# Wingham and Area Flood Plain Mapping

## TECHNICAL REPORT

Prepared for

**Maitland Valley Conservation Authority**
1093 Marietta St
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March 12, 2020
Project No. P2019-391

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# 1. Introduction

GeoProcess Research Associates Inc. (GRA) has completed floodplain mapping for the Maitland Valley Conservation Authority (MVCA) for the Maitland River and its tributaries in the Wingham area. Within Wingham, the Maitland River confluences with two large tributaries; the Middle Maitland River and the Little Maitland River. The three rivers total approximately 20 km in length within the study area. The limits of the study area and the rivers are shown in Figure A- 1.

Upstream of the study area, the Maitland River watershed drains approximately 1,600 km$^2$, which is primarily composed of agricultural lands. Drainage is conveyed by the Lower Maitland River to Lake Huron, at the Town of Goderich. The Town of Wingham (the Town) is one of the larger communities within Huron Country and is an important hub for emergency services, including fire, paramedic, and hospital (with an emergency room). As a result, characterizing the flood hazards in the area is not only important for the Town but also the surrounding communities.

For this project, GRA developed a hydrologic model of the watershed and a hydraulic model of the study area. These models were used to generate two-zone floodplain maps, identifying the estimated limits of flooding and potential hazard areas.

# 2. Field Investigation

Topographic survey of the rivers, bridges, and culverts was undertaken and combined with LiDAR data (provided by MVCA) into a single terrain file that formed the basis for the hydraulic model geometry. Surveys were completed collaboratively between GRA and MVCA, with GRA completing the surveys of the bridges and MVCA the culverts. Watercourses surveys were led by GRA and also completed jointly with MVCA staff.

Topographic surveys were completed using a combination of RTK (survey-grade) GPS, total station and an Acoustic Doppler Profiler (ADP). The survey focused on channel banks and bathymetry in the vicinity of bridges and dams because these are areas where LiDAR data is typically less accurate. The survey also included key elevations and dimensions the hydraulic structures.

Ten (10) bridges and one (1) dam were surveyed. At each bridge, five (5) cross-sections were surveyed: two upstream of the structure, two downstream of the structure and one beneath the structure. Where flow depths exceeded wadable conditions the ADP was used to obtain the channel bathymetry. The ADP was used at the Josephine Street Bridge, upstream of the Howson Dam and for a 4 km section of the Maitland River between Helena Street and Amberly Rd (i.e., Highway 86). Channel centreline surveys were completed using the RTK GPS along the Maitland River between B Line Road and the Optimist Ball Park (approximately 2.3 km), along the Middle Maitland River between Jamestown Road and London Road (approximately 3.3 km), and along the Little Maitland at the confluence with the Middle Maitland River (approximately 100 m). The survey was performed in NAD83(CSRS) UTM Zone 17N and referenced to the Canadian Geodetic Vertical Datum of 2013.

# 3. Hydrology

## 3.1. Regional Flood Flows

A hydrologic analysis was completed using the U.S. Army Corps of Engineers HEC-HMS model code to simulate the Regional flood through the study area. The details of this analysis are provided in Appendix E, with three key summary points discussed in Sections 3.2 to 3.4

## 3.2. 1:100-year Flows

The 1:100-year inflow hydrographs were scaled from the Regional inflow hydrographs. The scaling was done so that the peak flows matched the 1:100 year return period flow taken from the MNRF 2014 flood frequency analysis, published in Land Information Ontario (LIO).

## 3.3. Validation Flow Data (October 21, 2019)

Model validation was undertaken to compare surveyed water surface elevations (WSE) to the 2D hydraulic model output. The Water Survey of Canada (WSC) gauge station, Wingham B, is located at Helena Street and was operational on the day of the survey. The approved flow data was downloaded from the WSC website on January 7$^{th}$, 2020 and used at the inlet boundary condition for the validation model.

## 3.4. Calibration Flow Data (June 2017)

Model calibration was undertaken for the June 2017 flood. Flows were obtained from the WSC gauge stations upstream of the study area (i.e., Wingham A, Bluevale and Belgrave), and were routed to the model inlets. The magnitude and duration of the hydrographs derived from the gauge stations were used as the model input. The model was run from June 22$^{nd}$ to 26$^{th}$ (5 days).

