Increased 3X Base	+50%
006	Full	Base Particle Size Distribution	Increased 3X Base	-50%
007	Full	Decreased D50	Base	Base
008	Full	Decreased D50	Base	+50%
009	Full	Decreased D50	Base	-50%
010	Full	Decreased D50	Increased 3X Base	Base
011	Full	Decreased D50	Increased 3X Base	+50%
012	Full	Decreased D50	Increased 3X Base	-50%
013	Full	Increased D50	Base	Base
014	Full	Increased D50	Base	+50%
015	Full	Increased D50	Base	-50%
016	Full	Increased D50	Increased 3X Base	Base
017	Full	Increased D50	Increased 3X Base	+50%
018	Full	Increased D50	Increased 3X Base	-50%
019	Design	Base Particle Size Distribution	Base	Base
020	Design	Base Particle Size Distribution	Base	+50%
021	Design	Base Particle Size Distribution	Base	-50%
022	Design	Base Particle Size Distribution	Increased 3X Base	Base
023	Design	Base Particle Size Distribution	Increased 3X Base	+50%
024	Design	Base Particle Size Distribution	Increased 3X Base	-50%
025	Design	Decreased D50	Base	Base
026	Design	Decreased D50	Base	+50%
027	Design	Decreased D50	Base	-50%
028	Design	Decreased D50	Increased 3X Base	Base
029	Design	Decreased D50	Increased 3X Base	+50%
030	Design	Decreased D50	Increased 3X Base	-50%
031	Design	Increased D50	Base	Base
032	Design	Increased D50	Base	+50%

033	Design	Increased D50	Base	-50%
034	Design	Increased D50	Increased 3X Base	Base
035	Design	Increased D50	Increased 3X Base	+50%
036	Design	Increased D50	Increased 3X Base	-50%

Table 6-2: Summary of Sediment Transport Model Runs – Interim Conditions Design

Run No.	SDMA Initial Conditions	Sediment Bed Composition Alternatives	Erosion Rates Alternatives	Upstream Load Alternatives
001	Full	Base Particle Size Distribution	Base	Base
002	Full	Base Particle Size Distribution	Base	+50%
003	Full	Base Particle Size Distribution	Base	-50%
004	Full	Base Particle Size Distribution	Increased 3X Base	Base
005	Full	Base Particle Size Distribution	Increased 3X Base	+50%
006	Full	Base Particle Size Distribution	Increased 3X Base	-50%
007	Full	Decreased D50	Base	Base
008	Full	Decreased D50	Base	+50%
009	Full	Decreased D50	Base	-50%
010	Full	Decreased D50	Increased 3X Base	Base
011	Full	Decreased D50	Increased 3X Base	+50%
012	Full	Decreased D50	Increased 3X Base	-50%
013	Full	Increased D50	Base	Base
014	Full	Increased D50	Base	+50%
015	Full	Increased D50	Base	-50%
016	Full	Increased D50	Increased 3X Base	Base
017	Full	Increased D50	Increased 3X Base	+50%
018	Full	Increased D50	Increased 3X Base	-50%
019	Design	Base Particle Size Distribution	Base	Base
020	Design	Base Particle Size Distribution	Base	+50%
021	Design	Base Particle Size Distribution	Base	-50%
022	Design	Base Particle Size Distribution	Increased 3X Base	Base
023	Design	Base Particle Size Distribution	Increased 3X Base	+50%
024	Design	Base Particle Size Distribution	Increased 3X Base	-50%
025	Design	Decreased D50	Base	Base
026	Design	Decreased D50	Base	+50%
027	Design	Decreased D50	Base	-50%
028	Design	Decreased D50	Increased 3X Base	Base
029	Design	Decreased D50	Increased 3X Base	+50%
030	Design	Decreased D50	Increased 3X Base	-50%

031	Design	Increased D50	Base	Base
032	Design	Increased D50	Base	+50%
033	Design	Increased D50	Base	-50%
034	Design	Increased D50	Increased 3X Base	Base
035	Design	Increased D50	Increased 3X Base	+50%
036	Design	Increased D50	Increased 3X Base	-50%

6.1 Sediment Transport Model Results

6.1.1 Don Narrows and CNR Bridge

The amount of scour in the Don Narrows between the Eastern Avenue bridge and the CNR bridge impacts the hydrodynamics around the CNR bridge, and the water surface elevations at the flood protection measures upstream of the CNR bridge, which include the Eastern Avenue grading, the future Broadview Extinction grading, and the West Donlands FPL. The sediment transport model in this area was set up to allow for a maximum scour of up to 3.0 metres from the existing bed elevations. This is based on (a) the jet probe data provided by TRCA and (b) the borehole performed in the channel upstream of the CNR bridge (BHG18-206). The jet probe study does not provide specific sediment particle size classifications, but shows the general bed composition in 1 metre intervals from the existing riverbed. The jet probe results show that the bed consists of a sand-silt mixture, with some sections of clay and gravel. Sand was noted in 87.5 percent of the samples across all depth intervals. The clay and gravel content appears to increase with the deeper samples. All 40 of the jet probe locations were able to reach the 1-2 metre depth interval. There were 8 locations (20% of the total) where the results note that the jet probe was not able to probe to the full 3 metre depth. Four (4) of those locations noted rubble or a large stone as the reason for not being able to reach the depth. The remaining (4) four locations noted clay in the interval above. Locations that could not reach the full depth were general located closer to the river banks. The data from borehole BHG18-206 taken upstream of the CNR bridge shows 3.05 metres of loose sand with silt, and a very loose gravel pocket noted, above a layer of stiff clay.

Table 6-3: Summary of Jet Probe Survey Bed Materials by Depth Interval

Bed Material Type	Depth Interval	No. of Samples with Bed Material Noted	Percent of Samples with Bed Material Noted
Clay	0-1 m	3	8%
	1-2 m	20	50%
	2-3 m	27	84%
Silt	0-1 m	28	70%
	1-2 m	17	43%
	2-3 m	4	13%
Sand	0-1 m	37	93%
	1-2 m	32	80%
	2-3 m	29	91%
Gravel	0-1 m	6	15%
	1-2 m	10	25%

	2-3 m	20	63%

The sediment transport model results for the Don Narrows show that the majority of the channel is capable of scouring to the maximum allowed depth. A section of the predicted channel morphology within the Don Narrows is shown in Figure 6-1. The results across the entire channel show no variability with the sensitivity runs, leading to an increased confidence in the scour predictions in this channel section.

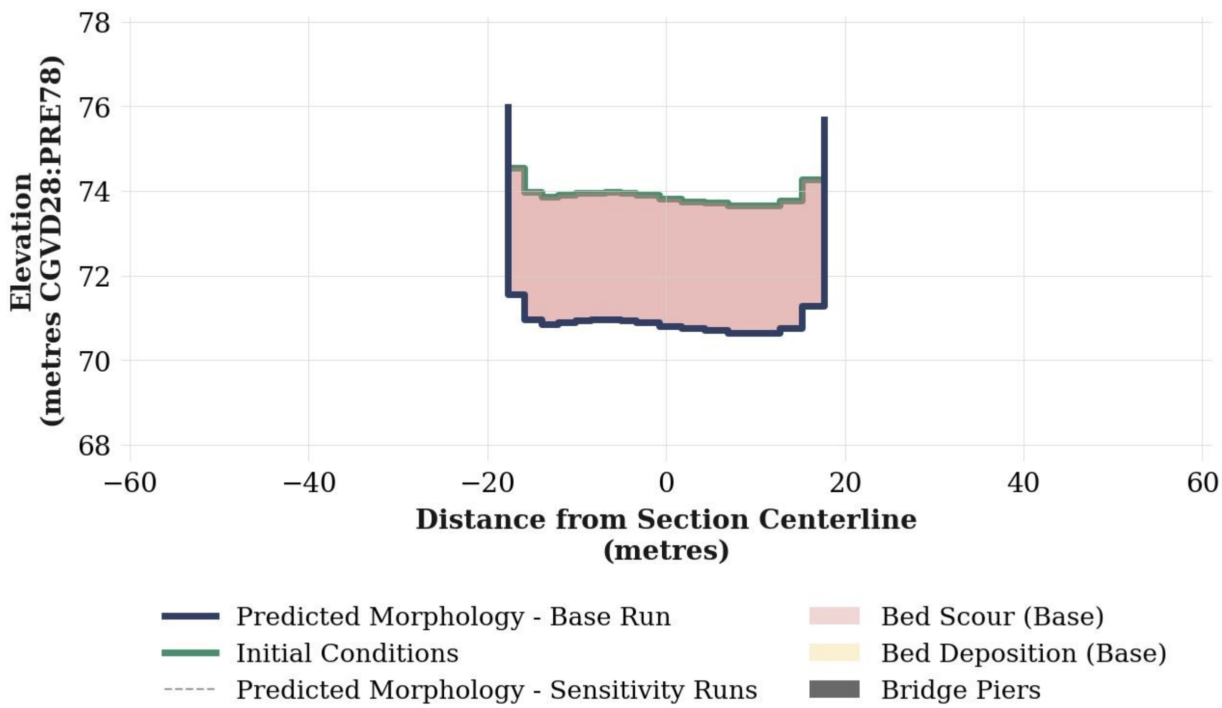


Figure 6-1: Predicted Bed Morphology Cross Section – Don Narrows 150 metres North of the CNR Bridge

The sediment transport model results for the river channel beneath the CNR Bridge show that the majority of the channel is capable of scouring the river bed to the maximum allowed depth of 3 metres. In western bay, where the channel was widened, there is some variability in the scour predicted under the sensitivity runs. The scour predicted under the base conditions in the western bay varies from 0.56 metres along the western bank to 3.0 metres adjacent to the bridge pier. The minimum scour along the western bank varies between 0.14 and 1.79 metres for all of the sensitivity runs. A section of the predicted channel morphology at the CNR Bridge is shown in Figure 6-2.

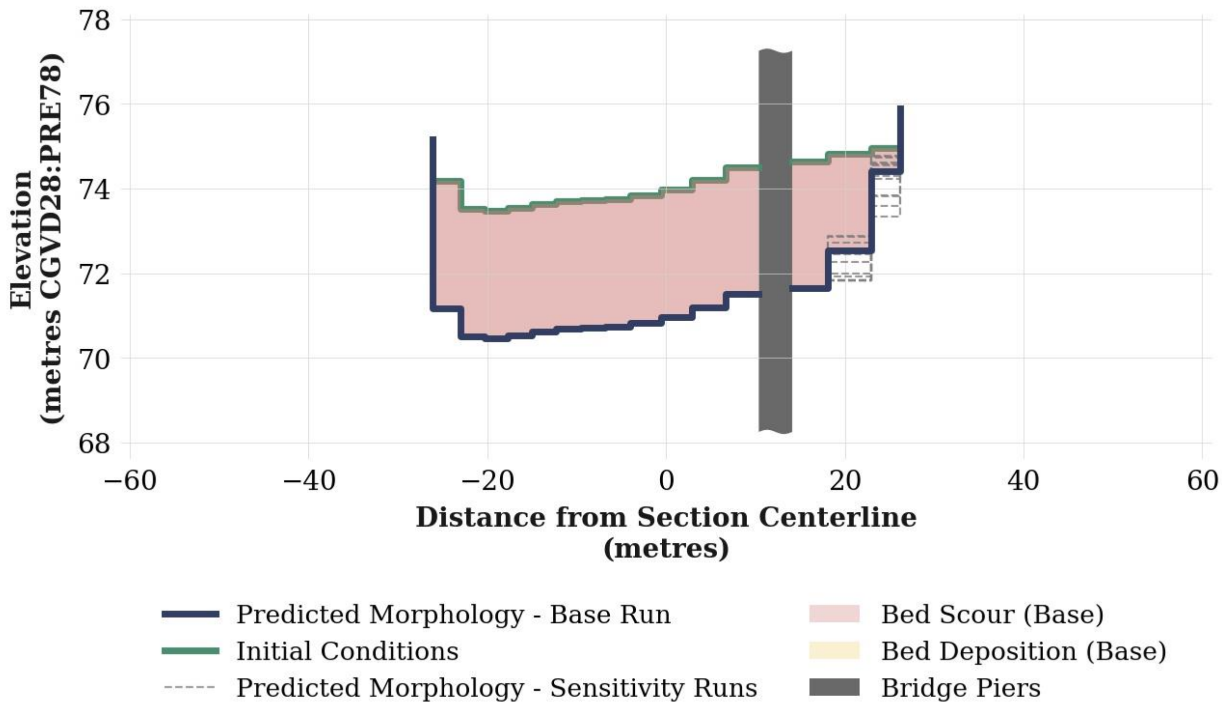


Figure 6-2: Predicted Bed Morphology Cross Section – CNR Bridge

The total area-averaged bed elevation change was analyzed for all the sediment transport sensitivity model runs, and the results for the Don Narrows and the CNR Bridge are shown in Figure 6-3 and Figure 6-4. The area-averaged bed elevation change in the Don Narrows for all model runs ranged from -2.95 metres, to -3.00 metres, with an average of -2.98 m for all model runs. The area-averaged bed elevation change in the channel under the CNR Bridge for all model runs ranged from -2.35 metres, to -2.61 metres, with an average of -2.48 m for all model runs. This area-based average is impacted by the lower scour in the western bay, along the western bank noted above.

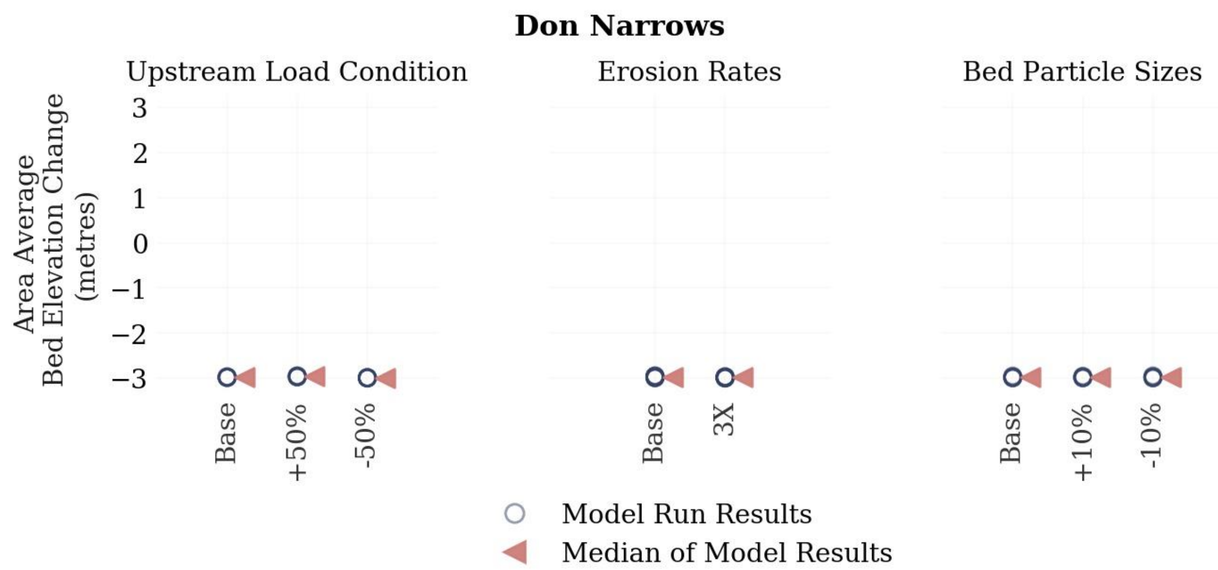


Figure 6-3: Area Average Bed Elevation Change Sensitivity Analysis – Don Narrows

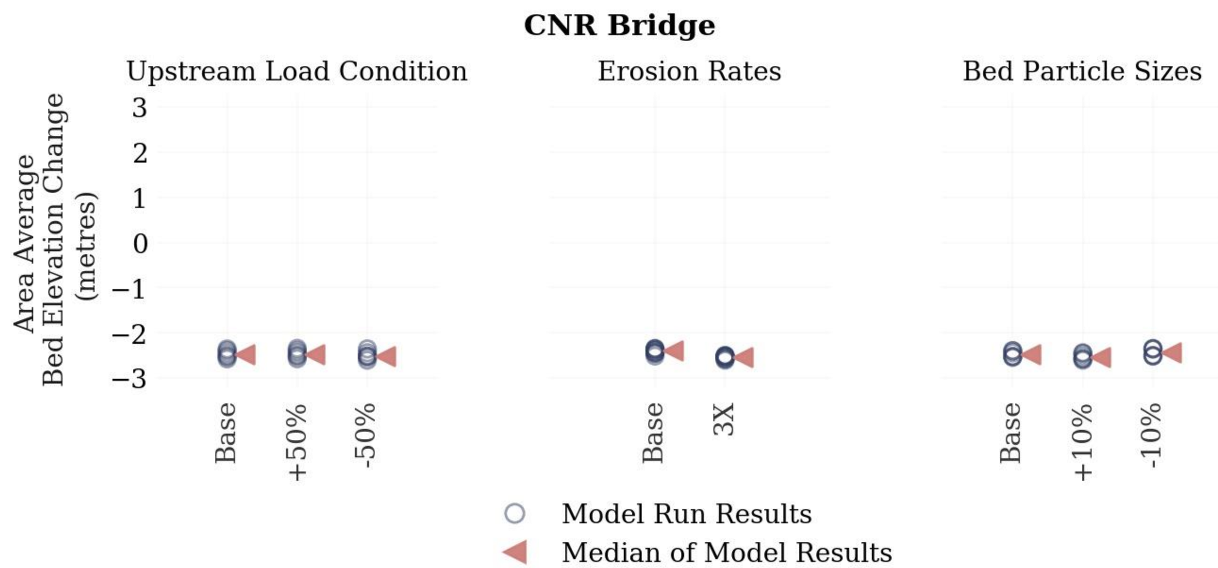


Figure 6-4: Area Average Bed Elevation Change Sensitivity Analysis – CNR Bridge

Based on the results of the jet probe survey showing that this area has erodible sediments to a depth of 3 metres over the majority of the channel, the characterization of the in-situ sediments in borehole BHG18-206, and the model results showing that the flows leading up to the peak of the regulatory flood event are capable of scouring the channel to 3 metres of depth, the sediment transport simulations were assumed to be able to scour a full 3 metres in this area. When the hydrodynamic models are run using the bed morphology at the peak of the regulatory flood event, the scour in this portion of the channel will be limited to a 1.5 metre depth as a conservative assumption. This is particularly conservative when combined with the morphology based on 3 metres of scour in this section, since the material that is scoured from the bed in this area can be deposited downstream in the PLFP site.

6.1.2 Sediment and Debris Management Area (SDMA)

The sediment dynamics vary within the SDMA due to the changing channel conditions, and were analyzed separately in Areas A, B, and C. These areas are illustrated in Figure 7-1. Area A is downstream of the CNR Bridge where the channel is highly constrained by the dock walls, and high velocities and shear stresses will cause substantial scour during the regulatory flood event. Within this area, the sediment transport model allowed to scour to elevation 69.0, based on the design plans for this section. The model results show that at the peak of the regulatory flood event, this area is predicted to scour to that elevation across the majority of the width. There is some variability on the west bank, as shown in Figure 6-5. The results across the majority of the channel show no variability with the sensitivity runs, leading to an increased confidence in the scour predictions in this channel section.

In SDMA Area B the center of the channel still has the potential to scour to elevation 69.0, but as the channel widens there is deposition near the walls, as shown in Figure 6-7. In addition, the model results show significant variability in the amount of deposition that could occur behind the bridge pier for the future Gardiner Ramps.

In SDMA Area C, as the primary flow path moves from the east bank, there is further deposition predicted in that area. There is some variability in the deposition predictions along the east bank within the sensitivity runs. The center of the channel still has the potential to scour to elevation 69.0, but as the channel widens there is deposition near the walls, as shown in Figure 6-9. The deposition in this area is also impacted by the upstream bridge pier for the future Gardiner Ramps.

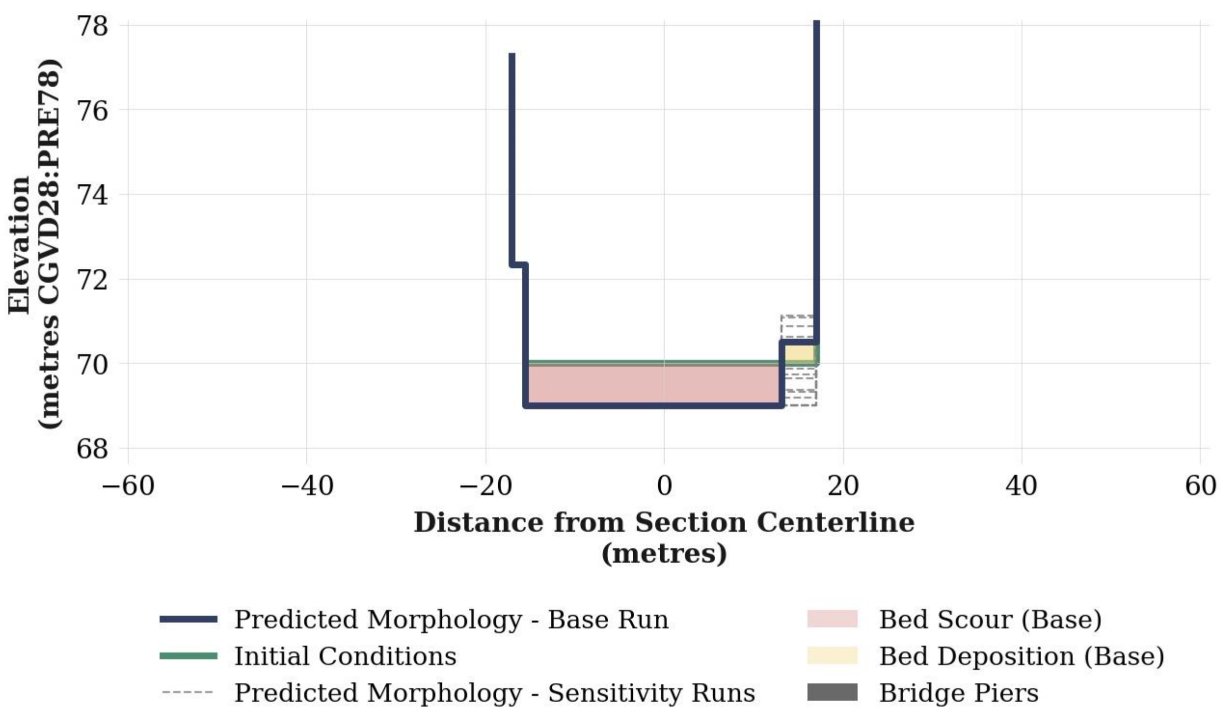


Figure 6-5: Predicted Bed Morphology Cross Section – SDMA Area A, SDMA Design Initial Conditions, looking downstream

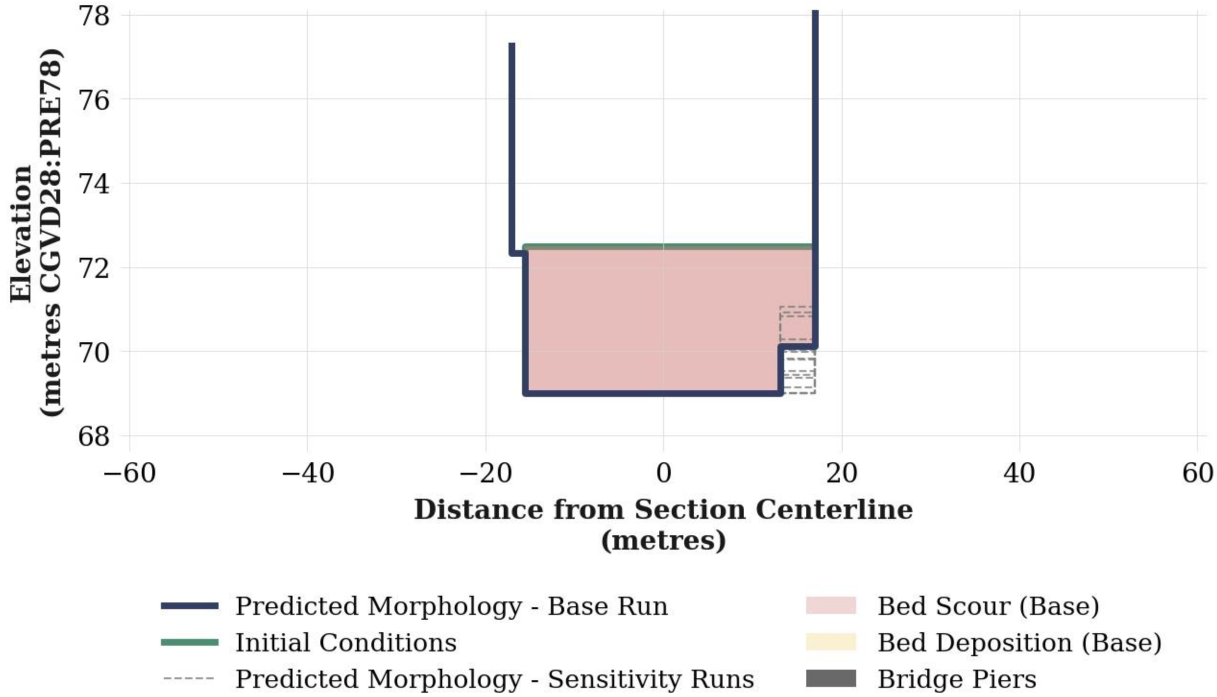


Figure 6-6: Predicted Bed Morphology Cross Section – SDMA Area A, SDMA Full Initial Conditions, looking downstream

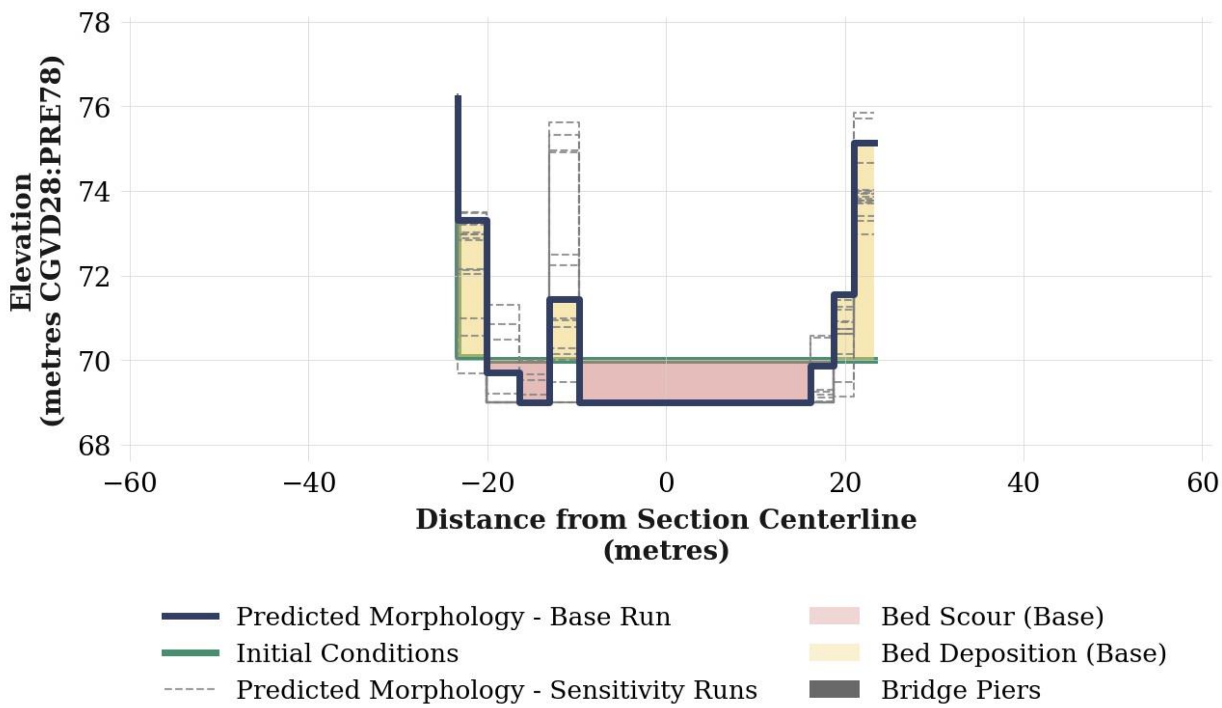


Figure 6-7: Predicted Bed Morphology Cross Section – SDMA Area B, SDMA Design Initial Conditions, looking downstream

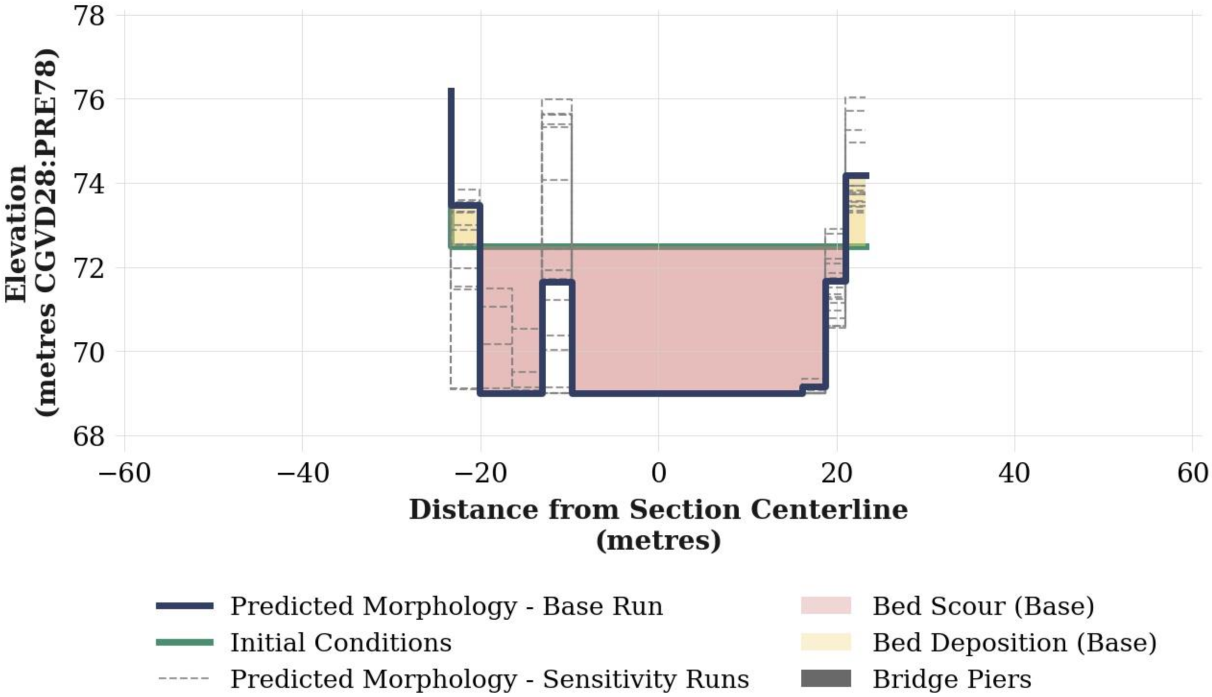


Figure 6-8: Predicted Bed Morphology Cross Section – SDMA Area B, SDMA Full Initial Conditions, looking downstream

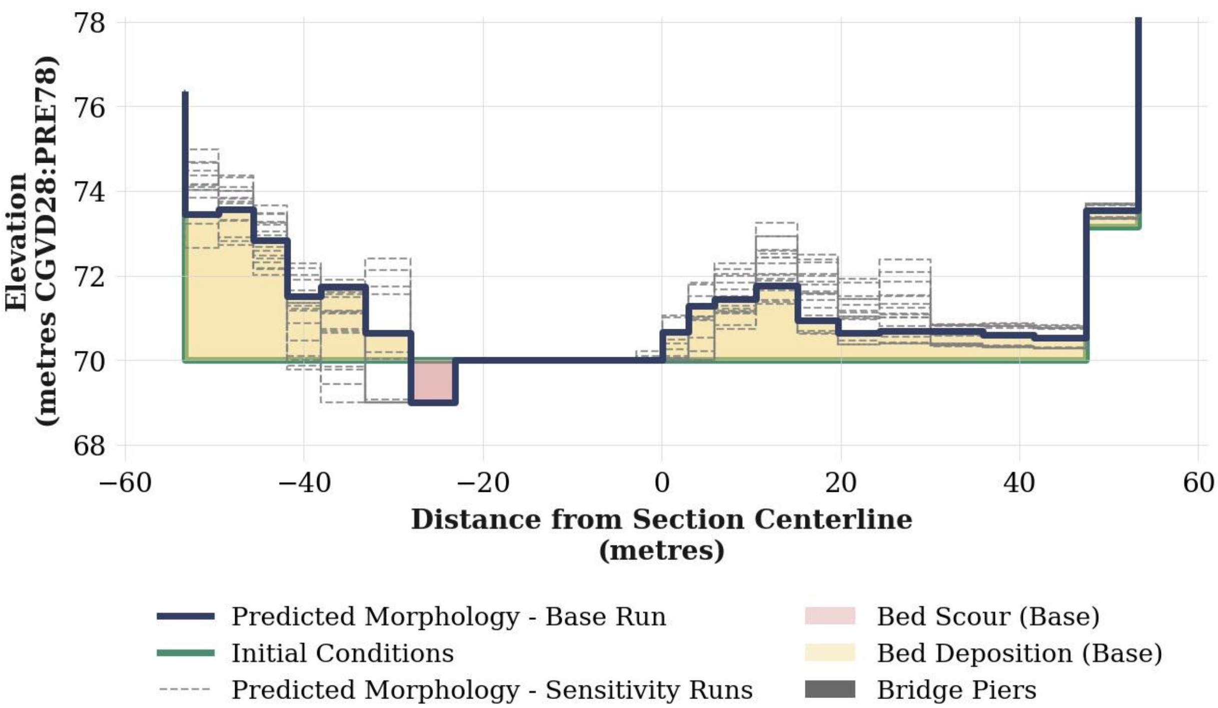


Figure 6-9: Predicted Bed Morphology Cross Section – SDMA Area C, SDMA Design Elevation Initial Conditions, looking downstream

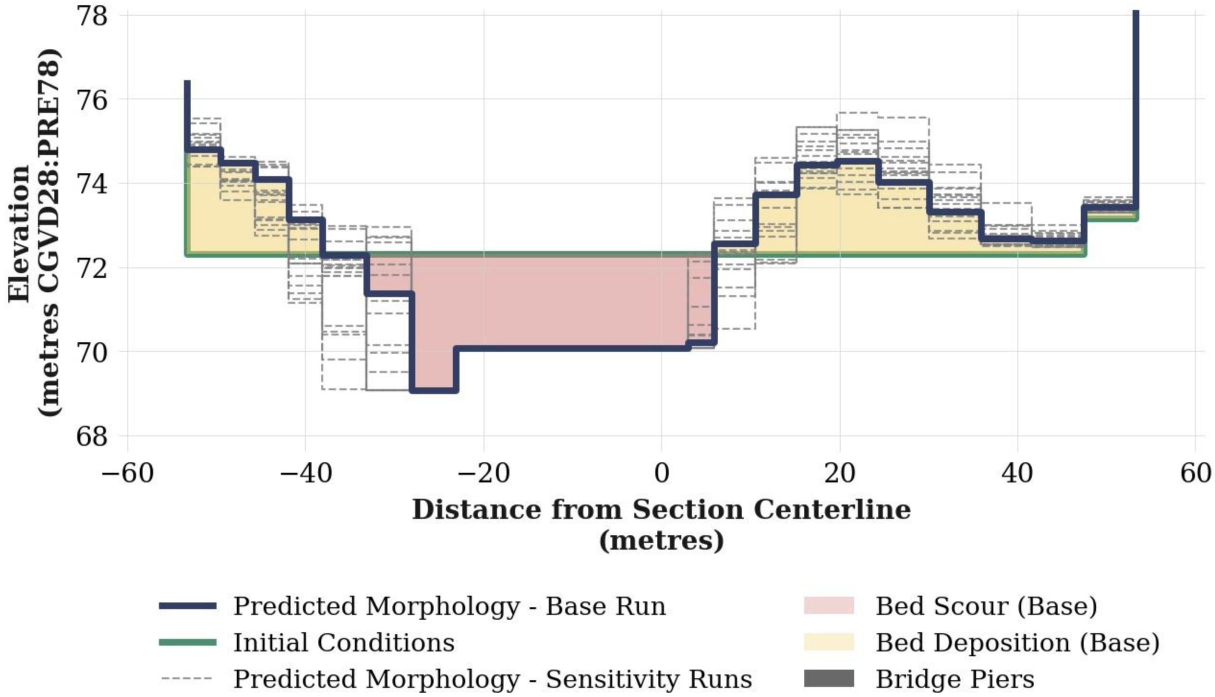


Figure 6-10: Predicted Bed Morphology Cross Section – SDMA Area C, SDMA Full Elevation Initial Conditions, looking downstream

6.1.3 Lake Shore Blvd. Bridge

Of the four bays at the Lake Shore Blvd. Bridge crossing, the western two bays would experience different sediment dynamics than the eastern two bays during the regulatory flood event in ways that influence flood levels. Portions of the western bays that discharge to the Keating Channel (Bays 3 and 4) will scour to elevation 69.0, which is the maximum amount of scour allowed in the model. Due to the hydrodynamics at this location, there may be less scour, or possibly event deposition at the eastern edge of these bays due to the wake in the flows created by the bridge piers and the flow changing directions as it passes through these bays.

In the eastern 2 bays, the reduction of velocities due to the flow split between the River Valley and the Keating Channel allows for deposition of sediment in these bays, and in the Ice Management Area immediately downstream.

