MEMORANDUM

Date:	March 21, 2017
To:	Poul Rosen, City of Nanaimo
From:	Glen Shkurhan, Taylor Swailes
File:	1296.0048.01
Subject:	**Colliery Middle Dam Hydraulic Assessment – FINAL**

1) Background

The City engaged Golder Associates to complete a study titled the "Colliery Dams, Nanaimo, BC Hydrology, Hydraulics and Middle Dam Breach Analysis", Golder, July 25, 2014 (the Golder Study).

The Golder Study conducted hydrologic calculations using the Soils Conservation Service (SCS) methodology which applies parameters of basin area, a runoff "curve number" based on land cover and soil type, a computed basin lag time, and a rainfall distribution. The SCS methodology outputs are sensitive to the estimated time to peak and selected curve number. The curve number is similar in nature to the runoff coefficient used in the Rational Method; in that it estimates only the portion of precipitation that results in direct surface runoff rather than the distribution between surface water and groundwater movement. In this context, groundwater refers to all forms of water movement through the soils and bedrock that eventually discharge as seepage into the creeks and reservoir system; including shallow groundwater, interflow, and potential deep groundwater movement.

The findings of the Golder Study predicted design flows that significantly exceed the capacity of the spillways, indicating a high risk of dam overtopping.

In late 2013 the City implemented a flow monitoring program, collecting continuous data at four stations; Lincoln Bridge, the upper reservoir, the middle reservoir (currently in question), and the lower reservoir which recently was upgraded with an auxiliary spillway. Also in recent years the City installed a continuous precipitation gauge in the vicinity of the upper reservoir providing data in closer proximity to the subject watershed.

The City of Nanaimo retained Urban Systems to revisit predicted design flows for the middle dam through the application of newly collected field data. This memo builds on the work previously completed by Golder. This memo notes information that is re-applied from the Golder study, it presents the field data and analysis undertaken, it presents revised predicted design flows that result from it, and it offers suggestions for further technical work before firm conclusion can be reached.

2) Summary of Findings

The analysis conducted for this assignment has made best use of available field data and measurements provided by the City of Nanaimo. Model calibration has demonstrated a very good fit to observed data. It appears clear that watershed hydrology is strongly governed by groundwater flow and has a time to peak that extends beyond a 24 hour period.

It is recognized, however, that the largest precipitation event observed to date, and calibrated to, has a return period of approximately 1:2 years. The calibrated model is therefore still making a very significant projection on how the watershed will respond under an extreme event.

There is a high degree of confidence for predictions for low return period events, however given the limited field data we strongly recommend that the projected peak flow for extreme events still be taken with caution. We suggest that the results of this assessment indicate less urgency than previously predicted, but we strongly recommend that further monitoring and analysis is required to reach firm conclusion on the peak design flow that should be used for risk assessment and design.

Given the demonstration that groundwater flow is significant, we recommend that the City engage a hydrogeologist to further explore the groundwater flow and snow melt regime of this watershed. The critical design flow for the dam will ultimately be generated by a combination of groundwater flow and surface water flow once the threshold capacity of the ground is reached. At this time, the threshold flow capacity of the ground is not well understood. A groundwater model developed by hydrogeologists would better inform the overall hydrologic model to provide a refined design flow. In addition, we recommend that the City continue its flow and water level monitoring program. We have separately discussed with the City opportunities to improve the monitoring program.

2) Observed Data & Spillway Rating Curves

a) Observed Data

i) Monitored water level data at each of the four locations was gathered from 2013-2016 by the City of Nanaimo. The data was recorded continuously using SCADA systems. Obvious data errors such as blanks and negatives were removed. In order to establish the datum of the SCADA measurements, the City provided field surveys of water surface elevation at known dates & times. This field survey data was compared with the reading from the automated data recorders to confirm the datum for each sensor, and establish a calibrated set of depth data.

ii) The City provided ACAD survey files of each spillway, as well as a detailed channel cross section at the point of measure in the creek at the Lincoln Bridge crossing.

iii) Using a portable hand held velocity meter, the City collected velocity measurements on two different occasions with different flow depths at each site. Each velocity measurement consisted of between 5 and 23 point measurements, spaced equally across the channel width. This data was used to develop an area weighted velocity for each incremental segment of the channel and then aggregated to produce an area weighted average velocity for the entire channel. This produced a calibrated relationship of flow velocity to flow depth, in turn producing a calibrated understanding of flow rate to flow depth.

b) Rating Curves

i) Using the two field measurements of velocity and flow depths as calibration points, the Manning's equation was applied to generate a rating curve relating recorded water surface elevation to flow rate, the results of which are presented in Figures 3 to 6 below.