# 4. Hydraulic Model

## 4.1. Model Setup

The Wingham area hydraulic model was developed using GeoHECRAS, a software that integrates the U.S. Army Corps of Engineers HEC-RAS version 5.0.7 software with GIS and AutoCAD tools. The model is fully compatible with native HEC-RAS file formats.

A step-wise modelling approach was used to develop the hydraulic model for the Wingham area. This method helped identify areas with more complex hydraulics, backwater, storage, or spills that have the potential to impact the limits of flooding and/or the level of flood risk. The modelling methodology steps were as follows:

1) Developed an initial 2D model, primarily using a preliminary version of the digital terrain model;
2) Identified hydraulically complex areas (e.g., spills, backwater, multiple channel flow, areas of erosional concern) and used these as the basis for delineating zones requiring increased terrain mesh resolution;

3) Refined digital terrain and mesh for the 2D model, based on the output of Step 2;
4) Evaluated (for a preliminary validation) and refined (as required) the model; and
5) Undertook final validation and calibration of the model.

Opportunities for 1D modelling were investigated throughout the modelling process. Earlier iterations of the model sought to couple 1D reaches (for more uniform hydraulic conditions) to 2D reaches (for more complex hydraulics), but the proximity of these areas to one another resulted in poor convergence between the elements and instabilities in running the model. As a result, the study area was modeled entirely in the 2D domain and the results reviewed and confirmed to be as representative as an integrated 1D/2D model.

### 4.1.1. 2D Mesh

The study area was initially modelled in HEC-RAS using a uniform 50 m grid. The mesh was then refined at many locations using breaklines that force adjoining cells to be resized, resulting in zones of higher resolution than the base grid. Breaklines were used to better define the hydraulics at bridges and culvert, channel banks, and other abrupt changes in the topography (e.g. trails and roads). Cell spacing along the breaklines varied from 0.5 m to 30 m. A summary of the mesh parameters is provided below in Table 1.

*Table 1: Summary of 2D mesh used for the hydraulic model.*

| 2D Mesh Components | Values |
|---|---|
| Total Mesh Area | 11.56 $km^2$ |
| Number of Cells | 15,947 |
| Number of Breaklines | 206 |
| Mesh Scheme | Uniform |
| Largest Cell Size | 50 m |
| Smallest Cell Size | 0.5 m |

### 4.1.2. Terrain

The terrain for the hydraulic model was developed from a combination of LiDAR data and topographic survey data. The LiDAR data was provided by MVCA and was collected in 2019, having a resolution of 1 m and an accuracy of ±0.1m. To define the river channel within the terrain, AutoCAD Civil 3D was used to develop a 3D corridor. The corridor incorporated the longitudinal profiles of the rivers taken from the centreline survey, bridge survey data and bathymetry data, with linear interpolation used to fill in the gaps. Top of banks were delineated from the LiDAR data. Individual surfaces were created at the hydraulic structure crossings using topographic survey data. Breaklines were added at the bottom and top of banks. At low-risk bridges (Section 4.1.6), abutments and piers were manually added from the survey data. Piers and abutments were not added at high-risk bridges. Finally, bars and islands within the main channel were delineated with LiDAR data used to represent these features.

The terrain was modified at the northwest corner of the intersection of Highway 4 and Highway 86, which is the location of ongoing development. The site plan of the development was used to create a surface in AutoCAD, which was incorporated into the modelling terrain. This incorporated a floodwall at the property limits with an elevation of 310.20 masl.

The LiDAR data was used to represent the terrain for all areas outside of the main channel and was ground-truthed using the survey data on several roads. A map of the terrain data used in the hydraulic model is presented in Figure A- 2.

### 4.1.3. Boundary Conditions

Several boundary conditions were required to accurately represent the flow through the Maitland River, including the many tributaries that drain into the river within the study area. The locations of all the boundary conditions are shown in Figure A- 3. Flow hydrographs were simulated for the Regional flood using the hydrologic model for the inlets of the Maitland River, the Little Maitland and the Middle Maitland River (see Section 3). For the 100-year event, the inflow hydrographs were scaled so that the peak flows matched the 1:100 year return period flow taken from the MNRF 2014 flood frequency analysis, published in LIO. The hydrographs used for the boundary conditions are included in Appendix D. At the downstream model extents, a normal flow boundary condition was assumed. This was determined to be representative of the flow conditions due to the straight and unobstructed nature of the reach.