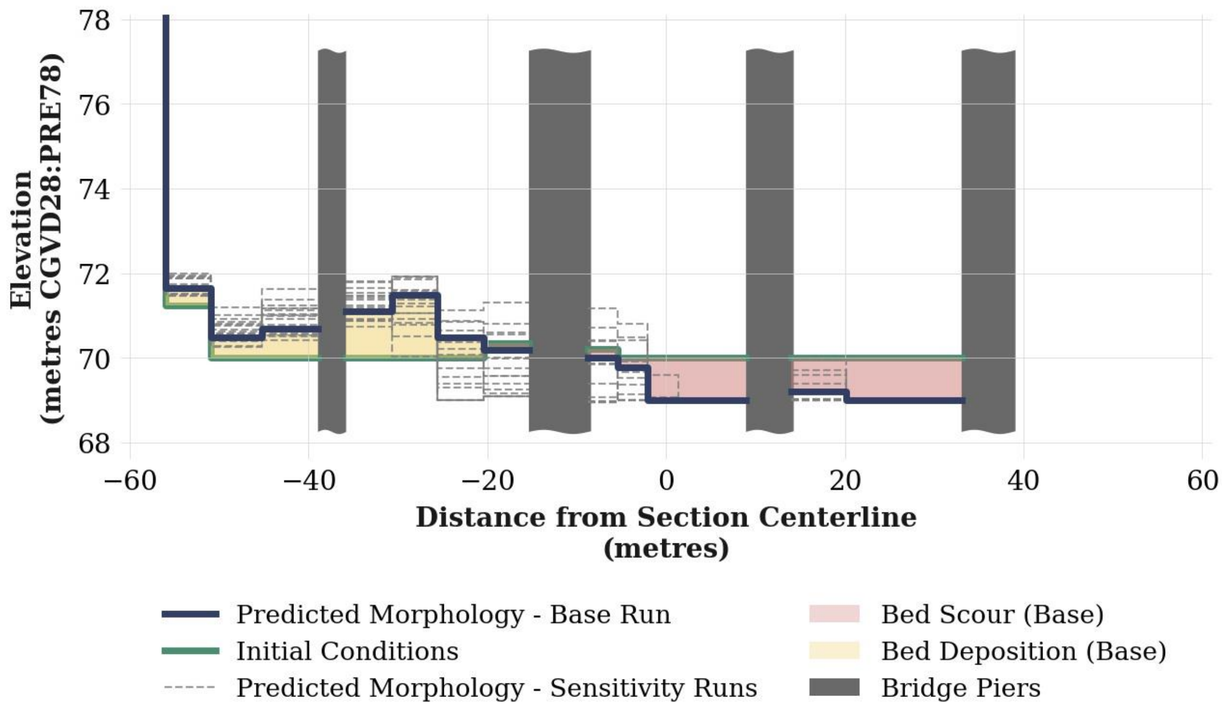


Figure 6-11: Predicted Bed Morphology Cross Section – Lake Shore Blvd. Bridge, SDMA Design Conditions, looking downstream

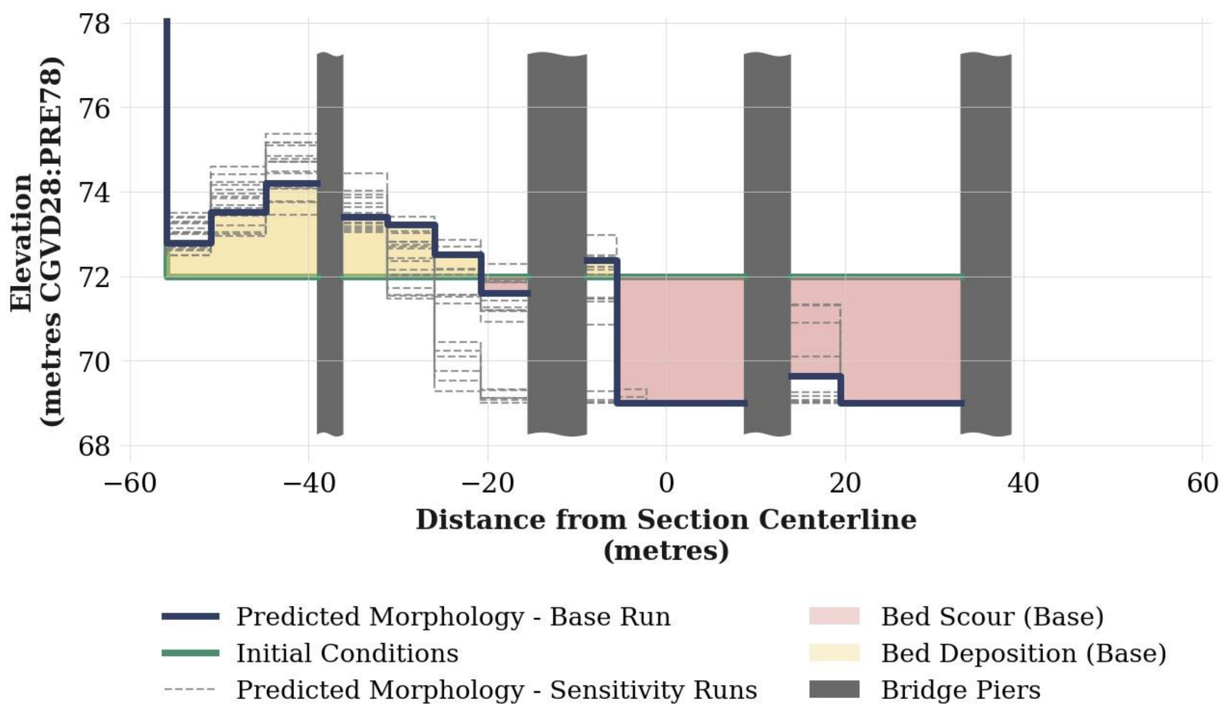


Figure 6-12: Predicted Bed Morphology Cross Section – Lake Shore Blvd. Bridge, SDMA Full Initial Conditions, looking downstream

6.1.4 Keating Channel

The flow in the upper Keating Channel immediately downstream of the Lake Shore Blvd. Bridge is concentrated along the southern bank, due to the sudden change in flow direction. This causes an area with low flow velocities that becomes depositional along the north bank, which can be seen in Figure 6-13. There is variability in the amount of sediment deposition in the sensitivity runs, with increasing loads and decreased erosion rates allowing for additional deposition in this area above the base model run, and decreased upstream loads and increased erosion rates showing decreasing deposition in this area.

Further downstream in the upper Keating Channel, the flows distribute more evenly across the channel, and there is less concentrated deposition. This can be seen in figure Figure 6-15, which is approximately 300 metres downstream of the Lake Shore Blvd. Bridge.

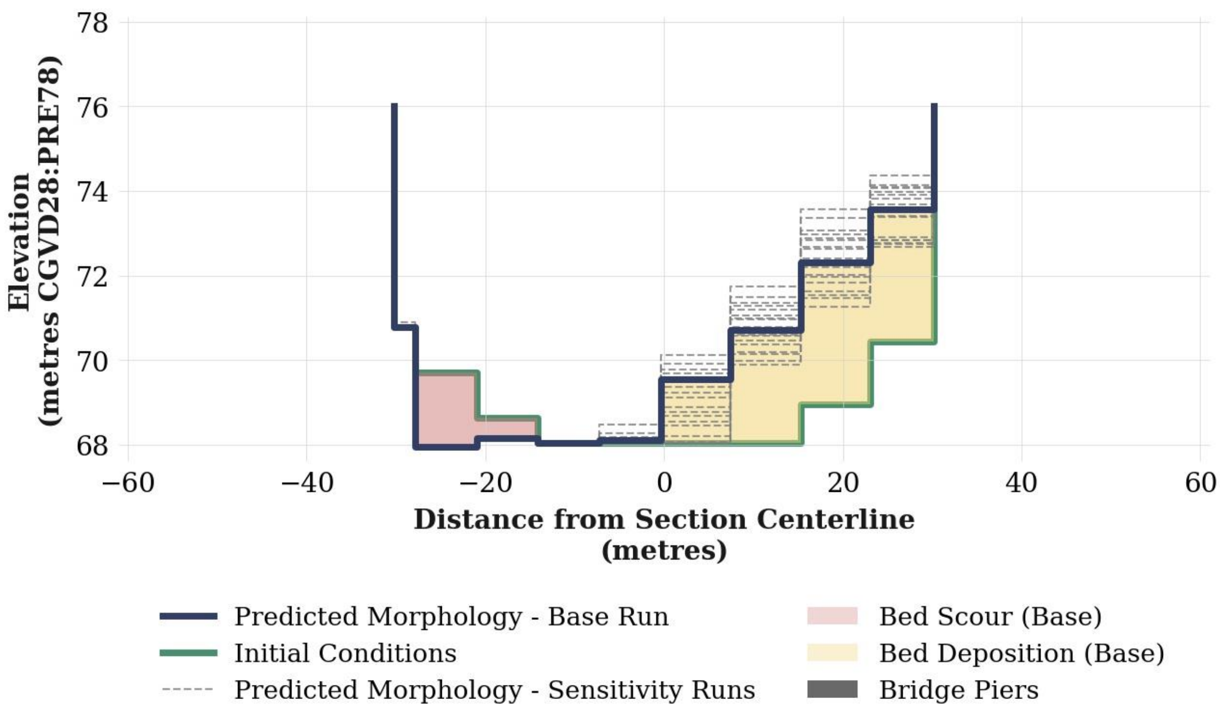


Figure 6-13: Predicted Bed Morphology Cross Section – Upper Keating Channel 85 metres downstream of Lake Shore Blvd. Bridge, SDMA Design Conditions, looking downstream

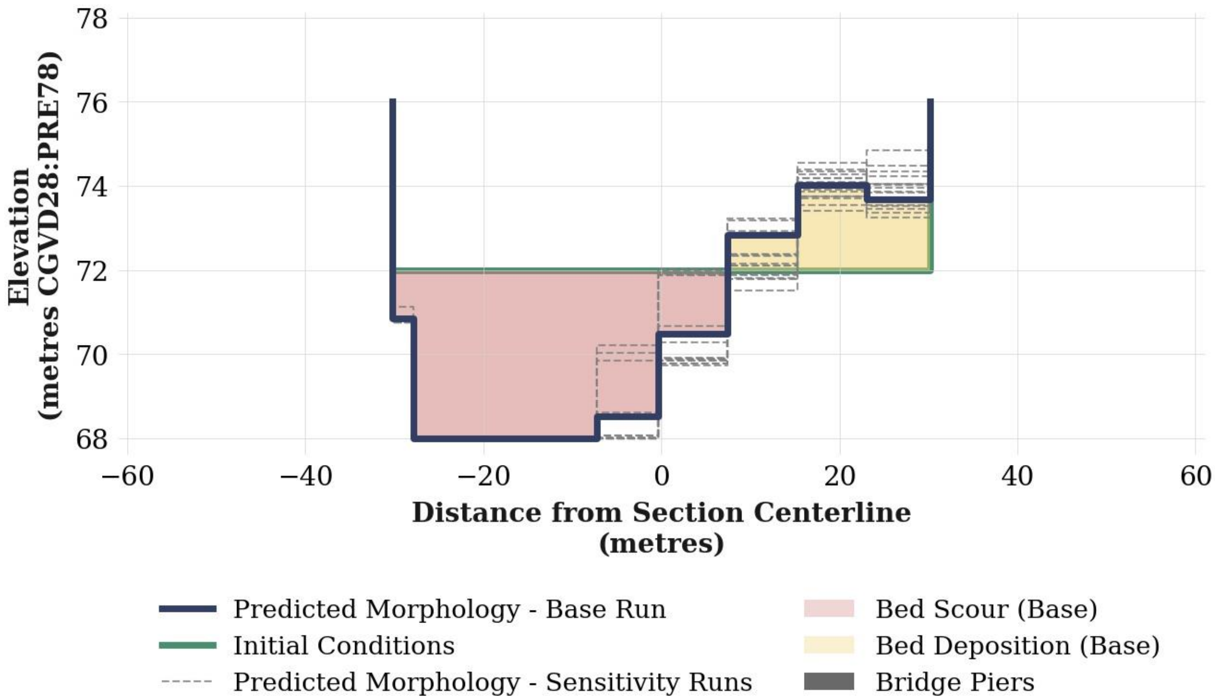


Figure 6-14: Predicted Bed Morphology Cross Section – Upper Keating Channel 85 metres downstream of Lake Shore Blvd. Bridge, SDMA Full Conditions, looking downstream

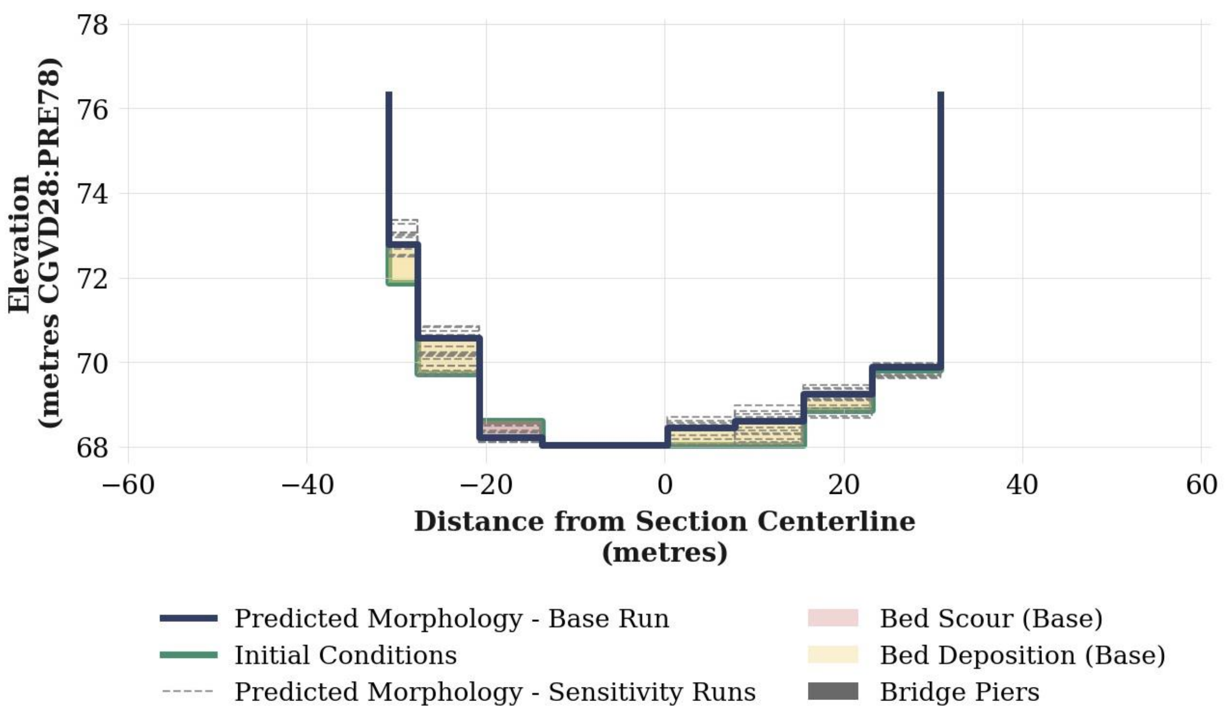


Figure 6-15: Predicted Bed Morphology Cross Section – Upper Keating Channel 300 metres downstream of Lake Shore Blvd. Bridge, looking downstream

The net deposition/scour for the Upper Keating Channel was analyzed for all of the sediment transport sensitivity model runs, and the results are shown in Figure 6-16. The net deposition/scour in the Upper Keating Channel for all design conditions model runs ranged from 3,200 cubic metres of deposition to 16,000 cubic metres, with an average of 8,800 cubic metres for all model runs. This deposition was primarily along the north side of the channel, in the ineffective flow area, so there was a minimal impact on flow conveyance.

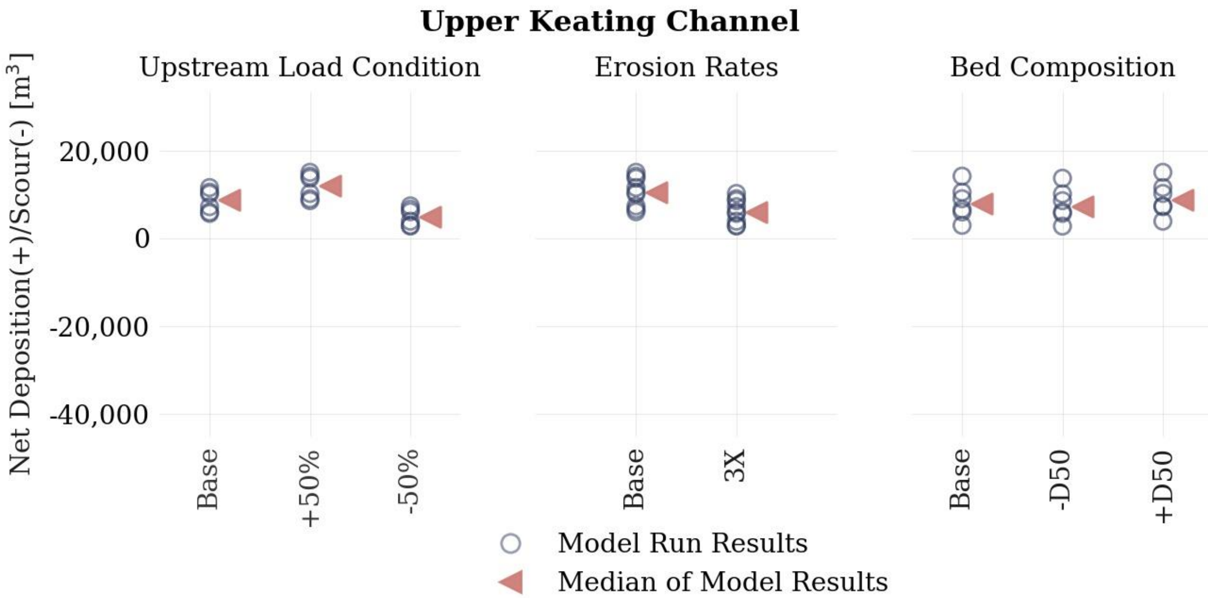


Figure 6-16: Upper Keating Channel Net Scour/Deposition Sensitivity Analysis, SDMA Design Initial Conditions

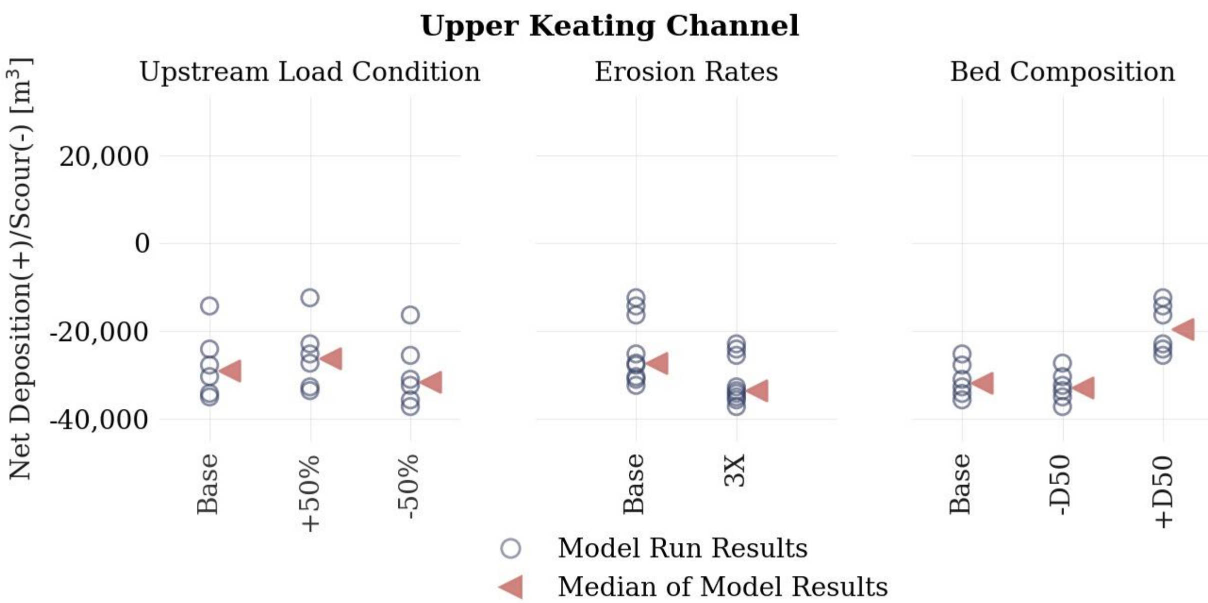


Figure 6-17: Upper Keating Channel Net Scour/Deposition Sensitivity Analysis, SDMA Full Initial Conditions

In the narrow portion of the Keating Channel, the flow is constricted, which significantly increases the velocities and shear stresses. In this portion of the model, the scour is limited at elevation 68.0. The construction dredging limits within the Keating Channel are at elevation 68.4 in the center of the channel. The exception to this is in the location of the existing pit slab for the Cherry Street bridge that will remain. This has an elevation of 69.8 metres, and was modeled as hardpan (no scour allowed). The initial conditions used in the model assumed that there are areas within 10 metres of the north and south banks of the channel where the maintenance dredging will be performed to elevation 69.6.

The limitation of the scour to elevation 68.0 in this portion of the model is likely a conservative assumption, since the existing sediment characterization near this area shows that the top several meters of sediment are loose silts with high organic content, which would be easily erodible under the velocities/shear stresses encountered during the regulatory flood event.

The sediment transport model results show that across the majority of the channel, the maximum scour depth was reached, with no variability in the sensitivity runs. Figure 6-18 and Figure 6-19 show the predicted bed morphology at the peak of the regulatory flood event. These sections are located upstream and downstream of the pit slab location, and along the southern bank, there is some variability in the scour predicted in the cells adjacent to the dock walls.

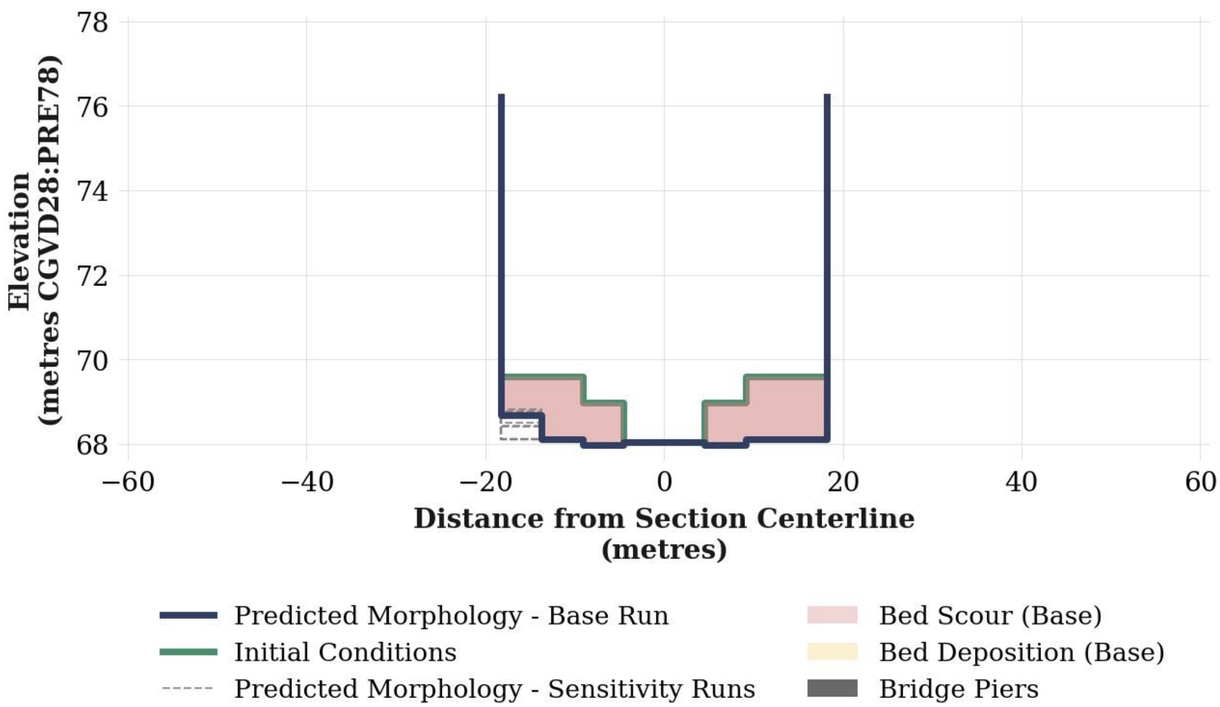


Figure 6-18: Predicted Bed Morphology Cross Section – Keating Channel Narrows 45 metres downstream of constriction, looking downstream

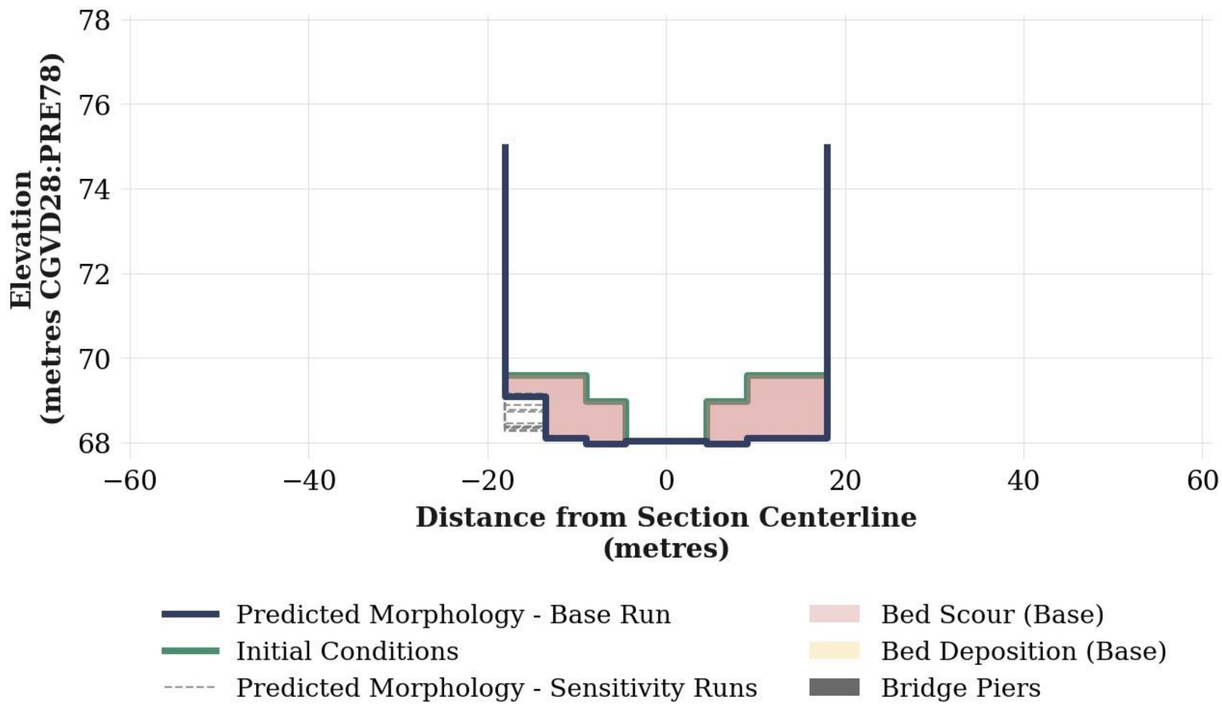


Figure 6-19: Predicted Bed Morphology Cross Section – Keating Channel Narrows 300 metres downstream of constriction, looking downstream

The sensitivity of the predicted scour in the southern portion of the Keating Channel Narrows was analyzed for all of the sediment transport sensitivity model runs, and the results are shown in Figure 6-20. There is very little variability in the predicted area-averaged bed elevation change based on the upstream load condition and the sediment bed particle sizes. There is some variability based on the erosion rates used in the model.

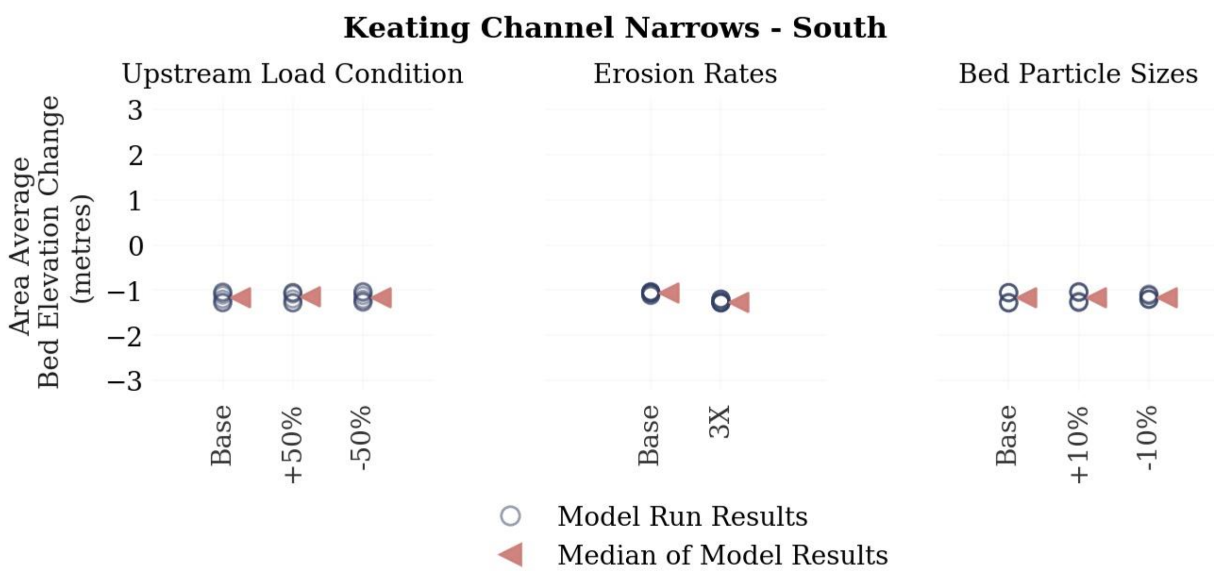


Figure 6-20: Area Average Bed Elevation Change Sensitivity Analysis – Keating Channel Narrows South

Based on the model results showing the scour in the portion of the channel going to the maximum scour allowed in the model over the majority of the channel, which is set at a conservative elevation based on the in-situ sediments adjacent to this portion of the channel, it is recommended that a scour assumption of a uniform bottom elevation of 68.0 metres in the Keating Channel Narrows be used in future hydrodynamic modeling. The exception to this would be the pit slab location, which should be modeled at its existing elevation of 69.8 metres.

6.2 Hydrodynamic Model Results

The morphology from the sediment transport runs was used to develop bathymetry for the river channel and floodplains, which was passed forward to the MIKE21 hydrodynamic model to simulate the hydraulics at the peak of the regulatory flood event. The only change in the bathymetry that was not brought forward was any scour in the Don Narrows upstream of the CNR bridge above 1.5 metres. This was a conservative assumption to limit the amount of scour only in the hydrodynamic runs, since allowing the sediment transport runs to scour to 3 metres supplies a larger amount of material to the PLFP site that can settle out.

Of all locations within the project areas, the Lake Shore Bridge ultimately was the most vulnerable to flooding, in part because of the wide cross-section at this location which tends to increase flood levels locally. For the four-bay scenario, there is more than 20 centimetres of freeboard within each of the Lake Shore Bridge bays. Figure 6-21 and Figure 6-22 illustrates the range of predicted flood levels at the LSB cross-section for the Full Vision design scenario, with the SDMA at design conditions, and full, respectively. Figure 6-23 and Figure 6-24 illustrates similar model results for the Interim Conditions scenarios. Each colored line represents model output from a separate scenario, and the dashed black line represents the low chord elevation of the bridge.

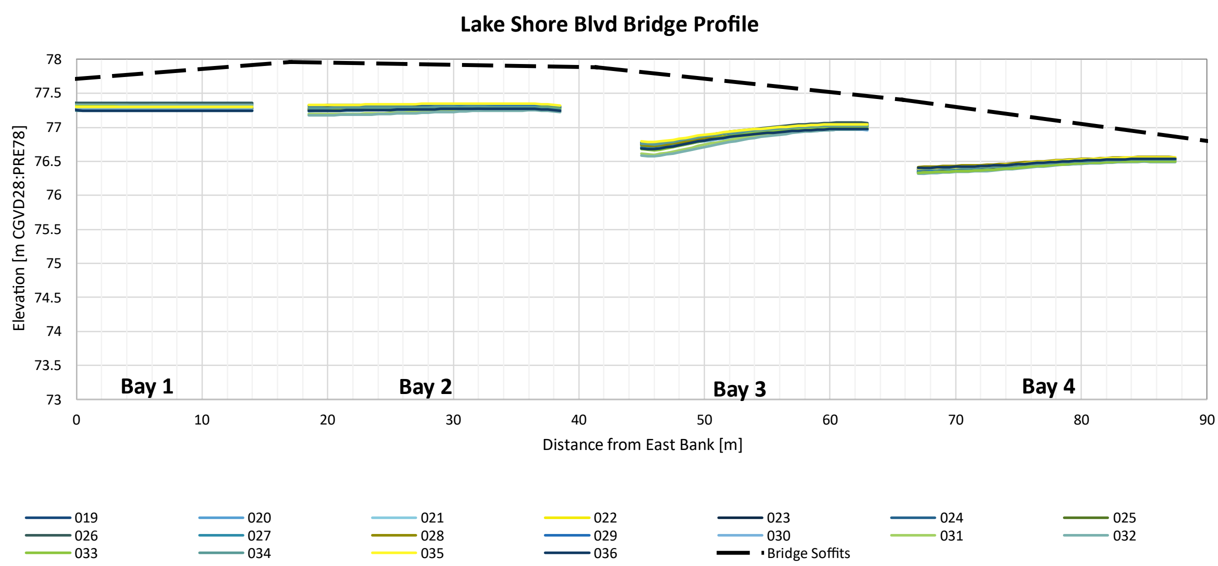


Figure 6-21: Predicted Flood Levels at Lake Shore Boulevard Bridge, Full Vision Design Scenario, SDMA Design Conditions, looking downstream

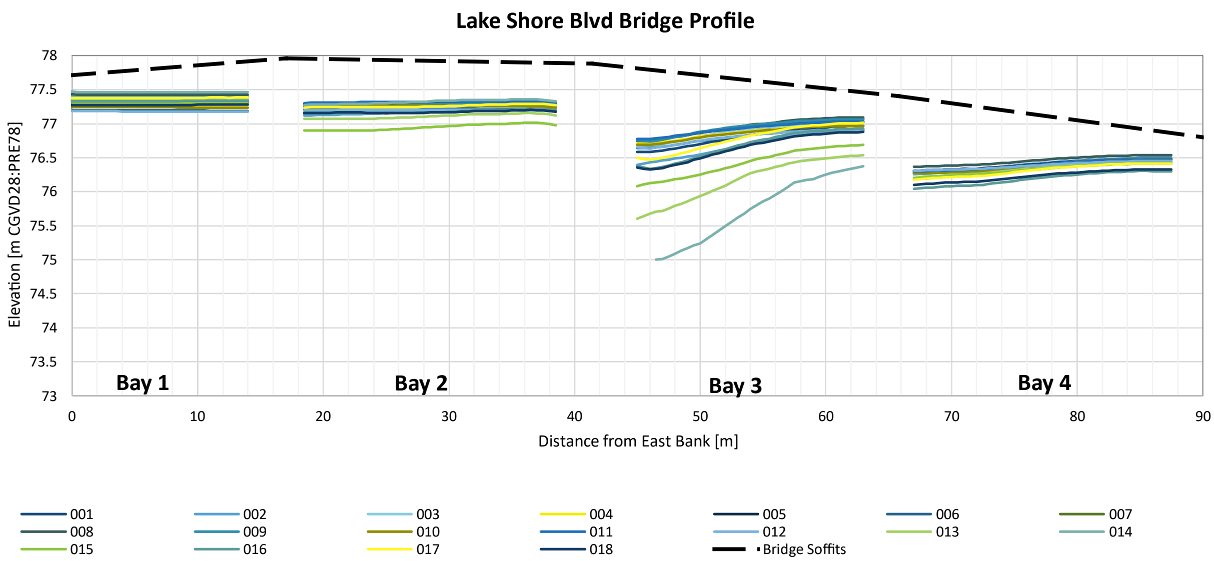


Figure 6-22: Predicted Flood Levels at Lake Shore Boulevard Bridge, Full Vision Design Scenario, SDMA Full Conditions, looking downstream

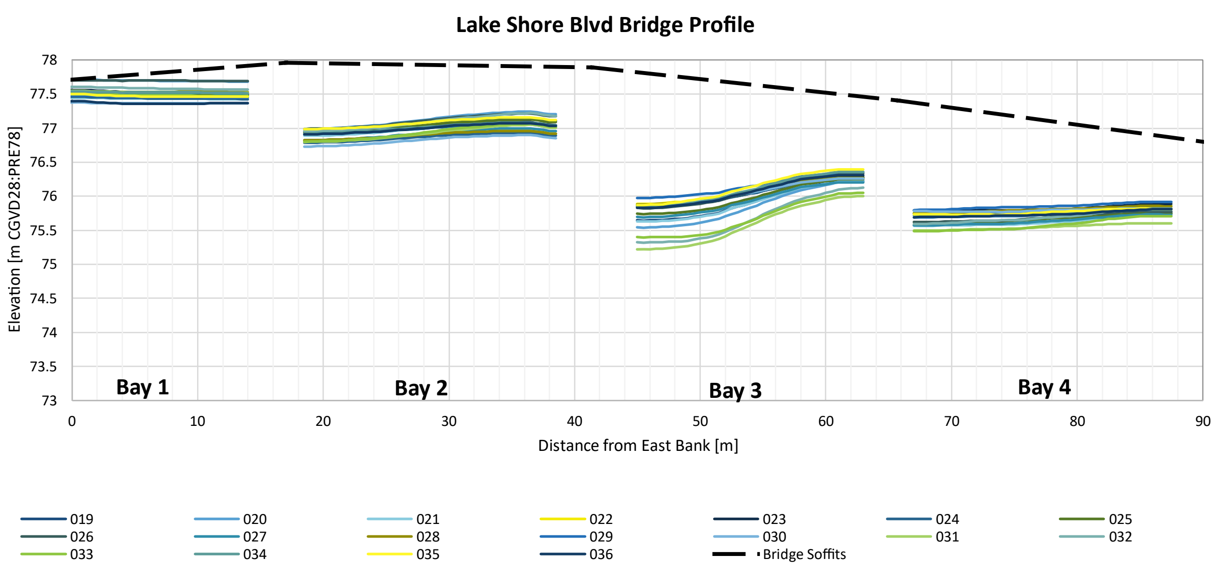


Figure 6-23: Predicted Flood Levels at Lake Shore Boulevard Bridge, Interim Conditions Scenario, SDMA Design Conditions, looking downstream

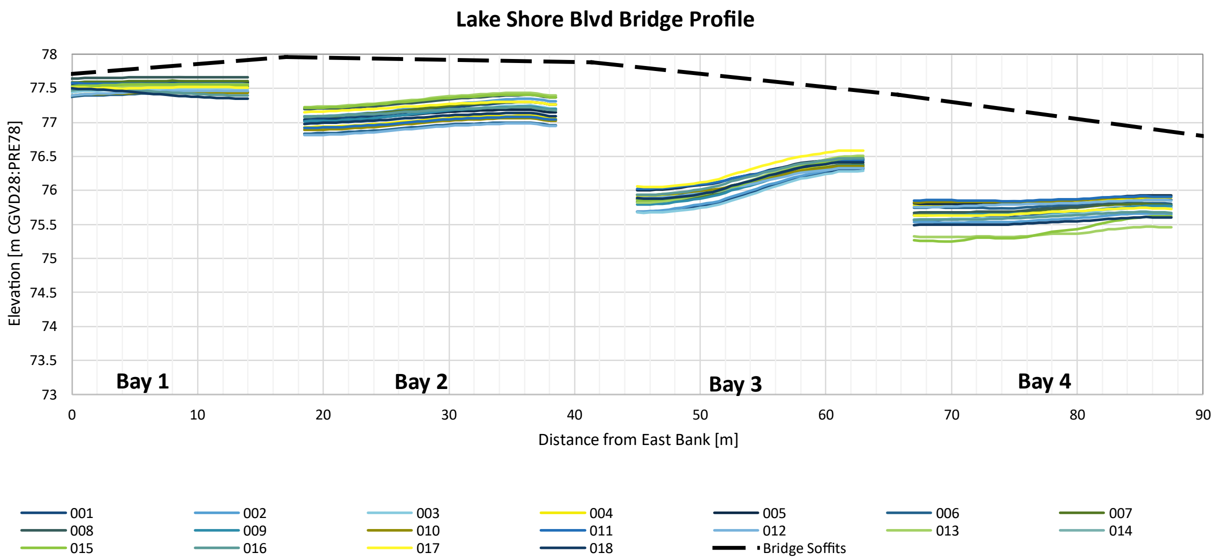


Figure 6-24: Predicted Flood Levels at Lake Shore Boulevard Bridge, Interim Conditions Scenario, SDMA Full Conditions, looking downstream

Predicted flood elevations at the four floodplain landform locations are summarized in Table 6-4. Maps of the sediment transport model results are included for three of the eighteen simulations Appendix A. For all tested model conditions for the Full Vision SDMA Full, Full Vision SDMA Design, and Interim Conditions SDMA Design scenarios, the models show that the PLFP design coveys the flood within the flood protection measures for the regulatory event. There were four events under the Interim Conditions SDMA Full scenarios where there was sediment deposited on the Don Roadway that caused shallow flooding down Lake Shore Boulevard. These events were associated with the scenarios where the load upstream was increased, and the three scenarios where the sediment mode was decrease, which increased the fraction of coarse particles in the sediment cores.