Figure 3: Stage vs Discharge Curve for Lincoln Road Bridge Crossing

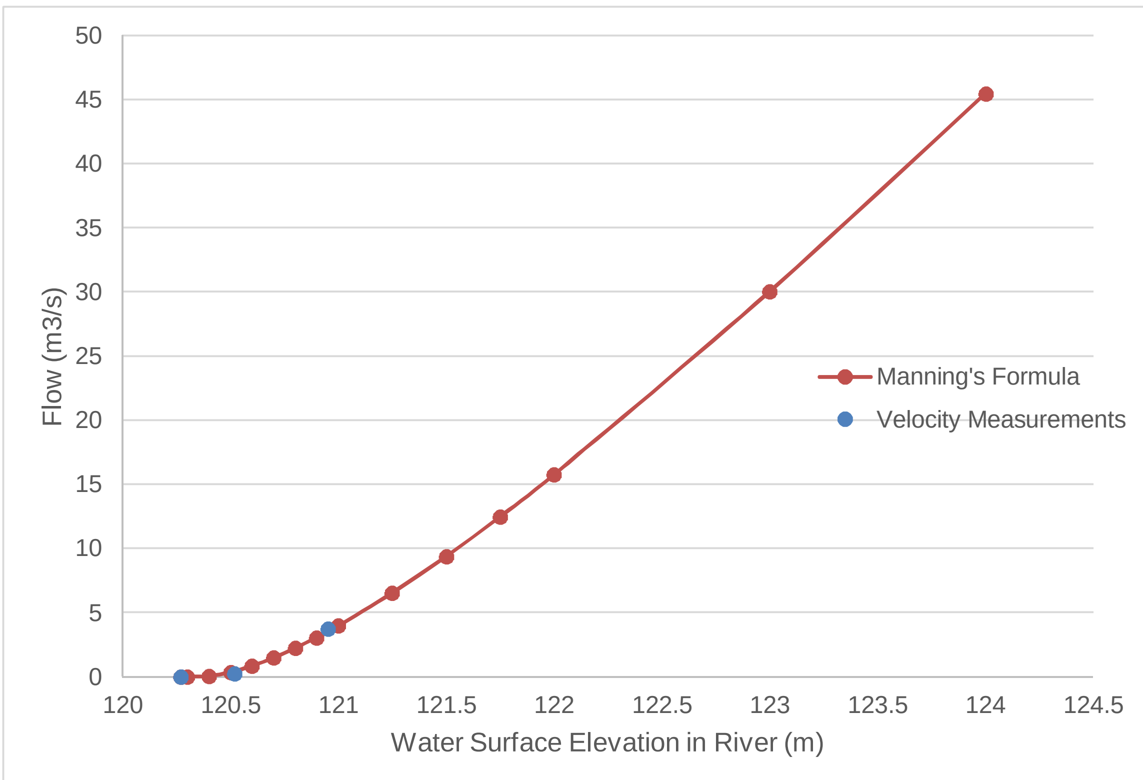


Figure 4: Stage vs Discharge Curve for Upper Dam Spillway

