RAVINE 10 ECOSYSTEM RESTORATION

APPENDIX I

HYDRAULICS AND HYDROLOGY

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CHAPTER 1. INTRODUCTION

The two issues present in Ravine 10 that were the focus of this project were addressing the exposure of the sanitary sewer line along the floor of the ravine and the removal of the defunct infrastructure present throughout the ravine, both of which contribute to stream fragmentation. A complicating factor, however, is that both of these elements (i.e. the sanitary sewer and the defunct infrastructure) are incidentally serving as grade control structures in some locations, so their removal from the system will require additional consideration.

CHAPTER 2. SITE DESCRIPTION

Ravine 10 is a dendritic ravine system which extends approximately 2,900 feet inland from Lake Michigan and has a tributary area of approximately 500 acres consisting primarily of single family homes. The ravine consists of two primary branches which divide approximately 385 feet upstream of the mouth of the ravine – one branch continuing west and the other running parallel to the shoreline before turning west. Relative to the confluence of the two branches, the project area includes 2,000 feet of the northern branch (North Reach) and 1,350 feet of the southern branch (South Reach).

The St Johns Avenue bridge serves as the upstream project limit of the North Reach. The channel downstream consists of a mix of fine grained sediments and small cobbles without the presence of a well-defined low-flow channel, as shown in Figure 2.1. The majority of the storm sewer outfalls tributary to this branch of the ravine discharge above or shortly downstream of the upstream limit of the project, so the entirety of the reach is relatively flashy and can quickly see a range of flow conditions.

Figure 2.1 Typical channel of the North Reach of Ravine 10

Approximately 1,650 feet downstream of the St. Johns Avenue bridge is the Sheridan Road bridge, at which point the ravine channel constricts, as shown in Figure 2.2. Erosion of the toe of the embankments can be seen, as well as gullies from the two storm sewer outlets discharging to the top of the banks.

Figure 2.2 North Reach channel below Sheridan Road bridge

A short distance upstream from the confluence, a failing gabion structure is slowly tilting into the channel, shifting conveyance against the opposing bank. As the toe of the bank erodes, bank sloughs are creating a narrowing channel, exasperating the erosional effects. The sanitary sewer, however, was not observed along this reach, suggesting that sufficient cover remains to prevent exposure of its crown.

Upstream from the confluence along the South Reach, however, the sanitary sewer quickly becomes exposed and begins affecting the channel morphology as it increasingly becomes exposed, as shown in Figure 2.3. The channel can be characterized as meandering via cut and fill alluviation of stone and cobble substrate. A shallow low-flow channel is readily identifiable while the overbank and adjacent slopes show signs of period high stages, as shown in Figure 2.4.

The project limit terminates at the Sheridan Road bridge crossing the South Reach. A short distance upstream of this point, however, is a rectangular weir constructed from gabions, shown in Figure 2.5. The upstream pool is partially drained by a 12” PVC pipe located just below the crest of the structure.

Figure 2.3 Sanitary Sewer in South Reach of Ravine 10

Figure 2.4 Typical channel within South Reach of Ravine 10

Figure 2.5 Typical channel within South Reach of Ravine 10

CHAPTER 3. HYDROLOGIC AND HYDRODYNAMIC DATA COLLECTION

Data collection and the initial estimation of parameters used to hydrologically model the ravine included in this study are discussed in this section. Data collection activities included subbasin delineation, current land-use, and soils information compilation.

3.1. Subwatershed Delineation

Subwatersheds were generated using HEC-GeoHMS and a 10-foot digital elevation model (DEM) that was provided by Lake County. The DEM was developed using deliverables from a LiDAR survey taken in April 2002 in support of the County's efforts to develop 2-foot contours. The resolution of the DEM was not sufficient to capture the curbs and gutters of the street, so the subwatersheds representing the streets were manually adjusted to reflect the sewers. They were then further divided to facilitate the development of the SWMM model. The original delineation is shown in Figure 3.1 while the final delineation is shown in Figure 3.2.

3.2. Soil Information Compilation

Soil maps created as part of the National Cooperative Soil Survey (NCSS), which is led by the USDA-NRCS, were used for classifying the hydrologic characteristics of the soils in the watershed. The USDA-NRCS soil categorization scheme contains the soils coverage Map Unit Symbol (MUSYM) attribute. The MUSYM is the same soil survey designation as the Soil Conservation Service (SCS) designation and corresponds to the hydrologic soil group for soil types A through D, but has four additional categories. The project area consists of hydrologic soil group type C.