Table 6-4: Full Vision with Initial Bed Condition at "Full" Predicted Flood Levels and Freeboard at Floodplain Landforms

Simulation Short ID	Water Surface Elevations (metres)		Minimum Freeboard (centimetres)	
	First Gulf	West Don	First Gulf	West Don
ST001	78.5	79.7	100.5	37.4
ST002	78.5	79.7	98.5	38.6
ST003	78.5	79.7	97.7	32.3
ST004	78.5	79.7	97.8	33.4
ST005	78.5	79.7	101.3	36.7
ST006	78.5	79.8	104.3	27.5
ST007	78.5	79.7	99.4	37.2
ST008	78.5	79.7	98.9	39.0
ST009	78.5	79.7	98.0	32.3
ST010	78.5	79.7	98.2	34.0
ST011	78.5	79.7	100.8	36.4
ST012	78.8	79.8	70.9	27.2
ST013	78.5	79.7	98.7	37.3
ST014	78.5	79.7	97.5	38.1
ST015	78.5	79.7	98.9	32.5
ST016	78.5	79.7	100.3	34.3
ST017	78.5	79.7	101.0	36.5
ST018	78.5	79.8	104.5	28.1

Table 6-5: Interim Condition with Initial Bed Condition at "Full" Predicted Flood Levels and Freeboard at Floodplain Landforms

Simulation Short ID	Water Surface Elevations (metres)		Minimum Freeboard (centimetres)	
	First Gulf	West Don	First Gulf	West Don
ST001	78.6	79.6	90.2	40.8
ST002	78.6	79.6	90.8	42.1
ST003	78.6	79.6	88.7	43.0
ST004	78.5	79.7	99.9	35.2
ST005	78.5	79.7	103.2	37.0
ST006	78.4	79.8	107.7	28.8
ST007	78.6	79.6	92.1	40.0
ST008	78.6	79.6	92.3	42.4
ST009	78.6	79.6	92.3	40.2
ST010	78.5	79.7	100.6	35.2

ST011	78.4	79.7	111.8	30.9
ST012	78.4	79.7	107.6	29.6
ST013	78.7	79.6	81.9	43.0
ST014	79.0	79.8	49.3	28.0
ST015	79.0	79.7	48.0	36.4
ST016	78.6	79.6	90.5	42.2
ST017	78.6	79.6	91.8	43.9
ST018	78.4	79.8	106.7	27.9

The sediment transport model results for the Interim Conditions were evaluated to test the impact of the area where the existing sediments are proposed to remain in place to support the existing dock walls on the east bank under the ramp from the Don Valley Parkway to the Westbound Gardiner highway. The hydrodynamic model results show that this causes high shear stresses in this area. The sediment transport model results show that this area remains net depositional until the 350-Year storm event is exceeded. Then existing sediment in this areas is eroded to increase the channel cross sections and prevent the development of the supercritical flow transition and hydraulic jump. Sediment transport modeling of the interim conditions design shows that this sloped area of existing sediments is net depositional for flows up to 730 cubic metres per second, which is in excess of the peak flow for the 350-Year recurrence storm (650 cubic metres per second), but that the sediment scours to the adjacent channel bottom elevation before the peak of the regulatory flood event. Figure 6-25 shows the simulated bed delta for the existing sediments in this area through the duration of the modeling to the peak of the regulatory flood event. This illustrates the range of conditions for the sensitivity runs, where the bed scour was constrained to elevation 69.0 metres, as well as a single run using the base parameterization where the bed was allowed to scour to the average bedrock elevation in the SDMA (66.5 meters).

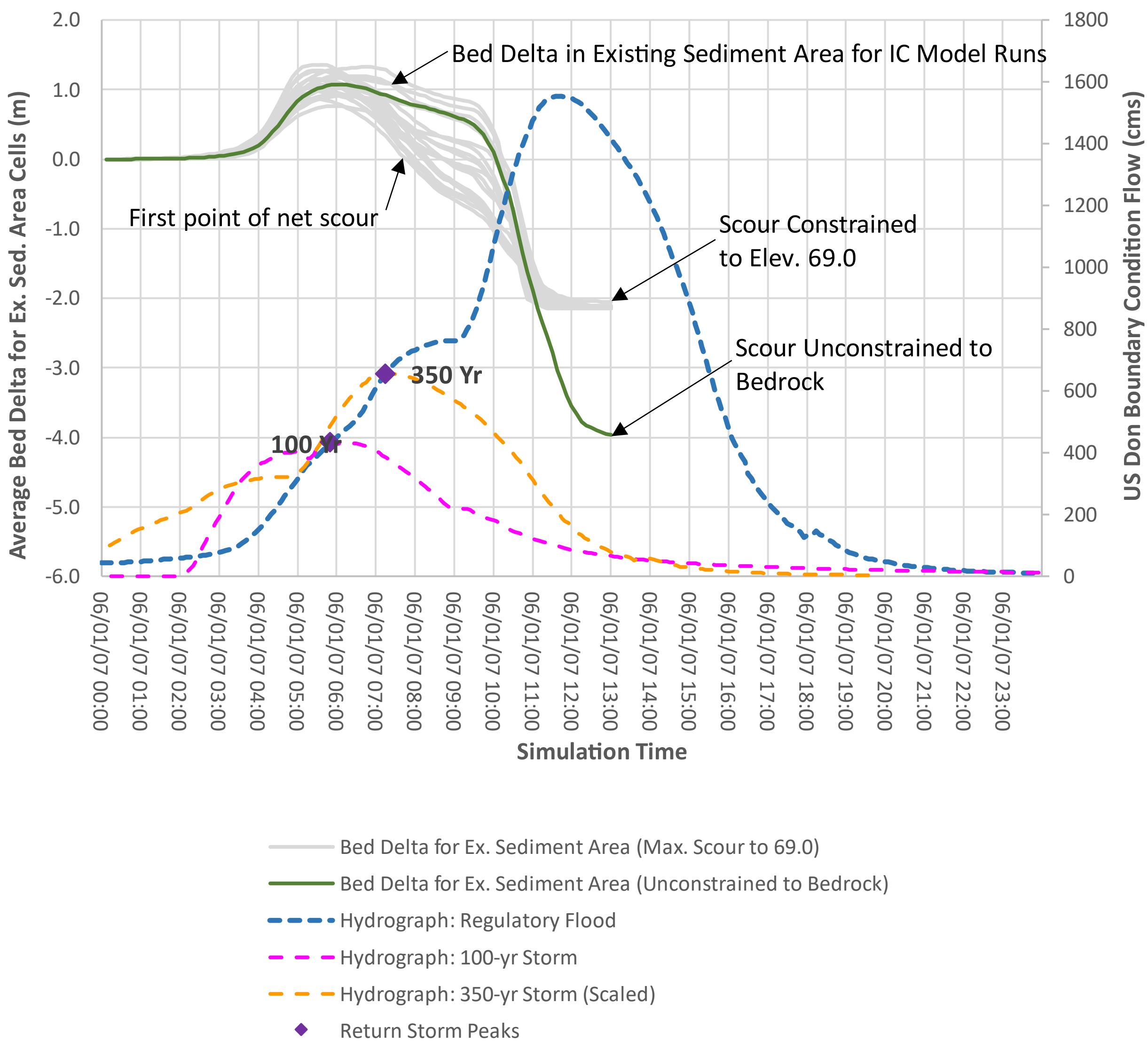


Figure 6-25. Bed Delta in Existing Sediment Area and Inflow Hydrograph for Interim Conditions Regulatory Model Runs

7 Model Application: Sediment Maintenance

The sediment transport model was applied to estimate long-term sediment dredging requirements in the SDMA and Keating Channel to sustain design conditions, reduce sediment deposition in the Keating Channel and Toronto Harbour, and reduce flood risk. Areas in which sediment deposition was summarized are illustrated in Figure 7-1 below.

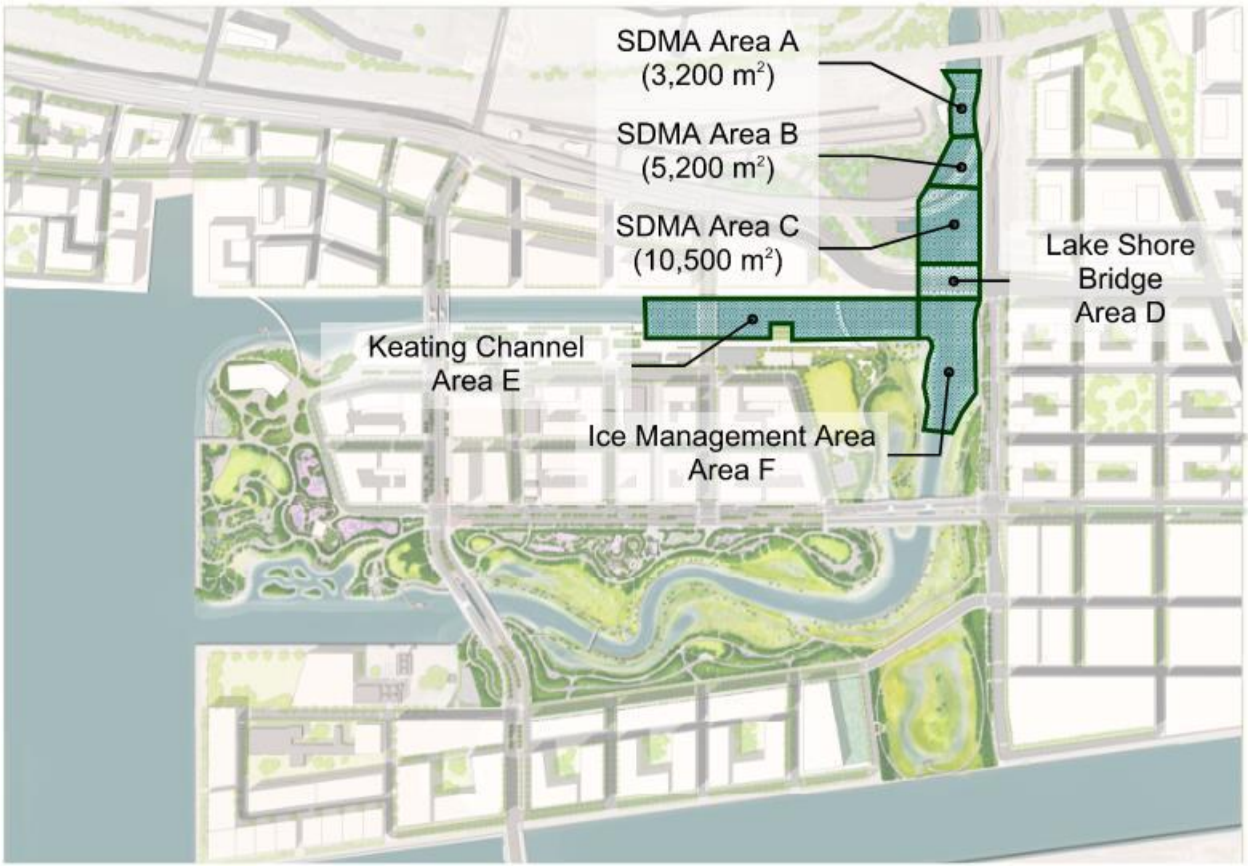


Figure 7-1: Areas for Summaries of Long-term Sediment Deposition

The degree of detail in the sediment transport model precludes the simulation of multi-year periods, so an approximation method for estimating long-term sedimentation was developed and validated. This method relates observed peak flow condition to predicted trap efficiency of watershed solids in each region of the project area, including each zone of the three SDMA zones and the Keating Channel. Simulations were conducted for seven synthetic hydrographs with peak flows of 20, 40, 80, 120, 160, 200, and 240 cubic metres per second. Modeled trap efficiency was computed in each zone as the total sediment volume trapped divided by the total upstream solids load. An example of such a relationship is included in Figure 7-2 below.

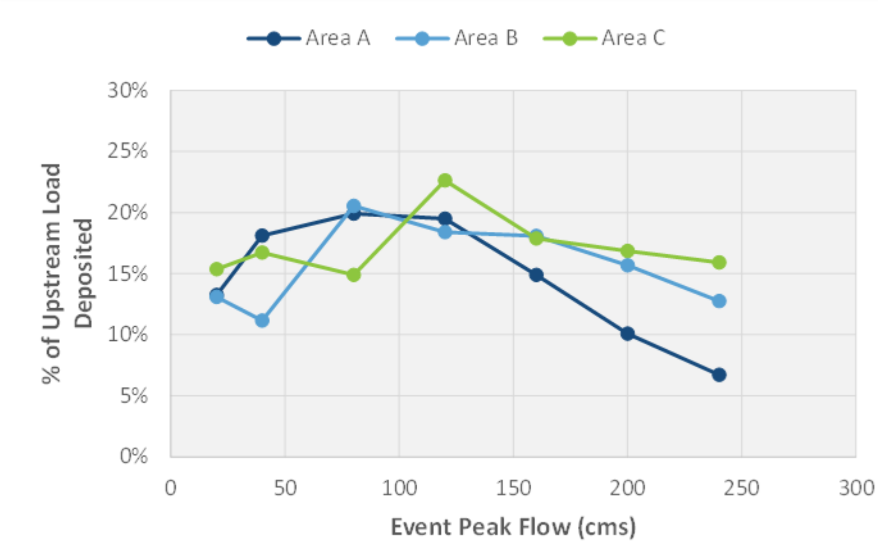


Figure 7-2: Predicted Relationship Between Event Peak Flow Rate and Trap Efficiency in the SDMA

Total sediment deposition in each zone per storm event was estimated by computing the total watershed sediment load per storm event and applying the modeled relationship between event peak flow and trap efficiency. For example, if the total sediment load for a single event with peak flow of 86 cubic metres per second was 10,000 kilograms and the modeled trap efficiency for an event with a peak flow of 80 cubic metres per second was 10% in the Keating Channel, deposition in the Keating Channel would be predicted as just over 1,000 kilograms for that event. Deposition during dry weather periods between storm events was estimated similarly.

Within each region of the project area, deposition was computed for a recent 18-year hydrologic period (2/23/2000 through 11/19/2017) and annualized sedimentation volumes were summarized. Figure 7-3 and Figure 7-4 illustrate annual average sedimentation volumes in each zone.

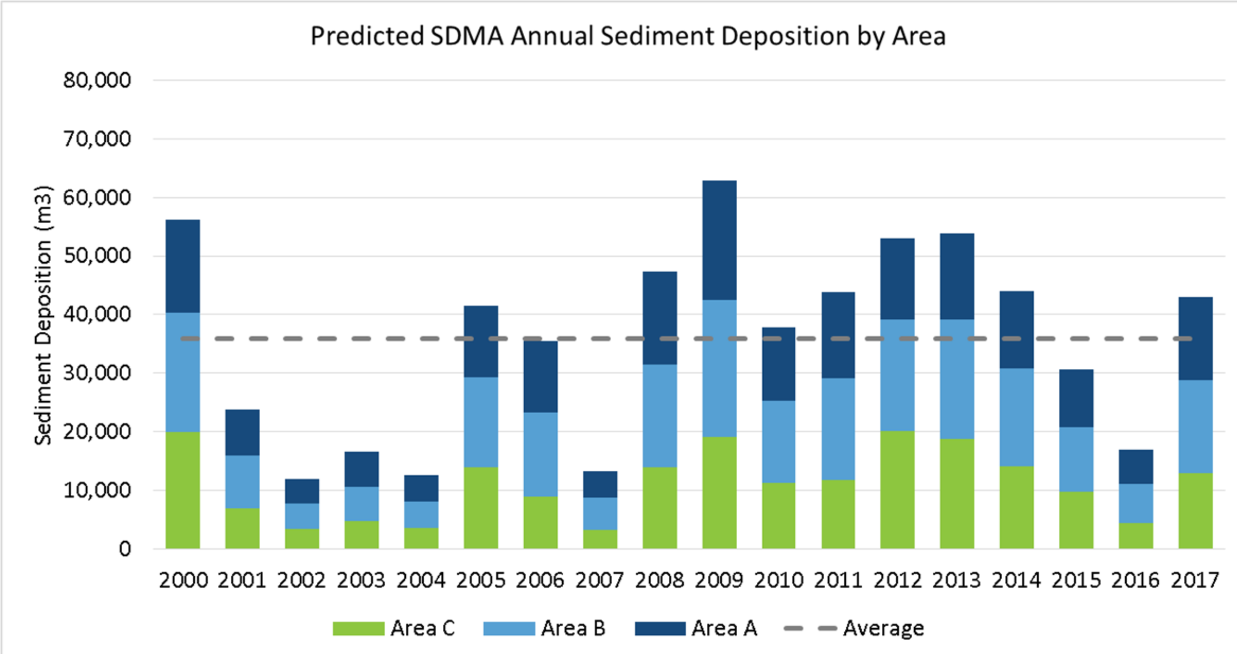


Figure 7-3: Full Vision Predicted Annual Sediment Deposition by Area: SDMA

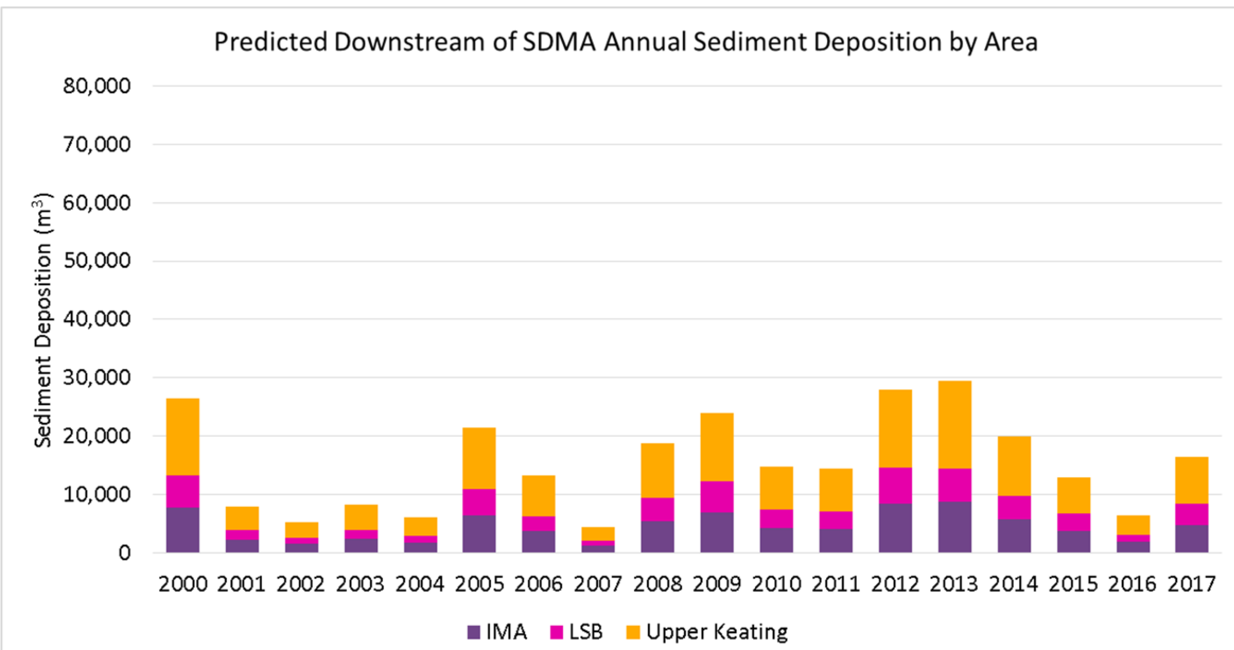


Figure 7-4: Full Vision Predicted Annual Sediment Deposition by Area: Downstream of SDMA

Dredge frequency requirements in the SDMA and Keating Channel were identified by establishing a critical threshold for dredging to occur based on regulatory event sediment transport modeling. In the SDMA, this threshold was established as a sediment bed elevation of 72.5 metres in sub-areas A and B, and a transition from 72.5 metres to 72 metres through Area C. This degree of sedimentation above the design elevation of 70 metres in these zones equals 34,000 cubic metres of sediment. Based on regulatory event sediment transport modeling, increases in sedimentation above this threshold would result in increased flood risk at the Lake Shore Boulevard Bridge.

In the Keating Channel, this threshold was established at a sediment bed elevation of 72 metres at the upstream end of the Keating Channel and would transition from 72 to the design condition within the broader, upstream section of the Keating Channel. This degree of sedimentation above the design elevation in the Keating Channel equals 60,000 cubic metres of sediment. Based on regulatory event sediment transport modeling, increases in sedimentation above this threshold would result in increased flood risk at the Lake Shore Boulevard Bridge.

Based on these assumed thresholds to initiate sediment maintenance, and the analysis of long-term sediment deposition, dredge frequencies were estimated as described in Table 7-1 for the Full Vision, and Table 7-2 for the Interim Conditions.

Table 7-1: Full Vision Predicted Dredge Frequency By Area for Reduction of Flood Risk

Area	Effective Sediment Volume Capacity (m^3)	Minimum Annual Sedimentation Volumes (m^3)	Average Annual Sedimentation Volumes (m^3)	Maximum Annual Sedimentation Volumes (m^3)	Dredging Frequency
SDMA Area A (Max. Elev. 72.5)	7,000	4,200	11,300	20,300	1-3x/year
SDMA Area B (Max. Elev. 72.5)	9,400	4,300	13,400	23,400	1-2x/year
SDMA Area C (Max. Elev. Slope from 72.5 at north end to 72.0 at LSB)	16,500	3,200	11,200	20,100	1-2x/year
SDMA Total	32,900	11,700	35,900	63,800	-
Lake Shore Blvd Bridge (Max. Elev. 72.0)	6,200	800	3,200	6,100	Every 2-3 years
Ice Management Area (Max. Elev. 72.0) [Area F]	9,200	1,300	4,500	8,800	Every 2-3 years
Upstream Section of Keating Channel (Max. Elev. 72.0) [Area E]	53,200	2,400	7,700	15,100	Every 4-5 years

Table 7-2: Interim Conditions Predicted Dredge Frequency By Area for Reduction of Flood Risk

Area	Effective Sediment Volume Capacity (m^3)	Minimum Annual Sedimentation Volumes (m^3)	Average Annual Sedimentation Volumes (m^3)	Maximum Annual Sedimentation Volumes (m^3)	Dredging Frequency
SDMA Area A (Max. Elev. 72.5)	6,600	4,000	10,800	19,300	1-3x/year
SDMA Area B (Max. Elev. 72.5)	8,400	4,300	7,200	12,400	1-2x/year
SDMA Area C	9,100	3,200	6,700	11,500	1-2x/year

(Max. Elev. Slope from 72.5 at north end to 72.0 at LSB)					
SDMA Total	24,100	11,700	35,900	43,200	-
Lake Shore Blvd Bridge (Max. Elev. 72.0)	6,200	1,600	5,100	8,800	Every 1-3 years
Ice Management Area (Max. Elev. 72.0) [Area F]	9,200	2,400	7,800	13,400	Every 1-2 years
Upstream Section of Keating Channel (Max. Elev. 72.0) [Area E]	53,200	2,900	10,300	20,100	Every 4-5 years

As previously stated, an approximation method was used for predicting long-term deposition rates and dredge frequencies. This approximation method was validated by comparison with a moderately long continuous simulation. A three-month simulation, which required seven days to run, was chosen for comparison with the approximation method. This period included 28 storm events with peak flows up to 116 cubic metres per second. Predicted sedimentation volumes from this continuous simulation were compared with predicted volumes from the approximation method, which synthesized the impacts of individual events. Volumes compared very well between the two methods, with the largest differences of 16% and 13%, which occurred in SDMA Area A and Area B.

Predicted sedimentation patterns were further evaluated for reasonableness by comparison with present-day sedimentation patterns in the Keating Channel. Currently, approximately 45,000 cubic yards of sediment are removed from the Keating Channel each year. For the design condition, a similar quantity of sediment is predicted to be deposited only in the SDMA (43,000 cubic yards per year). As designed, the SDMA effectively shifts the primary location of present-day sedimentation farther upstream to the SDMA.

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Appendices

Appendix A: Sediment Transport Model Results Maps

Appendix B: Sediment Transport Initial Bed Composition by Model “Core”

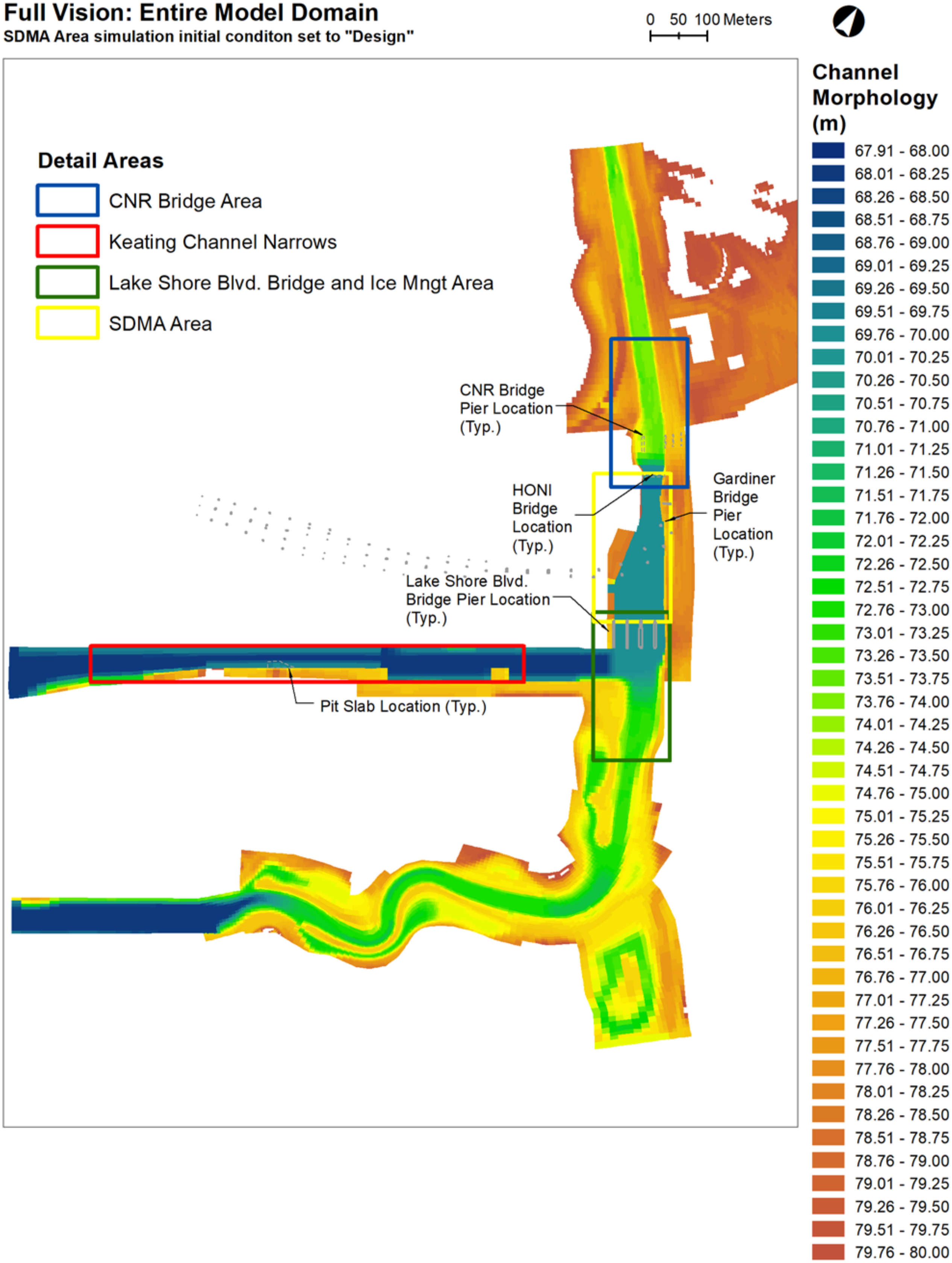
Appendix C: Sediment Transport Model Volumes of Deposition/Erosion

Predicted Channel Morphology & Bed Delta Plots for "Full" and "Design" Conditions

- 36 sediment transport simulations were conducted as sensitivity tests; 18 for "Design" conditions and 18 for "Full" conditions
- These 36 simulations were performed for both Full Vision and Interim Condition (for a total of 72 simulations)
- The regulatory event with a peak flow of 1,560 m^3/s was used in these simulations
- The table below provides the Plot ID corresponding to alternatives used in each sensitivity test

Plot ID	Bed Composition Alternatives	Erosion Rate Alternatives	US Load Alternatives
A	Initial Condition ("Full" or "Design")		
B	Base Particle Size Distribution	Erosion Rate 1x	US Load 1x
C	Base Particle Size Distribution	Erosion Rate 1x	US Load 1.5x
D	Base Particle Size Distribution	Erosion Rate 1x	US Load 0.5x
E	Base Particle Size Distribution	Erosion Rate 3x	US Load 1x
F	Base Particle Size Distribution	Erosion Rate 3x	US Load 1.5x
G	Base Particle Size Distribution	Erosion Rate 3x	US Load 0.5x
H	Decreased D50	Erosion Rate 1x	US Load 1x
I	Decreased D50	Erosion Rate 1x	US Load 1.5x
J	Decreased D50	Erosion Rate 1x	US Load 0.5x
K	Decreased D50	Erosion Rate 3x	US Load 1x
L	Decreased D50	Erosion Rate 3x	US Load 1.5x
M	Decreased D50	Erosion Rate 3x	US Load 0.5x
N	Increase D50	Erosion Rate 1x	US Load 1x
O	Increase D50	Erosion Rate 1x	US Load 1.5x
P	Increase D50	Erosion Rate 1x	US Load 0.5x
Q	Increase D50	Erosion Rate 3x	US Load 1x
R	Increase D50	Erosion Rate 3x	US Load 1.5x
S	Increase D50	Erosion Rate 3x	US Load 0.5x

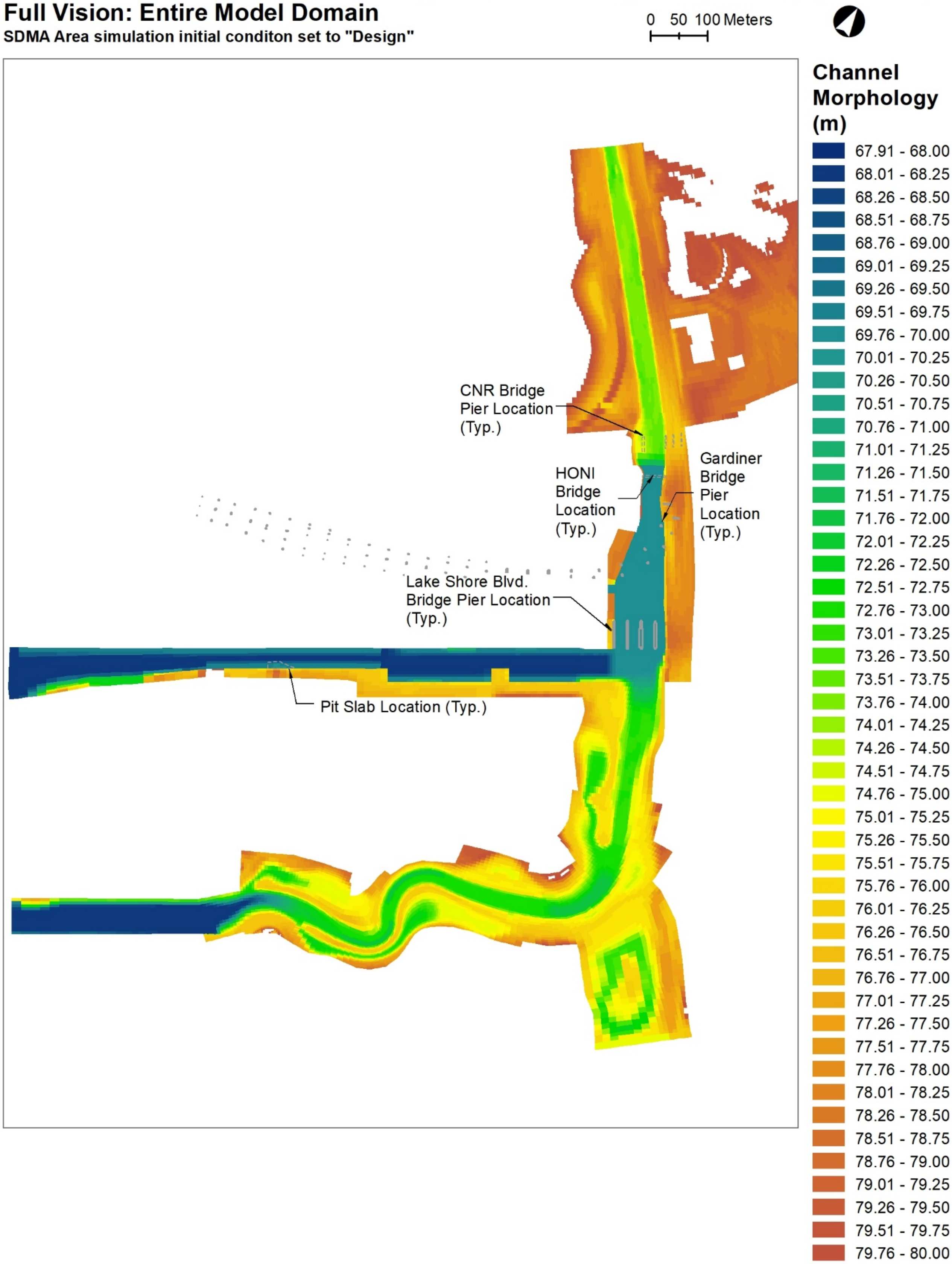
Design Channel Morphology with Extent of Detail Areas
Full Vision: Entire Model Domain
SDMA Area simulation initial conditon set to "Design"



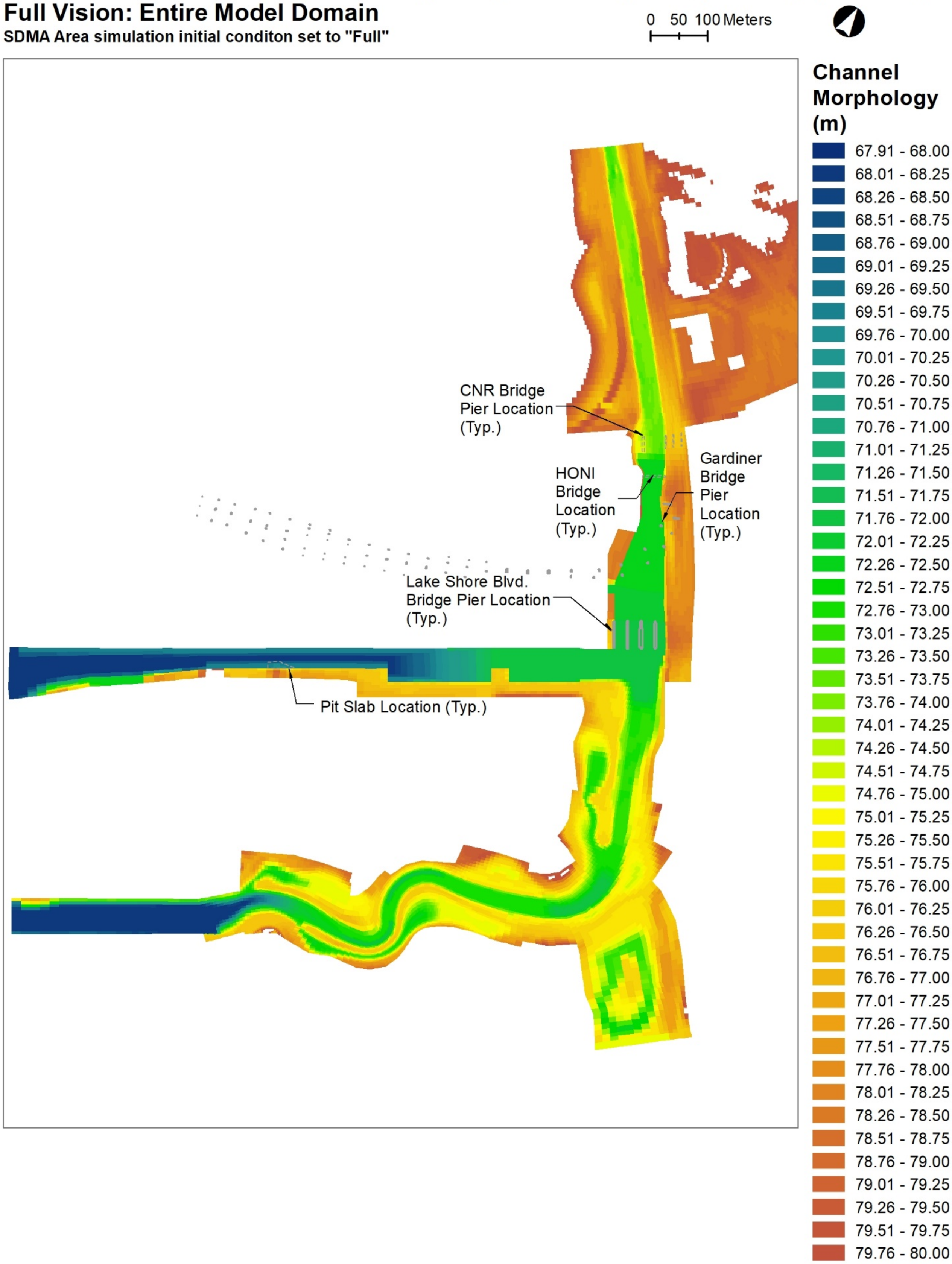
Full Vision Simulations

Initial Channel Morphology at the Beginning of the Regulatory Event [1,560 m^3/s]
Full Vision: Entire Model Domain