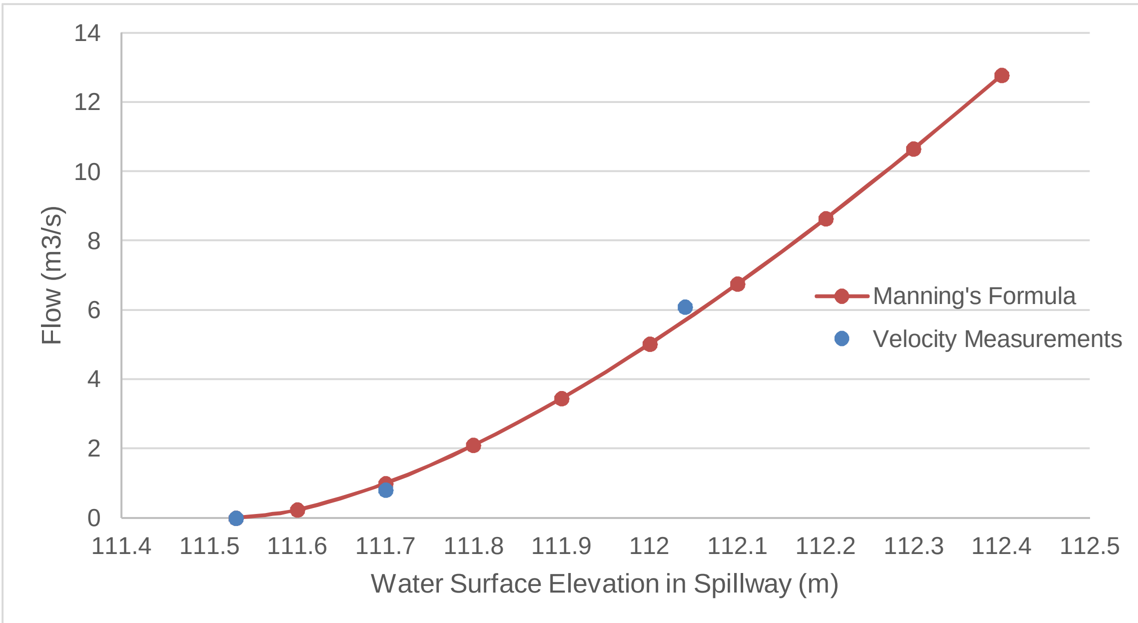


Figure 5: Stage vs Discharge Curve for Middle Dam Spillway

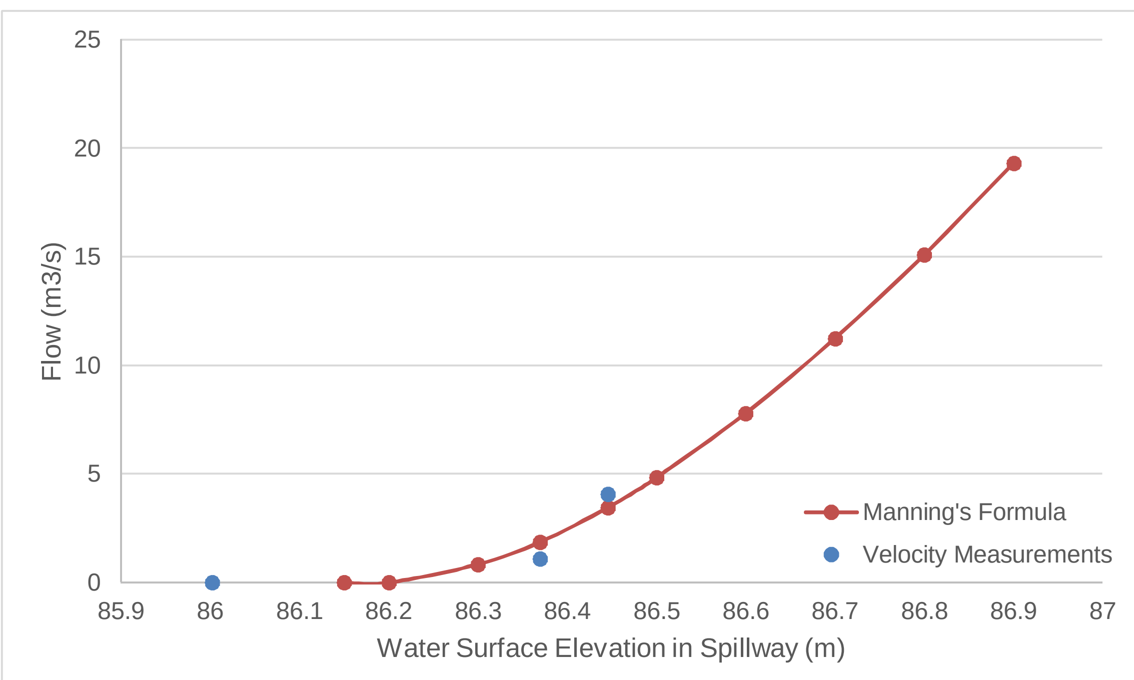
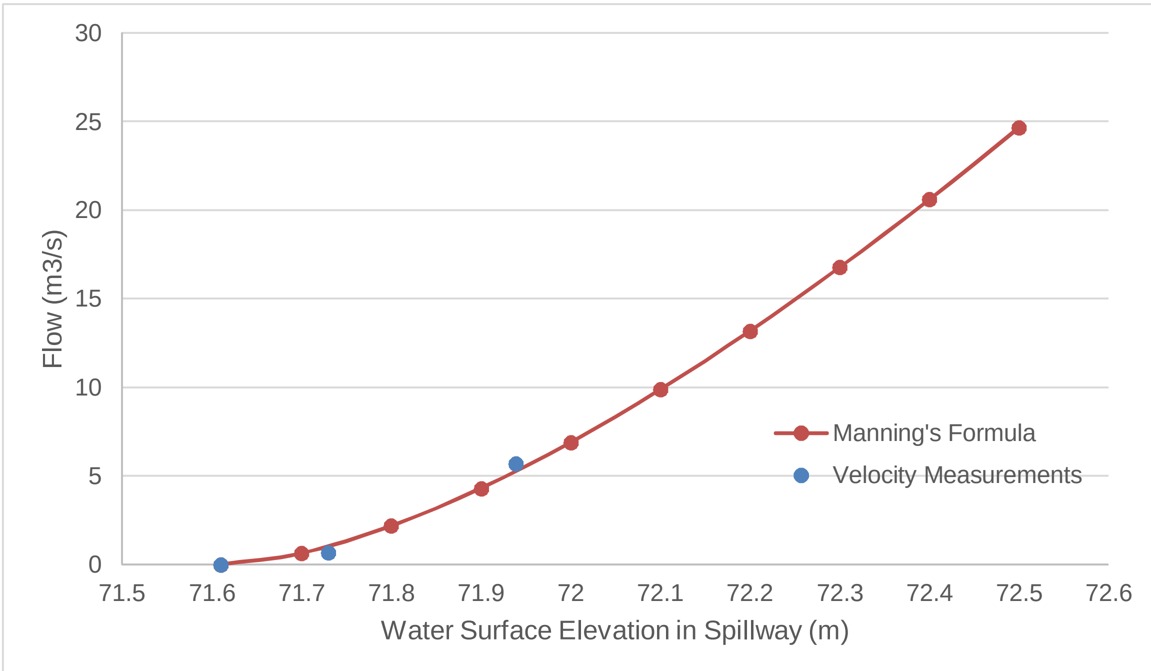