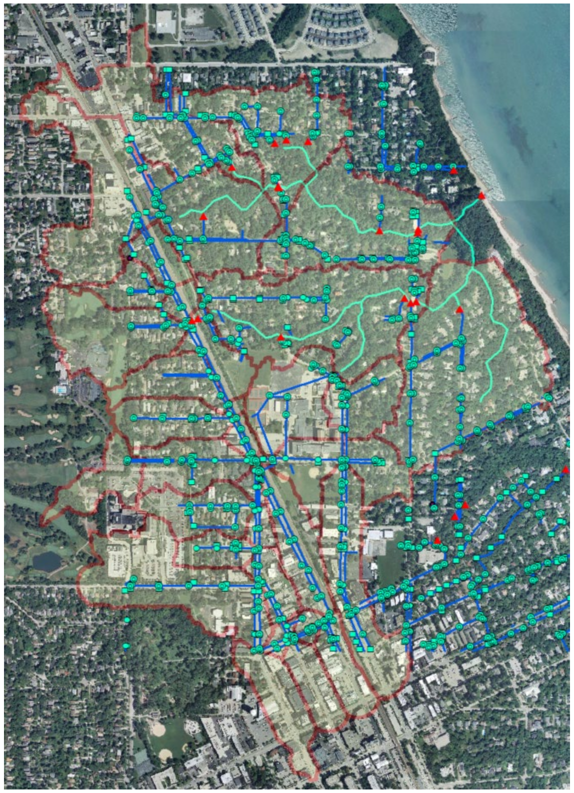


Figure 3.1 Original HEC-GeoHMS Subwatersheds

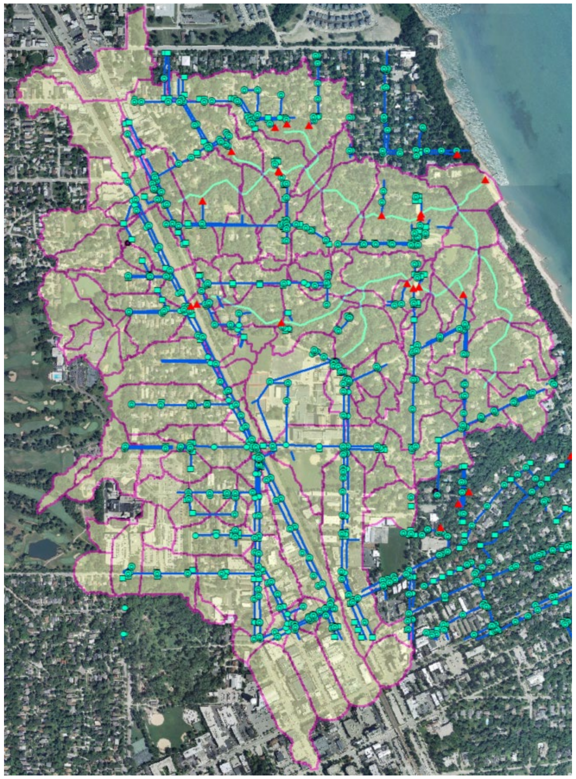


Figure 3.2 Adjusted Ravine 10 Subwatersheds

CHAPTER 4. HYDROLOGIC MODELING USING SWMM 5.0

The hydrology of the ravine's subbasins and the flow routing through the ravines were partially modeled using version 5.0 of EPA's SWMM model. The modeled portion of Ravine 10's watershed is delineated in Figure 4.1, indicated by the cross-hatched region. EPA-SWMM represents the watershed as an interconnected system of subbasins that simulate the precipitation runoff process and hydraulic components that connect the subbasins, and models a storm sewer network through a series of conduits and junctions. Each component is represented by a set of parameters that specify the physical processes. Inputs to the SWMM model include subbasin area and its interconnectivity, hydrologic parameters, and physical characteristics of the storm sewer infrastructure. The result of the model is the hydrographs at the outlets of the storm sewers where they discharge to the ravines.

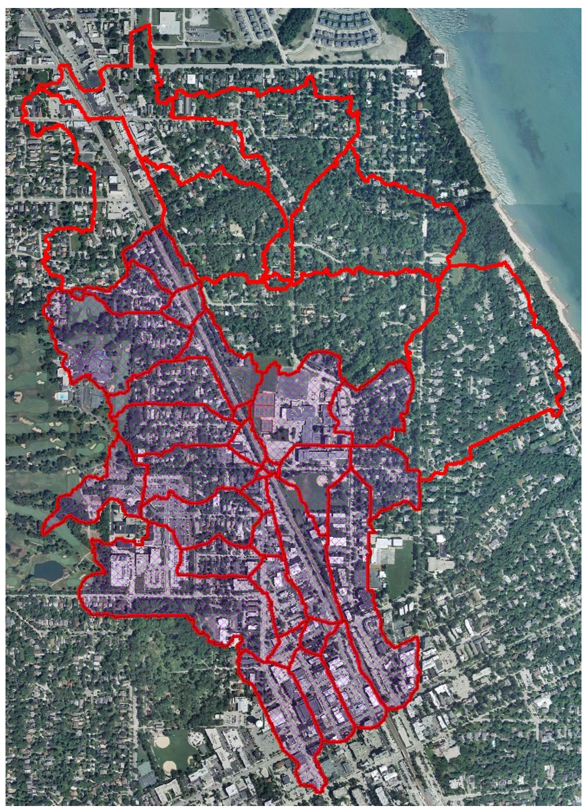


Figure 4.1 Portion of Ravine 10 Modeled in EPA-SWMM 5.0

4.1. Subcatchment Parameters

Subcatchment parameters included such things as overall area, characteristic width, percent slope, percent impervious, Manning's n, depth of depression storage, percentage of impervious area without depression storage, and SCS curve number. Subcatchment area and percent impervious were based on measurements taken from aerial photography in ArcGIS. The characteristic width is defined as the area divided by the longest flow path, which was estimated using topography and aerial photographs. The average percent slope was estimated using the DEM and longest flow path – elevations were taken at distances corresponding to 85% and 15% of the longest lengths and the slope defined using the distance between these points. The values for Manning's n were left at their default values of 0.01 and 0.1 for impervious and pervious areas, respectively. Similarly, the values for depth of depression were left at their default values of 0.05. These values can be used to calibrate the models should data become available later on. The SCS curve number applied to the pervious area only and was defined as 74.

4.2. Storm Sewer Data

Storm sewer data was provided by the local municipality and consisted of information relating to pipe size and invert elevations, structure rim elevations and outfall invert elevations. This information was used as input into the model.

4.3. Model Calibration

Model calibration was used to optimize model parameters to obtain a reasonable fit between observed storm events and modeled simulated events. Calibrating the hydrologic model involved comparing peak flows, volume, and times to peak. EPA-SWMM performs runoff and routing functions. The runoff function determines the amount of precipitation infiltrated, stored in depressions, and converted to runoff, as well as the timing of the runoff. A subbasin with a large percent imperviousness reduces the area available for infiltration, thus increasing the runoff of a subbasin. The subbasin width is used as a parameter to develop a relationship between a subbasin's area and overland flow length and affects the timing of the peak runoff generated by a subbasin. A subbasin with a smaller width would have a correspondingly longer overland flow length, and thus a more attenuated peak compared to the same watershed with a larger width (and therefore shorter overland flow length). The Manning's n value for the storm sewer network can also be adjusted within a certain range to assist in the model calibration. During the calibration process the subbasin width and Manning's n value were adjusted.

Two storm events with observed daily average discharges were available to calibrate the Ravine 10 EPA-SWMM model. The first event occurred on April 11-13, 1994 and was more than a 100% chance exceedance precipitation event (i.e. an exceedance probability likely to be exceeded several times in a given year). The second event occurred on June 23-24, 1994 and approximated a 20% chance exceedance precipitation event. The location of the stream gage is shown in Figure 4.2.

Figure 4.2 Location of Flow Gage in Ravine 10

The rain gage closest to Ravine 10 was Rain Gage #2 of the ISWS rain gage network maintained as part of the Lake Michigan Diversion Accounting program. Hourly rainfall data obtained from this gage was available for both storm events used during model calibration. The location of Rain Gage #2 relative to the Ft. Sheridan ravines and Ravine 10 is shown in Figure 4.3.