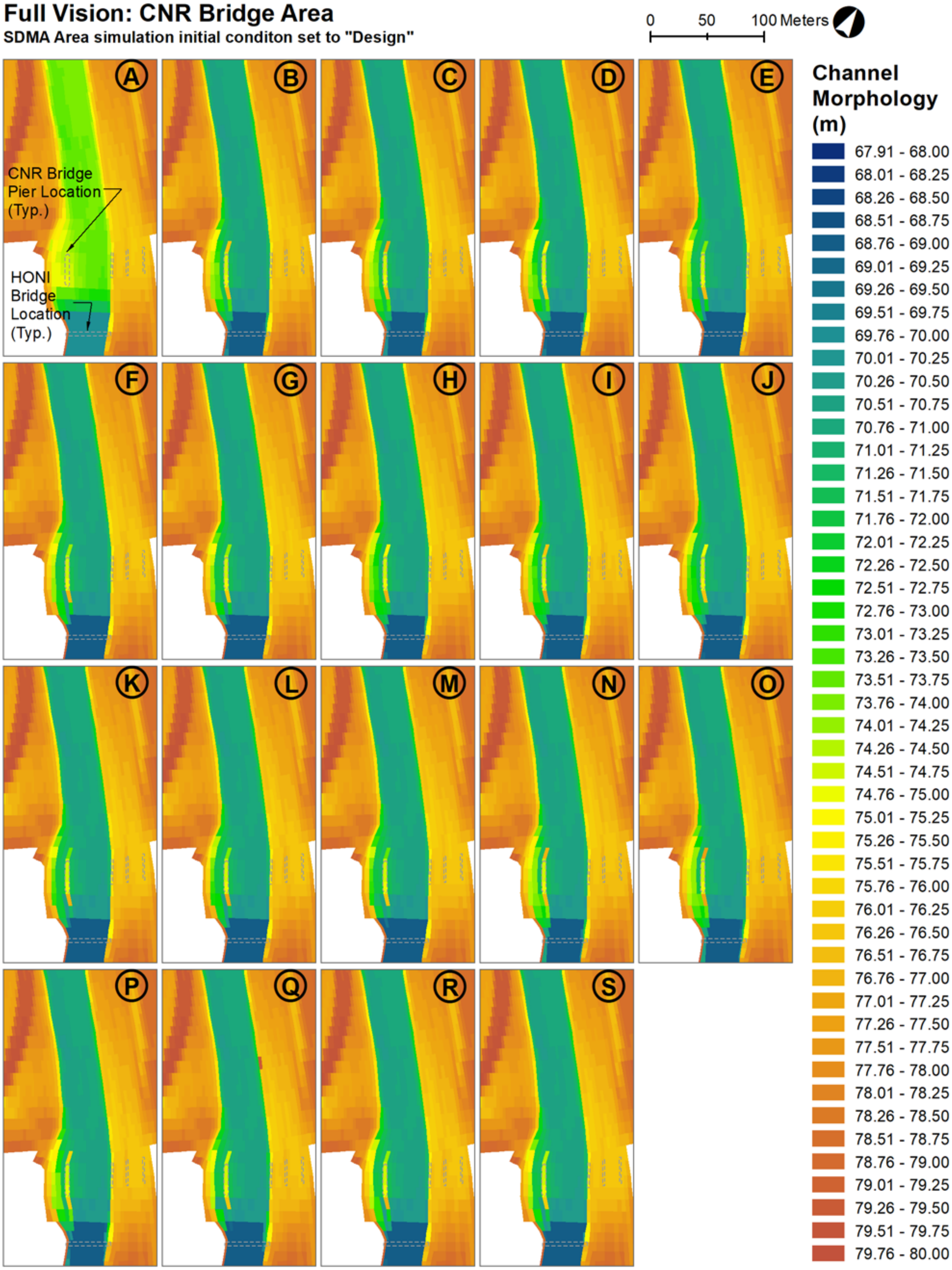
SDMA Area simulation initial conditon set to "Design"



Initial Channel Morphology at the Beginning of the Regulatory Event [1,560 m^3/s]
Full Vision: Entire Model Domain
SDMA Area simulation initial conditon set to "Full"



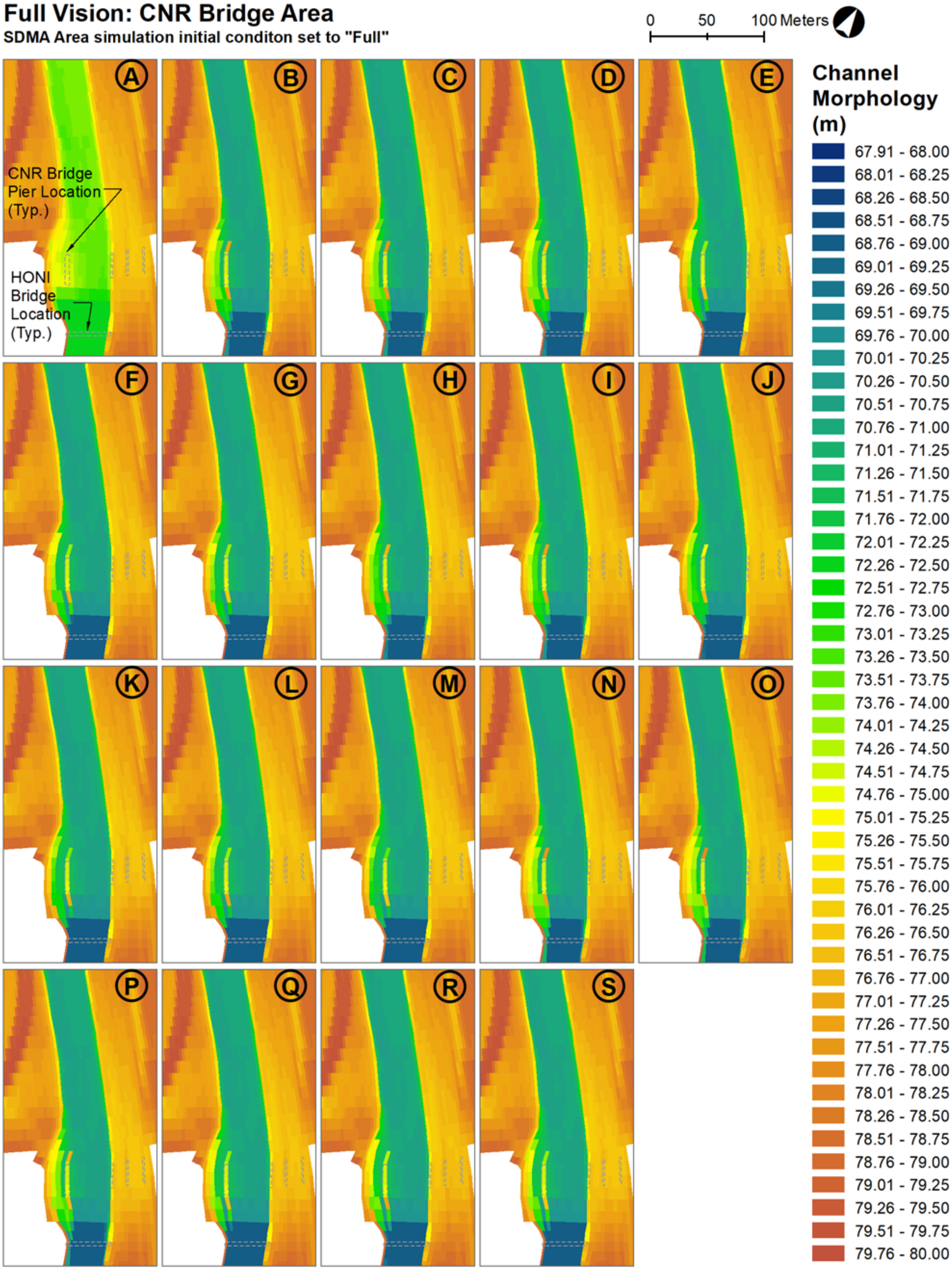
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: CNR Bridge Area
SDMA Area simulation initial conditon set to "Design"



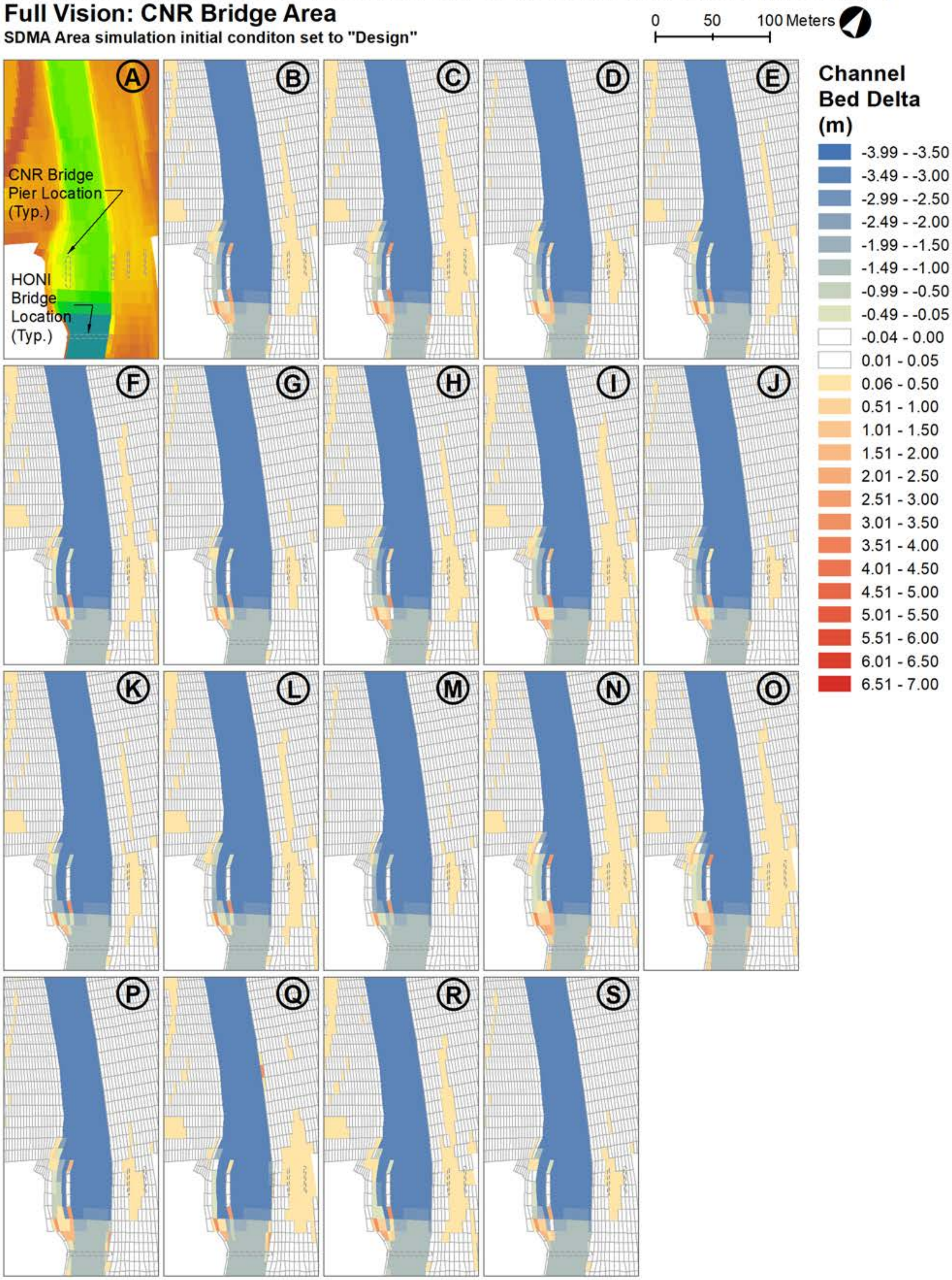
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

Full Vision: CNR Bridge Area

SDMA Area simulation initial conditon set to "Full"

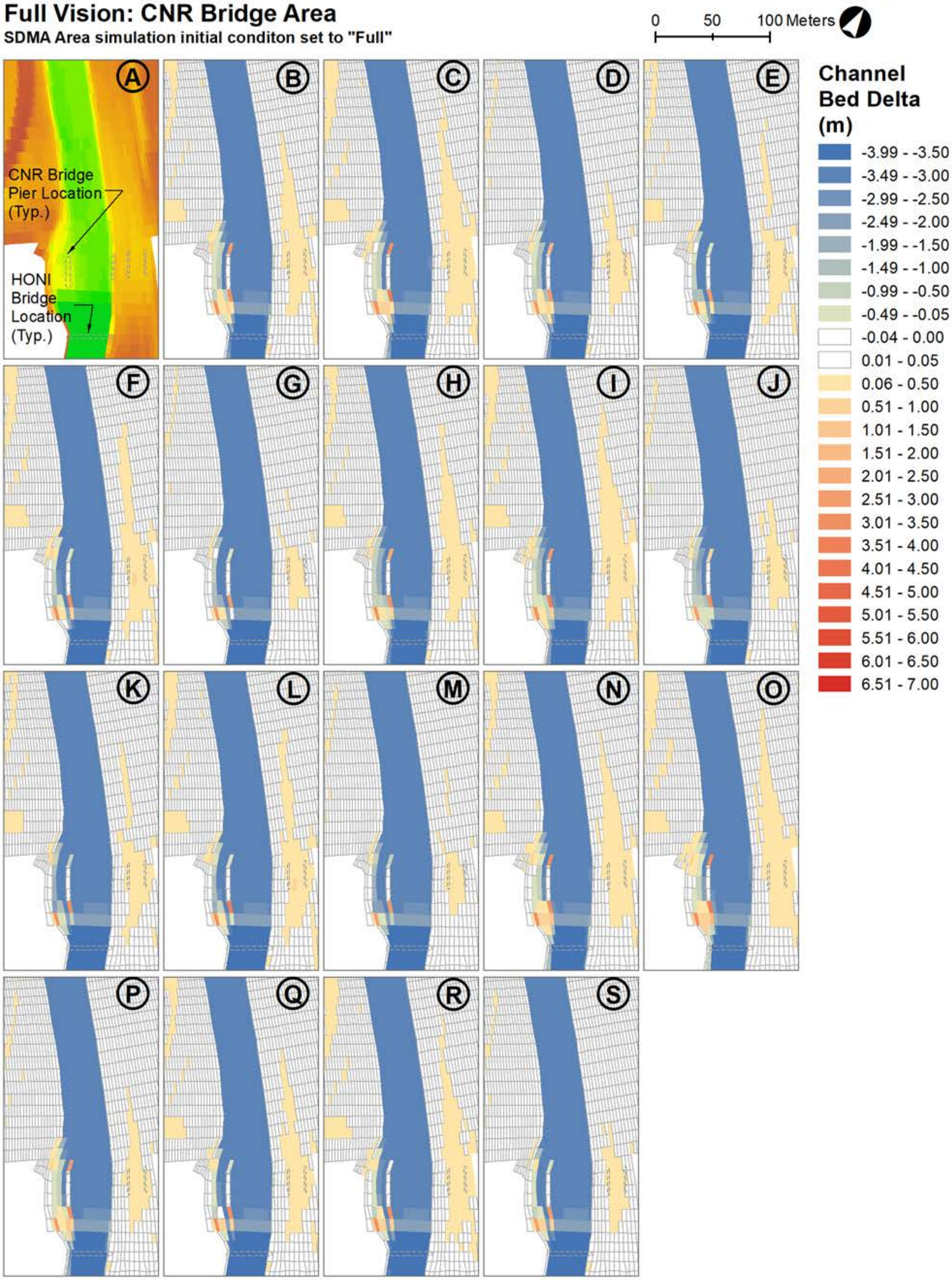


Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: CNR Bridge Area
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: CNR Bridge Area

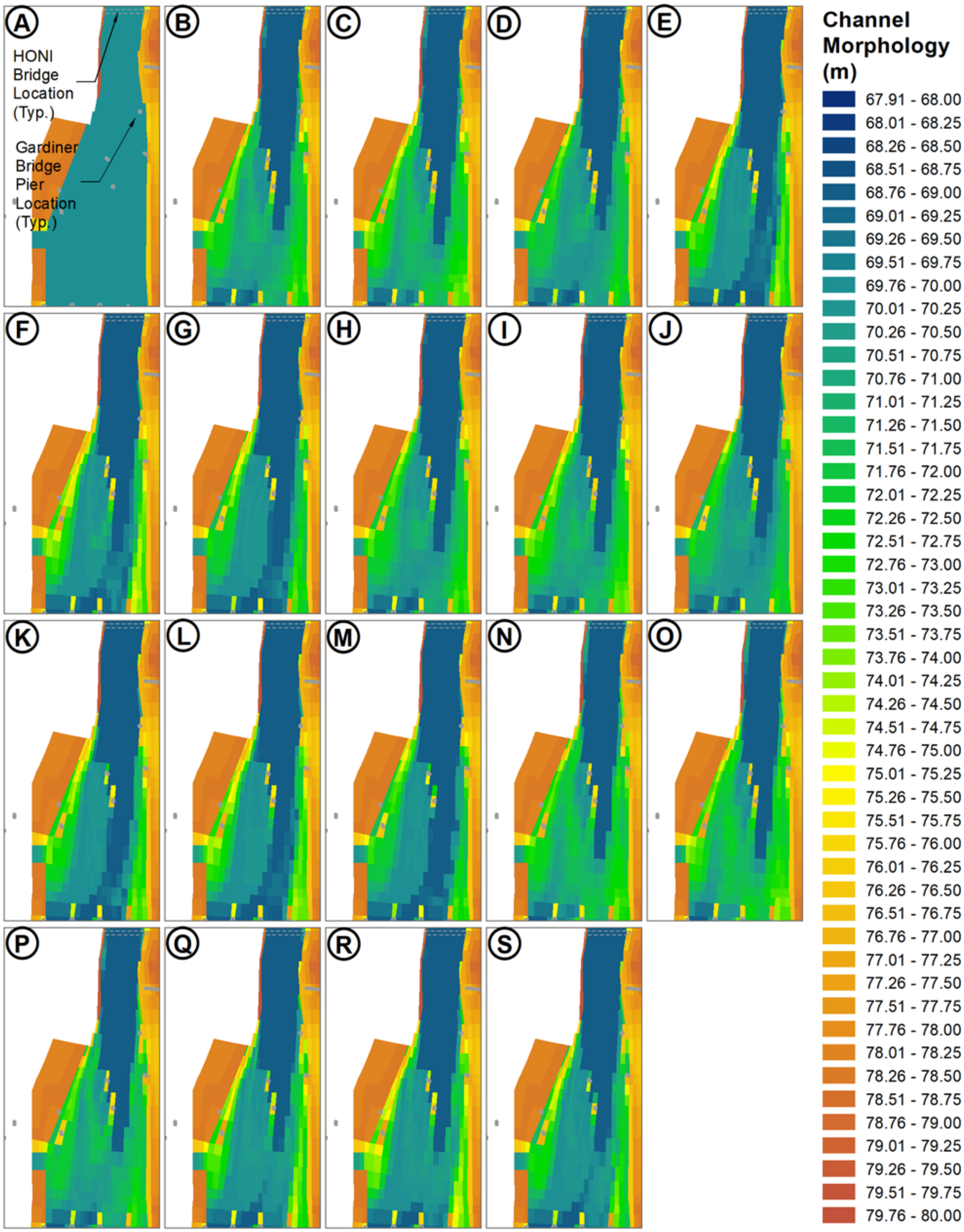
SDMA Area simulation initial conditon set to "Full"



Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: SDMA Area

SDMA Area simulation initial conditon set to "Design"

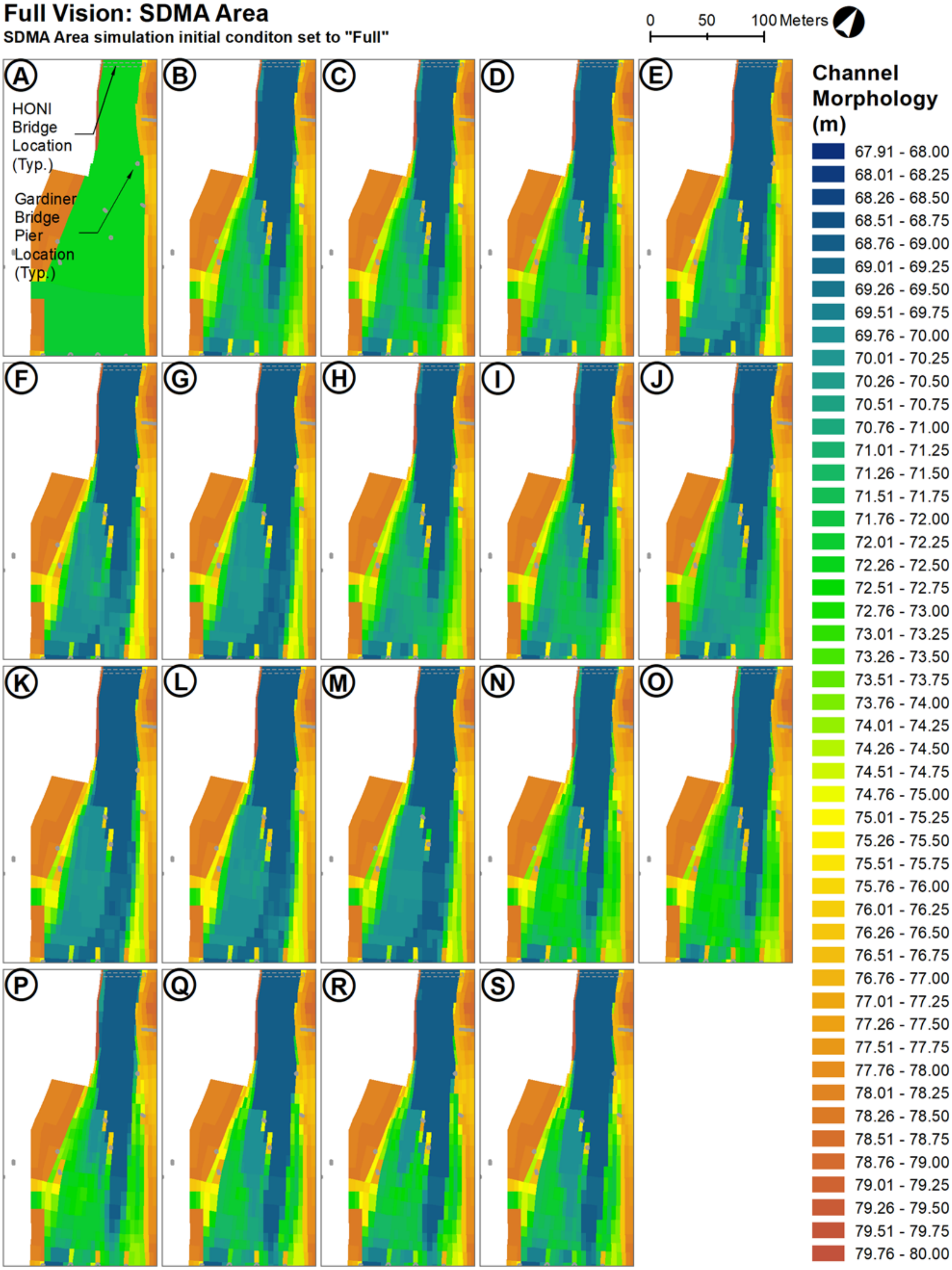
0 50 100 Meters



Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

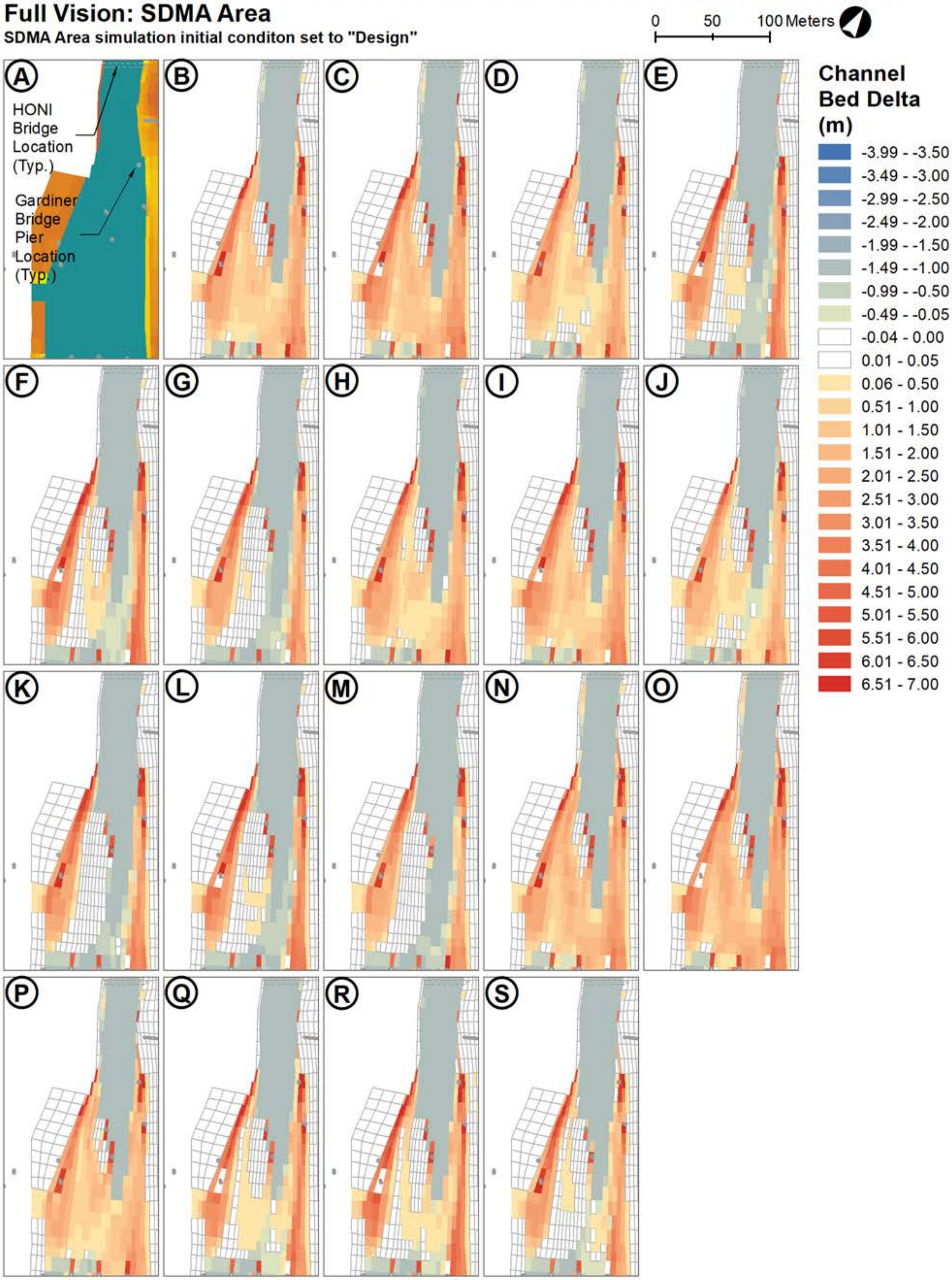
Full Vision: SDMA Area

SDMA Area simulation initial conditon set to "Full"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: SDMA Area

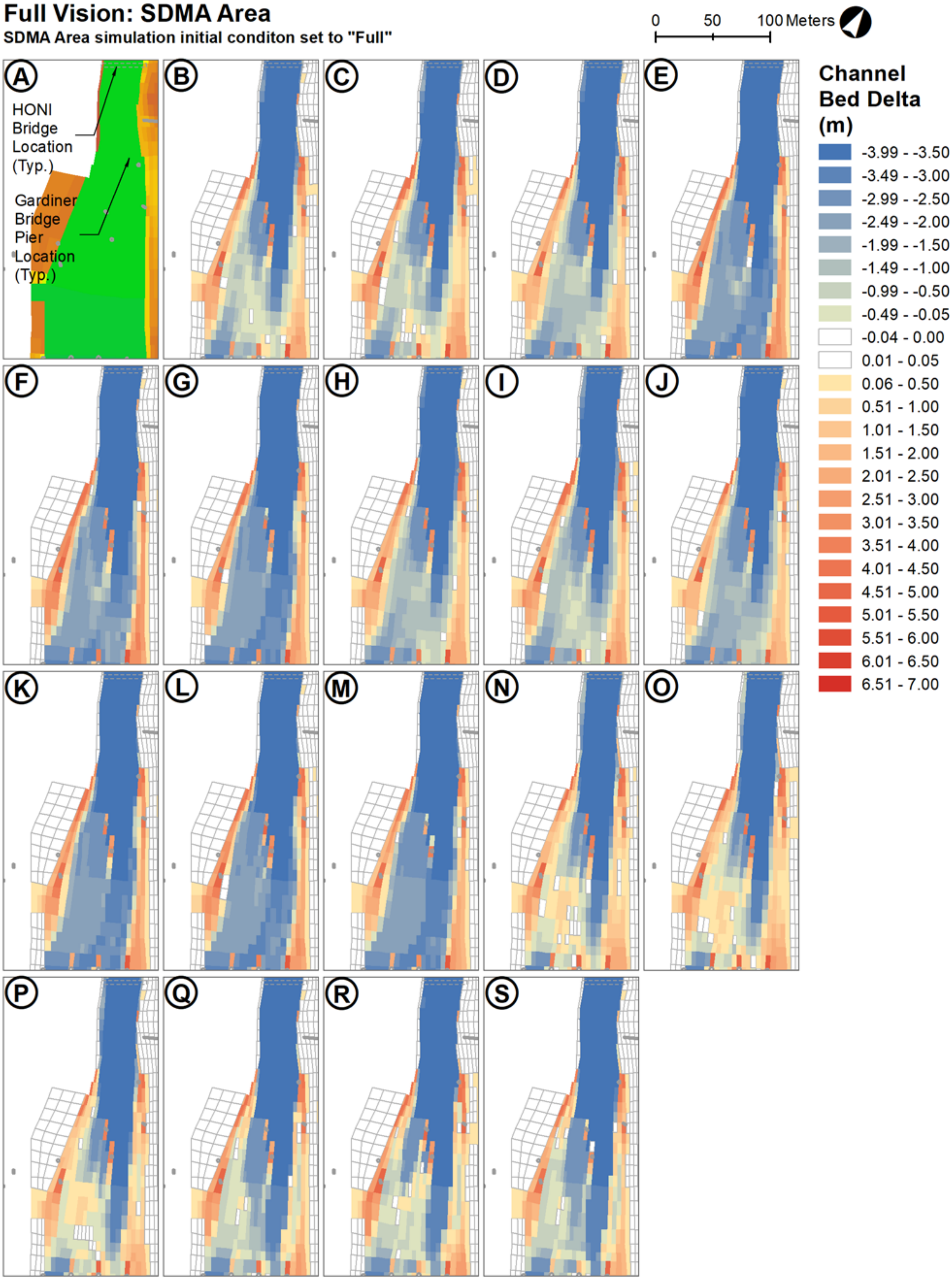
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m³/s]

Full Vision: SDMA Area

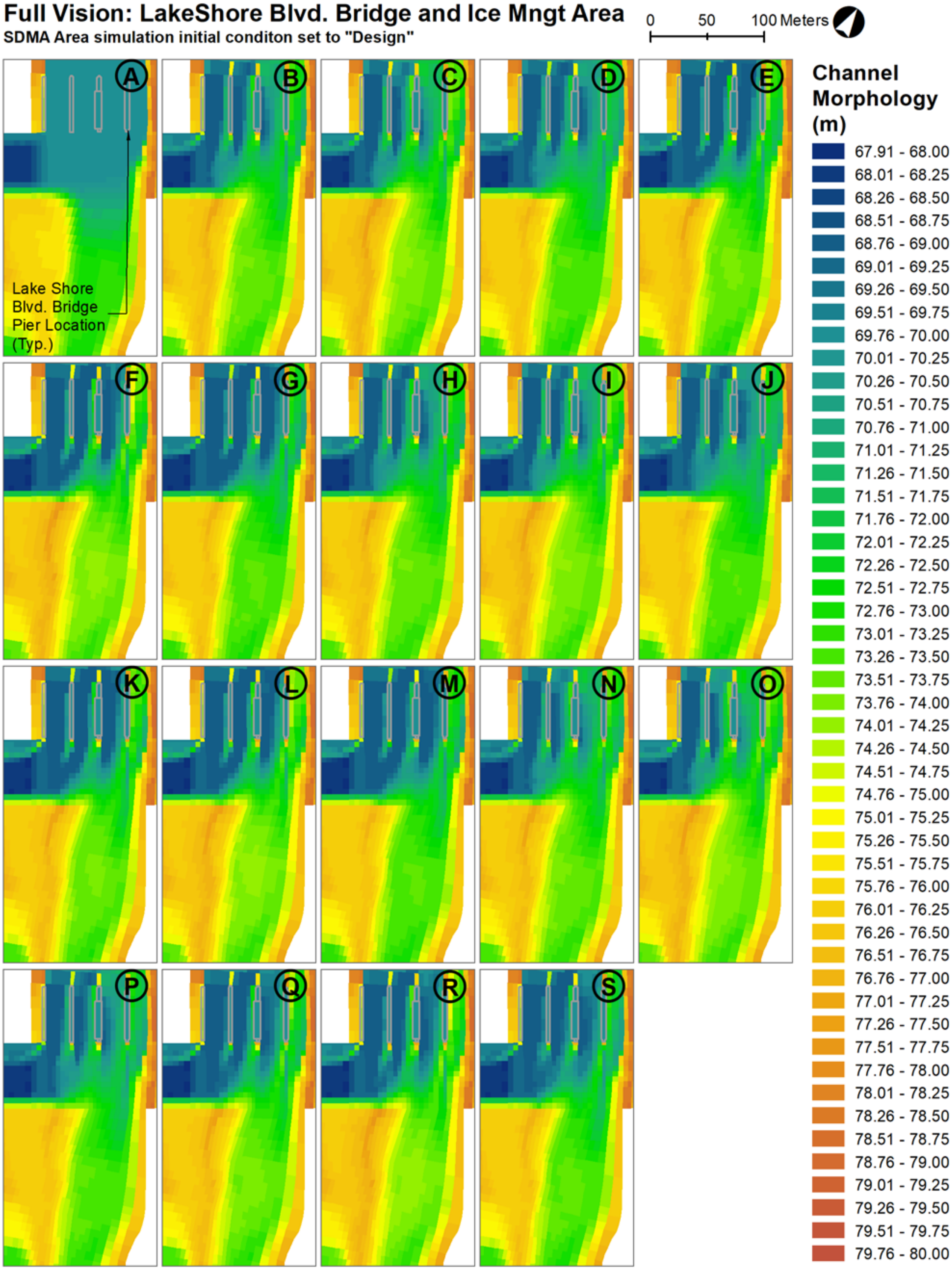
SDMA Area simulation initial conditon set to "Full"



Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

Full Vision: LakeShore Blvd. Bridge and Ice Mngt Area

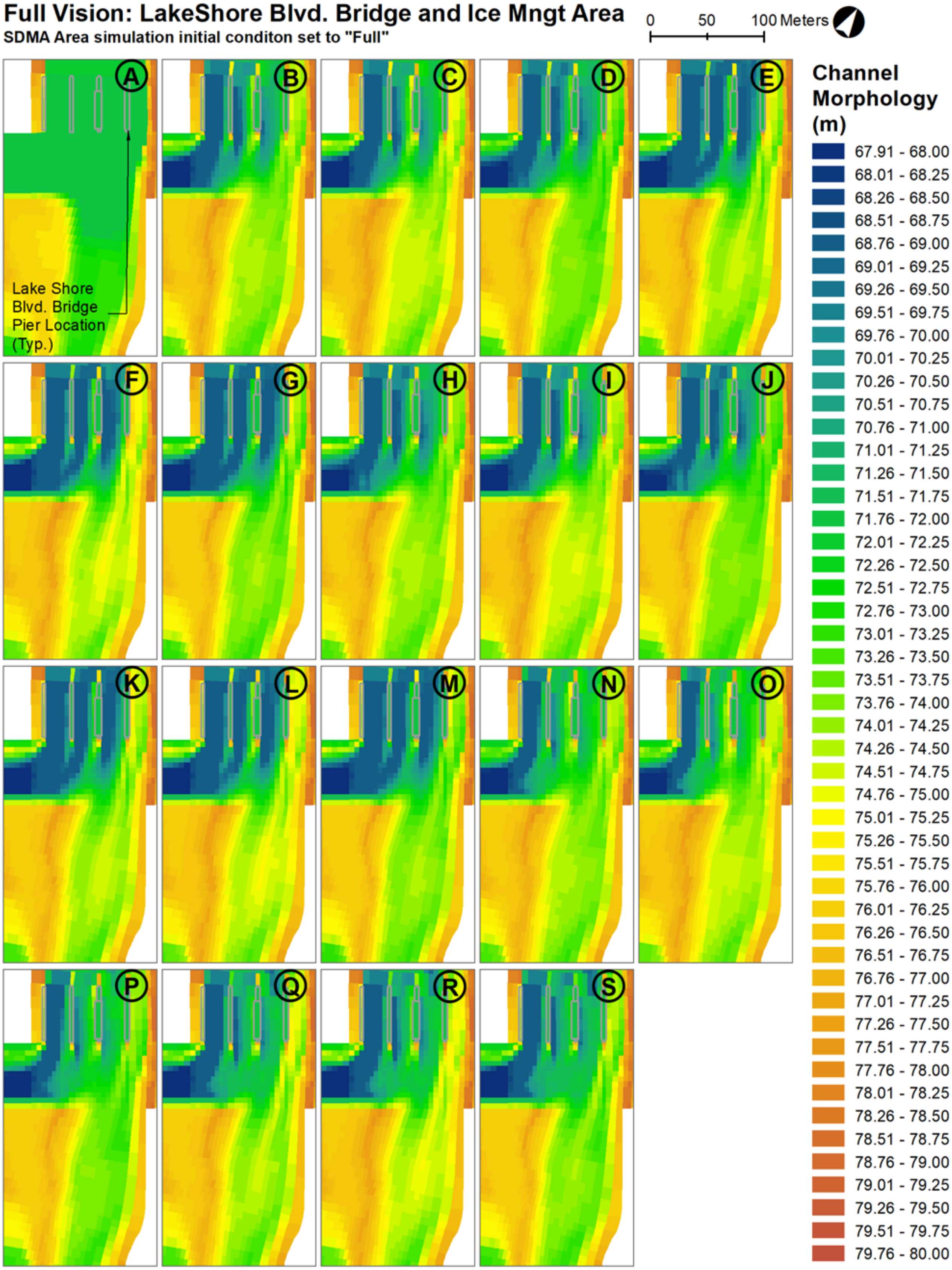
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