Preliminary assessments were undertaken to determine which of the smaller tributaries should be included in the model. Areas showing spills and backwater were identified, as were tributaries having catchment areas greater than 4 ha. A total of 17 tributaries were included in the model: four (4) along the Middle Maitland River and the remaining 13 along the Maitland River. Watershed area proportionality was used to create hydrographs for each of the tributaries, based on the main river hydrographs.

### 4.1.4. Simulation Settings

The Regional and 100-year flows where both modelled with a 2 second time step, under a mixed flow regime using the full momentum equation. The model run was started dry and was given 14 hours with a base flow to allow the channel to become wet. The average model run time for the Regional flood was 11 hours and 10 hours for the 100-year flood.

It was determined that to accurately represent the expansion and contraction of structures the full momentum equation was required. This was due to the total number of bridges and culverts in the model, particularly those near one another through the Wingham core area. Mixed flow was used as it was anticipated that some areas would experience supercritical flow (e.g. downstream of the dam, culvert outlets, flow over roads, etc.). The time step achieved acceptable Courant numbers for most of the model area; however, it was noted to be high for some of the smaller cells near the simulated bridges and dam. This created instability in some cells but the overall model was able to converge on a result. Time steps smaller than 2 seconds resulted in unacceptably long run times.

The model simulation period was nine days and 6 hours for both the Regional and 100-year hydrographs. This included enough time to wet the model with the base flow, capture the entire rising limb and most of the falling limb, to capture the effects of storage on the flooding limits and risks. Capturing the entire hydrograph allows for analysis into the wetting and drying fronts of the floods. While wetting and drying front analysis was beyond the scope of this project, MVCA could use this model for future such assessments.

### 4.1.5. Roughness Coefficients

A HEC-RAS Land Cover file was developed to spatially parameterize the Manning's roughness throughout the model area. Land use data were obtained from the Ministry of Natural Resources and Forestry's (MNRF) Ontario Flow Assessment Tool (OFAT) at 15 m resolution. Each land cover type was assigned a Manning's roughness value, per Table 2.

*Table 2: Manning's Roughness Coefficients by Land Cover*

| Land Cover Type | Manning's n |
|---|---|
| Agriculture and Undifferentiated Rural Lands | 0.04 |
| Swamp | 0.08 |
| Plantations – Treed Cultivated | 0.1 |
| Hedge Rows | 0.07 |
| Mixed Treed | 0.1 |
| Treed Upland | 0.1 |
| Clear Open Water | 0.035 |
| Community/Infrastructure | 0.02 |
| Marsh | 0.08 |
| Sand/Gravel/Mine Tailings/Extraction | 0.03 |

A test of the Manning's values was performed for the in-channel and adjacent overbank regions of the model. No noticeable changes to the WSE were observed when the channel roughness was changed by an order of magnitude. This was also the case for the overbank values, wherein a reduction in the Manning's value for agricultural lands did not significantly reduce flood elevations. These assessments, while performed at a high level, indicate that the WSE during major flood flows does appear to be sensitive to Manning's roughness values. To obtain a more quantitative understanding of the roughness value sensitivity, a detailed assessment of the low flow conditions would be necessary, and was beyond the scope of this study.

### 4.1.6. Hydraulic Structures

#### 4.1.6.1. Bridges (High-Risk)

High-risk bridges are defined when floodwaters are modelled to exceed the soffit elevation during the regulatory flood, which creates pressure flow through the bridge and which can also result in flow over the road. As these are complex, three-dimensional hydraulic processes, a 2D model is not able to accurately model bridges under pressure flow. This can be modelled in HEC-RAS in 1D using a series of equations, coefficients, and curves that represent the processes. Pressure flow can also be modelled using culverts in 1D and 2D HEC-RAS, however, this can sometimes overestimate the flow conveyed under the bridge.