Figure 4.3 Location of Rain Gage #2

The baseline modeling results showed that the uncalibrated model approximated the two simulated storm events, but peaked too late and underestimated the daily average flow. In order to decrease runoff attenuation and advance timing of the peak runoff, the Manning's n value and subbasin width were adjusted. The Manning's n value for the storm sewer network was decreased from 0.013 to the minimum published for concrete pipe of 0.011 in order to better align the timing of the peak runoff. The subbasin width was reduced by 50%. The results of the model calibration are shown in Figures 4.4 and 4.5. For the results of the calibration event, shown in Figure 4.4, the simulated results compared to the recorded had an S/R ratio of 1.04 and a correlation coefficient of 0.98. For the verification event, shown in Figure 4.5, the S/R ratio and correlation coefficient were 0.94 and 0.94, respectively. It is recognized that the statistical results for the verification event suggested that the model underperformed for the event, but the calibration event is more representative of the typical design storm event.

The larger of the two storms, the second event, was used to perform the model calibration. Although the daily average values appeared to not have adequately captured the peak discharge for this event, the two values bounding the peak appear to match well. When the verification event was run, it was observed that the model output slightly overestimated the daily average value, but the difference was considered acceptable. Any inaccuracies in the modeling results can be accounted for through a reasonable application of a safety coefficient for proposed measures.

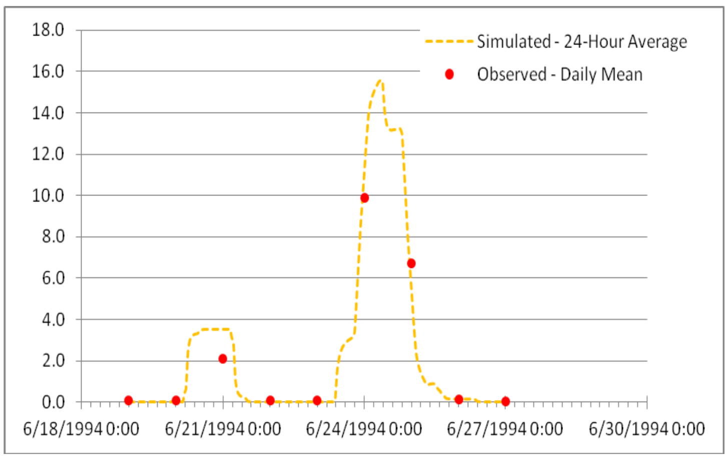


Figure 4.4 Calibration Event, June 23-24, 1994

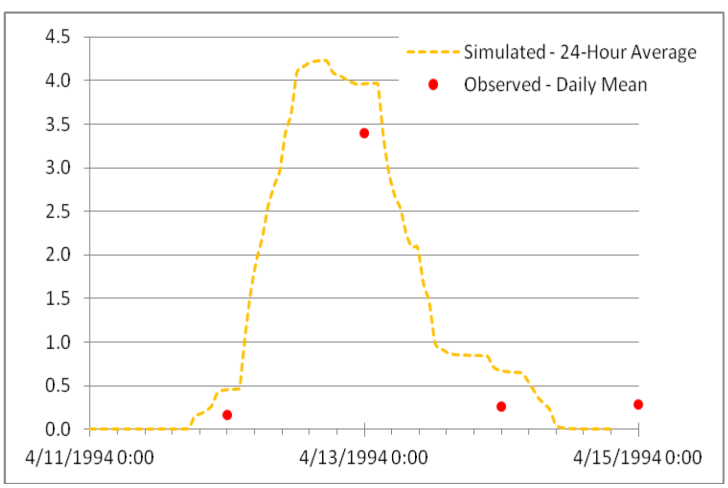


Figure 4.5 Verification Event – April 11-13, 1994

4.4. Synthetic Precipitation Data

While model calibration relied upon observed precipitation data as described in Section 4.3, input for the design of feasibility-level measures relied upon models using synthetic events to drive the output. Rainfall depths were developed using Bulletin 70 values and Huff quartile distributions applicable to areas of less than 10 square miles. The following convention was used when selecting quartiles for the time distribution of rainfall:

- First Quartile: Storms ≤ 6 hrs
- Second Quartile: Storms >6 hrs to ≤ 12 hrs
- Third Quartile: Storms >12 hrs to ≤ 24 hrs
- Fourth Quartile: Storms > 24 hrs

The tables used in the development of the synthetic rainfall depths and a table showing the temporal distributions are shown in Appendix A.

CHAPTER 5. RATIONAL METHOD TO ESTIMATE PEAK DISCHARGE

A portion of the watershed was modeled through EPA-SWMM, but due to time constraints the model of the full sewershed will be developed for final design. The peak discharge for the remaining watersheds must be therefore instead be estimated. For this the rational method was selected. The rational method is a simple technique used to estimate peak runoff from a small drainage area. Although criticized for its simplification of complex systems, it is a widely used and accepted approach to estimating peak discharge. The assumption made is that if a storm event with rainfall intensity continues indefinitely, the runoff will continue to increase until the entire watershed is contributing to the flow at the outlet. The duration for this to occur is called the time of concentration, t_c. The product of these two, with the inclusion of a runoff coefficient, C, is equal to the peak discharge from the watershed.

5.1. Time of Concentration

The time of concentration for each of the remaining watersheds was computed using the method outlined in TR-55. In this, the time of concentration is subdivided into the travel time, T_t, applicable to each of the components of the flow through the watershed. As runoff moves through the watershed, it typically transitions from sheet flow into shallow concentrated flow, until it finally is concentrated into open channel flow. Should a storm sewer network be included in the longest flow path, the travel time within the pipe must be estimated as well.

Sheet flow was defined using Manning's kinematic solution, which is defined as:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}}$$

where n is Manning's roughness coefficient, L is the flow length, P_2 is the 2-year, 24-hour rainfall in inches, and s is the slope of the hydraulic grade line. The maximum length for sheet flow was limited to 100 feet for grassed areas, but increased to 300 feet for paved surfaces.

Sheet flow typically becomes shallow concentrated flow after 100 to 300 feet. Once the watercourse slope has been determined, the average velocity for this flow can be determined from Figure 3.1 of TR-55.

It was assumed that shallow concentrated flow continued until the flow path entered the storm sewer network. The travel time for this component was estimated by determining the average velocity when the pipe was flowing at 80% capacity and multiplying it by the total flow length within the network. When the total run included pipes of differing diameters or slopes, the individual travel time was determined and ultimately summed.

For instances where the longest flow path traveled down a ravine slope, a second shallow concentrated flow computation was completed using the increased slope.

Once the flow path entered the channelized portion of the ravine floor, it was assumed to continue as channel flow. The average velocity for bank full flow was assumed to represent typical conditions during a storm event and determined using Manning's equation. For feasibility-level design a trapezoidal cross-section was assumed based on field photos and measurements taken from elevation data in ArcGIS.

After having found the travel times for each of these flow components, the total time of concentration was determined by summing the values. The table summarizing these computations can be found in Appendix C.