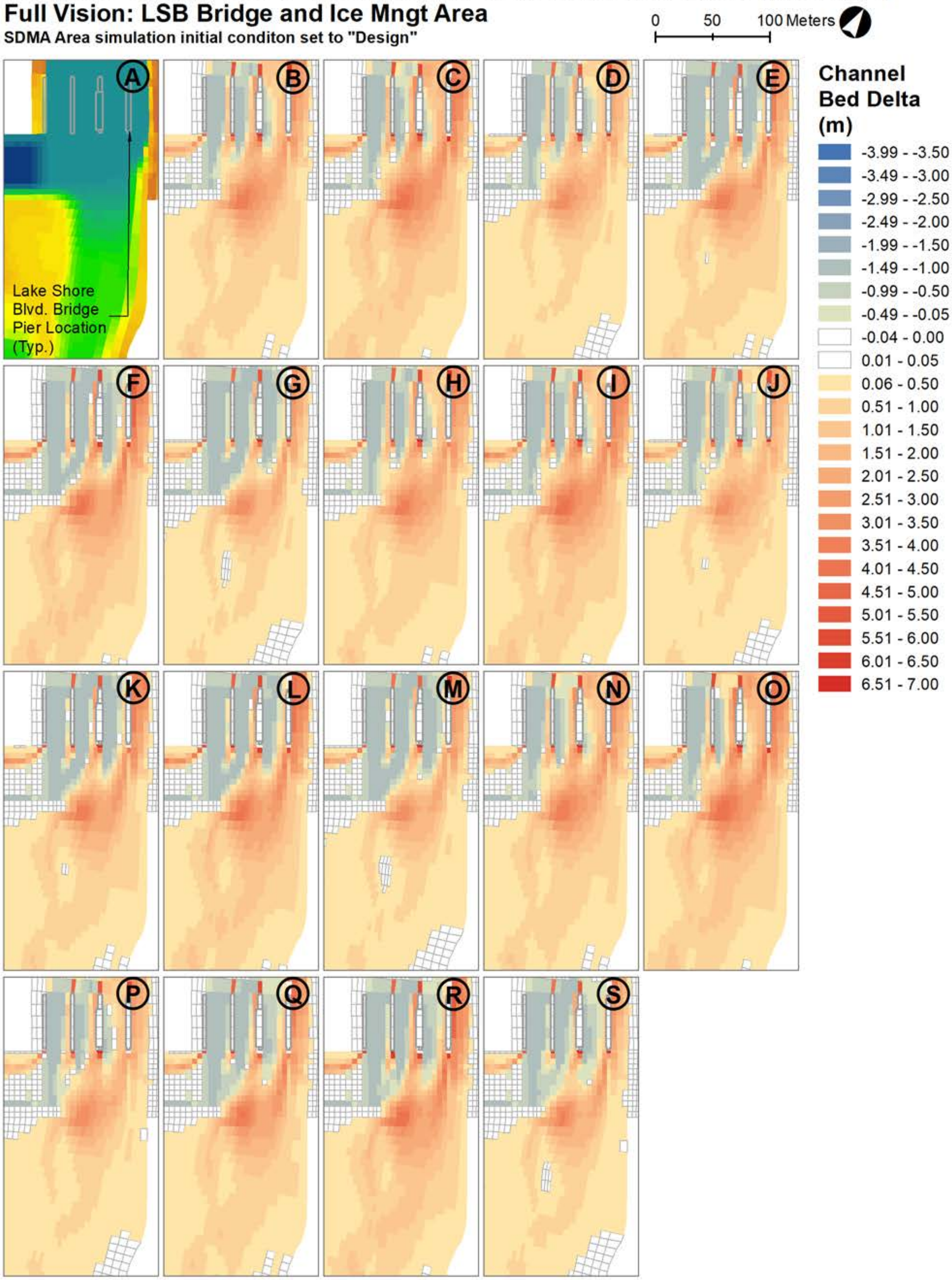
Full Vision: LakeShore Blvd. Bridge and Ice Mngt Area

SDMA Area simulation initial conditon set to "Full"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: LSB Bridge and Ice Mngt Area

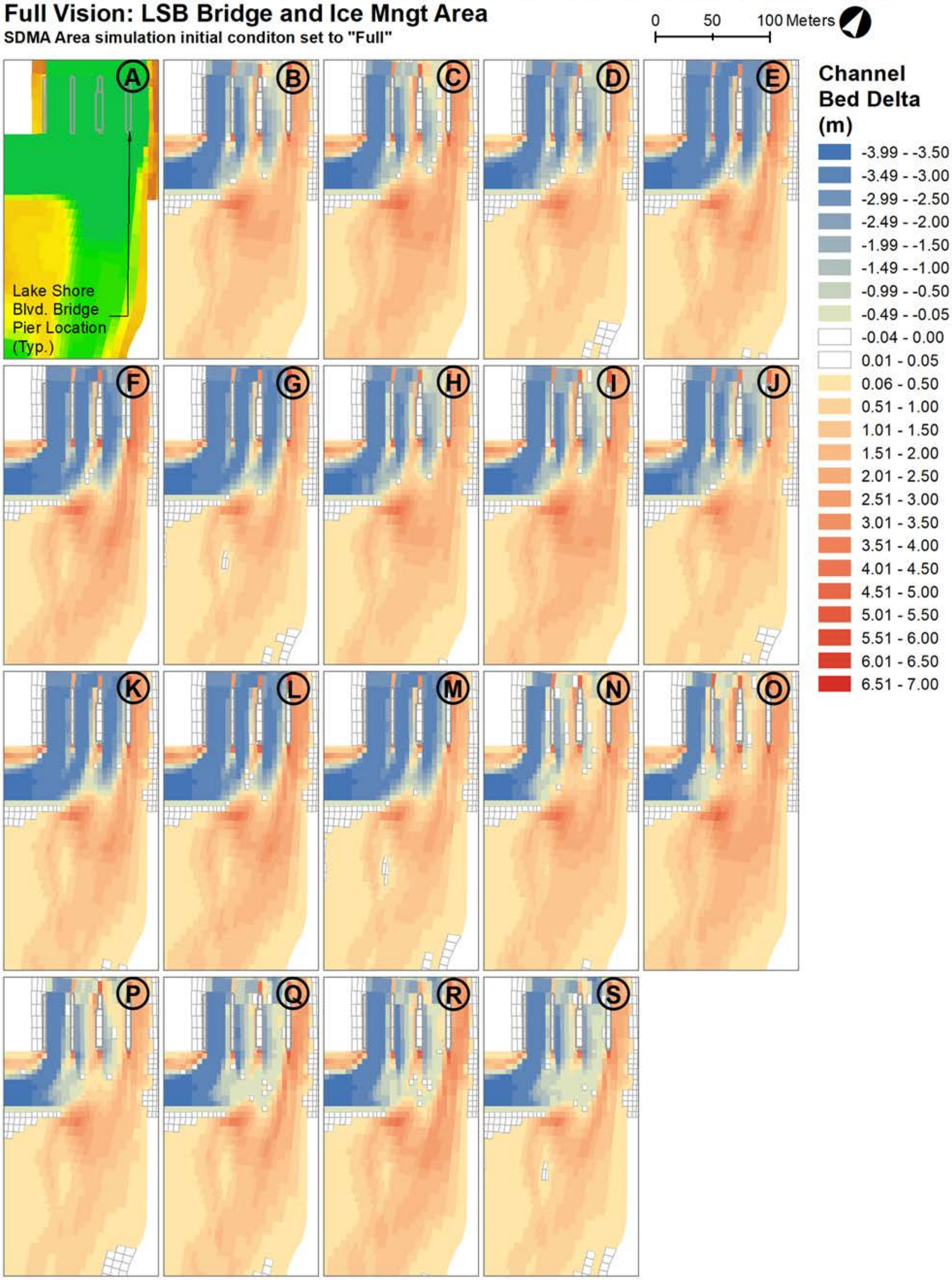
SDMA Area simulation initial conditon set to "Design"



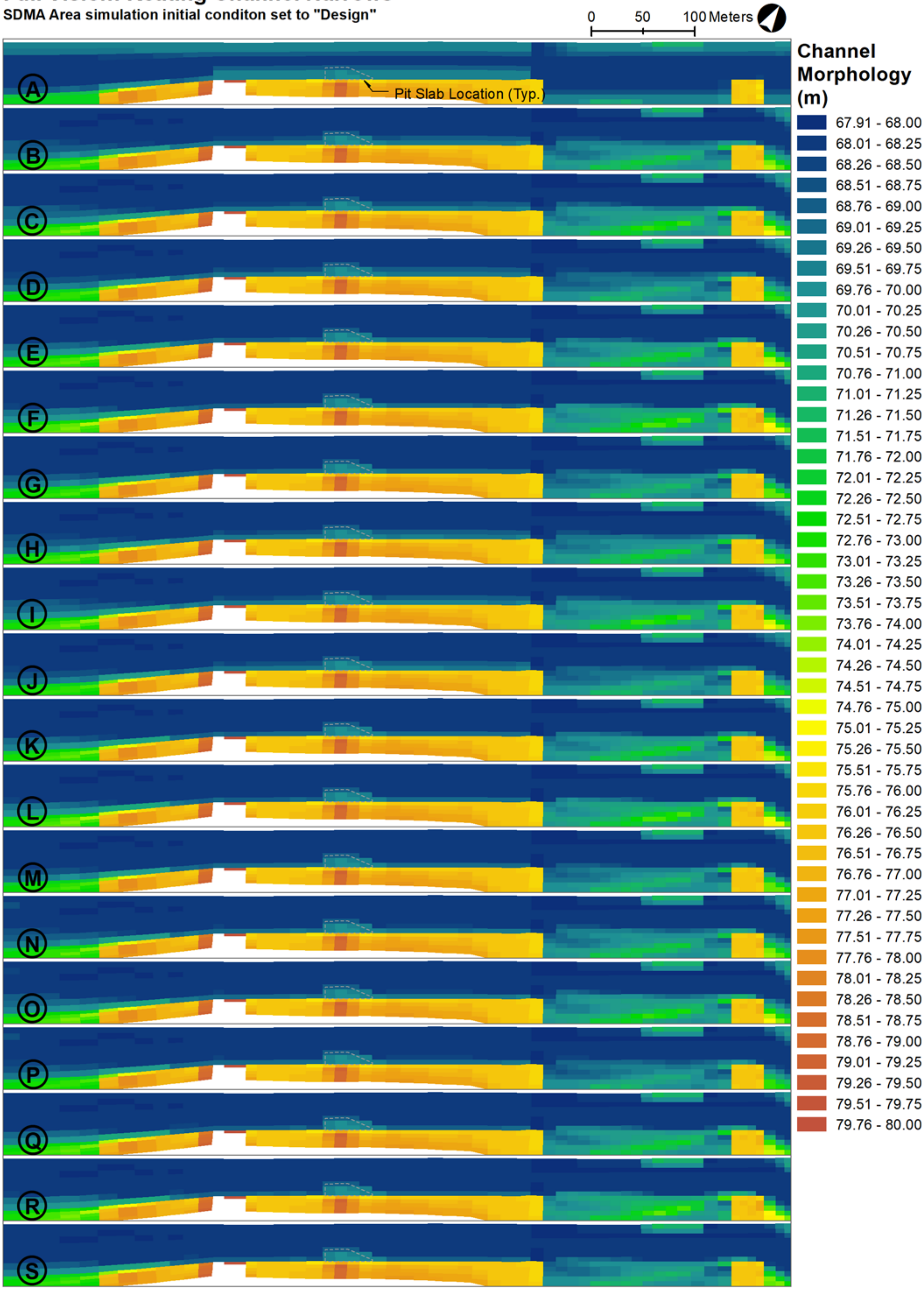
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]

Full Vision: LSB Bridge and Ice Mngt Area

SDMA Area simulation initial conditon set to "Full"

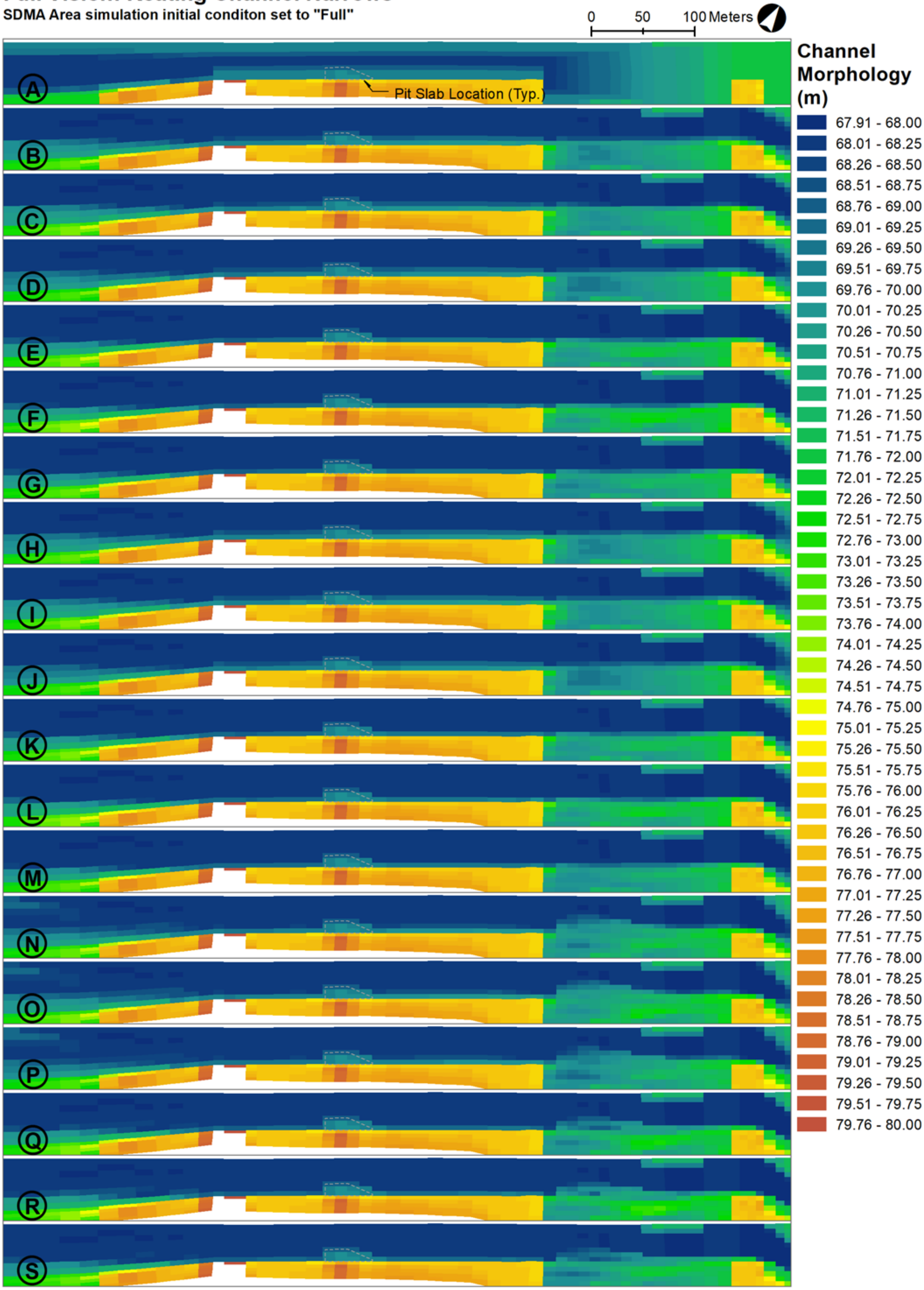


Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: Keating Channel Narrows
SDMA Area simulation initial conditon set to "Design"



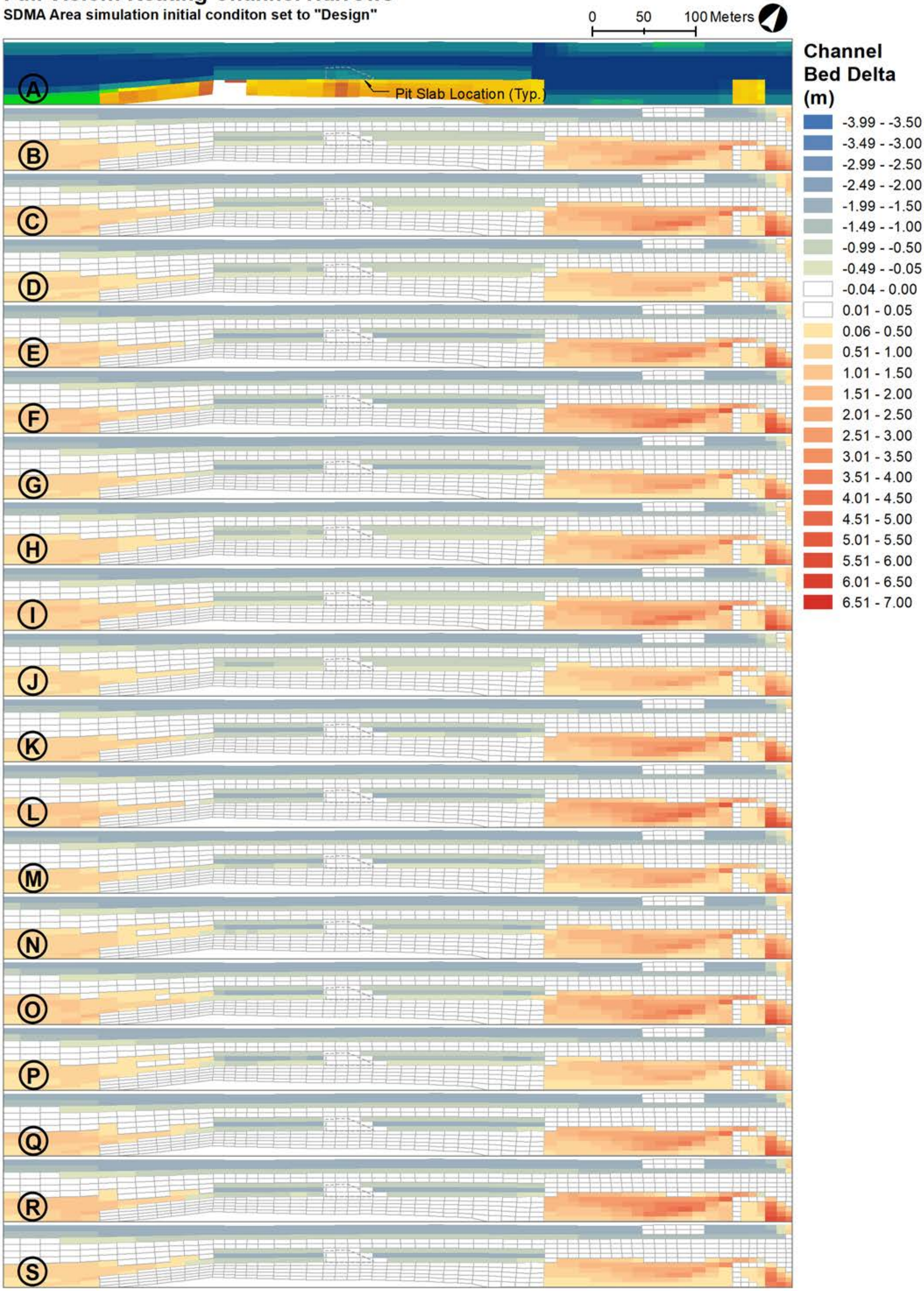
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Full"



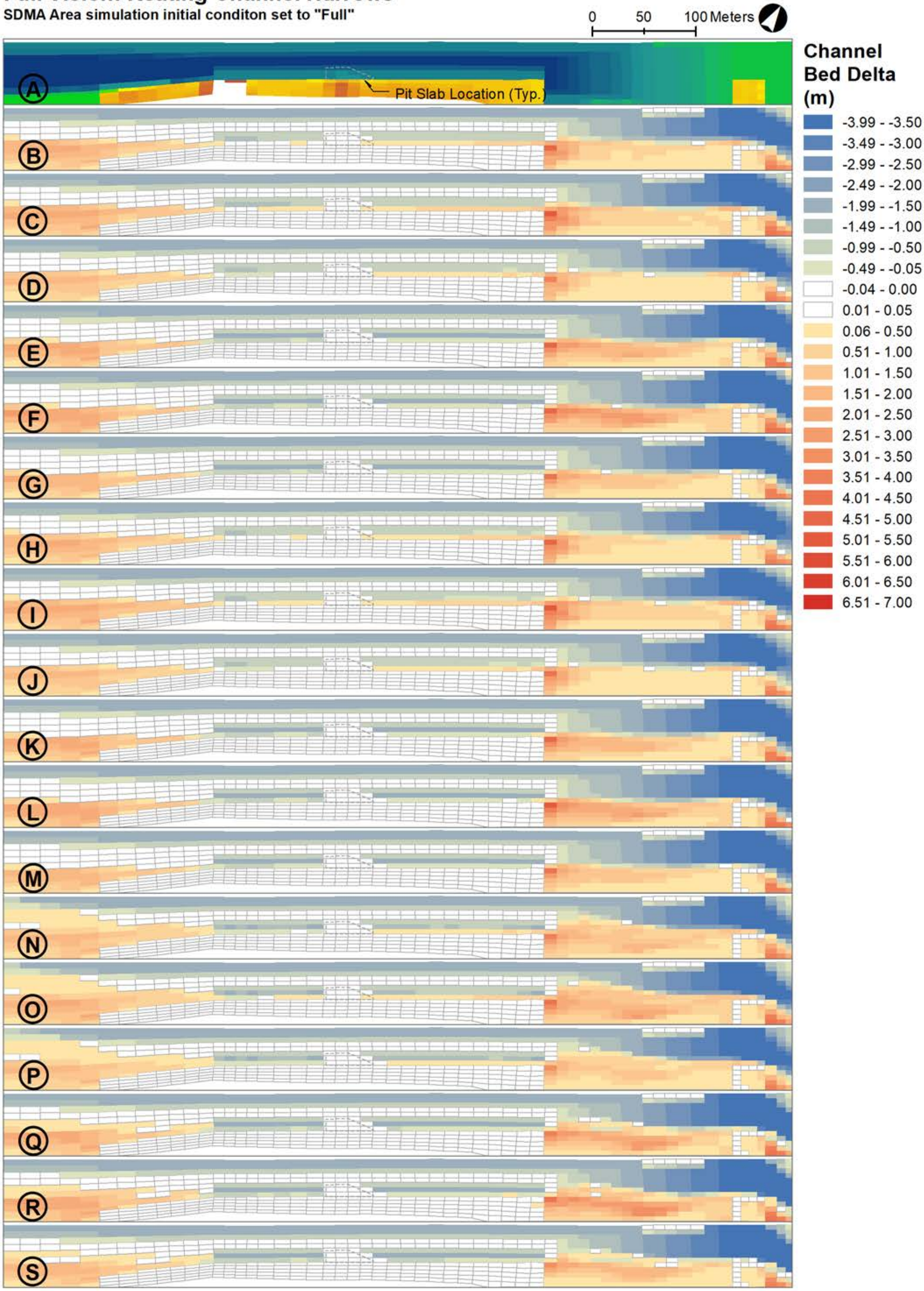
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m³/s]
Full Vision: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Full"



Predicted Channel Morphology & Bed Delta Plots for “Full” and “Design” Conditions

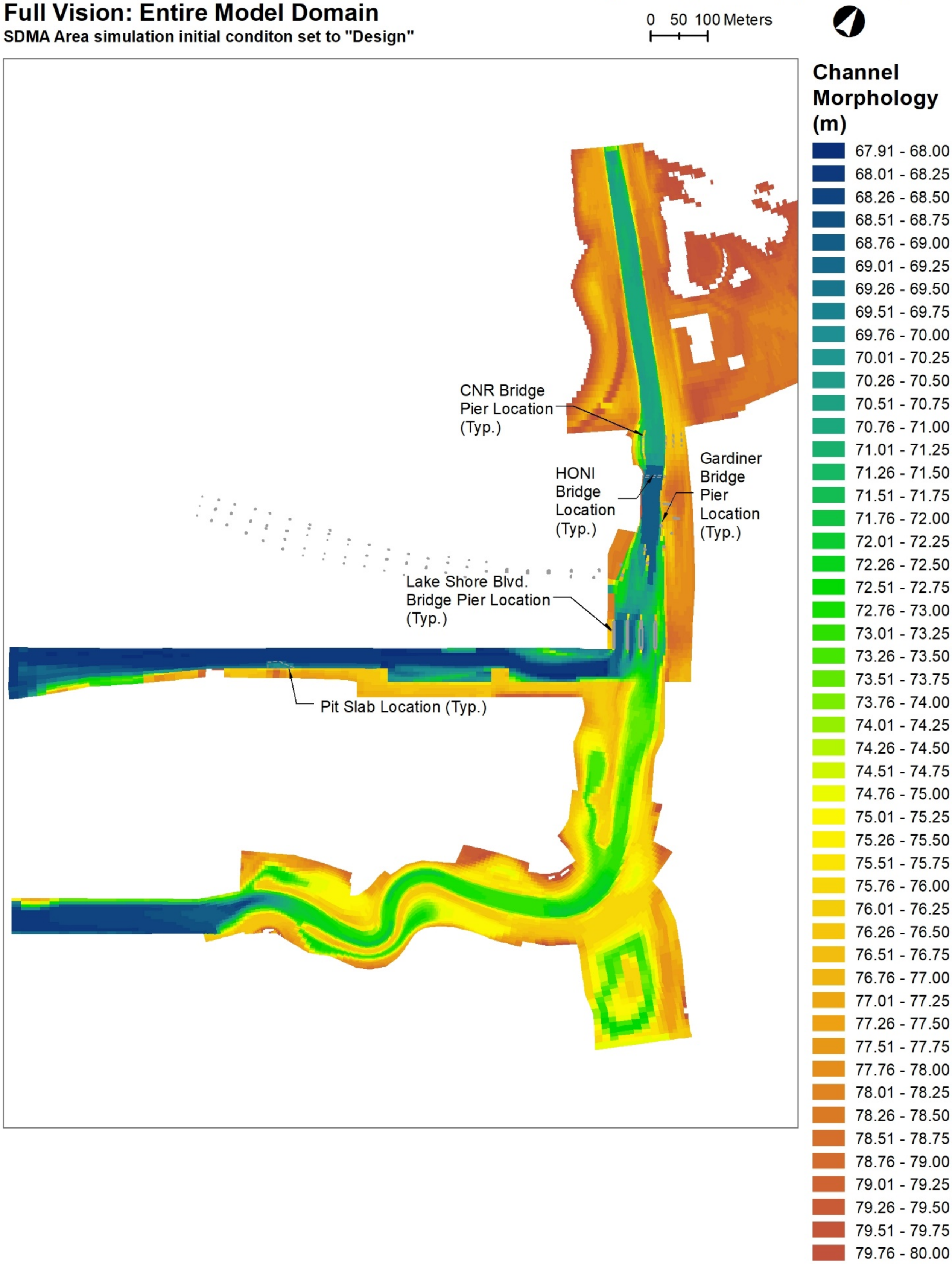
- The following plots show the entire EFDC model domain
- Only provided plots for the base alternative simulations for “Design” condition and “Full” condition (which corresponds to Plot ID “B” for the previous plots)

Plot ID	Bed Composition Alternatives	Erosion Rate Alternatives	US Load Alternatives
A	Initial Condition ("Full" or "Design")		
B	Base Particle Size Distribution	Erosion Rate 1x	US Load 1x
C	Base Particle Size Distribution	Erosion Rate 1x	US Load 1.5x
D	Base Particle Size Distribution	Erosion Rate 1x	US Load 0.5x
E	Base Particle Size Distribution	Erosion Rate 3x	US Load 1x
F	Base Particle Size Distribution	Erosion Rate 3x	US Load 1.5x
G	Base Particle Size Distribution	Erosion Rate 3x	US Load 0.5x
H	Decreased D50	Erosion Rate 1x	US Load 1x
I	Decreased D50	Erosion Rate 1x	US Load 1.5x
J	Decreased D50	Erosion Rate 1x	US Load 0.5x
K	Decreased D50	Erosion Rate 3x	US Load 1x
L	Decreased D50	Erosion Rate 3x	US Load 1.5x
M	Decreased D50	Erosion Rate 3x	US Load 0.5x
N	Increase D50	Erosion Rate 1x	US Load 1x
O	Increase D50	Erosion Rate 1x	US Load 1.5x
P	Increase D50	Erosion Rate 1x	US Load 0.5x
Q	Increase D50	Erosion Rate 3x	US Load 1x
R	Increase D50	Erosion Rate 3x	US Load 1.5x
S	Increase D50	Erosion Rate 3x	US Load 0.5x

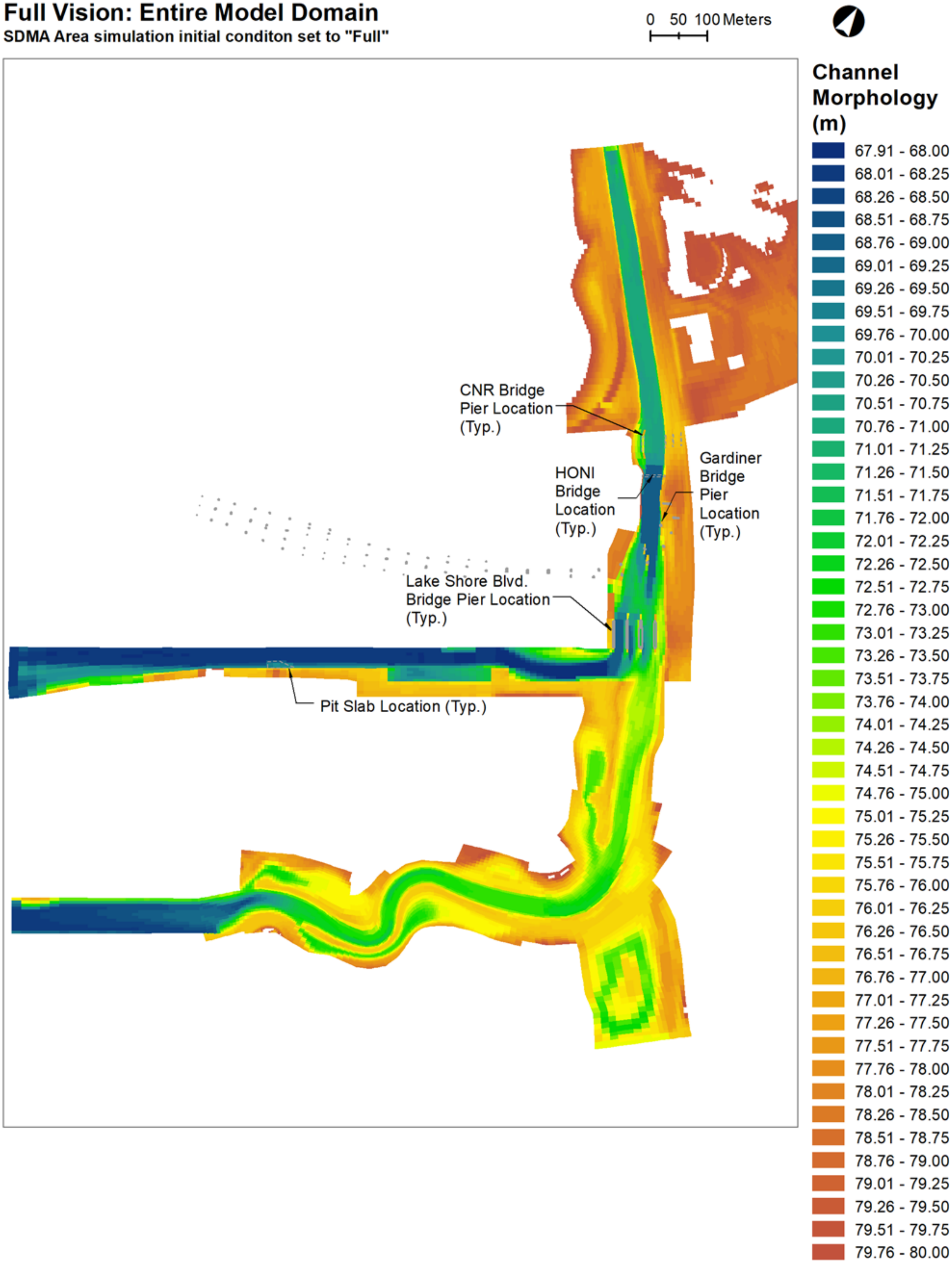
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

Full Vision: Entire Model Domain

SDMA Area simulation initial conditon set to "Design"

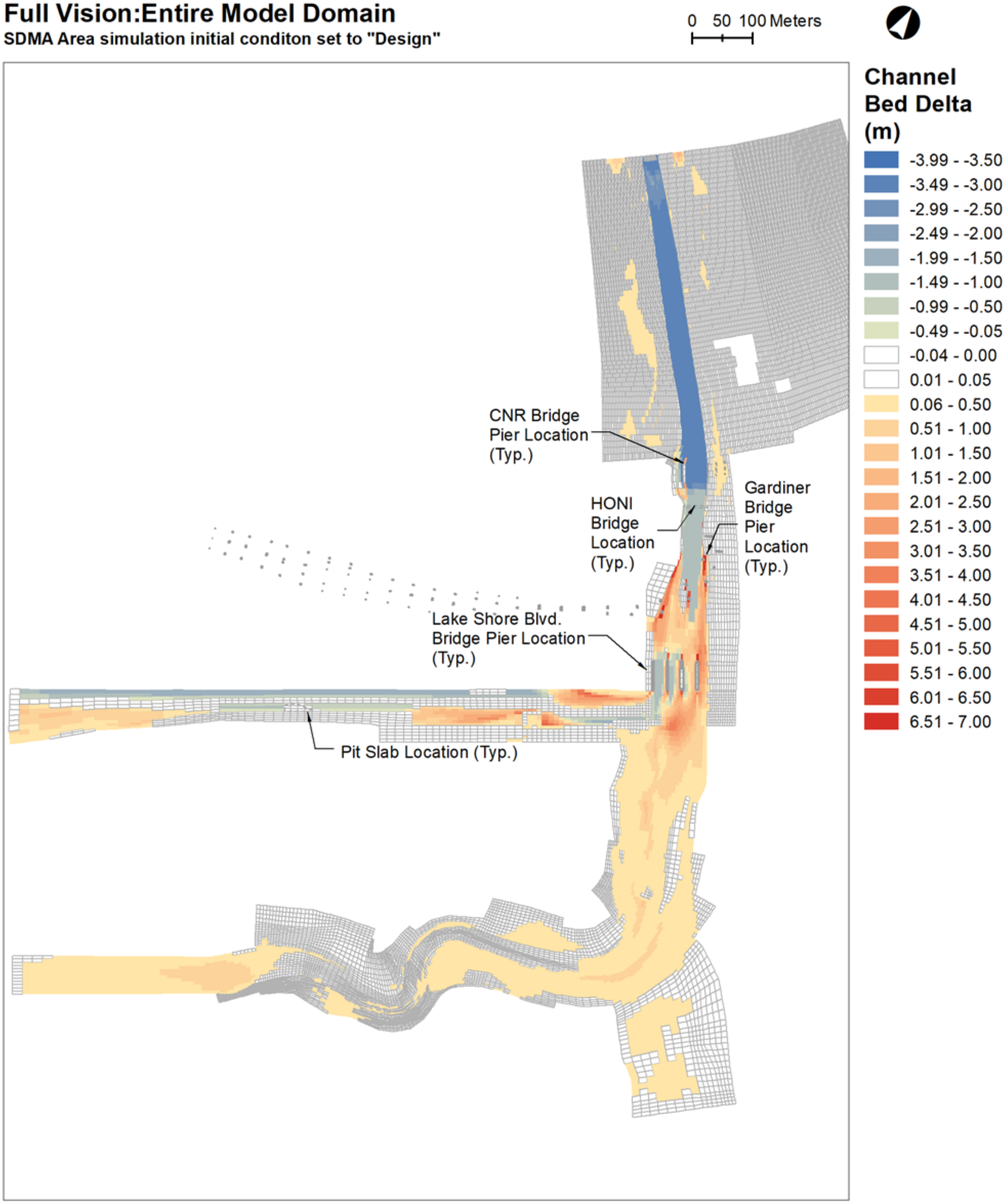


Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision: Entire Model Domain
SDMA Area simulation initial conditon set to "Full"

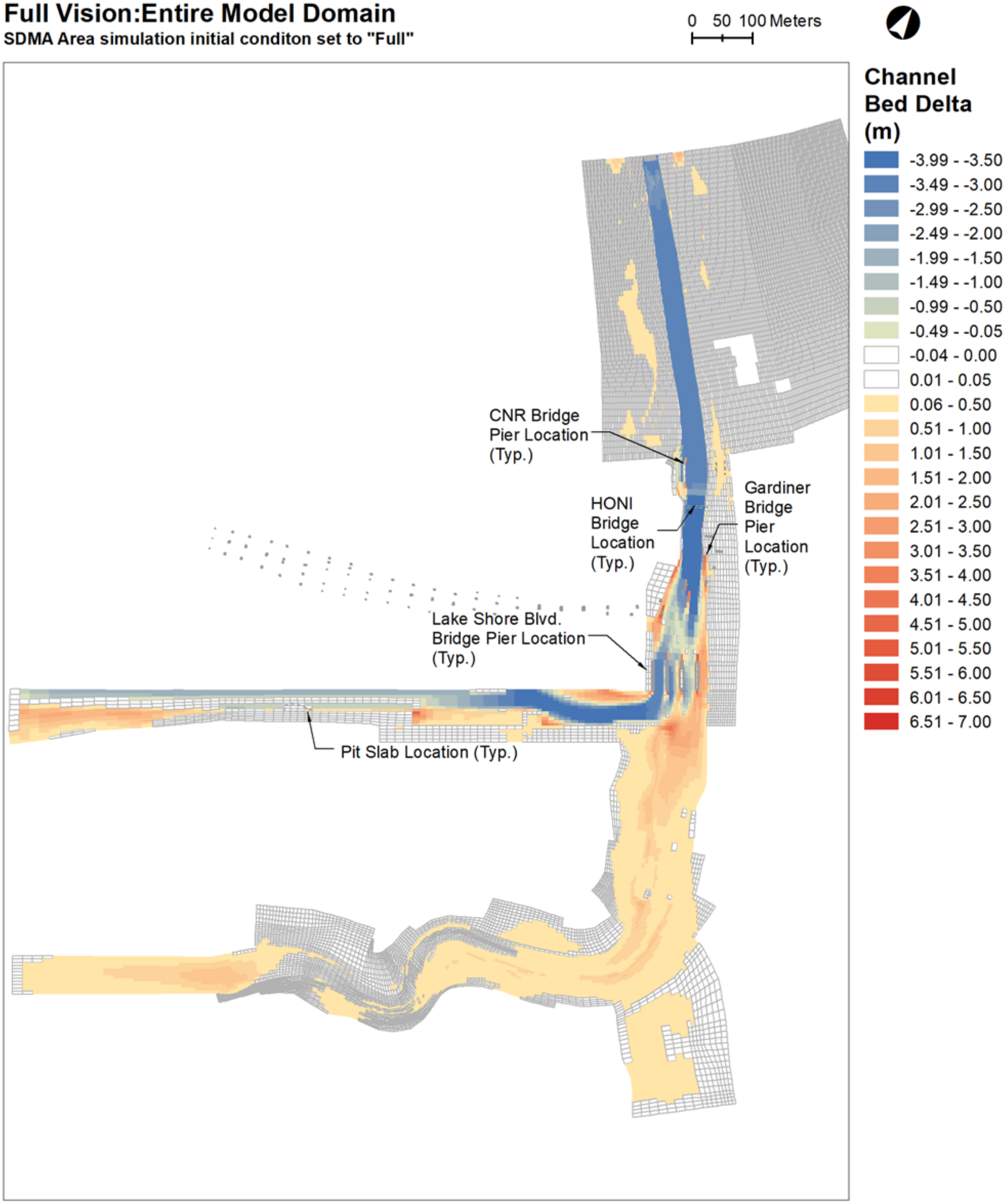


Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision:Entire Model Domain

SDMA Area simulation initial conditon set to "Design"