An iterative process was undertaken to determine the best method for modeling the high-risk bridges. An independent 1D model was investigated but determined to be difficult to calibrate to the 2D model. An integrated 1D/2D was also investigated but there were issues with stabilizing the model at the connections between 1D and 2D interfaces. Therefore, it was determined that the most representative approach would be to model the bridges as culverts in the 2D model. This involved modelling each opening as a culvert barrel and creating gaps between the culverts to represent piers. For example, a bridge with two piers would be modelled using three culverts.

*Table 3: Culvert Parameters at High-Risk Bridges*

| Parameter | Value |
|---|---|
| Entrance Loss Coefficient | 0.5 |
| Exit Loss Coefficient | 1 |
| Manning's n for Top | 0.013 |
| Manning's n for Bottom | 0.035 |

At the culverts, the size of the mesh cells was increased. This was required to ensure that there was sufficient volume of water for the culvert flow calculations at the cells that connect to the culverts. While the increased cell size affects the fine-scale hydraulic calculations, as does the use of culverts in general, it is necessary to accurately represent the conveyance through the bridge. It is expected that the culverts (and in-turn the coarser mesh) provide the most accurate representation of conveyance during the peak of the Regional flood.

#### 4.1.6.2. Bridges (Low Risk)

Low-risk bridges are defined by a low risk of floodwaters exceeding the soffit elevation during the 100-year or regional flood. These bridges were modelled by manually editing the terrain to include piers and abutments. The soffit and deck were not included in the terrain, as pressure flow is not anticipated at these locations.

#### 4.1.6.3. Culverts

Some culverts were included in the model, generally along the minor tributaries. These were areas where spills and backwater areas were poorly defined within the model or had complex hydraulics. In some cases, culverts were required to capture spills resulting from backwater by the Maitland River or to more accurately represent ponding at roadways.

The preliminary modelling efforts identified which culverts needed to be included in the model. MVCA completed the in-field survey and investigations of these culverts. Photos and dimensions were taken of the culverts and this data was used to select appropriate inlet coefficients, Manning's roughness coefficients, and culvert chart numbers.

The channel geometry of the tributaries was not manually surveyed, therefore there were instances where the LiDAR data was not able to accurately represent the channel bed elevations at the culvert inlets and outlets. To account for this, the culvert inverts were adjusted to match the LiDAR, where required. The surveyed slope of the culvert was maintained for all instances. Notes were made within the model where modifications to the culverts were required. A summary of all the culverts is provided in Appendix B, within the HEC-RAS report.

#### 4.1.6.4. Howson Dam

The modelling terrain was edited to include the geometry of the Howson Dam spillway, which was simulated as a low-risk bridge. The south portion of the dam consists of a series of openings that do not convey water except under high flows. These openings were modelled as gates, with ogee weirs. The gates remain open for the entire duration of the run. The Howson Dam is shown in Figure 1. The spillway and "gates" can be seen on the left and right, respectively.

*Figure 1: Downstream face of the Howson Dam (looking upstream).*

# 5. Hydraulic Analysis

## 5.1. Model Corroboration

Corroboration of the model was undertaken to understand if the model was performing as expected. Model corroboration is frequently completed by comparing the results of the new hydraulic model to the results of the previous hydraulic model (if available). While a previous hydraulic model does exist, corroboration to this model was not undertaken due to the extreme differences in the model layout (1D vs 2D), boundary conditions and changes to the peak flow values and regime (steady vs unsteady). Therefore, it was necessary to corroborate the results of the hydraulic model to that of the hydrologic model. This provides an understanding of how the flow routing is acting within the hydraulic model, in relation to the hydrologic model.

A hydrograph was generated for the Regional flood at the model outlet, which was then compared to the hydrograph at the outlet of the hydrologic model. The hydrographs are shown in Figure 2 and a comparison of the results is presented in Table 4.

The outlet analysis shows that the hydraulic model generally emulates the results of the hydrologic model. As expected, the peak of the flood within the hydraulic model is delayed by approximately 3 hours (~1.5% of the flood duration) and is approximately 28 $m^3/s$ (~2% of the peak flow) less than the results of the hydrologic model. This is expected due to the additional storage within the hydraulic model which occurs at road crossings and inline dam structures that are not represented within the hydrologic model.