5.2. Parameter Determination

The remaining parameters which needed to be defined include the rainfall intensity, watershed area, and the runoff coefficient. The rainfall intensity was found by interpolating between the intensities found in Atlas 14 corresponding to a 100-year event to find the duration equivalent to the time of concentration. The area was found using ArcGIS. The runoff coefficient was adopted using Table 15.1.1, Runoff coefficients for use in the rational method, in Chow's text, "Applied Hydrology". A coefficient for poor condition grassed areas, average slopes, and a 100-year return interval was adopted for each subwatershed.

5.3. Computation Results

Although the assumption is made that the storm duration is equivalent to the longest travel time in a watershed, the shortest duration available in Bulletin 70 is 1 hour; the 1 hour duration was adopted for any time of concentration shorter than 1 hour.

The results from rational method computation are summarized in Table 5.1.

Table 5.1 Results of Rational Method Computation

	CP #1	CP #2	CP #3	CP #4	CP #5	CP #6
Runoff Coefficient, C	0.53	0.53	0.53	0.53	0.53	0.53
Rainfall Intensity						
Time of Concentration (hr), T_c	0.29	0.32	0.41	0.18	0.18	0.44
Rainfall (in/hr)	4.03	4.03	4.03	4.03	4.03	4.03
Area (acres)	81.6	111.4	141.4	43.6	101.5	243.0
Discharge, Q (cfs)	**174.2**	**238.0**	**302.1**	**93.2**	**216.8**	**519.0**

CHAPTER 6. CLIMATE CHANGE IMPACTS

6.1. Literature Review

USACE is undertaking its climate change preparedness and resilience planning and implementation in consultation with internal and external experts using the best available and actionable climate science. As part of this effort, the USACE has developed concise reports summarizing observed and projected climate and hydrological patterns, at a HUC2 watershed scale cited in reputable peer-reviewed literature and authoritative national and regional reports. Trends are characterized in terms of climate threats to USACE business lines. The reports also provide context and linkage to other agency resources for climate resilience planning, such as downscaled climate data for sub-regions, and watershed vulnerability assessment tools.

The USACE literature review report focused on the Great Lakes Region was finalized in April 2015 (USACE, April 2015). The Ravine 10 watershed is located in the Great Lakes Region. Figure 6.1, taken from the Fourth National Climate Assessment's (NCA4) reported summary of the observed change in very heavy precipitation for the U.S., defined as the amount of precipitation falling during the heaviest 1% of all daily events. The NCA4 results indicate that 42% more precipitation is falling in the Great Lakes Region now as compared with the first half of the 20th century, and that the precipitation is concentrated in larger events.

The USACE literature review document summarizes several studies which have attempted to project future changes in hydrology. Based on a review of four studies, the projected total annual precipitation is expected to have a small increase when compared to the historic record and the precipitation extremes are projected to see a large increase. It is noted that consensus between the studies is low, and although most studies indicate an overall increase in observed average precipitation, there is variation in how these trends manifest both seasonally and geographically. Figure 6.2, taken from the USACE Climate Change and Hydrology Literature Reviews, summarizes observed and projected trends for various variables reviewed.

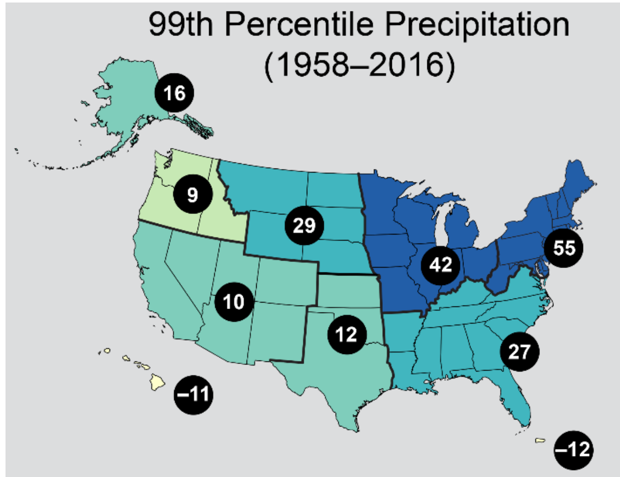


Figure 6.1 Percent changes in precipitation falling in the heaviest 1% of events from 1958 to 2016 for each region

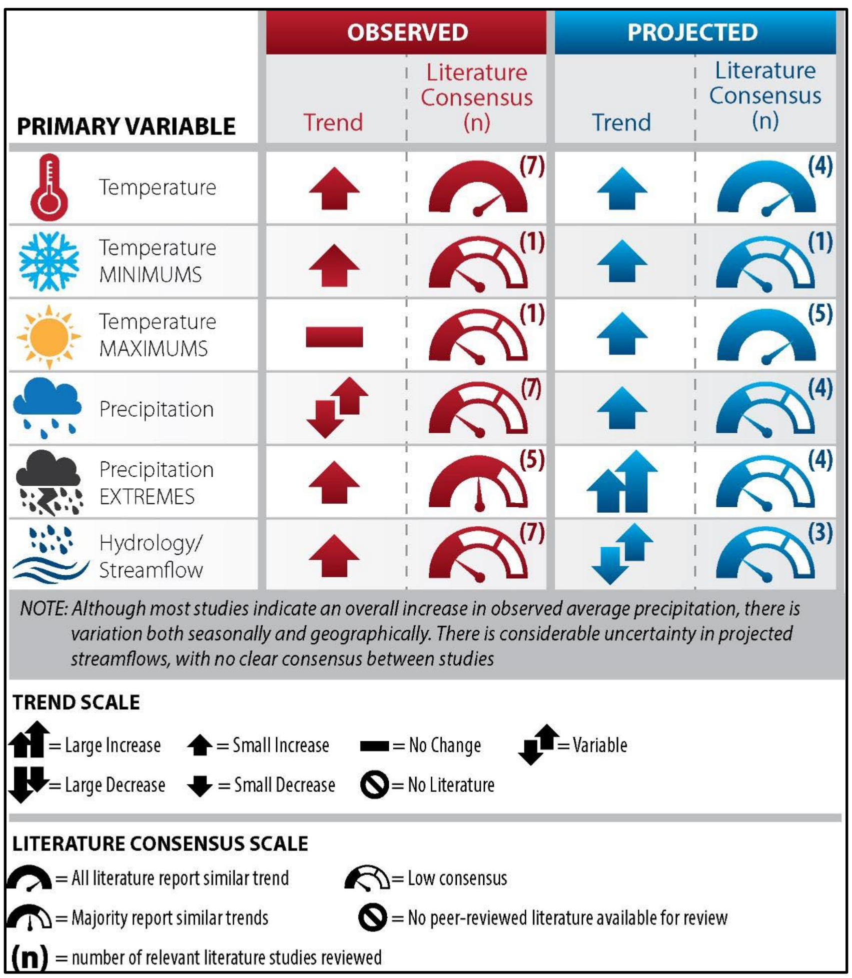


Figure 6.2 Great Lakes Region – Summary matrix of observed and projected climate trends and literary consensus

For the Great Lakes Regions, increase in temperatures have been observed and additional increases in temperature are predicted for the future. In addition, for the Great Lakes Region, “nearly all studies note an upward trend in average temperatures, but generally the observed change is small. Some studies note seasonal differences with possible cooling trends in fall or winter.” There is a strong consensus within the literature that temperatures are projected to continue to increase over the next century.