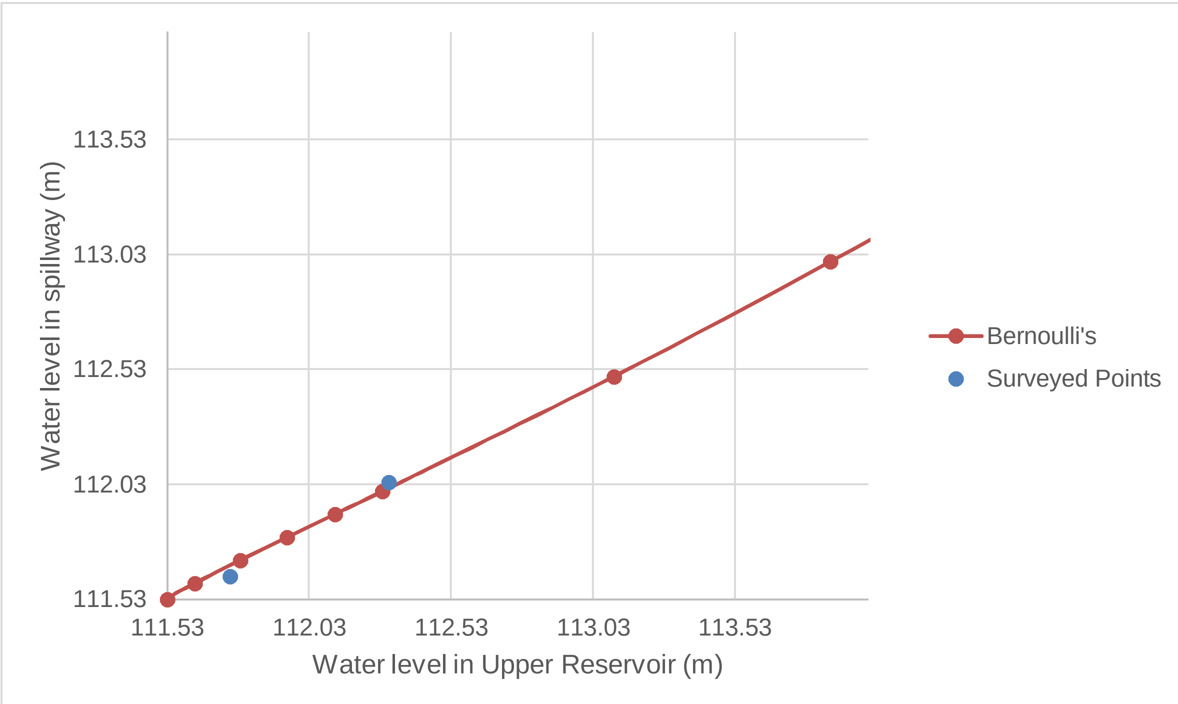
Figure 6: Stage vs Discharge Curve for Lower Dam Spillway



ii) Upper Reservoir

In the case of the Upper Reservoir, the automated water level is measured directly within the reservoir, whereas the other sites are measured directly within the river or spillway. As such, for the Upper Reservoir, an additional conversion was required to represent the change in water depth resulting from the acceleration of water from the reservoir into the spillway. This was done using a Bernoulli's theorem; assuming the velocity at the point of measure in the reservoir was zero, and assuming that friction was negligible compared to the change in velocity head. Calibration points were obtained by field measuring water depth in the spillway and comparing them against the water depth measured in the reservoir. The relationship between water elevations measured in the reservoir to the water elevation in the spillway is presented in Figure 7 below.

Figure 7: Water Surface Elevation in Flume vs Reservoir for Upper Dam



Using the calibrated rated curves presented above, a computed flow rate can be determined for each data point of the continuous water level data set recorded at each site.

3) Peak Flow Analysis

a. Annual Maximum Flows

i) The continuous water level data was converted into continuous flow rate data, from which the annual maximums were isolated at each site. As expected, the data shows a clear pattern of higher flows through the winter season, approximately mid-October to mid-April, with little to no flow occurring through the spillways from approximately May to October. Because of this pattern, using an annual maximum series based on calendar year created an arbitrary division midway through the wet season, where some wet seasons (2015-2016) had two “annual maximum” flows, where others (2013-2014) had none. In order to better reflect the periodicity of the observed data, a ‘year’ was taken to be August 1st to July 31st, so that the maximum series was represented by the winter season, not by calendar year.

ii) Observed data to date (late 2013 to early 2017) demonstrates peak flows ranging between 4 and 16 m^3/s. The monitoring data from the middle reservoir was excluded as it shows significant noise and scatter which brings its data into question. Urban Systems has separately advised the City to inspect and repair this monitoring station accordingly. Despite this, data at all other sites was found to be sound, and therefore applied.

b. Precipitation Data Set

i) In recent years the City installed a continuous precipitation gauge in the vicinity of the Upper Reservoir, named "RG_Reservoir 1". Data, recorded in 5 mm increments, was obtained for synchronization with flow data. In this case the rain gauge is directly within the study watershed and there is no succinct information to demonstrate how the observed precipitation varied across the watershed, therefore there was no adjustment for spatial distribution. The recorded data was applied evenly across the watershed during model calibration.