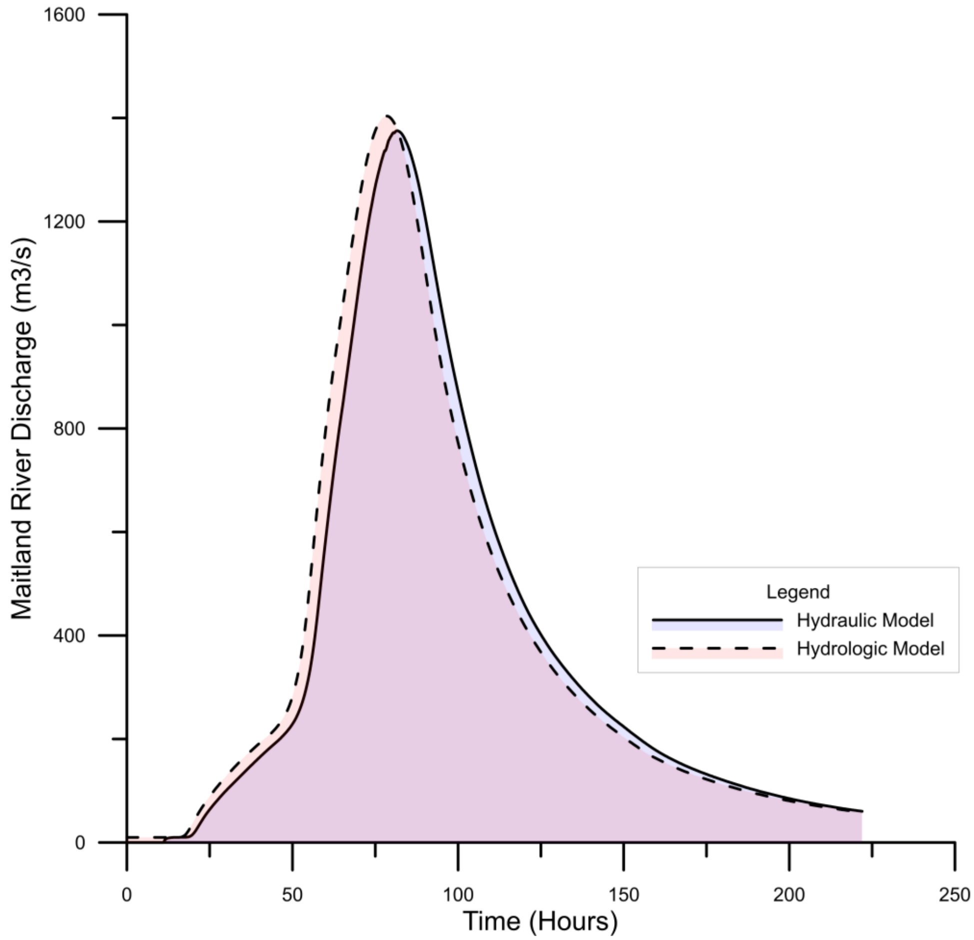


*Figure 2: Flow hydrographs from hydrologic and hydraulic model outlets for the Regional event.*

*Table 4: Comparison between hydraulic and hydraulic model outlets*

| Field | Units | Hydraulic Model Results | Hydrologic Model Results | Difference |
|---|---|---|---|---|
| Initial Base Flow | $m^3/s$ | 10.23 | 10 | 0.23 |
| Date & Time of Initial Rise in Flow | hours | 2000-01-01 13:20 | 2000-01-01 11:30 | 1.83 |
| Peak Flow | $m^3/s$ | 1375.25 | 1403.7 | 28.45 |
| Date & Time of Peak | hours | 2000-01-04 3:40 | 2000-01-04 0:30 | 3.17 |
| Final Base Flow | $m^3/s$ | 60.24 | 58.2 | 2.04 |
| Duration* | hours | 197.33 | 200 | 2.67 |

## 5.2. Errors and Warnings

Volume accounting was completed for both the Regional and 100-year simulations, summarized in Table 5. The percent error for both simulations was below 0.1%, indicating the model is managing the through-flow effectively.