In some parts of the region increases in streamflow have been observed. Future projections of streamflow rates are highly variable. For the Great Lakes region, trends in observed low and annual streamflow were variable, with slight streamflow increases observed at some gages, but other gages showing no significant changes. “Significant uncertainty exists in projected runoff and streamflow, with some models projecting increases and others decreases. Changes in runoff and streamflow may also vary by season. Projections of water levels in the Great Lakes also have considerable uncertainty, but overall lake levels are expected to drop over the next century.”

6.2. First Order Statistical Analysis and Nonstationarity Analysis

There is one stream gage close to the project area, 05535070, Skokie River near Highland Park, Illinois. The drainage area for this gage is 21.1 square miles. The gage has a period of record from 1986 to present day for various stream statistics including peak streamflow and daily discharge data.

6.2.1. Climate Hydrology Assessment Tool

As outlined in ECB No. 2018-14, an investigation of the trends in the annual maximum flow gage data was performed to qualitatively assess impacts of climate change within the watershed using the USACE Climate Hydrology Assessment Tool. For the Skokie River, Figure 6.3 below shows the instantaneous peak streamflow obtained from the USGS website for gage closest to the project site. The figures depict a trend towards increasing annual peak streamflow for the period of record, as represented by the gage trendline. The p-value for the gage trendline is 0.0002794, however, suggesting statistical significance to the possibility that this relationship could be a false positive. Figure 6.4 displays the projected annual maximum monthly trends from the USACE Climate Hydrology Assessment Tool. As expected for this type of qualitative analysis, there is a considerable, but consistent spread in the projected annual maximum monthly flows. This spread is indicative of the uncertainty associated with climate changed hydrology. The trend in the mean projected annual maximum monthly streamflow indicates an increase over time.

6.2.2. Nonstationarity Detection Tool

Stationarity is the assumption that the statistical characteristics of hydrologic time series data are constant through time. The stationarity assumption enables the use of well-accepted statistical methods in water resources planning and design in which the definition of future conditions relies primarily on the observed record. However, recent scientific evidence shows that in some locations climate change and human modifications of watersheds are undermining this fundamental assumption, resulting in nonstationarity (Milly et al., 2008, Friedman, et. al, 2016).