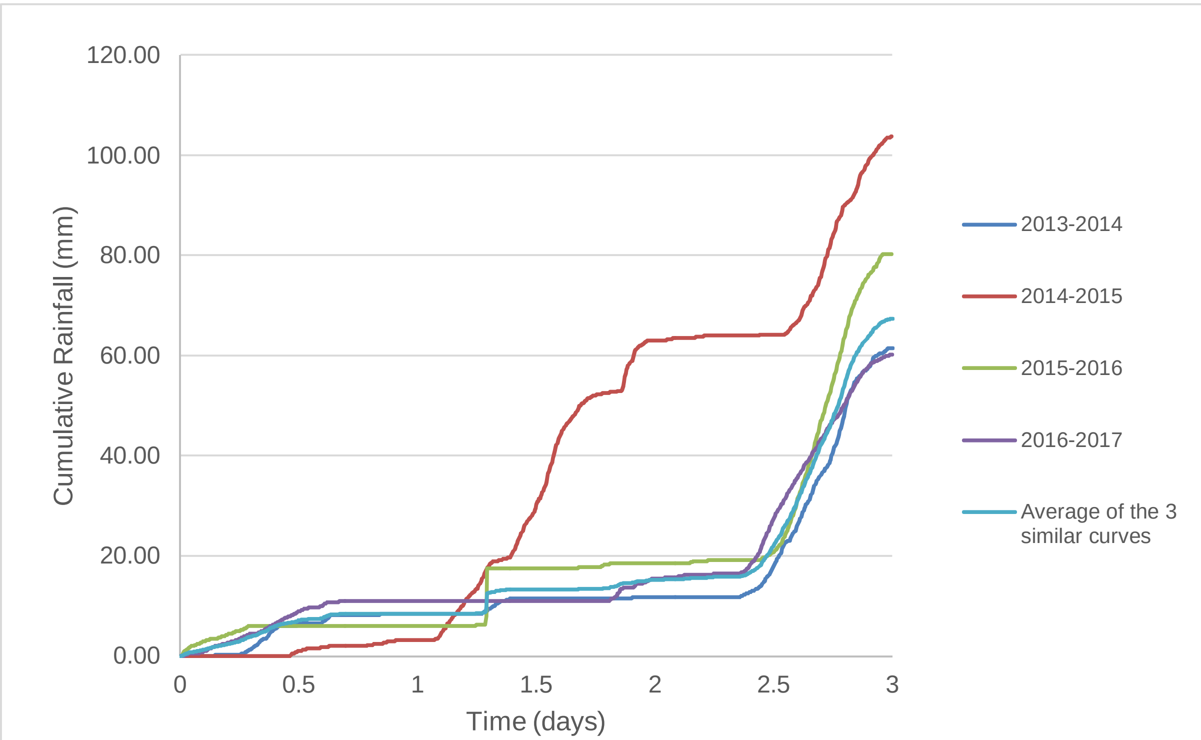
c. Correlation to Rainfall Depth

i) For the four largest observed flows the leading 24 hour, 48 hour, and 72 hour precipitation volumes were computed and plotted against the observed peak flows. The purpose of this was to see if there was a recurring "best fit" correlation between annual peak flow and precipitation duration. This initial review suggested that there is a more consistent positive correlation of peak flow to 48 hour and 72 hour precipitation than to 24 hour precipitation.

d. Developing Long Duration Synthetic Storms

i) For the 24 hour storm events the SCS Type 1A rainfall distribution was applied; consistent with that applied previously by Golder Associates. To generate 48 and 72 hour distributions, mass curves of the observed data were first developed. Mass curves for the four largest events recorded are presented in Figure 8 below. Three events produce virtually the same pattern, with a fourth being an outlier. The three similar events were used to produce a synthetic design storm distribution for 48 and 72 hour events (discussed below).

Figure 8: Observed 72 hour Cumulative Rainfall Volumes
(the four largest events recorded)



4) Continuous Simulation Model and Calibration

a. Model Set-Up & Assumptions

A continuous simulation model of the system was created using PCSWMM. The sub-catchment areas as previously developed by Golder Associates (Report on Colliery Dams, Nanaimo, BC, Hydrology, Hydraulics, and Middle Dam Breach Analysis, Golder, 2014) were applied. Physical catchment parameters such as impervious fraction, width, and slope were estimated based on Google Earth and available contour information.