*Table 5: Summary of volume accounting for Regional and 100-year models*

| Model Simulation | Cumulative Inflow Volume (1000 $m^3$) | Cumulative Outflow Volume (1000 $m^3$) | Ending Volume (1000 $m^3$) | Error (1000 $m^3$) | Percent Error (%) |
|---|---|---|---|---|---|
| Regional | 299,932 | 298,662 | 1,385 | 114.2 | 0.039 |
| 100-year | 253,749 | 252,380 | 1,480 | 111.3 | 0.044 |

## 5.3. Downstream Boundary Condition Validation

Model validation was undertaken to compare surveyed water surface elevations (WSE) to the 2D model output. A profile of the WSE was collected on October 21, 2019, between 15:00 and 17:00 by GRA and MVCA between Helena Street and Highway 86 (approximately 4 km). The Water Survey of Canada (WSC) gauge station, Wingham B, is located at Helena Street and was operational on the day of the survey. Using the WSC flow data as an inlet model boundary, the model was run using the same channel geometry and downstream boundary condition as the Regional and 100-year model. This validation was undertaken to evaluate and adjust the downstream boundary condition, which assumes normal flow depth. While this validation was completed using low flow data, it was anticipated that the normal flow boundary condition would be valid under high and low flow conditions and would still provide a frame of reference for the assumptions made.

Profiles showing the model results and the survey data are shown in Figure 3. Between 7+300 m and 9+300 m, there are no weirs nor obstructions to the terrain and a smooth profile of the water surface is seen in both the modelled and observed results. It is noted that the water surface slope for the modelled and observed results are equal (0.017%), indicating a corresponding similarity between the modelled and the observed downstream conditions. As this reach is immediately upstream of the outlet boundary condition, it is demonstrated that the boundary condition is an accurate representation of the observed conditions for the downstream reach.