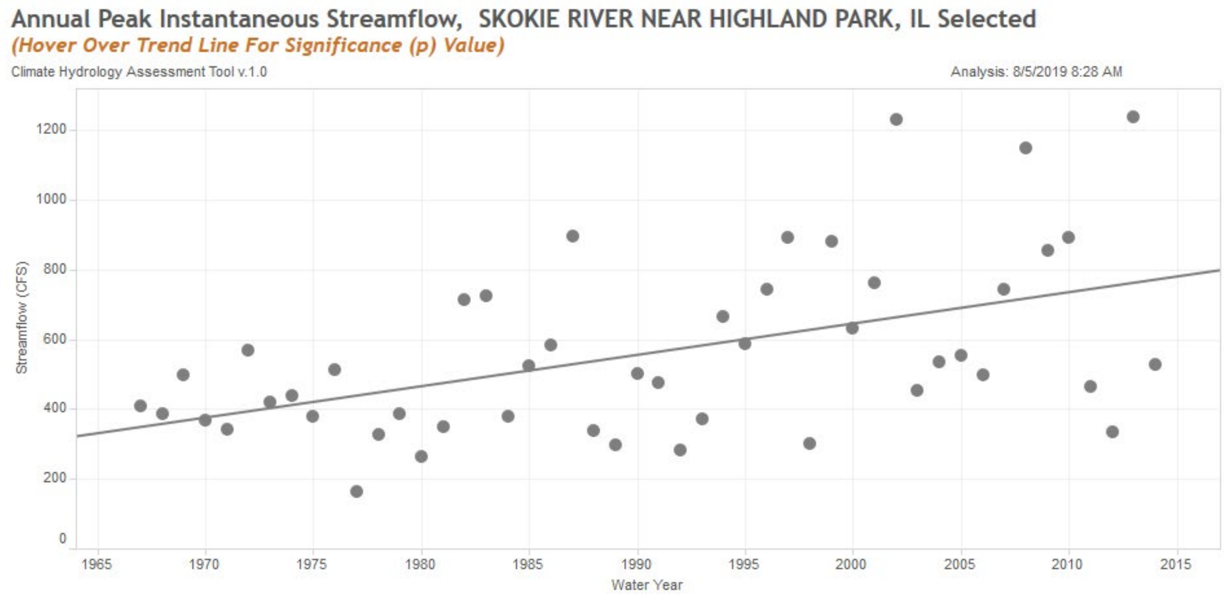


Figure 6.3 Peak Streamflow for Skokie River near Highland Park, IL

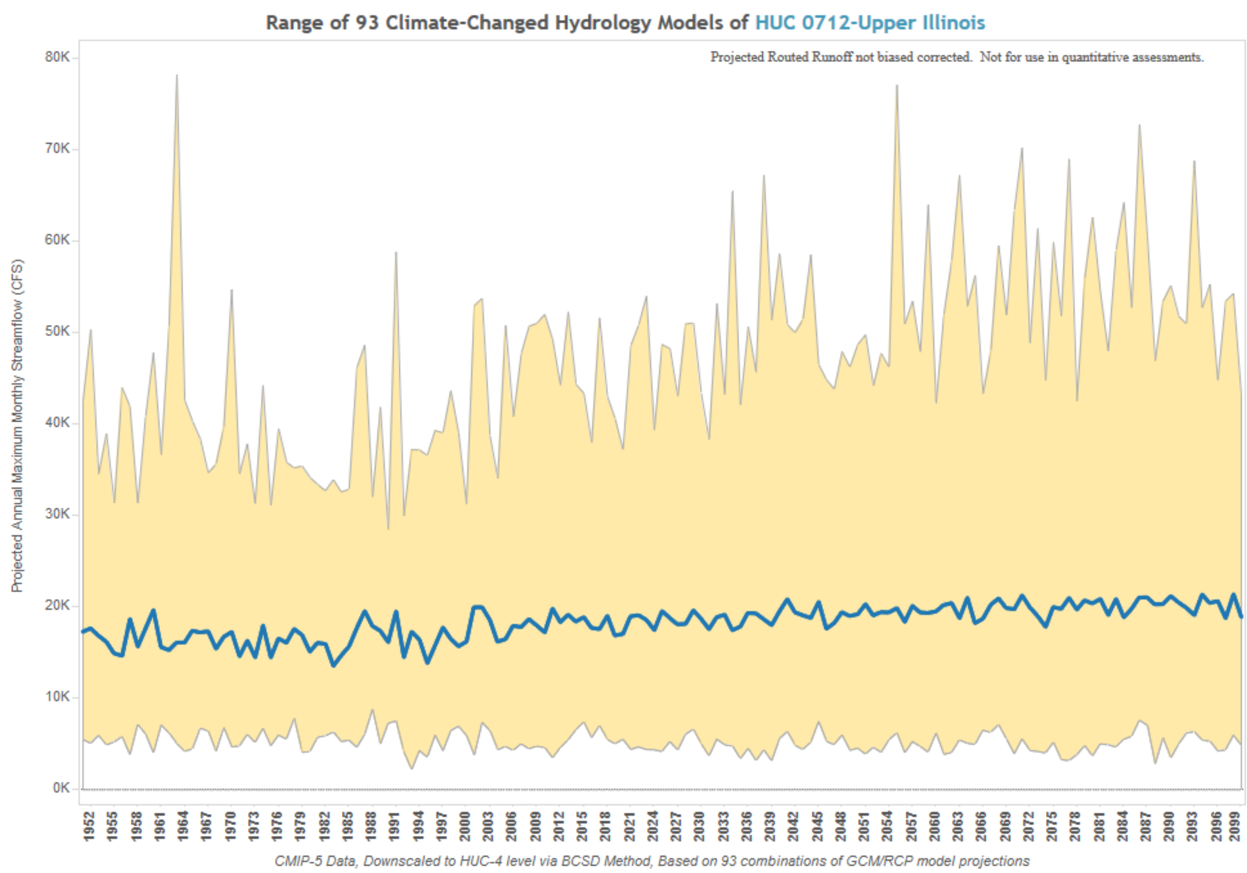


Figure 6.4 Peak Streamflow for Skokie River near Highland Park, IL

An assessment of historic gage records was performed to determine if nonstationarity exists within the Skokie River watershed. This was accomplished by carrying out a nonstationarity detection analysis using the USACE's Nonstationarity Detection (NSD) Tool. The nonstationarity analysis conducted as part of this study was generated using the default settings in the NSD tool. The USACE NSD tool uses twelve nonparametric and parametric tests to identify abrupt or smooth changes in the distribution, mean, and variance of annual flood time series data.

For USGS 05535070, Skokie River near Highland Park, IL gage, one abrupt nonstationarity was detected, as shown in Figure 6.5. Nonstationarities were detected at one point within the period of record: 1993. There is no consensus between the various tests for distribution, mean, and/or variance. In 1978, the nonstationarity detected corresponds to changes of about 268 cfs in mean. There is no change in standard deviation or variance.

Based on these results, since the dataset does not show consensus and cannot be considered robust, nonstationarities within the dataset do not exist. This is further supported when assessing monotonic trends within the record, as shown in Figure 6.6.

6.2.3. Vulnerability Assessment Tool

The USACE Vulnerability Assessment Tool was to be applied for the 0712-Upper Illinois HUC-4 to assess the project's vulnerability to climate change impacts relative to the other 201 HUC-4 watersheds within the United States. The USACE Watershed Climate Vulnerability Assessment (VA) Tool facilitates a screening level, comparative assessment of the vulnerability of a given HUC 04 watershed to the impacts of climate change relative to a maximum of 202 (depending on which business line is specified) HUC04 watersheds within the continental United States (CONUS). Assessments using this tool identify and characterize specific climate threats and sensitivities or vulnerabilities, at least in a relative sense, across regions and business lines. Ecosystem Restoration is the primary business line being assessed as part of this Feasibility Study.

The Watershed Vulnerability tool uses the Weighted Order Weighted Average (WOWA) method to represent a composite index of how vulnerable (vulnerability score) a given HUC04 watershed is to climate change specific to a given business line by using a set of specific indicator variables which relate to a particular business line. The HUC04 watersheds with the top 20% of WOWA scores are flagged as vulnerable. All vulnerability assessment analyses were performed using the National Standard Settings.

The USACE Climate Vulnerability Assessment Tool makes an assessment for two 30-year epochs centered at 2050 and 2085 to judge future risk due to climate change. These two epochs are selected to be consistent with many other national and international analyses related to climate. The Vulnerability tool assesses climate change vulnerability for a given business line using climate changed hydrology based on a combination of projected climate outputs from the general circulation models (GCM) and representative concentration pathway (RCPs) of greenhouse gas emissions resulting in 100 traces per watershed per time period. The top 50% of the traces is called "wet" and the bottom 50% of traces is called "dry." Meteorological data