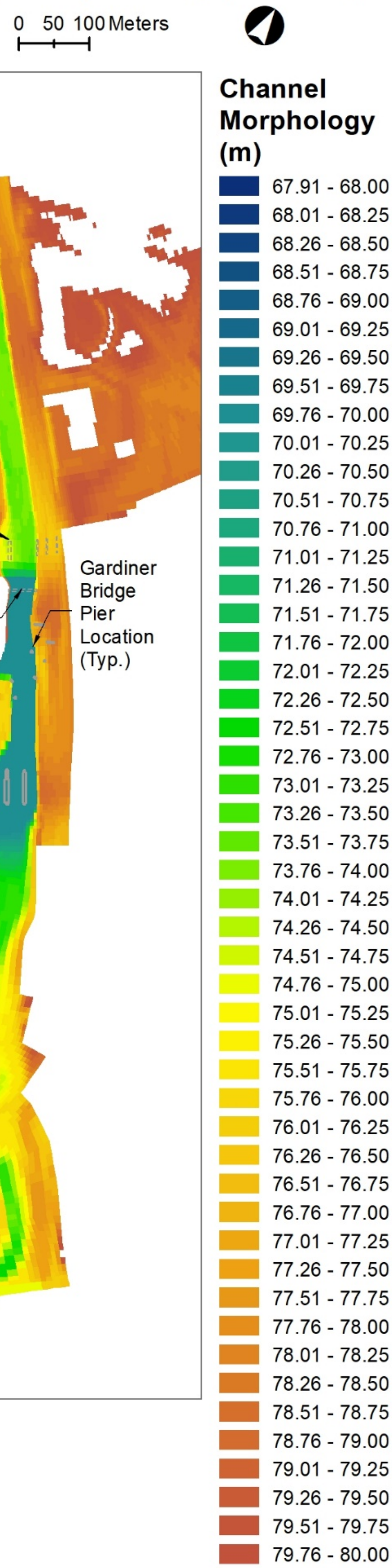
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Full Vision:Entire Model Domain
SDMA Area simulation initial conditon set to "Full"



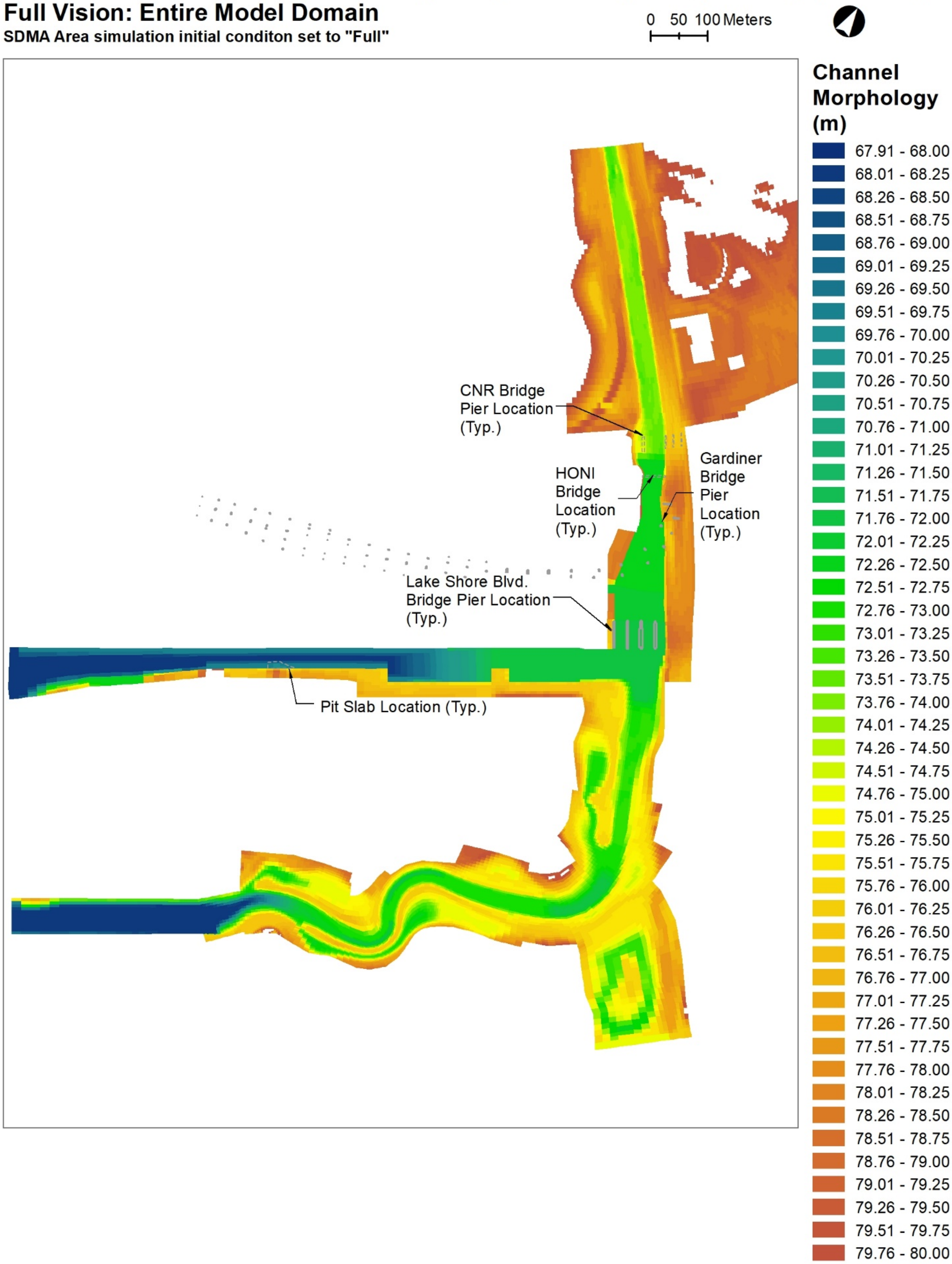
Interim Condition Simulations

Initial Channel Morphology at the Beginning of the Regulatory Event [1,560 m^3/s]
Interim Condition: Entire Model Domain

SDMA Area simulation initial conditon set to "Design"

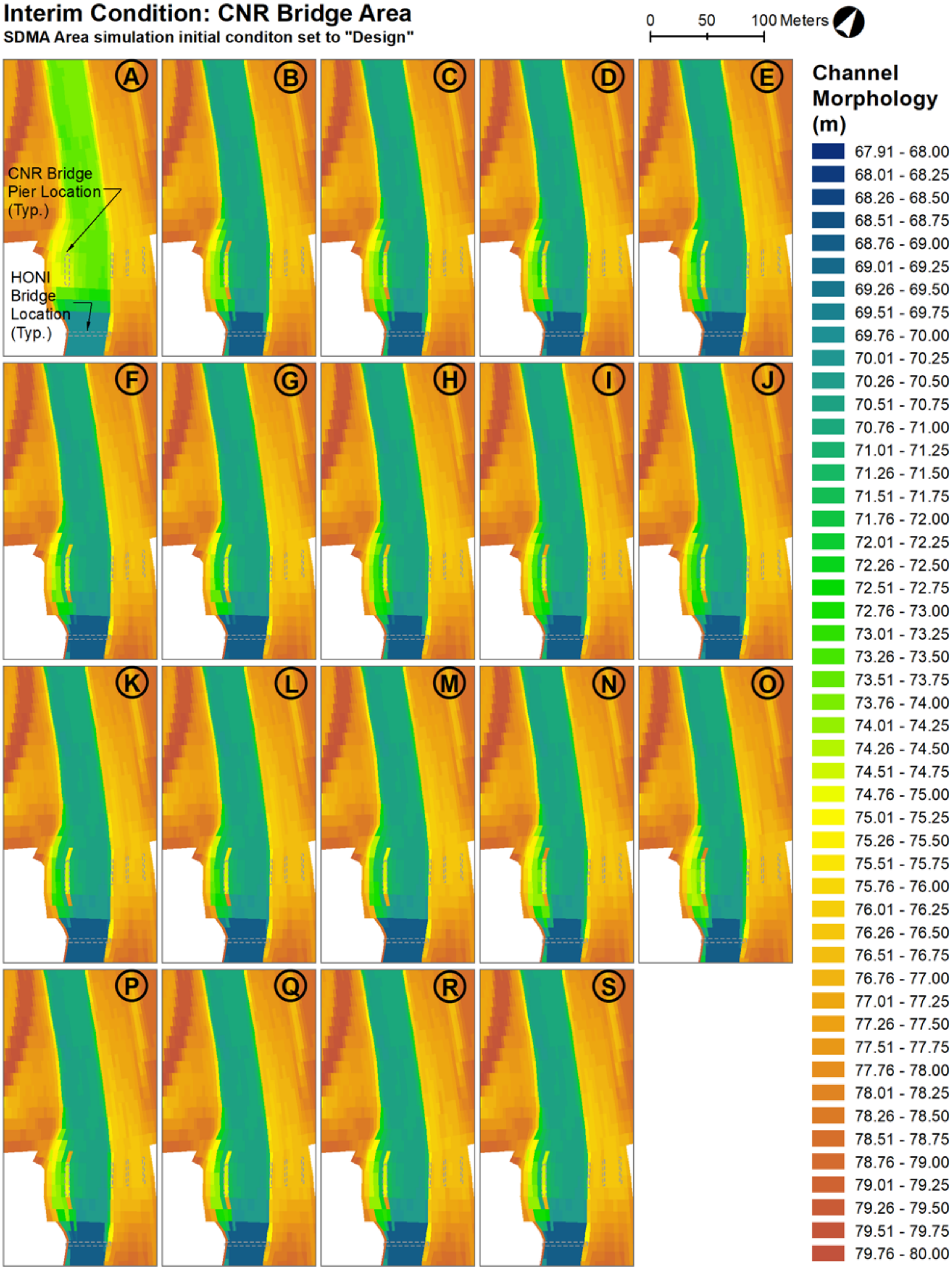


Initial Channel Morphology at the Beginning of the Regulatory Event [1,560 m^3/s]
Full Vision: Entire Model Domain
SDMA Area simulation initial conditon set to "Full"

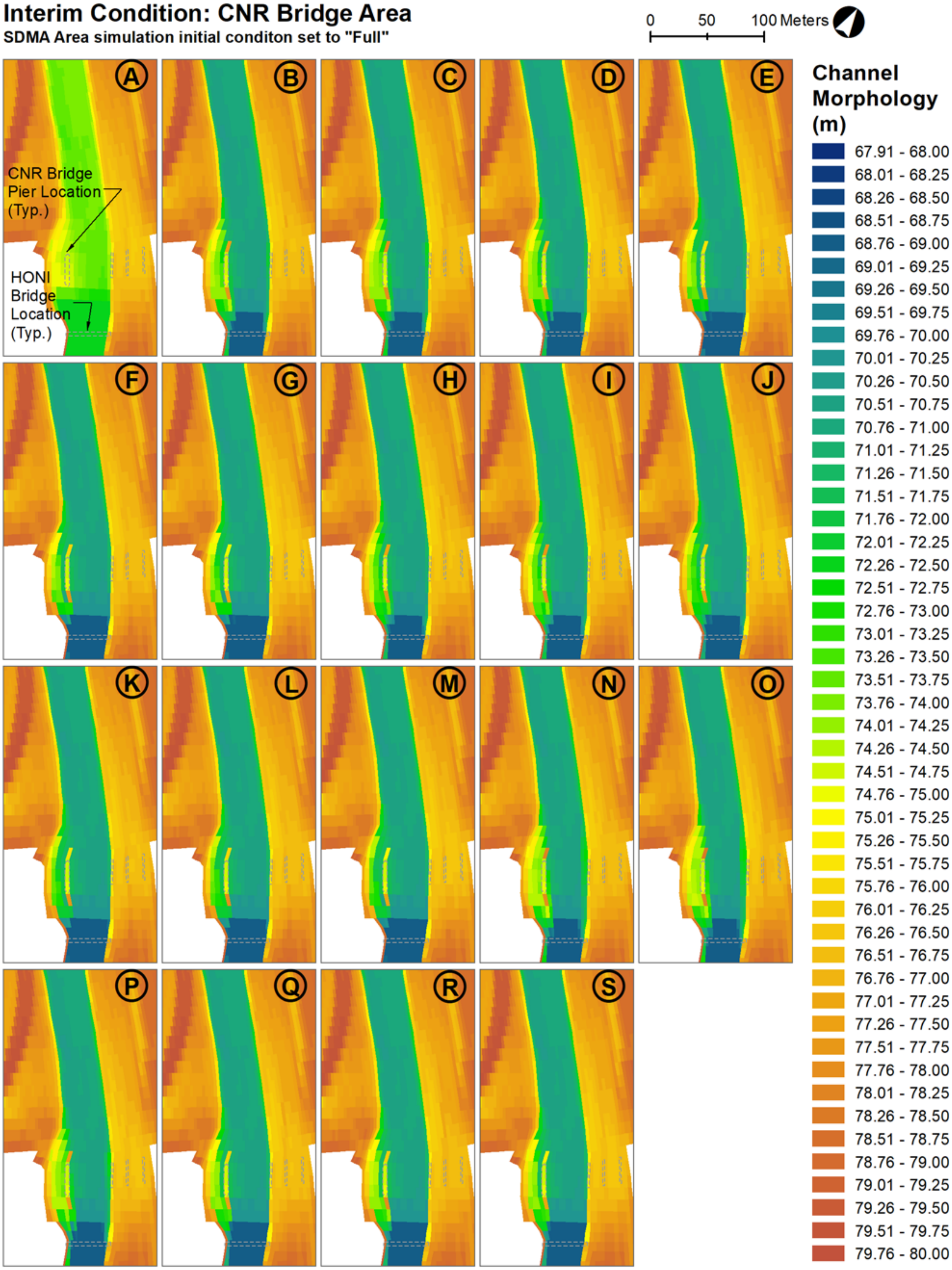


Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: CNR Bridge Area

SDMA Area simulation initial conditon set to "Design"

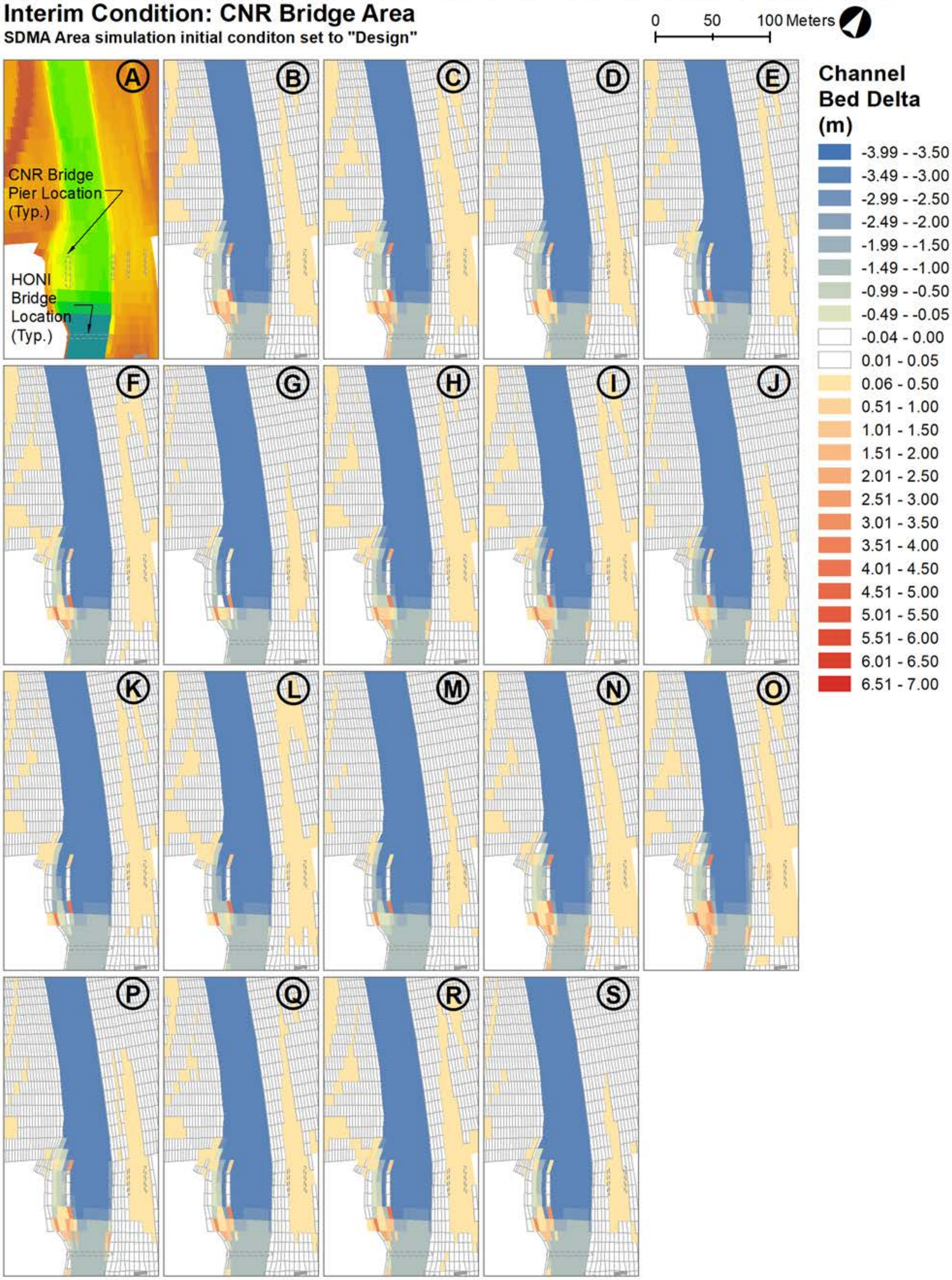


Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: CNR Bridge Area
SDMA Area simulation initial conditon set to "Full"



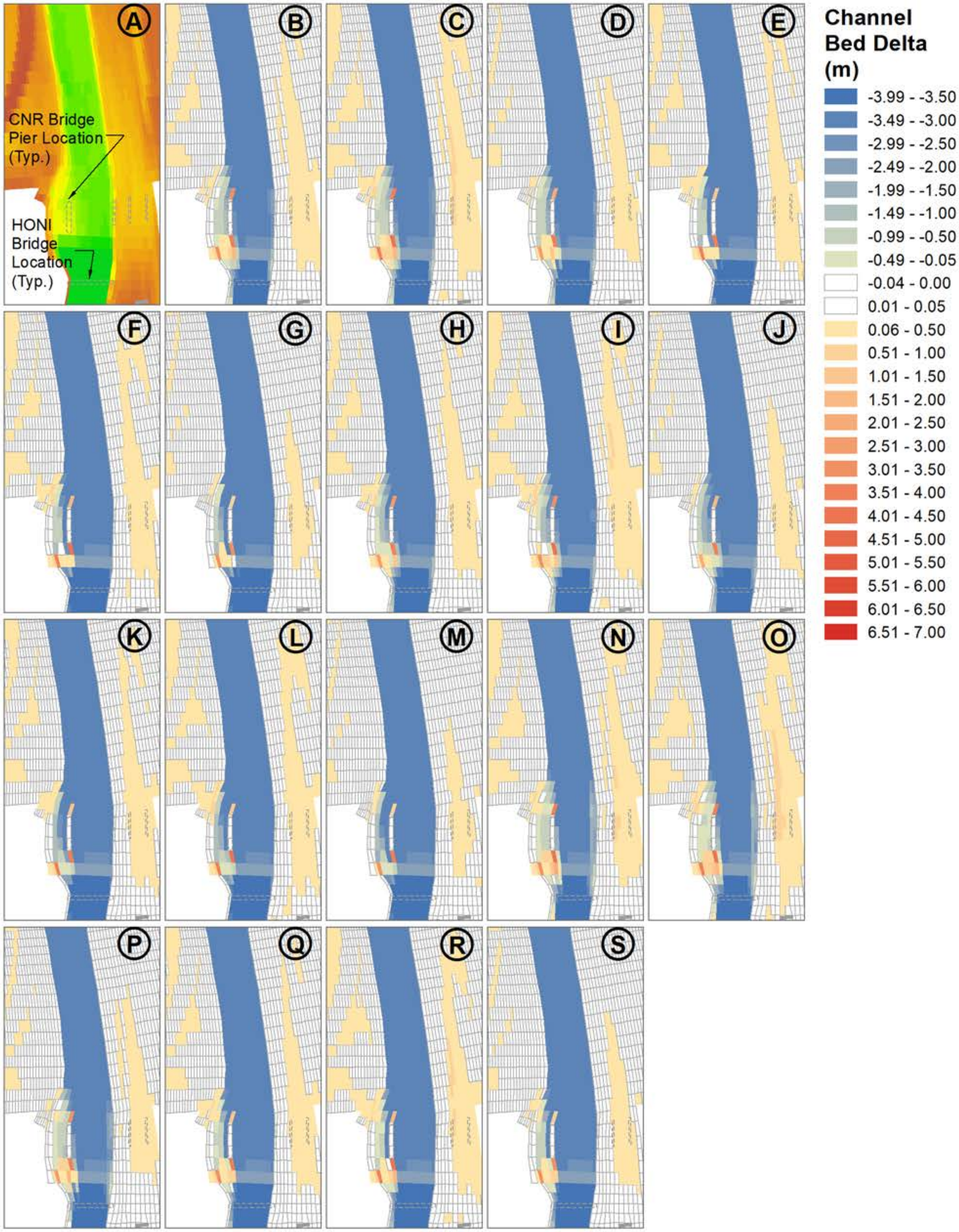
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: CNR Bridge Area

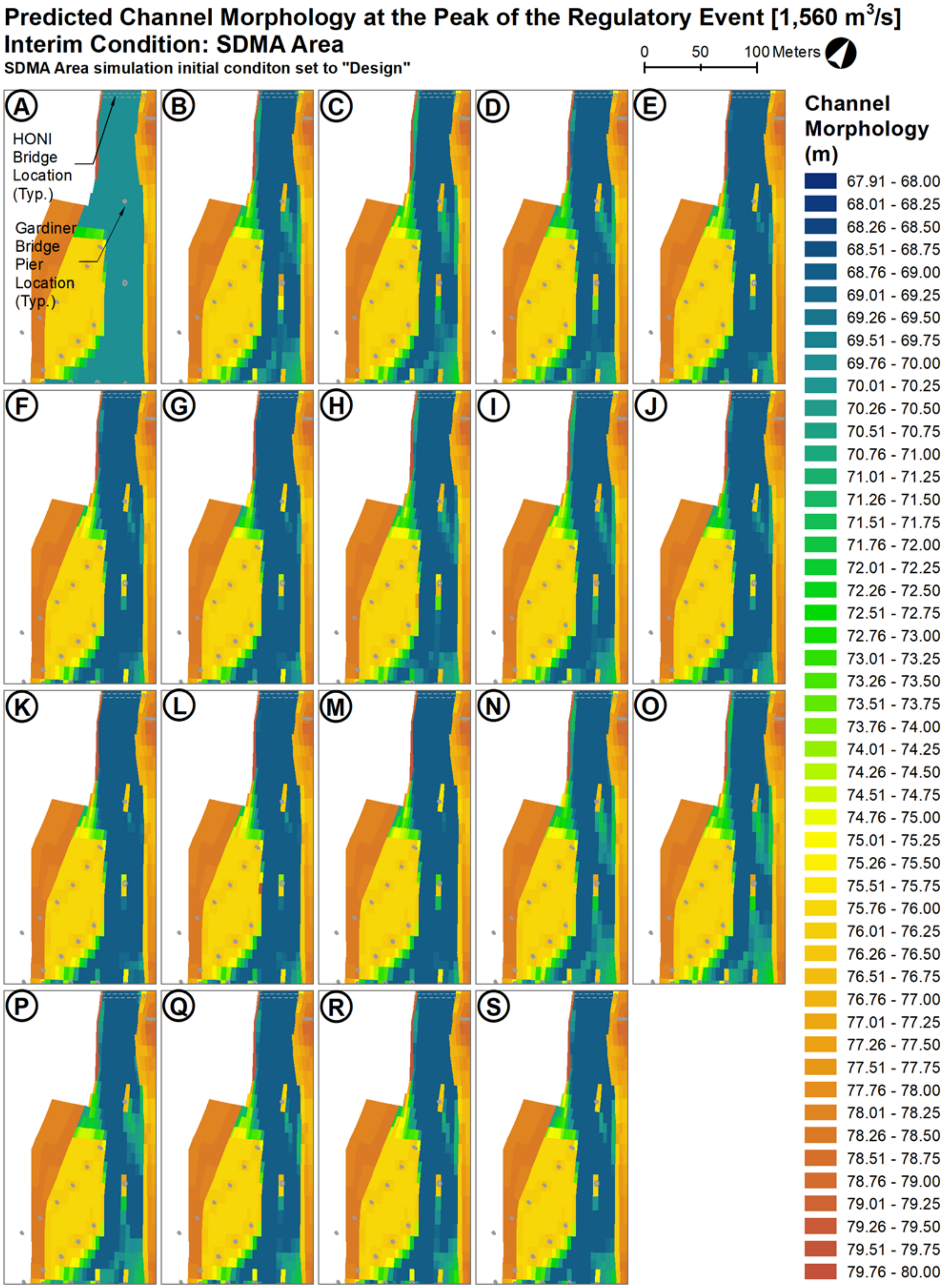
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: CNR Bridge Area

SDMA Area simulation initial conditon set to "Full"



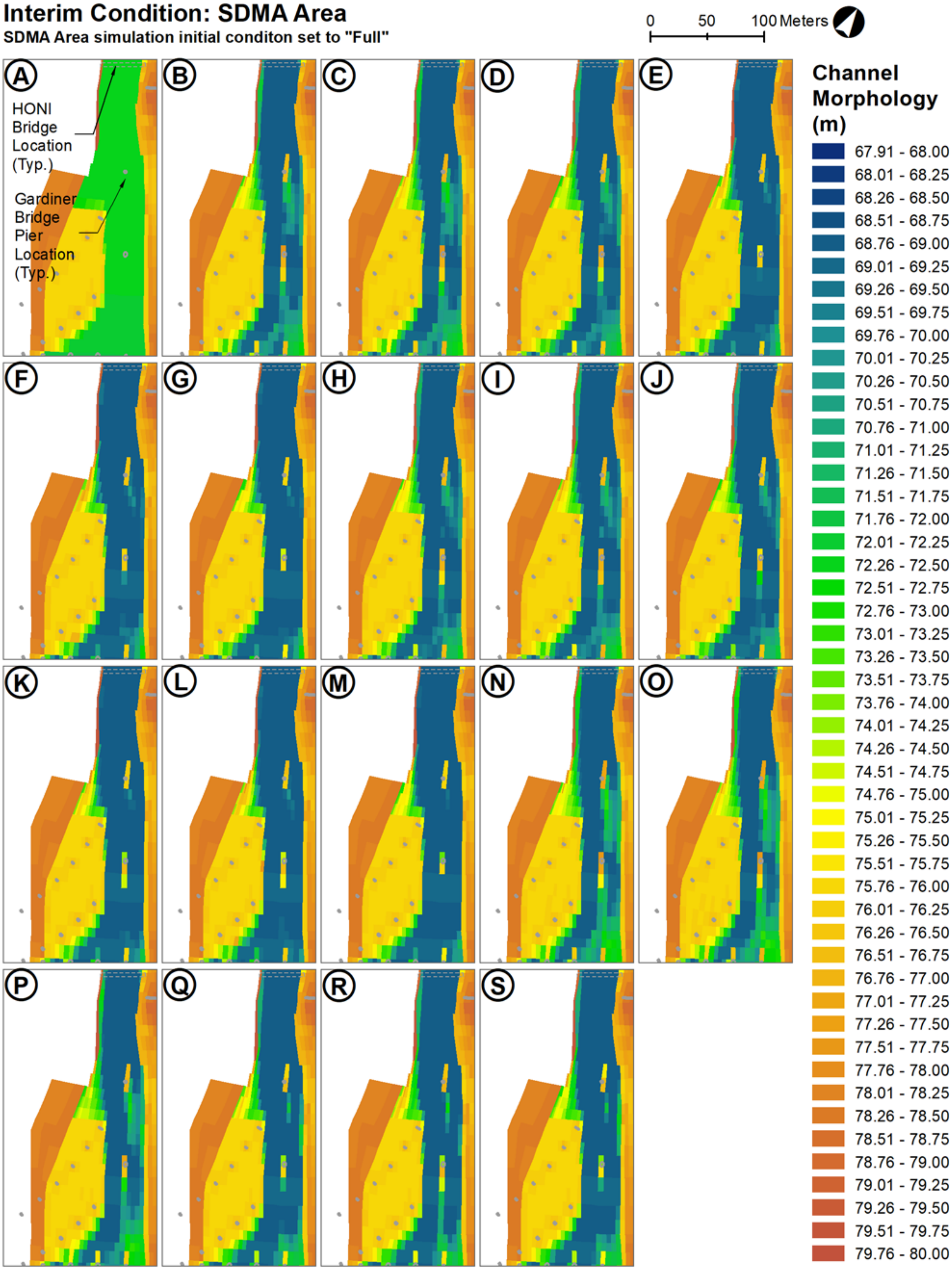


Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m³/s]
Interim Condition: SDMA Area
SDMA Area simulation initial conditon set to "Design"
0
50
100 Meters
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O
P
Q
R
S
HONI Bridge Location (Typ.)
Gardiner Bridge Pier Location (Typ.)
Channel Morphology (m)
67.91 - 68.00
68.01 - 68.25
68.26 - 68.50
68.51 - 68.75
68.76 - 69.00
69.01 - 69.25
69.26 - 69.50
69.51 - 69.75
69.76 - 70.00
70.01 - 70.25
70.26 - 70.50
70.51 - 70.75
70.76 - 71.00
71.01 - 71.25
71.26 - 71.50
71.51 - 71.75
71.76 - 72.00
72.01 - 72.25
72.26 - 72.50
72.51 - 72.75
72.76 - 73.00
73.01 - 73.25
73.26 - 73.50
73.51 - 73.75
73.76 - 74.00
74.01 - 74.25
74.26 - 74.50
74.51 - 74.75
74.76 - 75.00
75.01 - 75.25
75.26 - 75.50
75.51 - 75.75
75.76 - 76.00
76.01 - 76.25
76.26 - 76.50
76.51 - 76.75
76.76 - 77.00
77.01 - 77.25
77.26 - 77.50
77.51 - 77.75
77.76 - 78.00
78.01 - 78.25
78.26 - 78.50
78.51 - 78.75
78.76 - 79.00
79.01 - 79.25
79.26 - 79.50
79.51 - 79.75
79.76 - 80.00

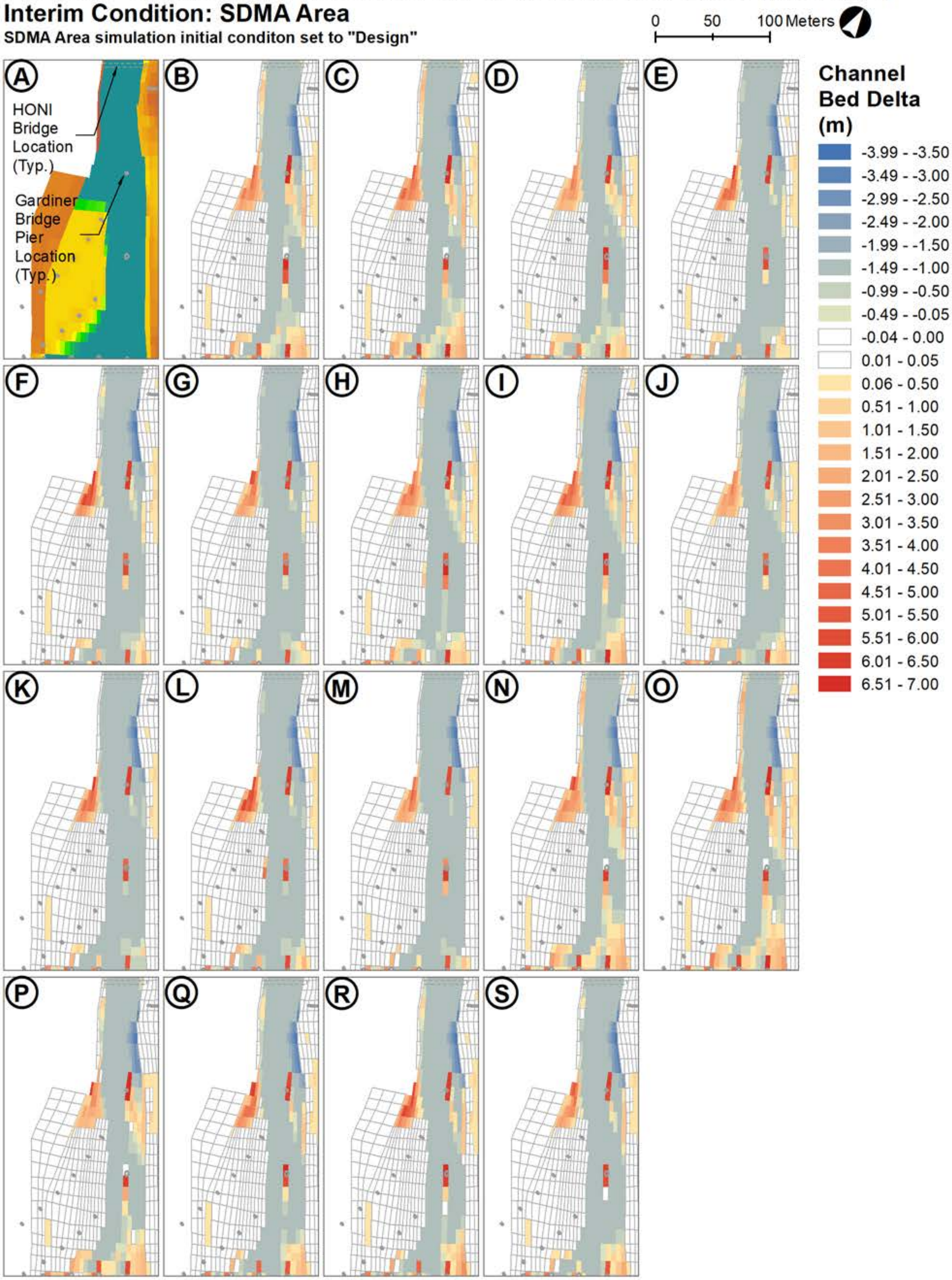
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

Interim Condition: SDMA Area

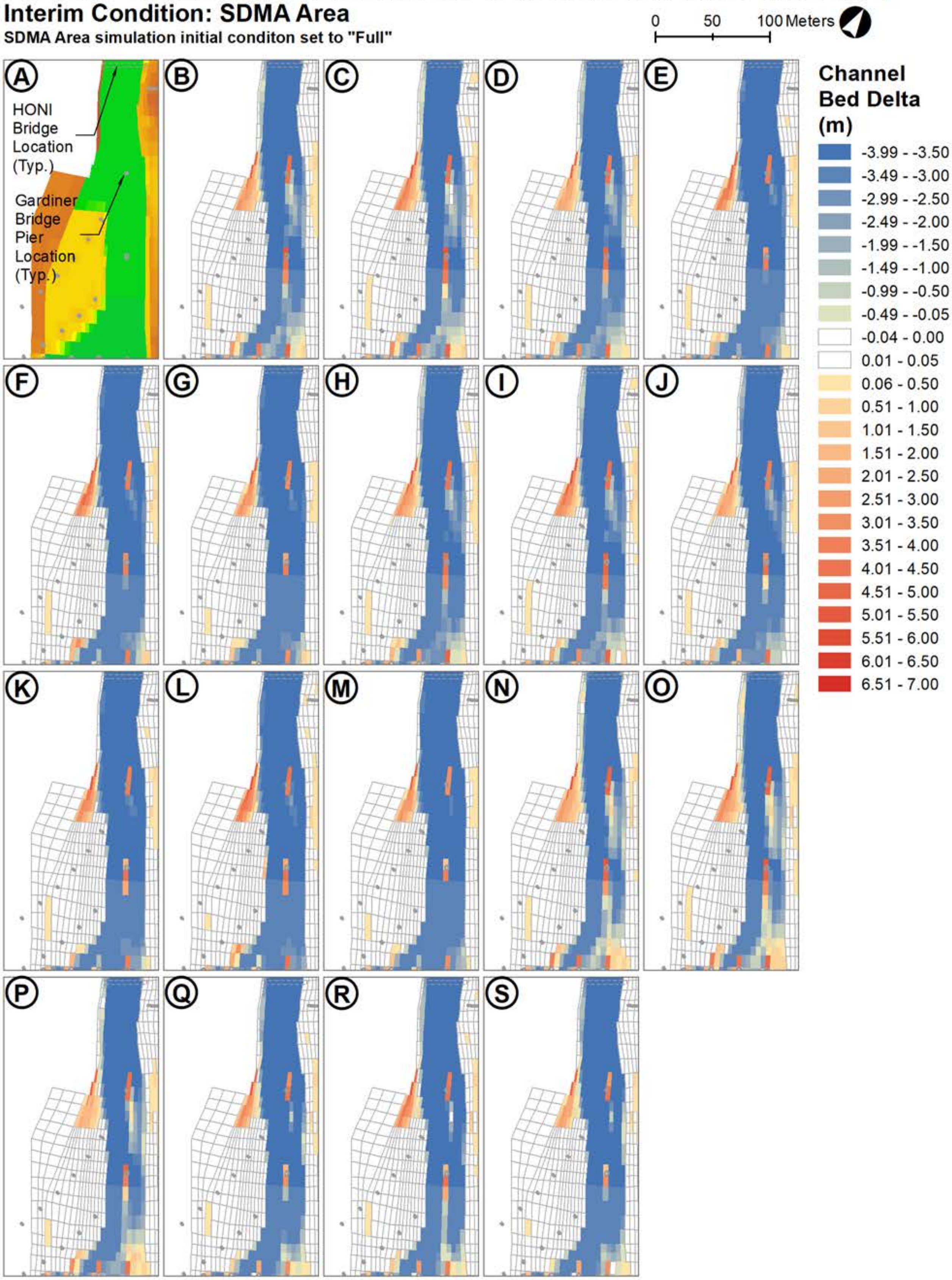
SDMA Area simulation initial conditon set to "Full"