Each of the reservoirs was modelled as a storage node, with stage area curves applied from the previous Golder Study. However, that prior study did not present a curve for the upper reservoir, therefore one was developed using aerial measurements from Google Earth.

The spillways from each reservoir, as well as the main channel at Lincoln Bridge, were modelled as control links by applying the stage versus discharge rating curves developed from observed data as discussed Section 2 above. The continuous rainfall data from the "RG_Reservoir 1" rain gauge collected by the City from 2013-2016 was also applied.

b. Calibration of Hydrologic Parameters

i) The model was first set up and run in absence of groundwater flow, yielding poor fit between observed and modelled hydrology. In order to better reflect the observed behaviour PCSWMM's groundwater module was applied. This module represents the distribution of water between that evaporated from the soil, percolation deep into the ground, and seepage out of the ground to reappear as flow in the defined hydraulic system (ie. channels, pipes, reservoirs, etc.). The module maintains a water balance throughout the analysis process, and simulates a groundwater table rising and falling depending on precipitation inputs relative to system losses.

Figure 9 below represents a comparison of modelled (blue) versus observed (green) flows at the Lincoln Bridge location continuously from late 2014 to early 2017. The observed data demonstrates a significantly diminishing base flow through the summer months. Focus of the analysis for this assignment is on winter conditions; therefore a fixed base flow value has been applied in the model to represent winter conditions only. For this reason, the model shows a summer base flow and precipitation response greater than observed. However, once the groundwater is recharged by fall rains, the observed and modeling hydrographs closely match.

Figure 9: PCSWMM Predicted Flows vs Observed Data at Lincoln Bridge Crossing

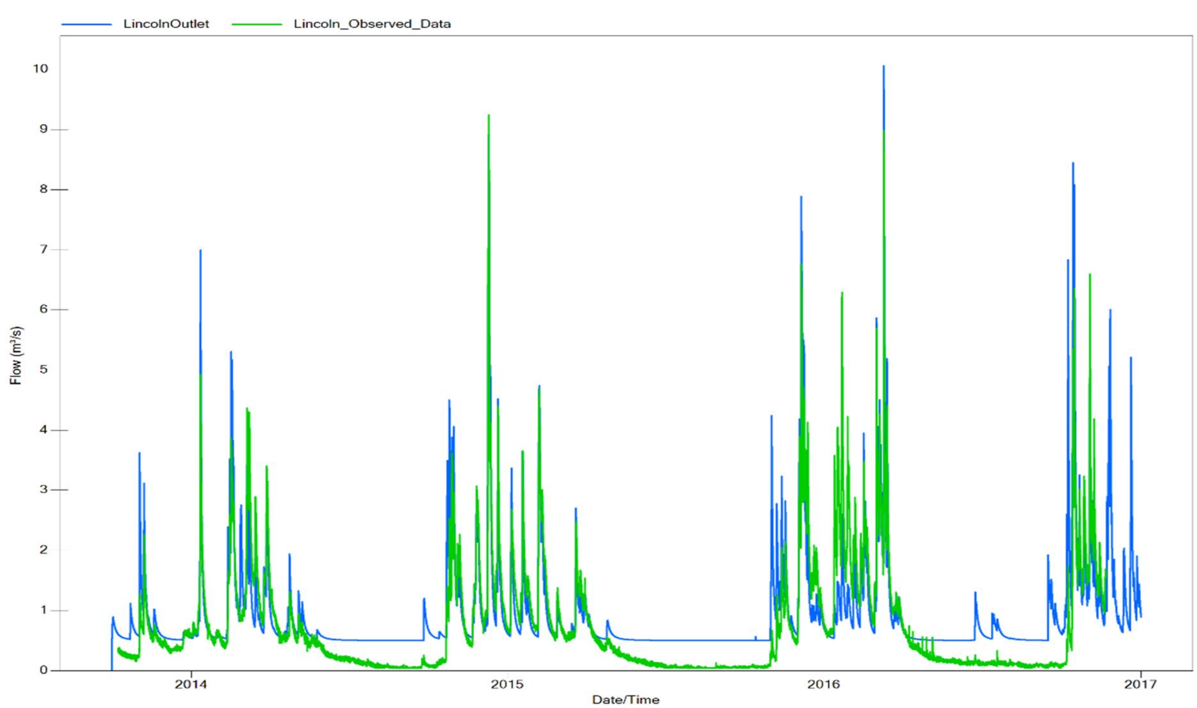
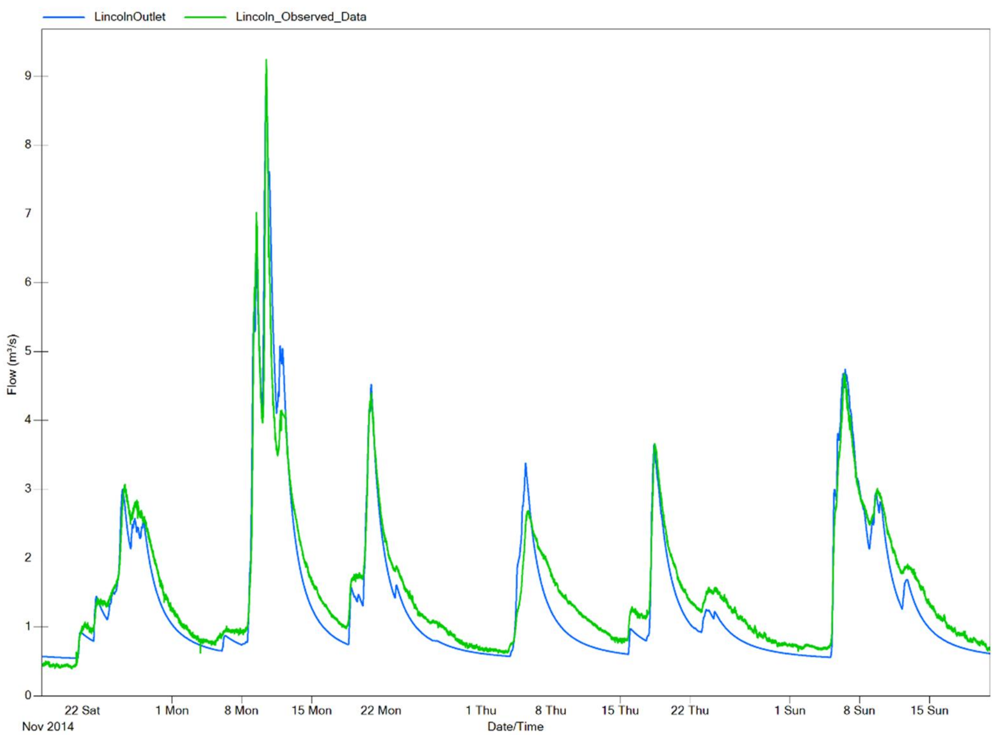