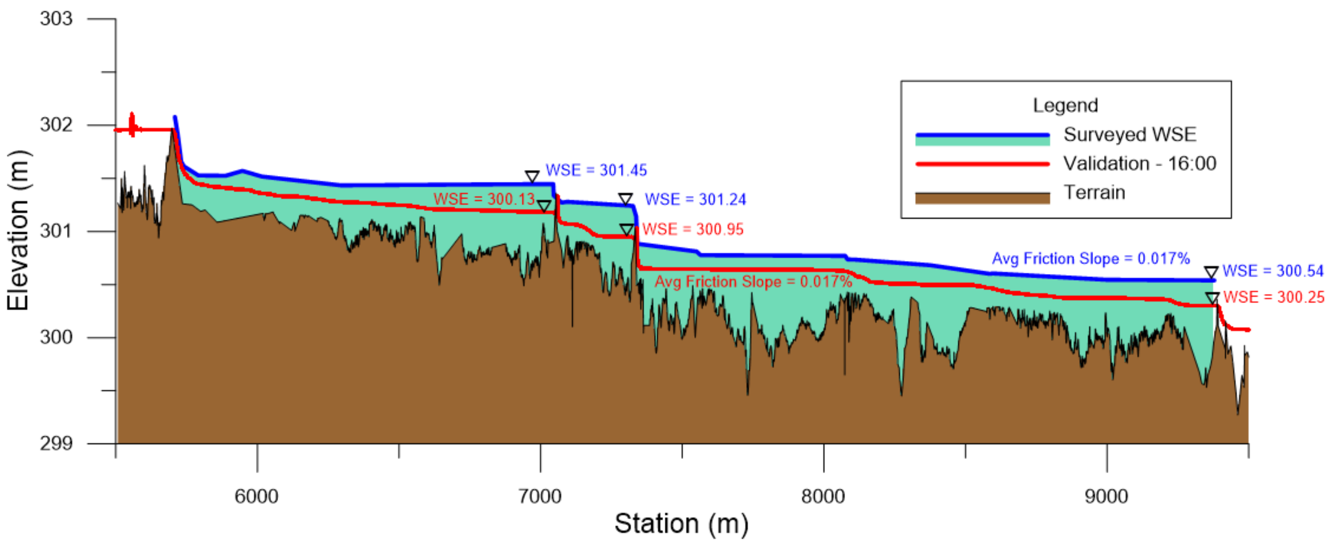


*Figure 3: Comparison of water surface profiles for the validation.*

The results indicate that the model was not able to accurately represent the backwater conditions at the boulder-weirs within the reach. This could be the result of issues with the low-flow terrain at these locations, or with the Manning's roughness used for the channel (0.035) was not high enough to adequately represent the low flow conditions. Relative roughness increases as the water depth decreases. Since the Manning's value here was selected for use in high-flow modelling (flood risk) scenarios, the assumed roughness might not be well suited to the lower flow validation. Because this model was only intended to simulate high flow conditions, it was not necessary to calibrate the terrain or roughness to the low flow conditions shown in Figure 3.

## 5.4. Calibration

Model calibration was undertaken for the June 2017 flood. Flows were obtained from the WSC gauge stations upstream of the study area (i.e., Wingham A, Bluevale and Belgrave), and were routed to the model inlets. The magnitude and duration of the hydrographs derived from the gauge stations were used as the model input. As with the Regional and 100-year model simulations, watershed area proportionality was used to create hydrographs for each of the tributaries from the main river hydrographs. The model was run from June 22$^{nd}$ to 26$^{th}$ (5 days).

A visual check was done using the imagery taken from a plane during the 2017 flood. A comparison of the aerial image and the model results from June 24 at 11:40 are presented in Figure 4 and Figure 5 respectively. The comparison showed the model results were able to accurately simulate the limits of flooding. Further, these photos were taken after the peak of the flood, when the flows had already started to recede. The model was able to accurately identify the flow recession from the area northeast of the highway intersections. As shown in the aerial imagery, neither Highway 86 or Highway 4 were overtopped at this point in the flood and the model showed the same result. Although not a quantitative comparison, it indicates that the model captures the general flow dynamics in the study area.

As a quantitative check, the results of the model were compared to high watermarks that were surveyed in 2019. The locations of the high-water marks were identified in the photographs taken during the 2017 flood. Also, the elevation of the water surface at the Howson Dam inlet was approximated based on the detailed structure survey. The results of the calibration are presented in Table 6.

At the Howson Dam inlet, the model simulated the water surface elevations within 0.3 m of the observed condition. Due to the complexities at this location, this result is considered a reasonable representation.

The model results at the two high water marks were significantly higher (0.70 m – 1.15 m) than the observed conditions. It is anticipated that the main cause of this discrepancy is the simulation of the inflow from the tributaries. Pro-rating the flows using area proportionality may have overestimated the inflows for the tributaries. There was no flow data for the tributaries and the geometry of these channels was not refined within the LiDAR. Approximately 300 m downstream of Helena Street, two tributaries (boundary condition IDs Trib10 and Trib04) confluence with the Maitland River and these flows may be contributing to the elevated results. To further verify, a sensitivity analysis would be required, and additional field data may be needed.

It is noted that there are limitations associated with calibrating to limited high-water marks. It is impossible to know if the model is representative throughout the entire reach with only three points, and difficult to adjust parameters through the entire model to match these points. Further to this, assumptions were made about the high-water marks as they were not surveyed at the time of the flood. The *time of the observation* for the high-water marks was taken from the digital data within the photographs (i.e., the date and time of file creation), which is based on the camera settings. This has not been further validated to confirm that these times are correct, which could mean that the photos were taken at different points along the wetting or drying fronts.

In summary, the model was able to confirm the visual observations from the 2017 flood, demonstrating that the model can accurately reproduce real events. Some discrepancies were noted during the calibration and further analysis and data will be necessary to further assess the calibration quantitatively. Based on this analysis, however, the model is expected to simulate the Regional and 1:100 year flood events with acceptable accuracy for the floodplain mapping.