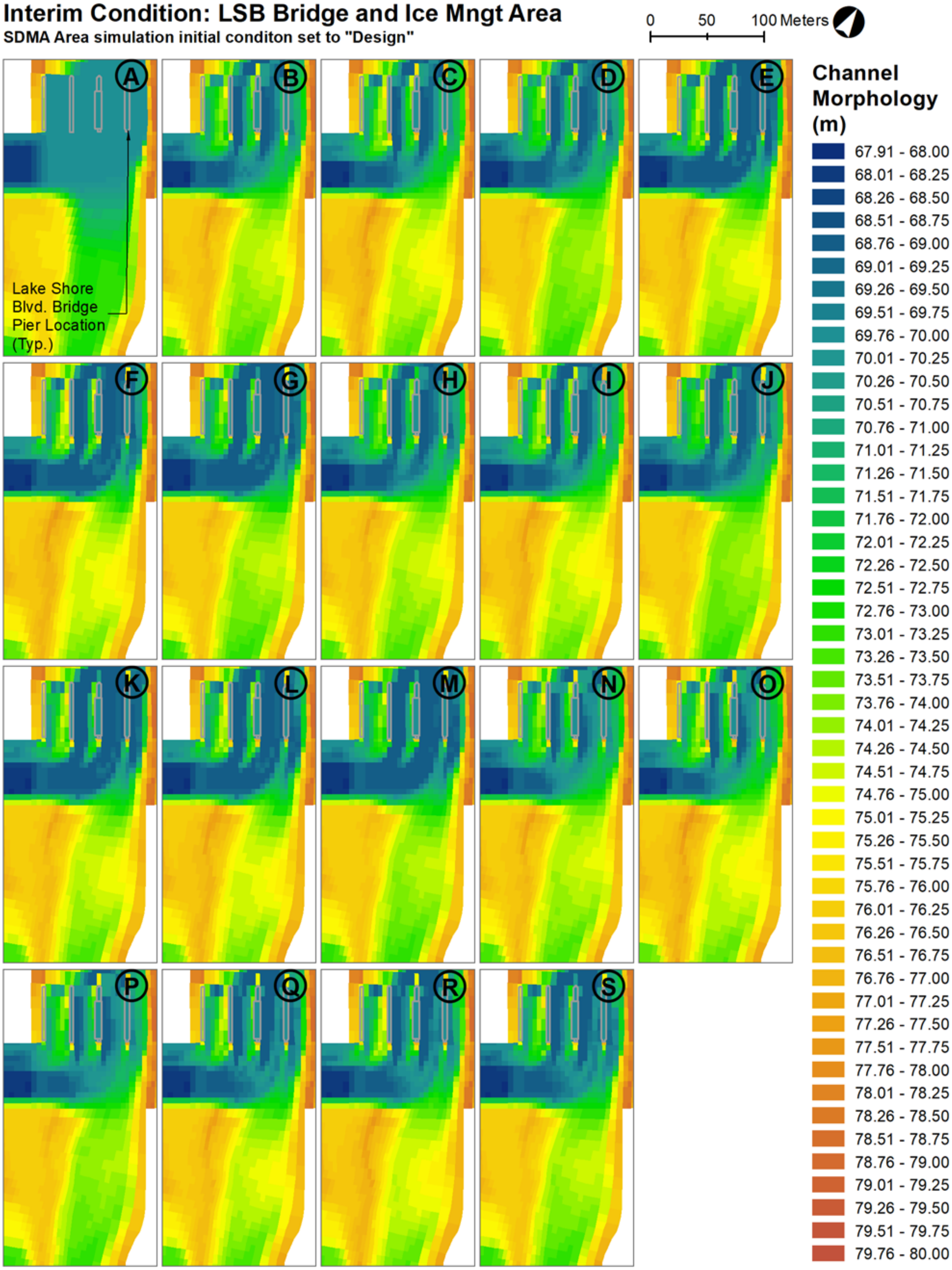
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: SDMA Area
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m³/s]
Interim Condition: SDMA Area
SDMA Area simulation initial conditon set to "Full"



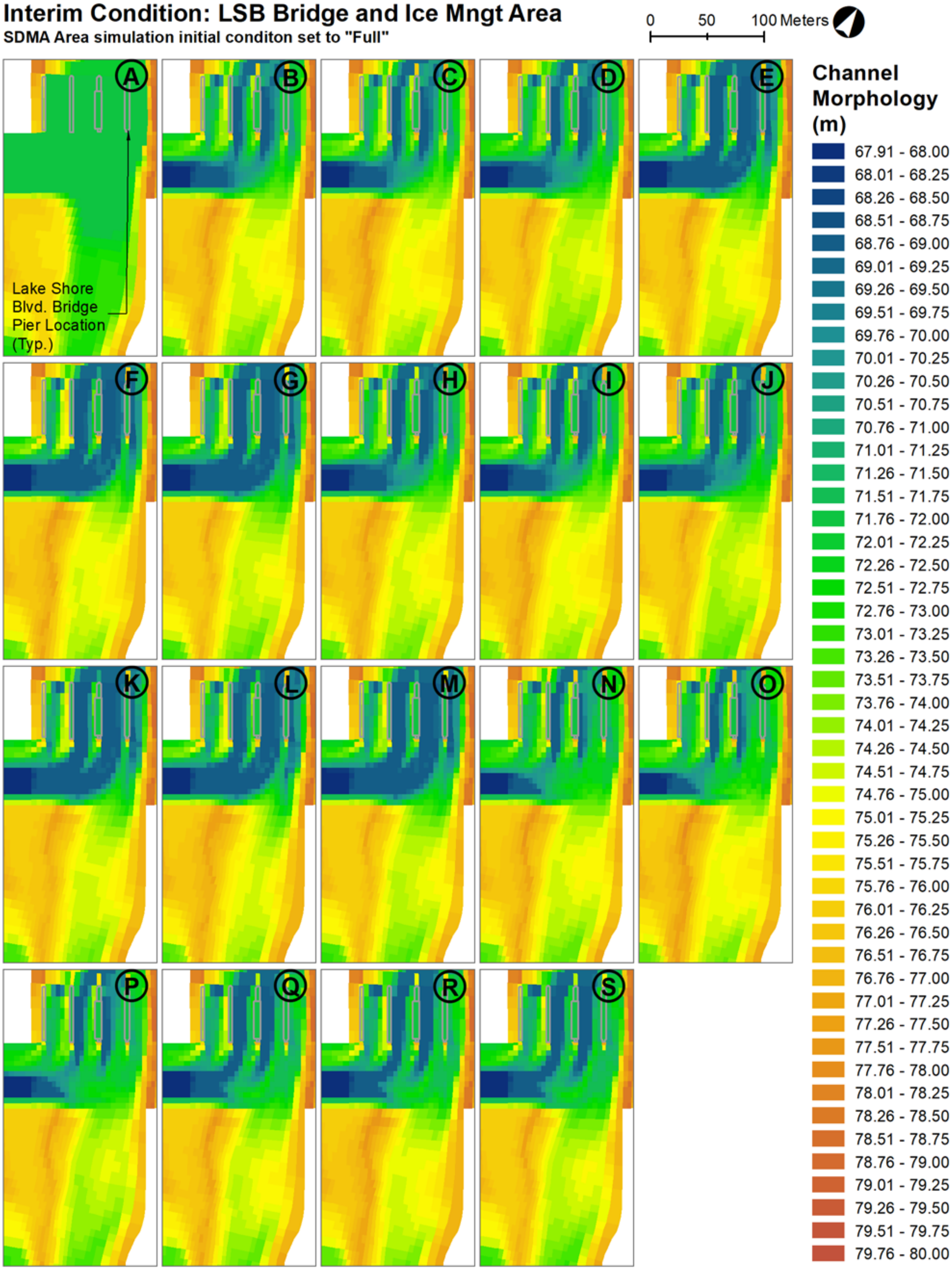
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: LSB Bridge and Ice Mngt Area
SDMA Area simulation initial conditon set to "Design"



Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]

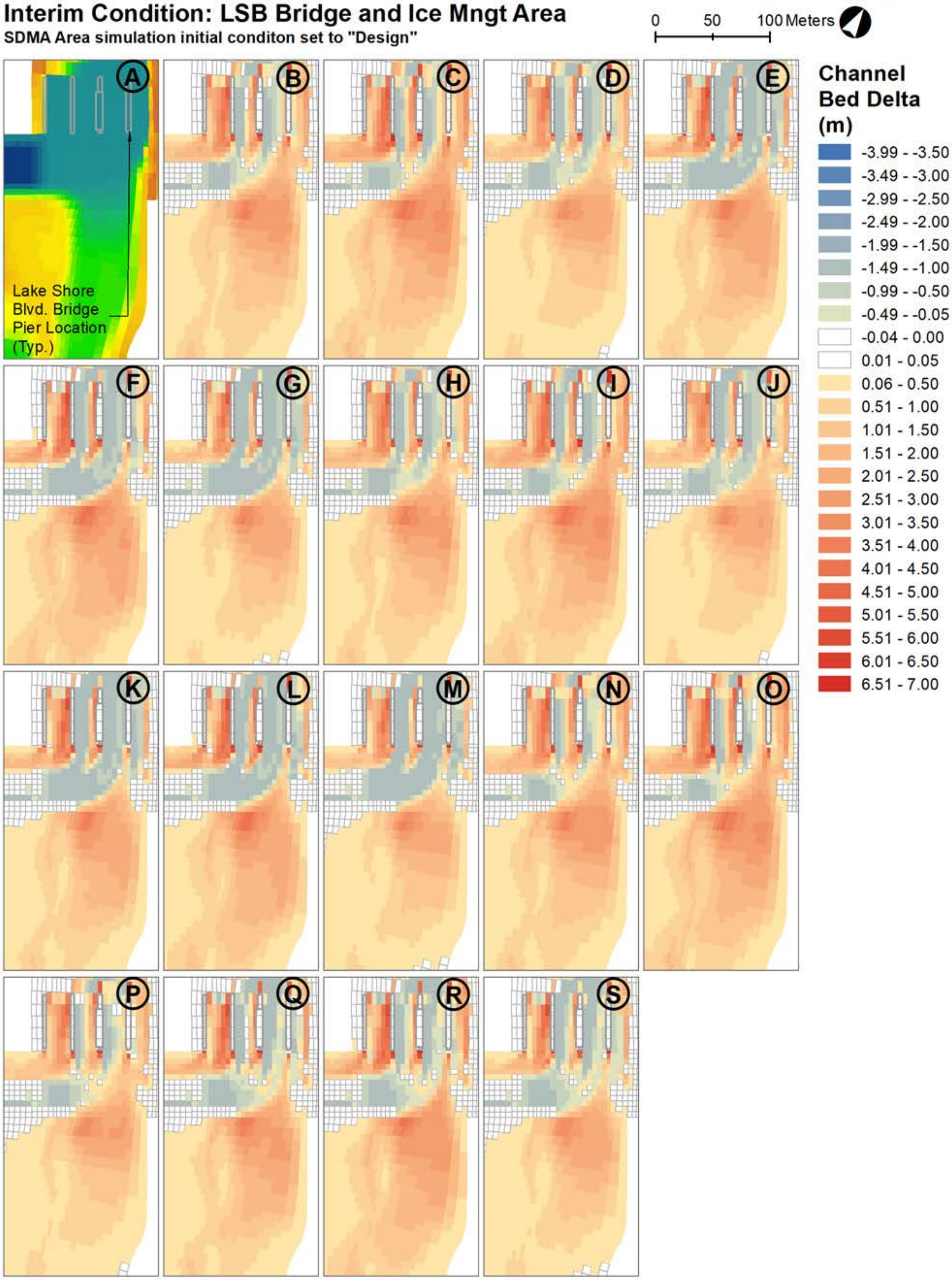
Interim Condition: LSB Bridge and Ice Mngt Area

SDMA Area simulation initial conditon set to "Full"



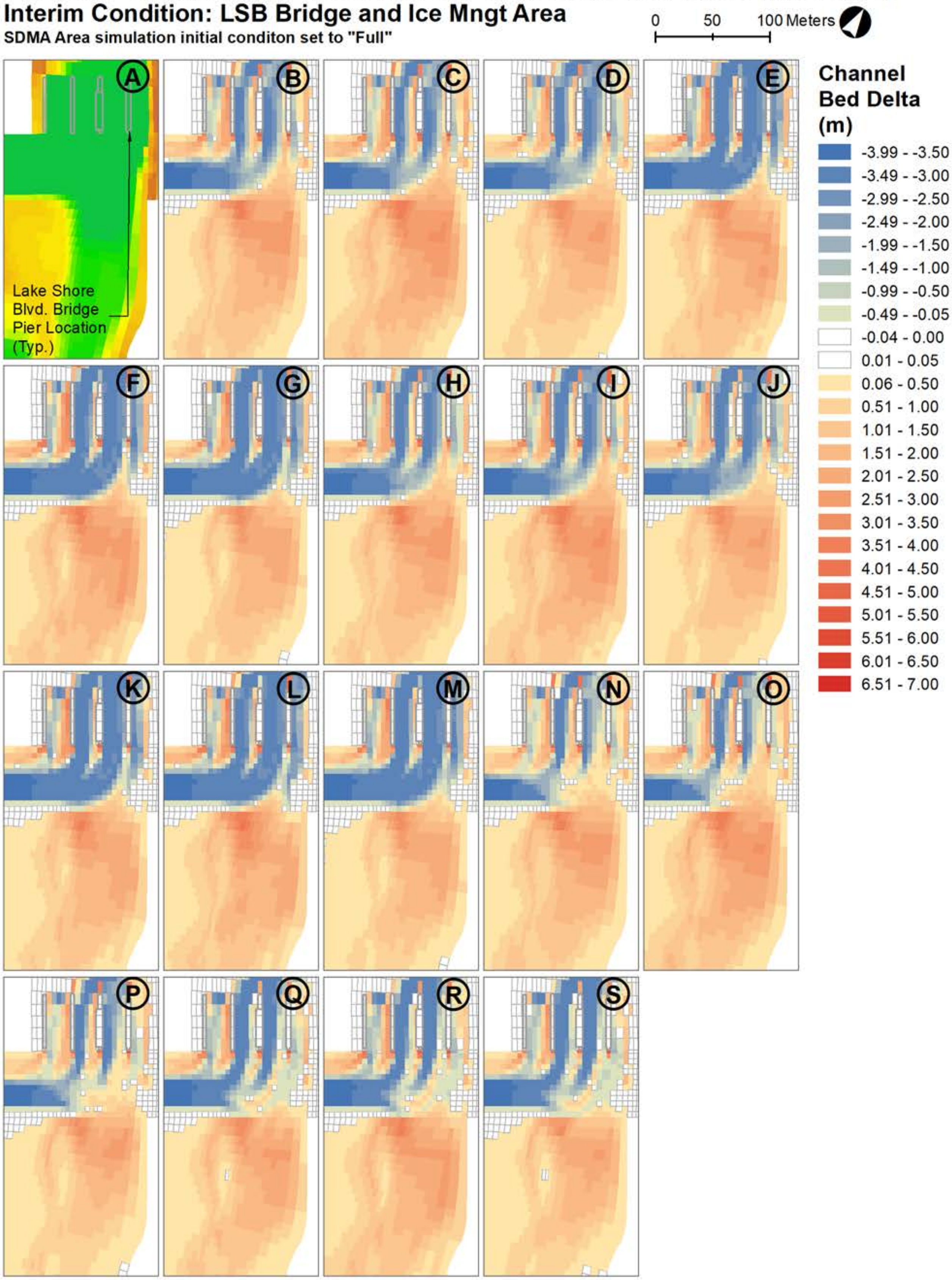
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m³/s]
Interim Condition: LSB Bridge and Ice Mngt Area

SDMA Area simulation initial conditon set to "Design"



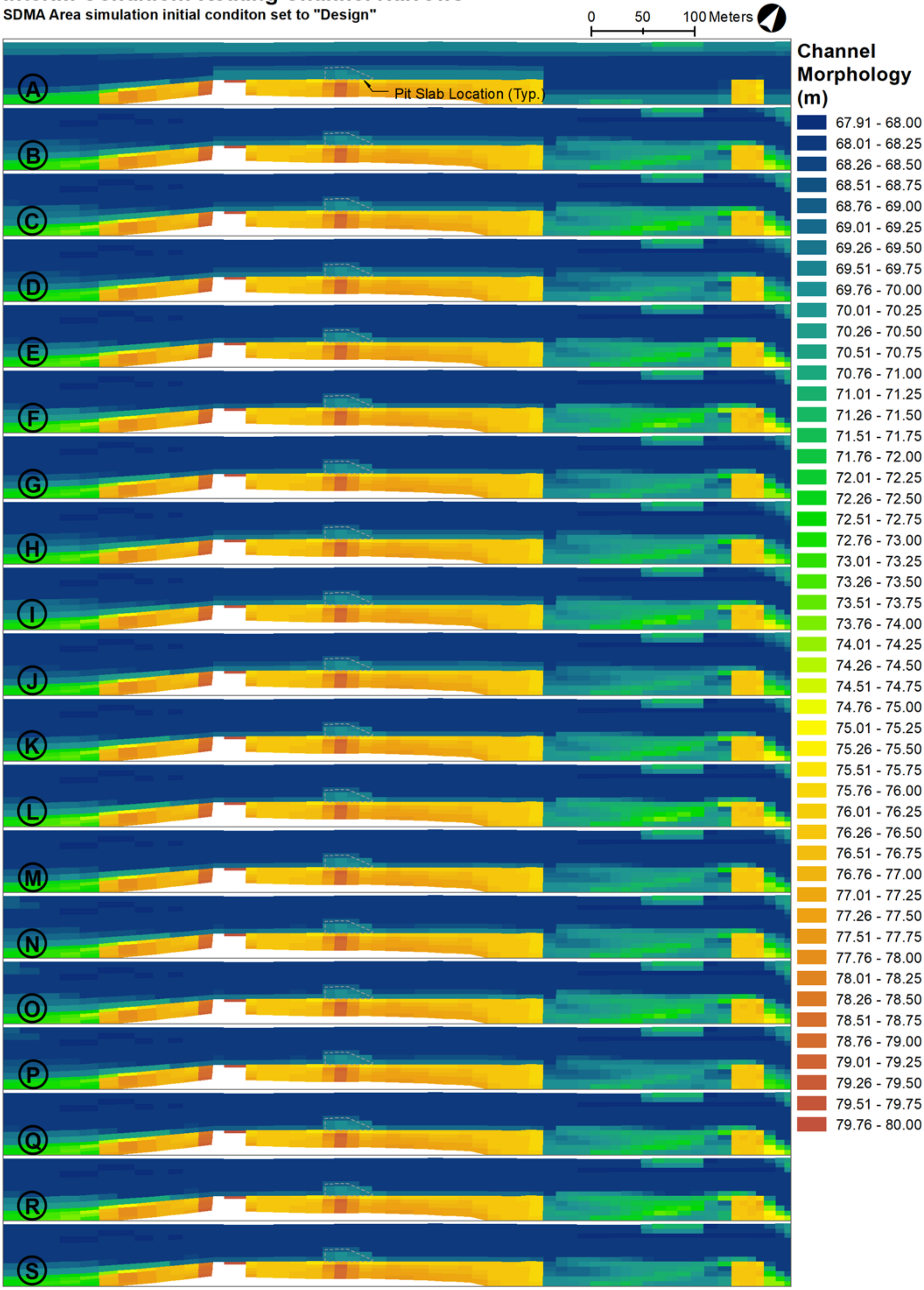
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: LSB Bridge and Ice Mngt Area

SDMA Area simulation initial conditon set to "Full"



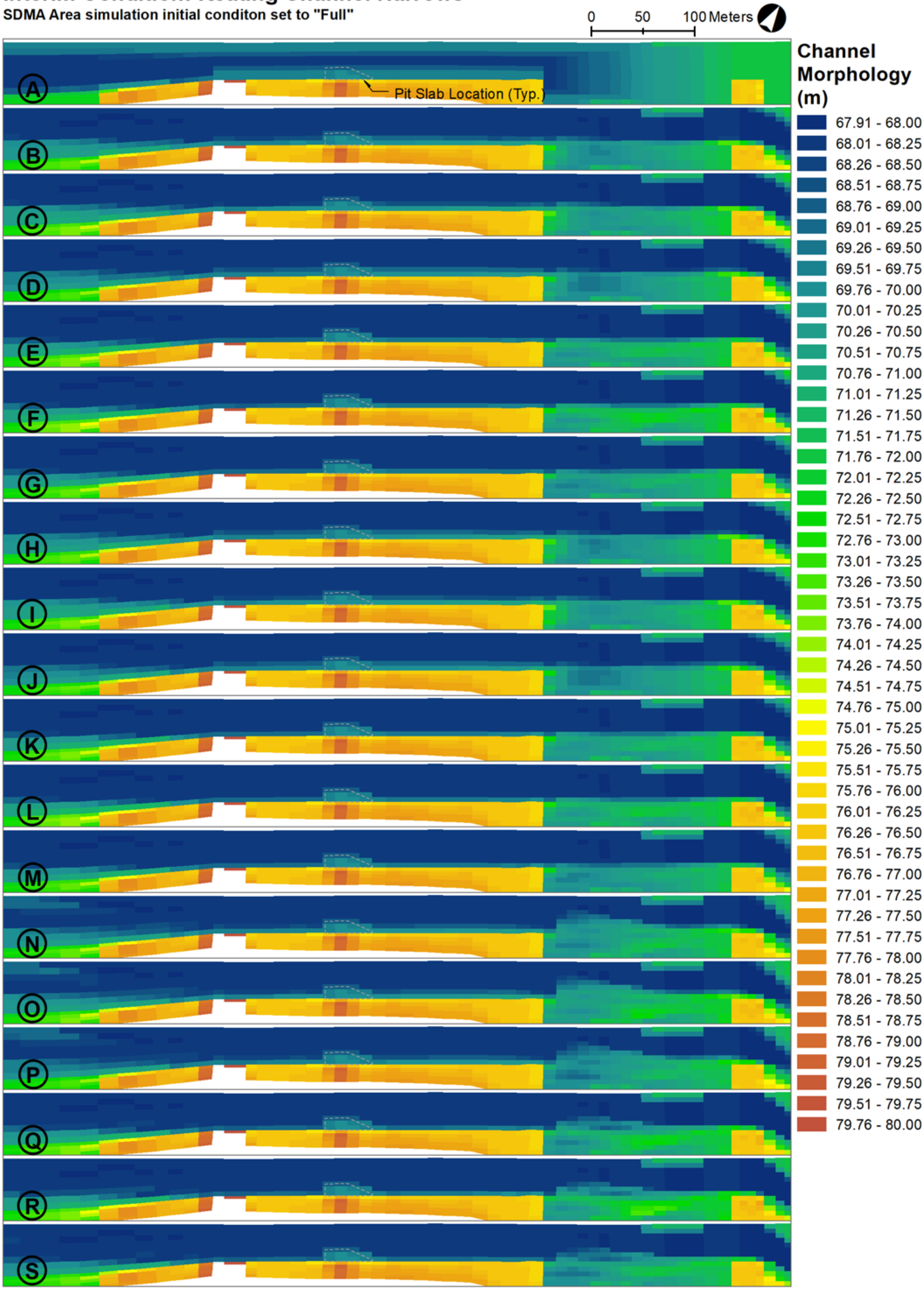
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Design"



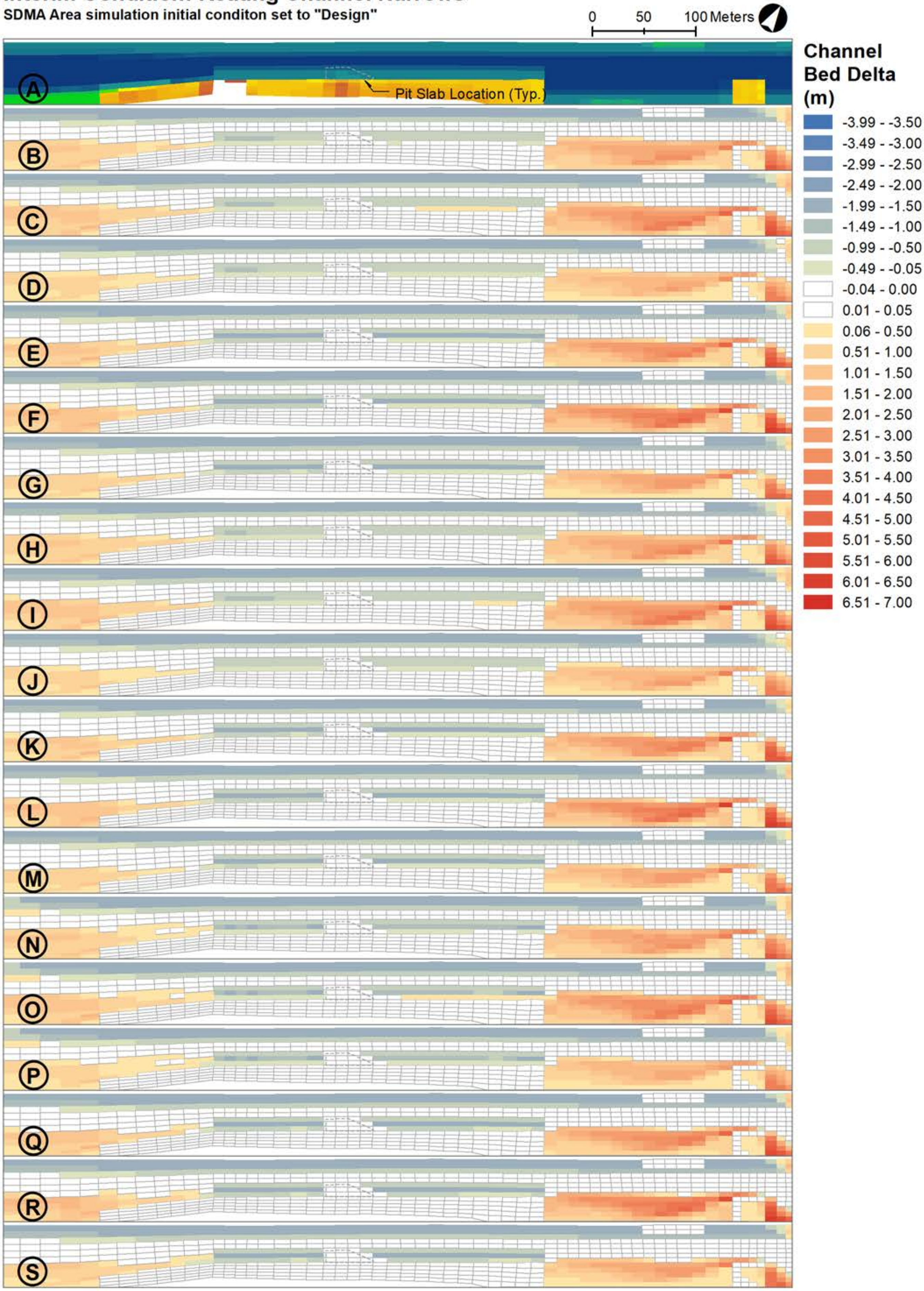
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Full"



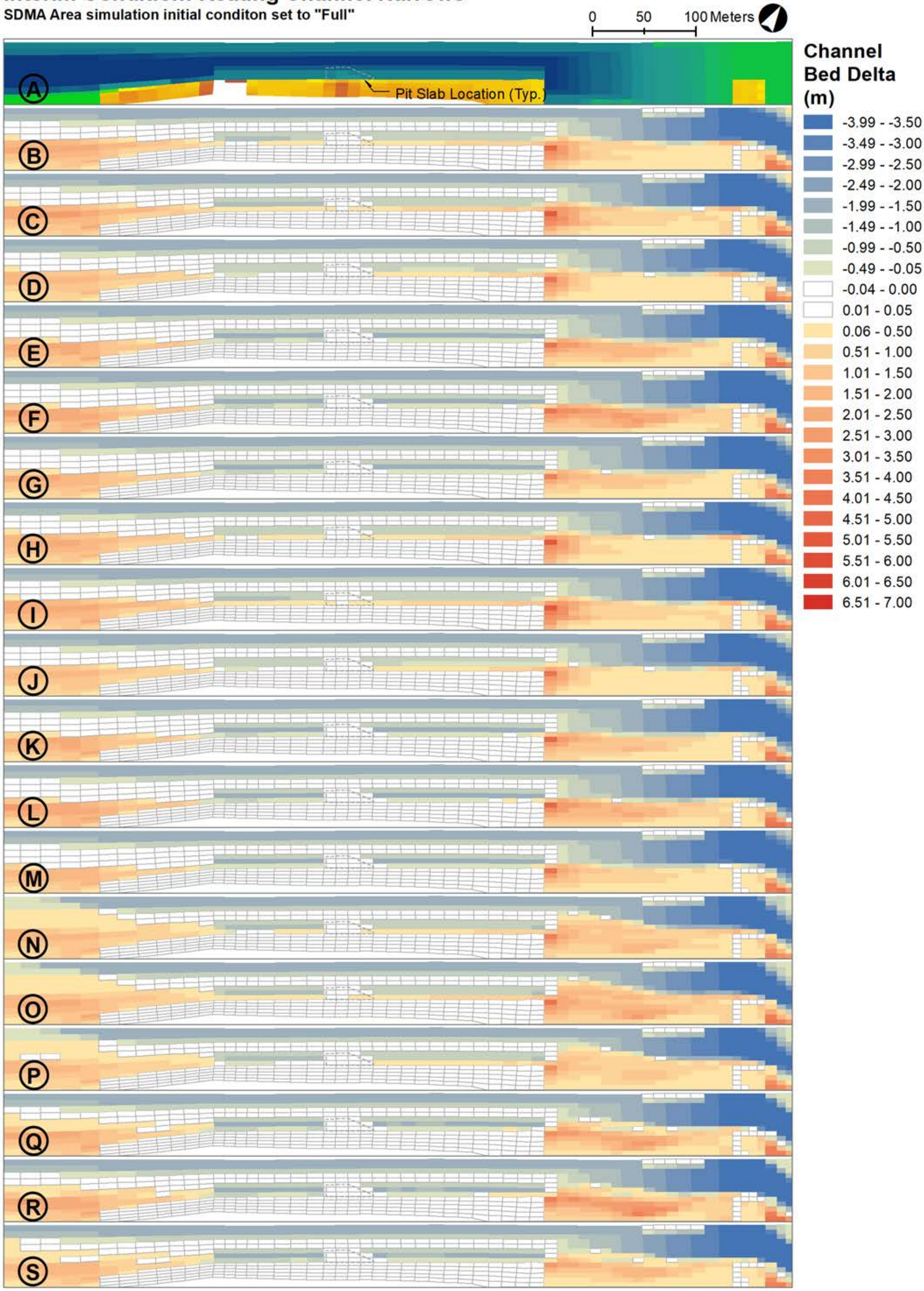
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Design"



Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Keating Channel Narrows

SDMA Area simulation initial conditon set to "Full"



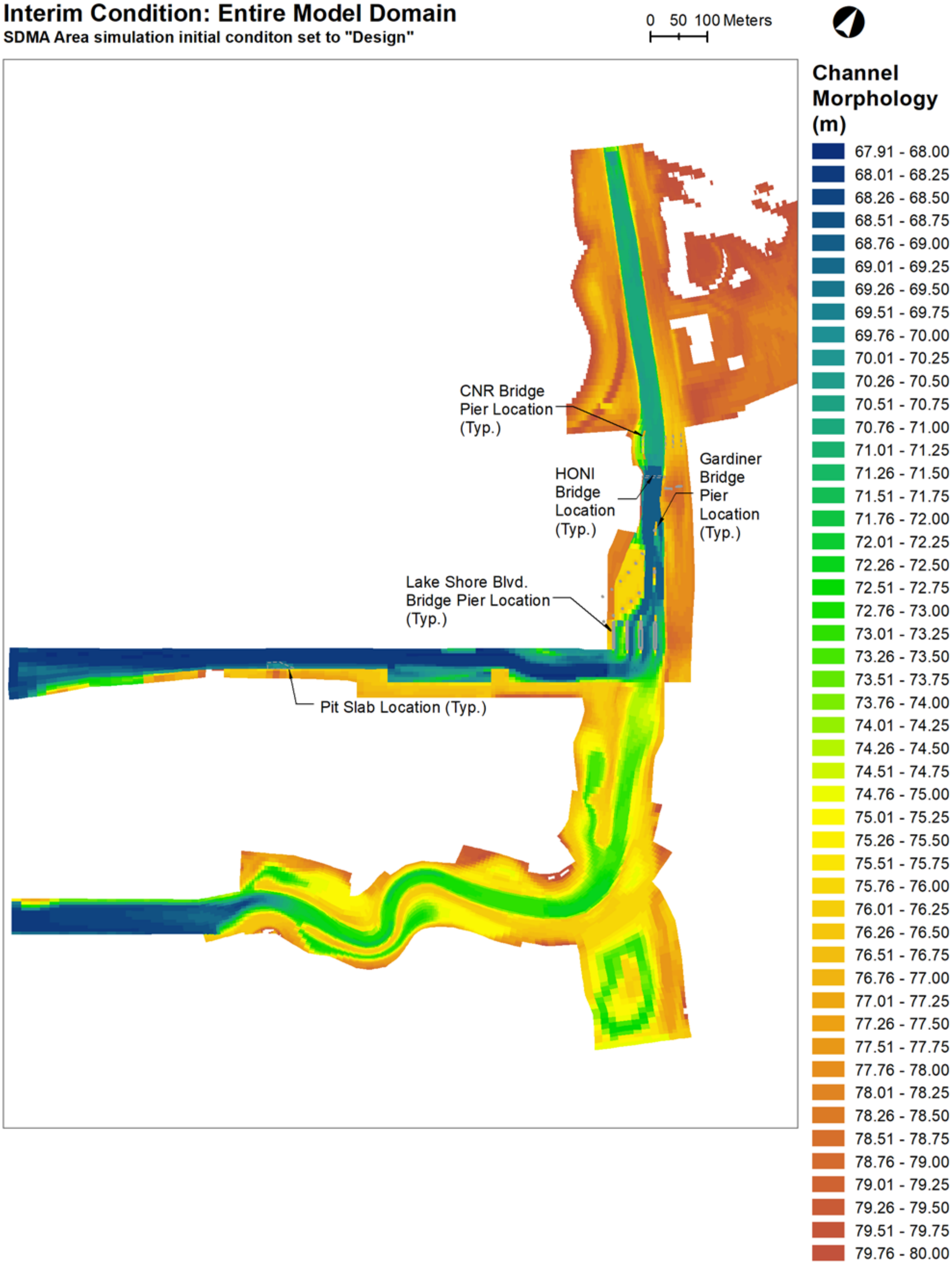
Predicted Channel Morphology & Bed Delta Plots for “Full” and “Design” Conditions

- The following plots show the entire EFDC model domain
- Only provided plots for the base alternative simulations for “Design” condition and “Full” condition (which corresponds to Plot ID “B” for the previous plots)

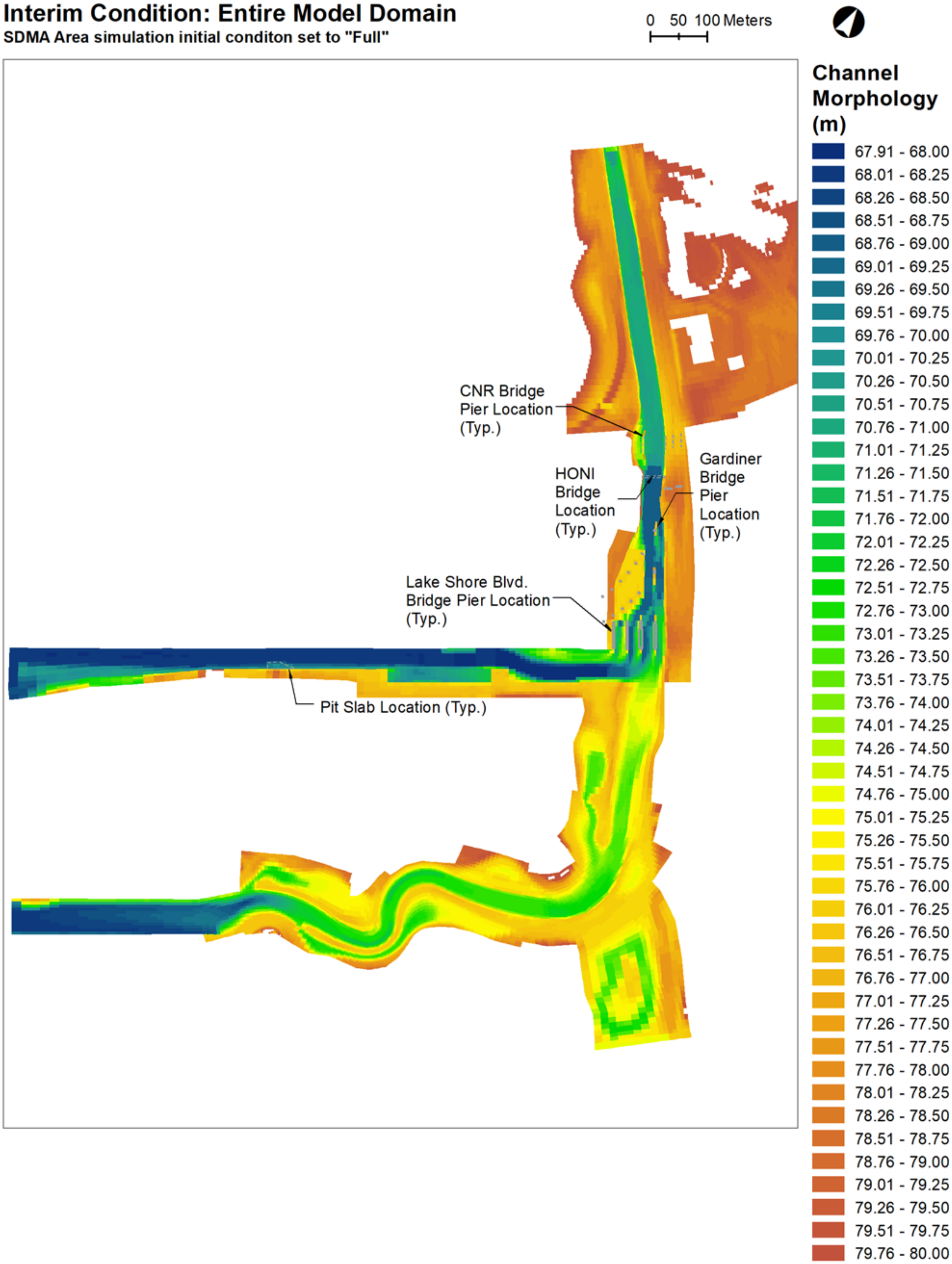
Plot ID	Bed Composition Alternatives	Erosion Rate Alternatives	US Load Alternatives
A	Initial Condition ("Full" or "Design")		
B	Base Particle Size Distribution	Erosion Rate 1x	US Load 1x
C	Base Particle Size Distribution	Erosion Rate 1x	US Load 1.5x
D	Base Particle Size Distribution	Erosion Rate 1x	US Load 0.5x
E	Base Particle Size Distribution	Erosion Rate 3x	US Load 1x
F	Base Particle Size Distribution	Erosion Rate 3x	US Load 1.5x
G	Base Particle Size Distribution	Erosion Rate 3x	US Load 0.5x
H	Decreased D50	Erosion Rate 1x	US Load 1x
I	Decreased D50	Erosion Rate 1x	US Load 1.5x
J	Decreased D50	Erosion Rate 1x	US Load 0.5x
K	Decreased D50	Erosion Rate 3x	US Load 1x
L	Decreased D50	Erosion Rate 3x	US Load 1.5x
M	Decreased D50	Erosion Rate 3x	US Load 0.5x
N	Increase D50	Erosion Rate 1x	US Load 1x
O	Increase D50	Erosion Rate 1x	US Load 1.5x
P	Increase D50	Erosion Rate 1x	US Load 0.5x
Q	Increase D50	Erosion Rate 3x	US Load 1x
R	Increase D50	Erosion Rate 3x	US Load 1.5x
S	Increase D50	Erosion Rate 3x	US Load 0.5x

Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Entire Model Domain

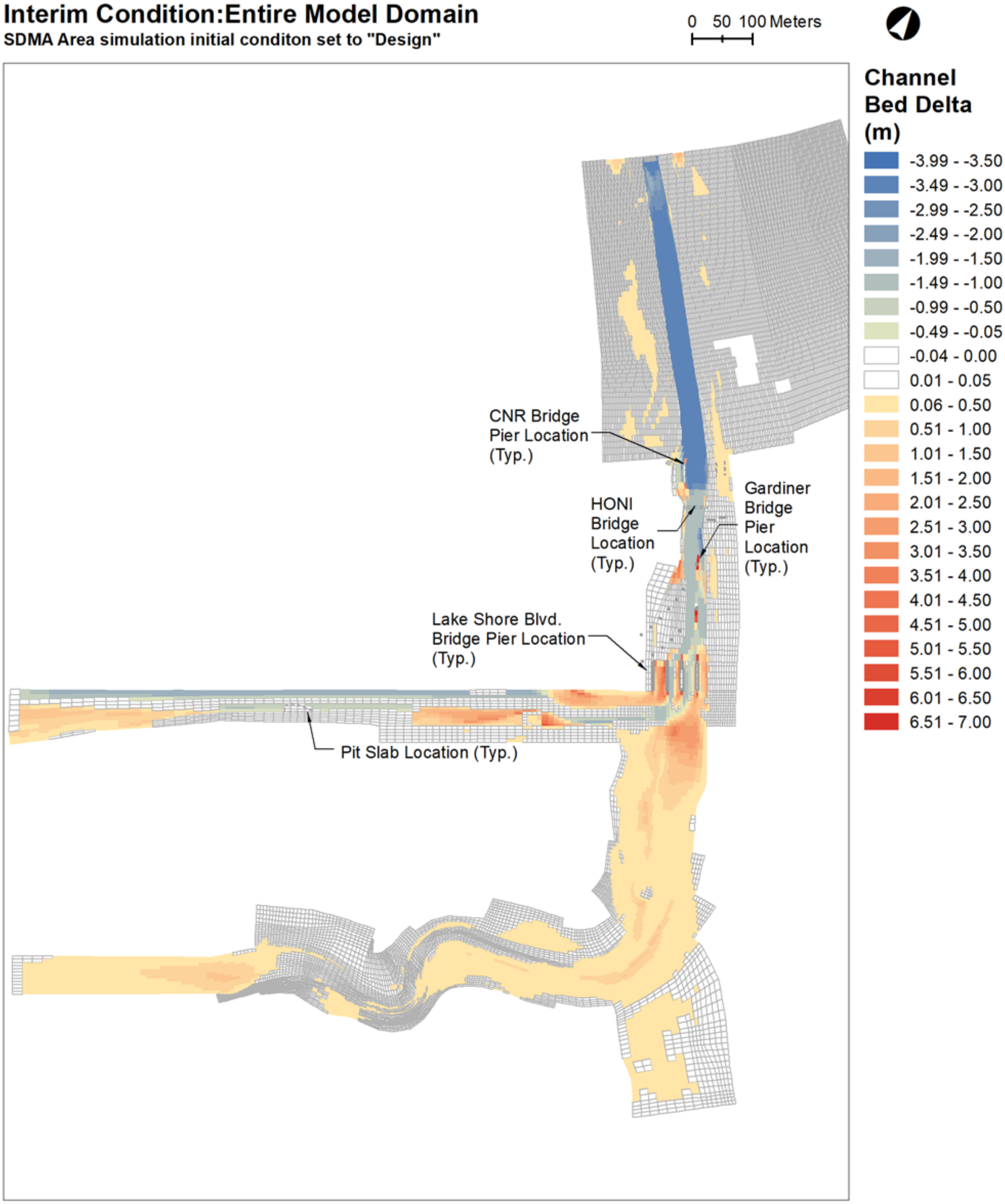
SDMA Area simulation initial conditon set to "Design"



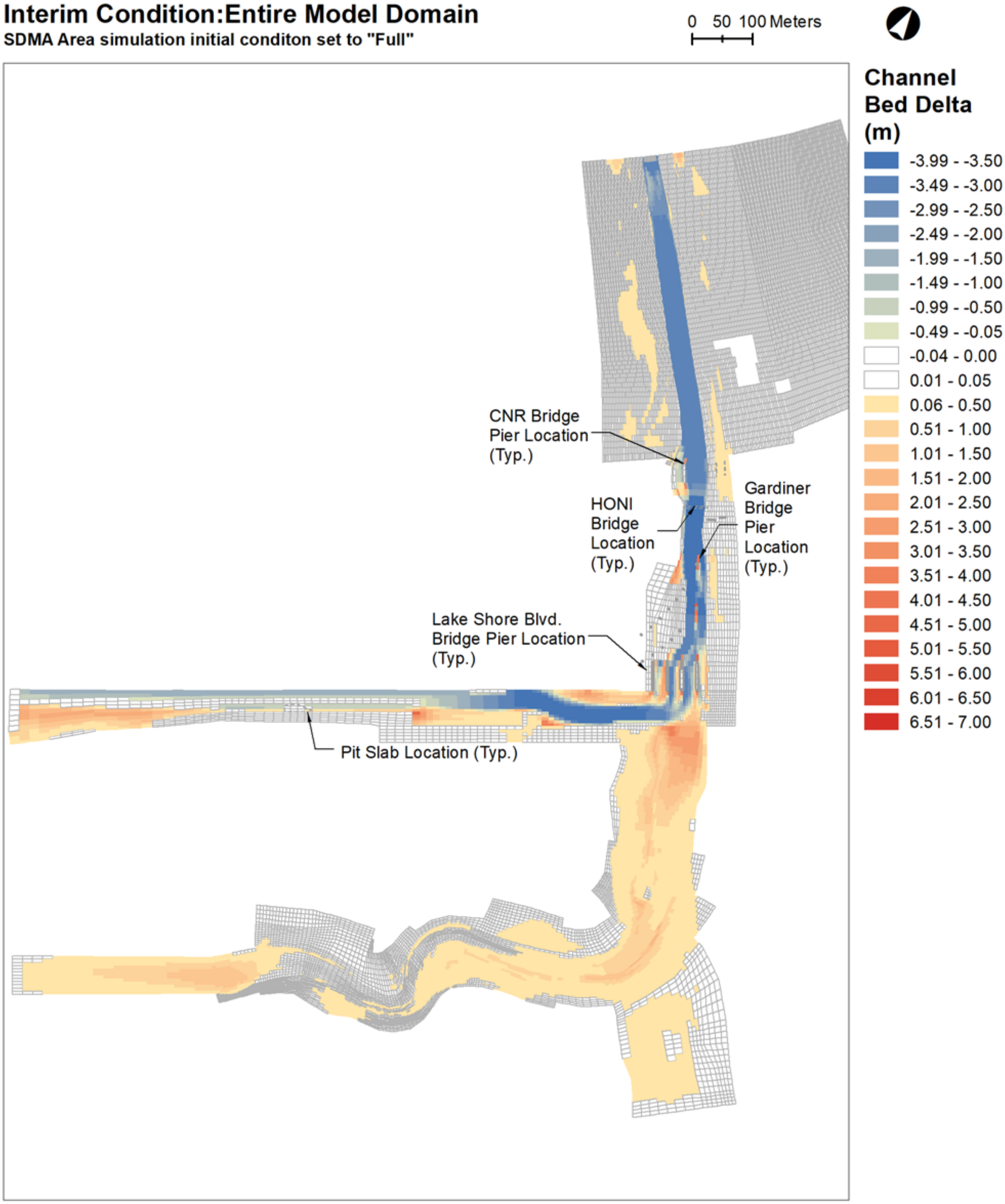
Predicted Channel Morphology at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition: Entire Model Domain
SDMA Area simulation initial conditon set to "Full"



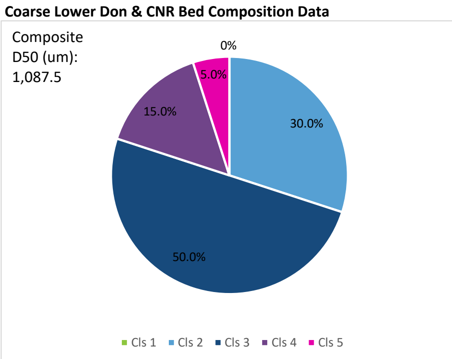
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition:Entire Model Domain
SDMA Area simulation initial conditon set to "Design"



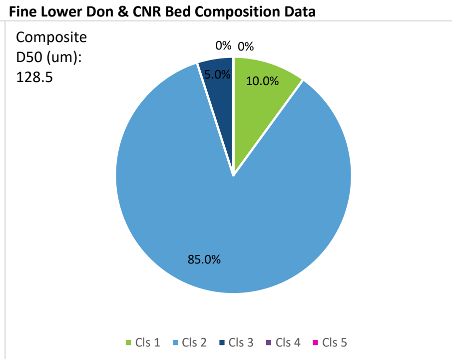
Predicted Channel Bed Delta at the Peak of the Regulatory Event [1,560 m^3/s]
Interim Condition:Entire Model Domain
SDMA Area simulation initial conditon set to "Full"



Coarse Lower Don & CNR Bed Composition Data

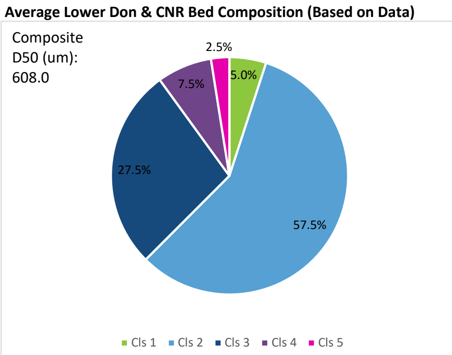


Fine Lower Don & CNR Bed Composition Data

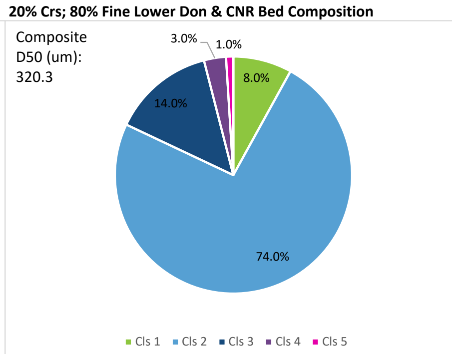


Note: The Coarse and Fine data core in this row was used to develop the Average, and Weighted Average cores in the next row which were used in the sediment transport model.

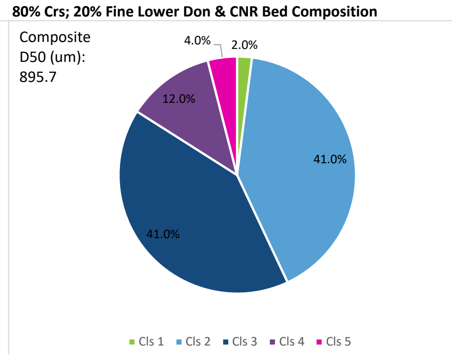
Average Lower Don & CNR Bed Composition (Based on Data)



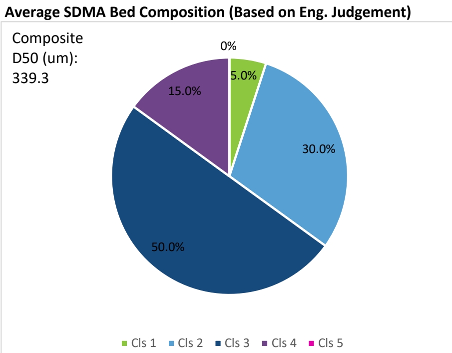
20% Crs; 80% Fine Lower Don & CNR Bed Composition



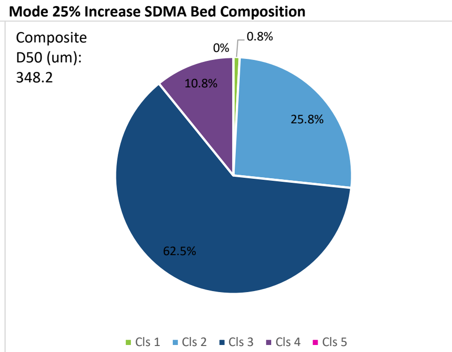
80% Crs; 20% Fine Lower Don & CNR Bed Composition



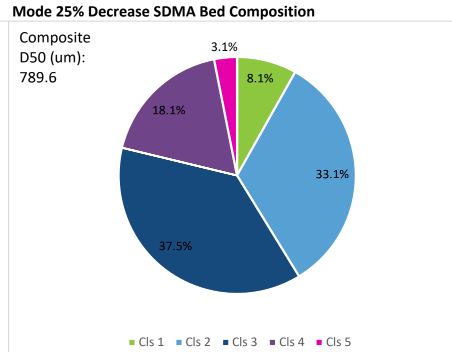
Average SDMA Bed Composition (Based on Eng. Judgement)



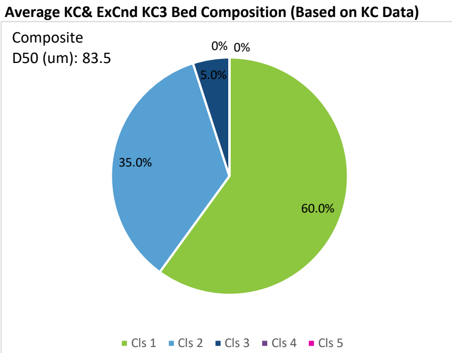
Mode 25% Increase SDMA Bed Composition



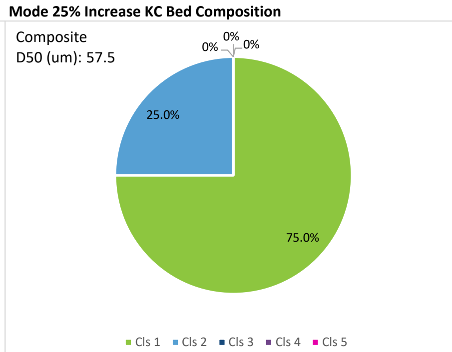
Mode 25% Decrease SDMA Bed Composition



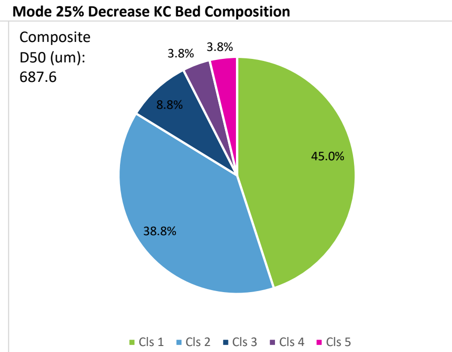
Average KC& ExCnd KC3 Bed Composition (Based on KC Data)



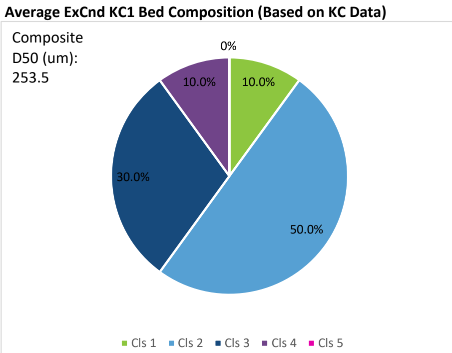
Mode 25% Increase KC Bed Composition



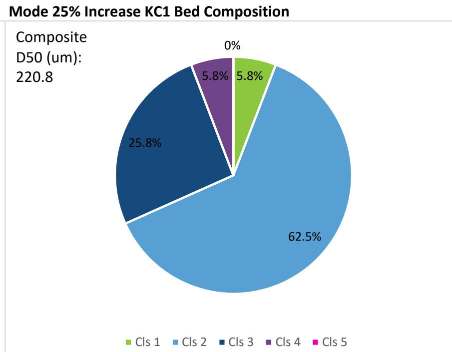
Mode 25% Decrease KC Bed Composition



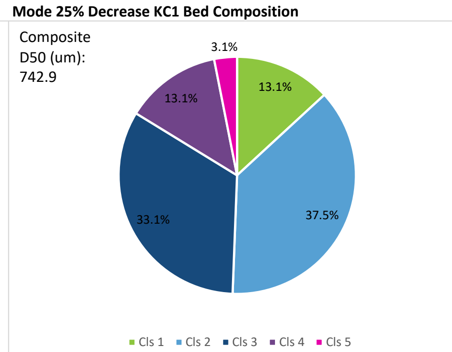
Average ExCnd KC1 Bed Composition (Based on KC Data)



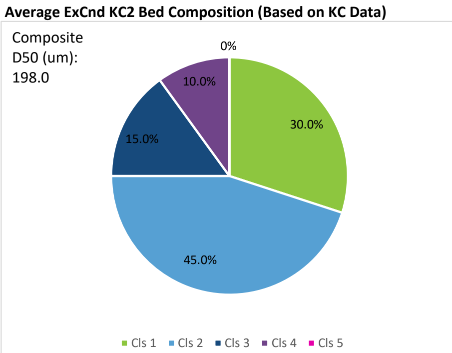
Mode 25% Increase KC1 Bed Composition



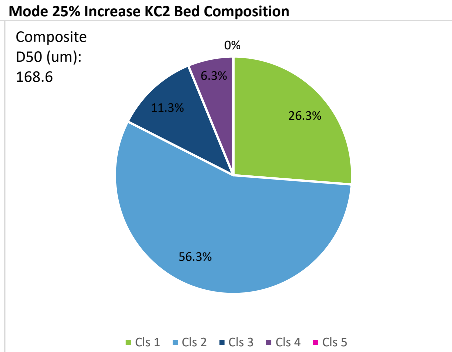
Mode 25% Decrease KC1 Bed Composition



Average ExCnd KC2 Bed Composition (Based on KC Data)



Mode 25% Increase KC2 Bed Composition



Mode 25% Decrease KC2 Bed Composition

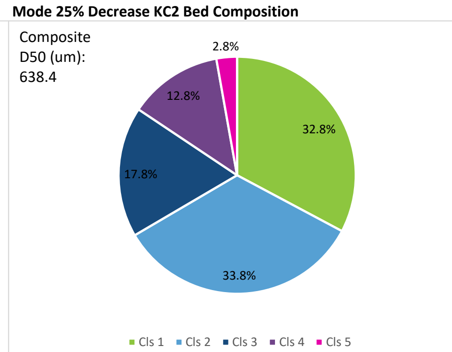


Table C-1: Full Vision with Initial Condition of "Design" Sediment Volumes (m^3) [Columns Correspond with Letter from Appendix A Plots; Also Provided Simulation ID]

Sedment Transport Zone	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
	019	020	021	022	023	024	025	026	027	028	029	030	031	032	033	034	035	036
CNR	-9,222	-9,068	-9,450	-9,971	-9,889	-10,055	-9,588	-9,427	-9,786	-10,164	-10,111	-10,314	-8,788	-8,708	-9,167	-9,974	-9,733	-9,926
Floodplain_East_CNR	225	339	112	157	232	81	179	267	96	144	220	72	234	337	117	250	267	98
Floodplain_West_CNR	164	191	130	148	169	117	151	187	115	145	172	112	179	202	142	145	173	126
Area A	-2,598	-2,498	-2,661	-3,072	-3,001	-3,136	-2,839	-2,777	-2,938	-3,120	-3,077	-3,242	-2,216	-2,189	-2,392	-2,827	-2,860	-2,927
Floodplain_East_Area A	34	52	22	37	54	19	36	52	18	40	49	18	39	54	25	24	53	22
Floodplain_West_Area A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area B	4,723	5,424	3,985	4,271	4,685	3,639	4,097	5,053	3,356	3,897	4,357	2,964	5,185	5,789	4,458	4,326	4,703	3,741
Floodplain_East_Area B	79	105	50	65	80	41	70	104	46	66	92	36	86	122	49	75	103	45
Floodplain_West_Area B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area C	11,642	13,752	8,627	7,479	9,303	5,958	10,078	12,559	7,254	6,891	7,980	4,804	13,164	15,101	10,295	9,102	10,179	7,567
Floodplain_East_Area C	52	66	35	53	80	35	43	60	22	47	67	28	57	74	43	58	75	38
Floodplain_West_Area C	32	45	17	38	54	21	34	48	20	41	57	26	29	41	15	37	53	20
Lakeshore Blvd	1,615	3,073	444	298	1,139	-560	1,596	2,685	311	232	1,017	-491	2,280	3,571	880	352	1,321	-473
Floodplain_East_LSB	25	37	16	21	30	14	24	32	14	22	28	13	28	38	17	24	31	14
Floodplain_West_LSB	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ice_Management_Area	12,658	16,073	9,101	11,154	14,076	7,427	12,502	15,862	8,987	10,616	14,148	7,251	13,157	16,739	9,529	11,838	14,830	8,172
Floodplain_East_IMA	107	184	53	102	160	49	110	179	54	101	168	50	105	169	52	99	164	48
Floodplain_West_IMA	2,660	3,452	1,761	2,607	3,374	1,800	2,579	3,366	1,640	2,560	3,285	1,745	2,694	3,451	1,830	2,660	3,460	1,887
KC_Upper	12,320	16,942	7,487	11,988	16,266	7,451	12,661	17,387	7,834	12,104	16,409	7,533	11,920	16,532	7,186	12,024	16,374	7,510
Floodplain_South_KC_Upper	535	751	320	534	756	326	532	739	311	536	745	326	532	747	317	545	752	323
KC_Narrows	-8,374	-8,396	-8,323	-9,667	-9,719	-9,591	-8,278	-8,316	-8,220	-9,602	-9,667	-9,528	-9,233	-9,185	-9,260	-9,761	-9,737	-9,765
Floodplain_South_KC_Narrows	2	3	2	2	4	1	3	3	2	2	4	1	1	2	2	3	5	1
KC_Mouth	243	2,315	-1,824	218	2,179	-1,622	84	2,153	-1,968	-41	1,796	-1,951	1,630	3,517	-241	685	2,607	-1,098
Floodplain_South_KC_Mouth	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NC_UpperBend	7,529	10,675	4,487	7,501	10,950	4,524	7,482	10,639	4,298	7,426	10,574	4,469	7,544	10,660	4,454	7,479	10,936	4,604
NC_MiddleBend	1,294	1,872	750	1,268	1,797	773	1,291	1,867	723	1,263	1,760	783	1,276	1,851	729	1,248	1,790	764
NC_Spillway	1,744	2,670	870	1,699	2,595	835	1,719	2,631	866	1,664	2,542	824	1,736	2,647	860	1,728	2,666	844
NC_Mouth	385	524	241	425	571	277	418	564	270	437	592	287	361	499	217	396	536	256
NC_Wetland1	3,114	4,492	1,719	3,035	4,381	1,709	3,066	4,436	1,686	3,023	4,298	1,727	3,085	4,448	1,695	3,038	4,442	1,708
NC_Wetland2	159	233	81	145	213	75	157	232	80	142	208	74	157	229	80	145	217	76
NC_Wetland3	1,776	2,629	973	1,692	2,515	904	1,710	2,544	939	1,683	2,449	937	1,764	2,623	944	1,712	2,575	897
NC_Wetland4	113	169	57	100	151	52	112	167	56	99	146	51	113	165	57	100	153	52
Polson	7,872	10,771	4,854	8,824	12,086	5,509	8,019	10,961	4,937	8,853	12,100	5,630	7,561	10,427	4,573	8,435	11,704	5,233

Table C-2: Full Vision with Initial Condition of "Full" Sediment Volumes (m^3) [Columns Correspond with Letter from Appendix A Plots; Also Provided Simulation ID]

Sedment Transport Zone	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
	001	002	003	004	005	006	007	008	009	010	011	012	013	014	015	016	017	018
CNR	-9,318	-9,165	-9,503	-10,272	-10,152	-10,379	-9,681	-9,497	-9,985	-10,519	-10,351	-10,664	-8,963	-8,735	-9,181	-10,064	-9,948	-10,116
Floodplain_East_CNR	282	401	163	249	359	124	276	390	161	244	346	114	279	416	156	285	401	167
Floodplain_West_CNR	159	195	119	148	176	124	155	194	116	154	187	126	172	210	139	150	177	121
Area A	-10,996	-10,856	-11,082	-11,507	-11,447	-11,548	-11,144	-11,005	-11,304	-11,579	-11,522	-11,632	-10,477	-10,375	-10,726	-11,405	-11,339	-11,433
Floodplain_East_Area A	42	60	26	29	50	13	44	62	23	28	57	15	33	48	23	47	70	24
Floodplain_West_Area A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area B	-5,976	-5,636	-6,139	-6,972	-6,674	-7,492	-5,964	-5,704	-6,654	-7,376	-7,200	-7,743	-5,283	-4,741	-5,964	-6,436	-6,197	-6,794
Floodplain_East_Area B	105	154	67	78	100	53	117	156	84	80	117	50	119	172	75	105	174	60
Floodplain_West_Area B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area C	-2,408	-1,429	-3,526	-8,328	-7,103	-9,871	-3,888	-2,873	-5,784	-9,591	-9,331	-10,331	-163	731	-1,352	-5,231	-3,896	-6,071
Floodplain_East_Area C	66	87	47	68	99	43	65	89	36	58	80	36	55	68	37	58	75	39
Floodplain_West_Area C	40	54	24	48	67	32	40	56	24	50	64	38	35	48	22	56	81	26
Lakeshore Blvd	-2,924	-1,788	-4,010	-5,468	-5,029	-6,004	-3,519	-2,864	-4,386	-5,670	-5,101	-6,210	-557	410	-1,600	-2,566	-2,297	-3,144
Floodplain_East_LSB	38	48	24	24	34	17	28	39	18	23	32	14	44	51	32	30	41	25
Floodplain_West_LSB	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ice_Management_Area	11,501	14,753	7,873	8,429	10,884	5,427	11,342	14,679	8,289	8,494	11,228	5,366	12,948	16,621	9,228	10,642	13,662	7,732
Floodplain_East_IMA	148	239	66	139	230	62	150	261	71	144	248	62	149	252	68	155	261	69
Floodplain_West_IMA	3,597	4,255	2,764	3,193	3,776	2,454	3,516	4,228	2,668	3,090	3,794	2,414	3,541	4,239	2,715	3,132	3,857	2,415
KC_Upper	-25,436	-24,456	-25,815	-26,758	-24,260	-29,288	-22,785	-22,550	-24,352	-27,420	-24,997	-29,630	-20,595	-17,478	-23,439	-24,126	-21,804	-26,631
Floodplain_South_KC_Upper	719	975	452	661	862	428	706	953	434	650	859	420	652	853	426	593	806	372
KC_Narrows	-7,569	-7,502	-7,747	-9,442	-9,390	-9,469	-7,507	-7,320	-7,694	-9,416	-9,373	-9,409	-8,409	-8,165	-8,609	-9,569	-9,482	-9,641
Floodplain_South_KC_Narrows	16	19	7	5	6	3	16	25	8	5	6	3	10	9	6	3	7	1
KC_Mouth	6,441	8,566	3,979	6,426	8,375	4,248	6,303	8,567	3,669	5,925	8,013	3,726	8,316	10,357	6,013	7,134	8,997	4,934
Floodplain_South_KC_Mouth	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NC_UpperBend	11,954	16,066	8,005	12,164	16,440	8,160	12,157	16,349	8,075	12,283	16,515	8,543	11,377	15,457	7,345	11,686	15,940	7,621
NC_MiddleBend	1,892	2,628	1,200	1,695	2,355	1,091	1,973	2,723	1,246	1,720	2,409	1,143	1,741	2,471	1,041	1,604	2,243	1,021
NC_Spillway	2,454	3,693	1,240	2,053	3,097	1,034	2,429	3,662	1,243	2,031	3,055	1,017	2,300	3,488	1,143	2,125	3,235	1,046
NC_Mouth	538	687	391	701	860	537	618	766	472	797	955	626	469	604	322	555	711	402
NC_Wetland1	3,980	5,432	2,379	3,623	4,920	2,180	3,964	5,456	2,353	3,540	4,895	2,152	3,848	5,368	2,248	3,605	5,005	2,140
NC_Wetland2	181	270	93	149	227	75	181	273	93	145	224	74	176	266	90	154	233	79
NC_Wetland3	2,494	3,596	1,421	2,088	3,026	1,162	2,386	3,469	1,336	2,045	2,955	1,161	2,372	3,403	1,329	2,147	3,148	1,181
NC_Wetland4	125	186	64	99	152	49	125	188	63	93	146	48	123	187	62	101	154	51
Polson	10,666	13,745	7,409	12,390	15,982	8,648	11,545	14,795	8,147	12,664	16,382	8,907	9,522	12,659	6,267	11,025	14,510	7,434

Table C-3: Interim Condition with Initial Condition of "Design" Sediment Volumes (m^3) [Columns Correspond with Letter from Appendix A Plots; Also Provided Simulation ID]

Sedment Transport Zone	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
	019	020	021	022	023	024	025	026	027	028	029	030	031	032	033	034	035	036
CNR	-8,497	-8,361	-8,794	-9,434	-9,277	-9,628	-8,991	-8,833	-9,355	-9,863	-9,725	-9,992	-8,046	-7,835	-8,235	-9,172	-9,054	-9,245
Floodplain_East_CNR	338	493	171	306	439	176	315	452	165	310	445	174	371	548	196	309	456	174
Floodplain_West_CNR	171	220	123	191	239	138	181	220	118	201	253	149	181	224	126	187	230	141
Area A	-3,264	-3,048	-3,526	-3,897	-3,664	-3,950	-3,360	-3,213	-3,657	-3,945	-3,897	-3,974	-3,047	-2,863	-3,286	-3,610	-3,547	-3,685
Floodplain_East_Area A	40	61	21	51	76	26	43	68	25	59	82	27	42	66	24	45	67	23
Floodplain_West_Area A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area B	807	1,153	325	-208	137	-634	342	832	-245	-407	-79	-820	1,165	1,718	685	6	525	-281
Floodplain_East_Area B	214	288	147	195	278	118	227	284	136	192	282	124	215	313	125	191	269	124
Floodplain_West_Area B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area C	-372	-106	-1,079	-2,162	-1,991	-2,408	-1,163	-831	-1,978	-2,450	-1,984	-2,423	150	608	-339	-1,360	-991	-1,770
Floodplain_East_Area C	66	53	29	31	58	19	68	40	22	22	81	22	51	56	28	42	61	19
Floodplain_West_Area C	5	8	3	4	10	2	12	9	3	5	15	2	5	10	2	4	7	3
Lakeshore Blvd	3,763	4,514	2,673	1,762	2,507	1,338	3,172	3,982	2,328	1,535	1,951	1,073	4,486	5,185	3,609	2,477	3,040	2,000
Floodplain_East_LSB	43	29	35	38	45	35	40	26	32	13	45	30	47	30	37	46	47	38
Floodplain_West_LSB	2	1	1	2	3	2	2	2	2	2	3	2	2	1	1	2	3	2
Ice_Management_Area	15,843	19,574	11,796	13,601	16,033	10,168	15,211	19,123	11,236	13,081	15,468	9,507	16,681	20,082	12,730	14,900	17,433	11,572
Floodplain_East_IMA	208	307	125	251	377	137	220	310	119	261	384	142	195	306	104	244	336	126
Floodplain_West_IMA	2,915	3,780	1,936	2,850	3,644	1,947	2,783	3,607	1,767	2,767	3,558	1,854	3,027	3,866	2,083	2,924	3,625	2,049
KC_Upper	13,173	17,635	8,501	12,725	16,960	8,036	13,567	17,824	8,825	12,690	16,953	8,012	13,112	17,680	8,311	12,660	16,877	8,074
Floodplain_South_KC_Upper	506	723	294	493	689	288	479	703	279	484	678	274	501	705	305	497	692	291
KC_Narrows	-8,186	-8,136	-8,168	-9,599	-9,599	-9,553	-8,115	-8,137	-8,087	-9,558	-9,579	-9,485	-8,854	-8,674	-8,969	-9,661	-9,651	-9,668
Floodplain_South_KC_Narrows	3	3	2	1	4	1	2	2	2	1	4	1	1	0	1	1	3	1
KC_Mouth	1,357	3,717	-972	1,036	3,090	-1,091	1,169	3,426	-1,090	653	2,735	-1,482	3,007	5,287	746	1,636	3,590	-374
Floodplain_South_KC_Mouth	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NC_UpperBend	9,860	13,902	6,065	10,229	14,959	6,076	10,081	13,801	5,983	10,301	14,890	6,207	9,760	13,927	5,852	10,299	14,435	6,064
NC_MiddleBend	1,622	2,333	979	1,581	2,184	1,006	1,644	2,295	959	1,583	2,186	1,013	1,598	2,300	949	1,592	2,167	1,021
NC_Spillway	2,031	3,085	1,033	1,937	2,950	974	2,018	3,064	1,020	1,902	2,881	954	2,014	3,055	1,012	1,934	2,913	966
NC_Mouth	402	527	266	457	613	308	469	606	329	509	668	351	361	489	224	412	555	262
NC_Wetland1	3,433	5,032	1,887	3,261	4,739	1,787	3,346	4,879	1,803	3,195	4,612	1,752	3,443	5,047	1,896	3,309	4,766	1,808
NC_Wetland2	184	276	94	164	241	84	183	268	92	162	237	83	185	279	94	168	244	87
NC_Wetland3	2,129	3,097	1,219	2,012	2,961	1,145	2,080	3,005	1,159	1,999	2,893	1,163	2,101	3,059	1,180	2,011	2,912	1,128
NC_Wetland4	135	208	68	117	177	59	134	197	66	114	171	58	138	212	69	122	181	62
Polson	8,684	11,734	5,545	10,054	13,628	6,505	9,170	12,305	5,898	10,206	13,780	6,620	8,204	11,230	5,079	9,548	12,970	6,028

Table C-4: Interim Condition with Initial Condition of "Full" Sediment Volumes (m^3) [Columns Correspond with Letter from Appendix A Plots; Also Provided Simulation ID]

Sedment Transport Zone	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
	001	002	003	004	005	006	007	008	009	010	011	012	013	014	015	016	017	018
CNR	-8,579	-8,281	-8,898	-9,675	-9,644	-9,891	-9,144	-8,822	-9,519	-10,140	-9,986	-10,292	-7,522	-7,343	-8,049	-9,323	-9,212	-9,402
Floodplain_East_CNR	418	622	222	342	474	193	368	565	200	352	495	200	505	720	268	379	555	206
Floodplain_West_CNR	177	240	129	212	247	147	193	254	124	219	269	156	203	250	138	211	266	159
Area A	-10,759	-10,601	-10,916	-11,702	-11,708	-11,744	-10,977	-10,799	-11,180	-11,749	-11,683	-11,782	-10,415	-10,343	-10,587	-11,127	-11,019	-11,142
Floodplain_East_Area A	43	64	21	47	76	26	44	66	23	57	79	30	35	58	22	40	58	20
Floodplain_West_Area A	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Area B	-6,996	-6,509	-7,445	-8,594	-8,376	-8,850	-7,542	-7,202	-8,017	-8,682	-8,446	-8,965	-5,985	-5,576	-6,718	-8,042	-7,970	-8,141
Floodplain_East_Area B	250	325	164	208	295	133	246	343	161	217	308	128	252	321	187	248	333	166
Floodplain_West_Area B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area C	-7,948	-7,395	-8,611	-10,613	-10,119	-10,819	-8,872	-8,353	-9,616	-10,601	-10,251	-11,099	-6,641	-6,234	-7,396	-9,670	-8,950	-9,905
Floodplain_East_Area C	59	102	48	34	70	29	53	85	26	69	103	27	91	109	54	58	101	28
Floodplain_West_Area C	9	14	4	8	11	4	7	12	4	7	14	4	7	12	4	8	13	5
Lakeshore Blvd	-2,467	-1,984	-3,001	-5,331	-5,071	-5,489	-3,254	-2,773	-3,778	-5,419	-5,256	-5,678	-1,750	-1,343	-2,400	-3,848	-3,688	-3,997
Floodplain_East_LSB	21	24	41	38	49	41	10	17	18	43	0	38	43	16	39	36	0	37
Floodplain_West_LSB	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ice_Management_Area	12,585	15,290	9,416	8,269	9,885	5,835	12,041	14,820	8,770	7,425	8,464	5,261	13,712	16,374	10,525	10,377	12,156	8,150
Floodplain_East_IMA	310	438	172	378	486	246	310	449	188	398	551	250	260	390	128	339	520	185
Floodplain_West_IMA	3,719	4,371	2,882	3,270	3,880	2,580	3,635	4,336	2,770	3,290	4,045	2,509	3,767	4,479	2,935	3,444	4,158	2,697
KC_Upper	-25,698	-23,370	-27,454	-26,504	-23,999	-29,033	-25,764	-24,043	-26,931	-27,157	-25,019	-29,290	-18,751	-15,789	-21,707	-23,119	-20,625	-25,764
Floodplain_South_KC_Upper	622	809	386	544	749	338	619	830	369	533	716	330	555	747	377	563	762	345
KC_Narrows	-7,589	-7,698	-7,553	-9,398	-9,367	-9,398	-7,421	-7,512	-7,500	-9,336	-9,317	-9,325	-7,921	-7,680	-8,118	-9,416	-9,312	-9,506
Floodplain_South_KC_Narrows	9	11	5	4	4	3	10	12	5	5	4	4	6	8	4	4	7	2
KC_Mouth	7,079	9,369	4,559	6,449	8,415	4,230	6,749	9,062	4,267	6,093	8,201	3,870	9,205	11,372	6,965	7,500	9,260	5,314
Floodplain_South_KC_Mouth	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NC_UpperBend	14,391	19,180	9,692	15,444	20,343	10,576	14,781	19,685	10,116	15,936	21,211	10,765	13,665	18,621	9,081	14,582	20,064	9,527
NC_MiddleBend	2,233	3,037	1,457	2,002	2,704	1,367	2,336	3,164	1,565	2,125	2,907	1,419	2,152	2,981	1,399	2,008	2,799	1,367
NC_Spillway	2,648	3,914	1,375	2,184	3,159	1,139	2,654	3,943	1,379	2,221	3,174	1,126	2,521	3,752	1,297	2,255	3,417	1,161
NC_Mouth	524	661	384	704	913	520	627	769	481	795	995	618	422	557	290	559	731	381
NC_Wetland1	4,234	5,785	2,519	3,677	4,969	2,230	4,176	5,741	2,473	3,685	4,992	2,197	4,273	5,931	2,551	3,949	5,429	2,370
NC_Wetland2	202	300	105	162	236	86	199	296	104	164	241	85	216	317	111	180	267	95
NC_Wetland3	2,713	3,788	1,600	2,263	3,116	1,344	2,621	3,751	1,523	2,317	3,159	1,319	2,601	3,653	1,542	2,407	3,432	1,430
NC_Wetland4	144	218	73	110	158	58	138	209	71	110	160	57	161	242	82	135	200	70
Polson	11,541	14,747	8,226	13,456	17,398	9,581	12,884	16,151	9,403	14,210	18,383	10,191	10,161	13,437	6,910	12,027	15,892	8,168