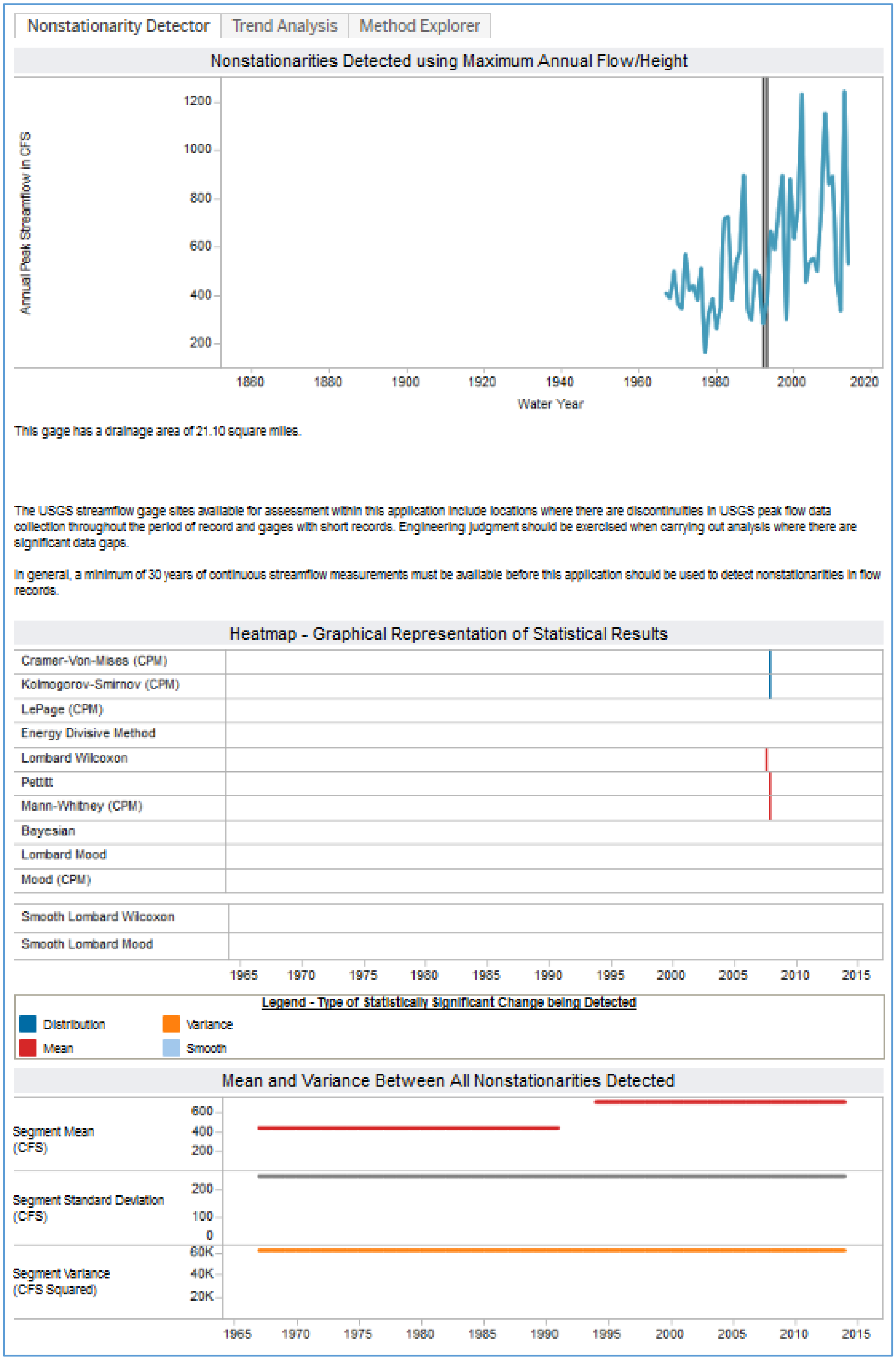


Figure 6.5 Nonstationarity Analysis, Skokie River near Highland Park, IL

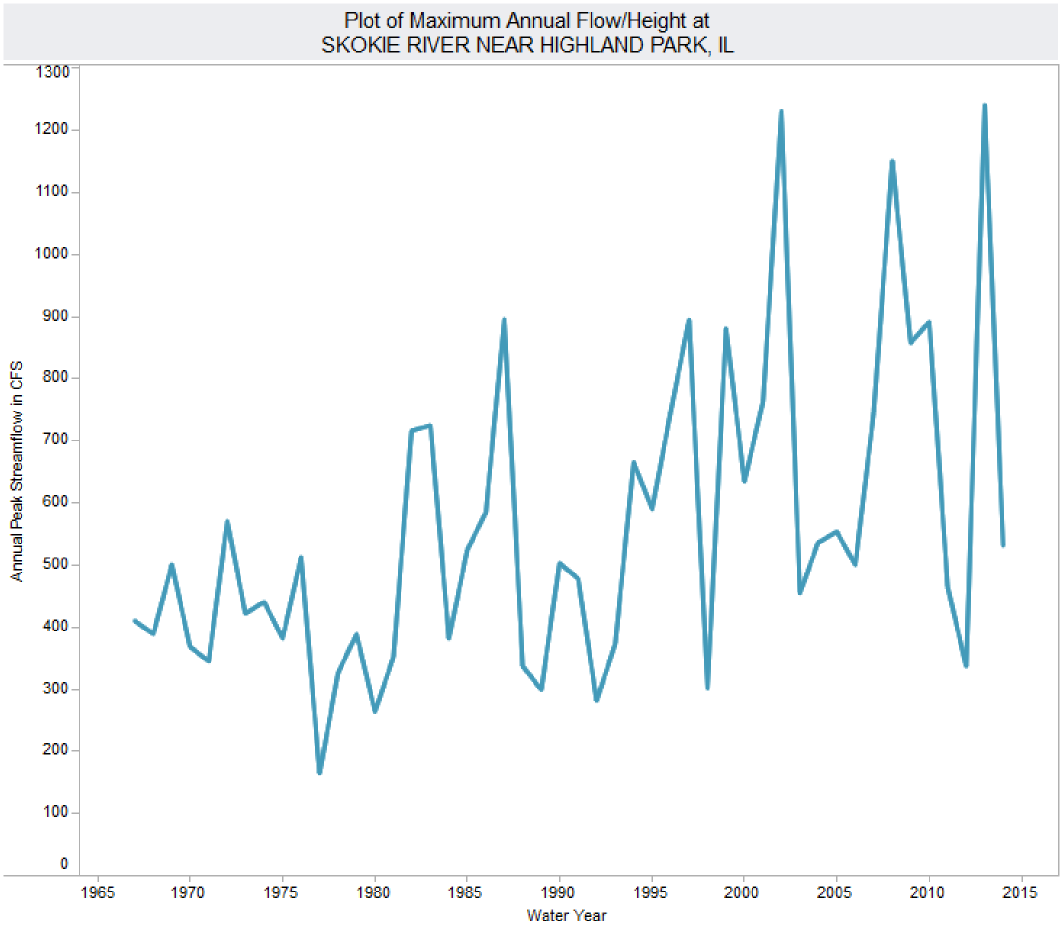


Figure 6.6 Trend Analysis for Skokie River near Highland Park, IL

projected by the GCMs is translated into runoff using the Variable Infiltration Capacity (VIC) macroscale hydrologic model. The VIC model applied to generate the results used by the Vulnerability Assessment Tool was developed by the U.S. Bureau of Reclamation and is configured to model unregulated basin conditions.

At the time of this report, the USACE Watershed Climate Vulnerability Assessment (VA) Tool was not available for use; an assessment of the HUC-4 watershed 0712-Upper Illinois was not possible.

Based on the tools utilized above, it does not appear that the project area will be significantly impacted by climate change.