Figure 10 below provides an example of the model calibration during a period of the year when the groundwater has been recharged, clearly demonstrated a good, repeated fit. A similar fit was obtained for other winter periods. Based on the currently available information, a calibrated model is achieved.

Figure 10: Sample Calibration of November 2014 to January 2015 at Lincoln Bridge Crossing (blue is modeled, green is observed)



5) Synthetic Storm Development

a) Precipitation Depths by Return Period and Duration

24 hour synthetic storms as previously applied by Golder Associates in their 2014 study were reapplied herein; whereas precipitation depth projections were newly developed for the 48 hour and 72 hour durations. To do this, IDF (intensity duration frequency) curves from the "City Works Yard" station were plotted and then extrapolated to longer durations for return periods of 1:2 year to 1:100 years. However, IDF curves do not exist for return periods beyond 1:100 year. As such, precipitation depths as previously developed for extreme 24 hour durations were projected to longer durations based on the pattern established for more frequent return periods. The resulting table of precipitation depths is presented in Table 1 below.

Table 1: Design Precipitation Depths (mm)

Return period	Duration (min) 5	10	15	30	60	120	360	720	1440 (24 hour)	2880 (48 hour)	4320 (72 hour)
2	3.1	4.7	5.8	7.8	10.4	14.8	31.2	43.2	57.6	89.6	111.9
5	4.8	7.3	8.9	11.4	13.7	18.4	37.2	52.8	74.4	113.6	142.7
10	5.9	9.1	11.0	13.7	16.0	20.8	40.8	58.8	84.0	126.9	159.3
25	7.3	11.3	13.6	16.7	18.8	23.8	45.6	67.2	96.0	144.5	181.5
50	8.4	12.9	15.6	18.9	20.9	26.0	49.2	73.2	105.6	158.4	199.3
100	9.4	14.5	17.5	21.1	22.9	28.4	52.8	79.2	115.2	171.7	215.9
200									126.2	187.1	235.5
500									139.5	206.0	259.4
1,000									149.6	220.2	277.5
2,000									159.6	234.5	295.5
5,000									172.9	253.4	319.4
10,000									182.9	267.6	337.5
50,000									206.3	300.7	379.4

Values from City Work Yard Station (Flowworks)

Values from Golder Associates memo February 7, 2014, Appendix D of the Report on Collieries Dams Hydrology and Hydraulics

Extrapolated values from Urban System.

NOTE: GOLDER REPORTED THE 1:50,000 APPROXIMATING THE PMP

b) Precipitation Depth by Sub-Catchment

The 2014 Golder Study also provided a scaling factor to adjust precipitation for each sub-catchment based on its centroid elevation. As such, beyond the precipitation projections presented in Table 1 above, a scaling factor was also applied consistent with those previously applied. Resulting precipitation depths for sub-catchment are presented in Table 2 below.