*Figure 4: Image of the June 2017 flood at 11:39, showing the intersection of Highway 86 and Highway 4 (provided by MVCA).*

*Figure 5: Isometric view of the results from the calibration model at 11:40 on June 24, 2017.*

(NOTE: terrain for the model include the proposed floodwall at the new development at the intersection of Highway 86 and Highway 4)

*Table 6: Summary of calibration results.*

| Location ID | Photo of Location | Time of Observation | Time of Model Results | Surveyed Elevation (m) | Model Results (m) | Difference (m) |
|---|---|---|---|---|---|---|
| 2017-HWM-01 (within the soccer fields west of Helena Street) |  | June 24, 2017, 08:48 | June 24, 2017, 08:50 | 306.13 | 307.260 | 1.13 |
| 2017-HWM-02 (upstream and east of the MacIntosh Road bridge inlet) |  | June 24, 2017, 08:39 | June 24, 2017, 08:40 | 306.77 | 307.488 | 0.72 |
| Howson Dam Inlet |  | June 24, 2017, 11:27 | June 24, 2017, 11:30 | 310.50 – 310.60 | ~310.79 | 0.19 – 0.29 |

## 5.5. Tributary Conveyance and Lateral Spills

For discretionary reasons pertaining to model performance and usibility, certain laneway and minor roadway culverts are not included in the hydraulic model. While the absence of these culverts is not expected to significantly alter the maximum floodplain elevation and extents, it does increase the predicted flood extents and duration during the receding limb of the hydrograph, upstream of the missing culverts. This is because the roads (which are missing the culverts) restrict the outflow. For example, including the London Road culverts on Trib05 and Trib06 would allow for better characterization of the flood progression between London Road and Josephine Street.

There are also some areas where the flood extent reaches the study area boundary. These areas include:

- Downstream of Amberly Road
- Adjacent to the following boundary condition lines: MainMaitland, Trib12, Trib14/15, Trib16, and Trib17.

As a consequence, the spill dynamics at these two locations are not fully captured within the hydraulic model. Given the limited extent of these spill areas, the impact on flooding within the study area is expected to be minimal, but further model refinement at these locations could be possible with an extended LiDAR dataset.

# 6. Floodplain Mapping

Floodplain mapping was completed using the results of the HEC-RAS model and a digital copy of the maps is provided in Appendix C. The maps contain SWOOP 2015 orthoimagery and show 1 m contours generated from the modelling terrain (per Section 4.1.2). The results provided are in the CGVD2013 vertical datum and CSRS NAD 83 horizontal datum for UTM Zone 17N.

At the request of MVCA, the two-zone flood risk mapping approach was applied when delineating the floodplain maps. The two-zone concept involves delineating a floodway where most of the flood is conveyed (and presents a higher associated risk), and a flood fringe, where the risk of flooding is present, but the risk considered reduced.

The floodway was delineated using a combination of depth and velocity criteria to identify the portion of the floodplain which poses a significant risk to life. The criteria were taken from Appendix 6 of the River & Stream Systems: Flooding Hazard Limit Technical Guide (MNR, 2002). Table 7 lists the criteria used for the delineation.

*Table 7: Threshold Criteria for Floodway Delineation*

| Hydraulic Parameter | Threshold | Explanation |
|---|---|---|
| Depth (m) | 0.98 | Depth sufficient to float young children |
| Velocity (m/s) | 1.7 | Recommended threshold value |
| Depth x Velocity ($m^2/s$) | 0.372 | Equal to 4 $ft^2/s$, which is considered a suitable risk threshold for most individuals |

The portions of the floodplain for which any of these three thresholds are exceeded were classified as a floodway. Where none of the criteria were exceeded, the floodplain was classified as flood fringe. The floodway and flood fringe are clearly delineated in the mapping in Appendix C.

# 7. Conclusion

On behalf of MVCA, GRA has completed a floodplain mapping study for flood-prone areas of the community of Wingham, Ontario. Hydrologic and hydraulic modelling was completed using HEC-HMS and HEC-RAS, respectively. Regional (Hurricane Hazel) and 1:100 year flood lines have been delineated and mapped at 1:2500 scale.

This report and the associated mapping have been reviewed and sealed by a professional engineer.

# 8. References

Ontario Ministry of Natural Resources (MNR), 2002. Technical Guide – River & Stream Systems: Flooding Hazard Limit

Ontario Ministry of Natural Resources (MNR), 2014. Flood Flow Statistics: For the Great Lakes- St. Lawrence Watershed System. Queen's Printer for Ontario

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March 12, 2020

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