Table 2: Design Precipitation Depths for Each Sub-Catchment

24 hour Duration				
Return Period	Upper Hwy 19	Lower Hwy 19	Middle Res	Lower Res
2	70.3	60.8	58.9	58.3
5	88.8	76.8	74.4	73.5
1000	180.5	156.1	151.3	149.6
1/3 1000 to PMP	203.3	175.8	170.4	168.5
50000 (PMP)	249	215.3	208.6	206.3
48 hour Duration				
Return Period	Upper Hwy 19	Lower Hwy 19	Middle Res	Lower Res
1000	322.9	279.2	270.7	267.6
1/3 1000 to PMP	336.3	290.8	281.8	278.7
50000 (PMP)	363.0	313.9	304.1	300.7
72 hour Duration				
Return Period	Upper Hwy 19	Lower Hwy 19	Middle Res	Lower Res
1000	407.2	352.2	341.3	337.5
1/3 1000 to PMP	424.1	366.8	355.4	351.5
50000 (PMP)	458.0	396.0	383.7	379.4

(green) Values from Golder Associates memo February 7, 2014, Appendix D of the Report on Collieries Dams Hydrology and Hydraulics

(blue) Extrapolated values from Urban System.

c) Significance of Observed Precipitation

As noted in Table 2 above, the statistical 1:2 year, 24 hour precipitation depth ranges from 58.3 mm to 70.3 mm depending on the sub-catchment. In comparison, the largest observed 24 hour precipitation depth recorded at the RG_Reservoir 1 station is 62 mm, which is generally equivalent to the statistical 1:2 year 24 hour depth. The 48 hour precipitation depth associated with this same event only increased marginally to 74 mm and increased marginally again to about 80 mm over a 72 hour period. As such, this observed event is reflective of an event between 24 hour and 48 hours.

The largest observed 48 hour precipitation event occurred in December of 2014 with a total depth of 101 mm at the RG_Reservoir 1 station. Referring to Table 1 above, this event is statistically between a 1:2 year and 1:5 year level.

Both of these observed events were valuable in model calibration and being able to compare against past model predictions.

6) Projected Design Flows

Based on the above, the PCSWWM model was used to generate projected design flows and reservoir water levels, as summarized in Table 3 on the following page for the Middle Reservoir. Where possible comparative flow values from the 2014 Golder Study have been presented.

Table 3: Peak Flow and Level Results for Middle Reservoir

24 Hour Duration					
	Modelled Peak Flow	Golder's 2014 Predicted Peak Flow	Estimated Spillway Capacity	Modelled Max HGL	Top of Dam Elevation
Return Period	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)
2	7.5	22.7	68.2	86.7	88.3
5	10.6	34.9	68.2	86.9	88.3
1000	27.4	103.6	68.2	87.4	88.3
1/3 1,000 to PMP	30.0	121.5	68.2	87.5	88.3
50,000 (PMP)	34.5	-	68.2	87.6	88.3
48 Hour Duration					
Return Period	Modelled Peak Flow	Golder's 2014 Predicted Peak Flow	Estimated Spillway Capacity	Modelled Max HGL	Top of Dam Elevation
	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)
2	16.6	-	68.2	87.1	88.3
1,000	42.1	-	68.2	87.8	88.3
1/3 1,000 to PMP	43.4	-	68.2	87.9	88.3
50,000 (PMP)	46.0	-	68.2	87.9	88.3
72 Hour Duration					
	Modelled Peak Flow	Golder's 2014 Predicted Peak Flow	Estimated Spillway Capacity	Modelled Max HGL	Top of Dam Elevation
Return Period	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)
2	20.0		68.2	87.2	88.3
1,000	46.7	-	68.2	88.0	88.3
1/3 1000 to PMP	49.4	-	68.2	88.0	88.3
50,000 (PMP)	54.9	-	68.2	88.2	88.3

The governing design event is highlighted in yellow.

Consistent with the calibrated model, projected multi day precipitation events yield higher flows than the 24 hour event. For example, the 2014 Golder Study predicated a peak flow of 22.7 m^3/s at the middle dam for the 1:2 year, 24 hour event. However, based on the calibrated model now developed a 1:2 year 24 hour event would generate a substantially smaller peak flow of only 7.5 m^3/s, but then increase significantly to 16.6 m^3/s and 20.0 m^3/s for the 48 hour and 72 hour duration events, respectively.

It is understood that based on the Dam Safety Regulation, the middle dam must serve the event 1/3 from the 1,000 to the PMP event. For that event, the predicted peak flow based on the above is currently estimated at 49.4 m^3/s (based on a 72 hour duration event), with a minimum available freeboard in the middle reservoir of 0.3 meters.

We thank you for the opportunity to assist the City with this initiative and we look forward to receiving any comments or questions you have at this time.

Sincerely,

URBAN SYSTEMS LTD.

PROVINCE OF G. SHKURHAN BRITISH COLUMBIA ENGINEER

March 21, 2017

Glen Shkurhan P.Eng.
Senior Engineer, Principal

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