CHAPTER 7. DETERMINATION OF DESIGN VELOCITY

Preliminary design of the proposed project features is based on a combination of design flow rates and velocities. For the purposes of this analysis, a simple representation of the channel conditions is adequate. From this assumed channel geometry, a design velocity can be determined.

7.1. Combination of Peak Flow Estimates

Although it is recognized that this simplification of the channel hydraulics will result in a conservative estimate of peak discharge rates at a given point, it is assumed that the peak discharges determined by the EPA-SWMM model and through the Rational method are cotemporal at each of the computation points. The results of combining these peak flows is summarized in Table 6.1.

Table 7.1 Cumulative Peak Flow Rates

	CP #1	CP #2	CP #3	CP #4	CP #5	CP #6
Rational method Peak Flow	174.2	238.0	302.1	93.2	216.8	519.0
EPA-SWMM Peak Flow	-	-	-	55.7	55.7	55.7
Total Discharge, Q (cfs)	**174.2**	**238.0**	**302.1**	**148.9**	**272.5**	**574.7**

7.2. Cross-Section Geometry

The geometry data used in the velocity computation was developed using field observations. For simplicity of calculations during the feasibility level analysis, the channel was assumed to consist of a shallow trapezoidal low-flow channel with a wider overflow channel, also trapezoidal in shape. The low-flow channel was assumed to be approximately 0.7 feet deep, 3 feet wide at its base, and have 2:1 side slopes. The full channel width was approximately 15 feet wide with 2:1 side slopes.

7.3. Friction Roughness Factors

Manning's n values were determined using notes taken from field observations and photographs taken of the site. The median Manning's n value for a cobble substrate surface, 0.035, was selected as the initial value.

7.4. Channel Slope

The slope of the channels was based on a 10-foot DEM developed using Lake County LiDAR elevation data. For the purposes of the feasibility study, the channel slope identified between the elevation corresponding to points at the upstream limit of the project and immediately upstream of the confluence near the mouth to be representative of the total reach. It is recognized, however, that there are elements within the reach which are already serving as grade control features, so it is likely that shallower slopes currently exist.

7.5. Design Velocity

A maximum permissible velocity, corresponding to clear water flow with a channel bed consisting of graded colloidal silts and cobbles, of 4.0 ft/s was adopted for this analysis. When determining the design velocity to use in the stone and structure sizing calculations, the same typical cross-section was identified for each reach. The channel slope and corresponding normal flow depth were varied to attain the velocity within the maximum permissible velocity. In both reaches, a channel slope of 0.005 ft/ft corresponded to velocities just within the permissible limit. It is recognized that the design velocities will not be sufficient to halt bank erosion and will likely result in the suspension and transport of fine material.

The computations of the design velocity and corresponding cross-sections are shown in Appendix D.

CHAPTER 8. PROPOSED REMEDIATION METHODS

This chapter details the toolbox of measures employed in this study and any assumptions that may have been made. Individual design calculations for the measures included in the recommended plan can be found in Appendix D.

8.1. Step-Pool Design

Grade control features provide a mechanism for mitigating the erosive effects of flow over an excessively steep slope, relative to one which is free of downcutting. They also provide the additional benefit of providing an engineered scour hole at the base of the structure that assists in the diffusion of energy. It must be noted that although the computations performed are for step-pools, an alternative structure accommodating a similar elevation drop may later be selected.

The consideration for the inclusion of step-pools within a reach of a ravine was based on a velocity determined using Manning's equation, a representative cross-section for the given reach, and an average channel slope. It was assumed that flow velocities exceeding 4.0 feet per second would result in the suspension of fine sediments.

The design methodology adopted for this project is detailed in D.B. Thomas' paper titled "A Design Procedure for Sizing Step-Pool Structures". In this method, the step-pool height is determined. This is established based on the resulting elevation difference between the existing slope and a limiting slope, S_L, for a given reach. Based on field observations of the apparent quality and size of the channel substrate, an assumption was made of the limiting slope. Once it had been confirmed that the resultant velocity for these cross-sections was below the threshold value, the channel slope was measured and used as the limiting slope. The variables considered are illustrated in the figure below.

Having identified a limiting slope, the relationship used to compute the amount of drop removed between the riffles is as follows:

$$H = (S_O - S_L)x$$

In this equation, H is the amount of drop removed in the reach, S_O is the original bed slope, S_L is the limiting bed slope, and x is the length of the reach. The maximum total drop across the length of a riffle was typically based on the depth of channel incision within the reach. The total drop removed in the reach, H, was divided by the drop across each structure to determine the required number of structures.

Next, the active channel width and weir width are specified. There is not significant variation along the floor of the ravine within a given reach, allowing for an assumption that a representative active channel width can be estimated based on discharge resulting from the 50% chance exceedance event.

The pool length can then be found with the following equation:

$$\frac{L_2}{L_3} = 0.409 + 4.211\frac{H}{L_3} + 87.341\frac{S_o q_{25}}{\sqrt{g(L_3)^{2/3}}}$$

where L_2 is the pool length, L_3 is the average channel width, H is the weir height, q_{25} is the unit discharge for the 4% chance exceedance event and S_o is the channel slope.

The estimated scour depth is computed with the following equation:

$$\frac{S}{L_3} = -0.0118 + 1.394\frac{H}{L_3} + 5.514\frac{S_o q_{25}}{\sqrt{g(L_3)^{2/3}}}$$

where S is the scour depth.

The effective width at the downstream control is defined by the equation:

$$B_5 = 0.92 \cdot B_1$$

The maximum pool width is defined by the equation:

$$B_3 = 1.2 \cdot B_1$$

The distance to the maximum scour depth, L_1, was assumed to occur at a distance of 1/3 the total pool length, L_2.

Although the equations are based on the 4% chance exceedance event discharge, due to the risk associated with failure of the structures the pool dimensions and stone sizing is based on 1% chance exceedance events. A sanitary sewer runs below the floor of the ravine; should one of the structures fail, the sewer line would be at risk to exposure and failure should corrective action not be taken in an adequate amount of time. Failure of a structure would likely result in a headcut that would migrate upstream, eventually either naturally stabilizing as substrates sort, eroding back to the next upstream feature where it propagates further, or exposing the sanitary sewer which then shores up the slope but again fragments the stream. It is expected that the pools will naturally fill in with alluvial material to be more consistent with events of smaller return periods. Should a larger event occur, the fill would flush out, exposing the armor stone below.

Once the physical parameters of the step-pool have been defined, the proposed design is checked to ensure that the specified structure height and spacing meet the required elevation drop based on the difference between the proposed and existing channel slopes. If necessary, the elevation drop per structure or overall number of structures can be adjusted so that the overall required elevation drop can be accommodated within the available horizontal distance.

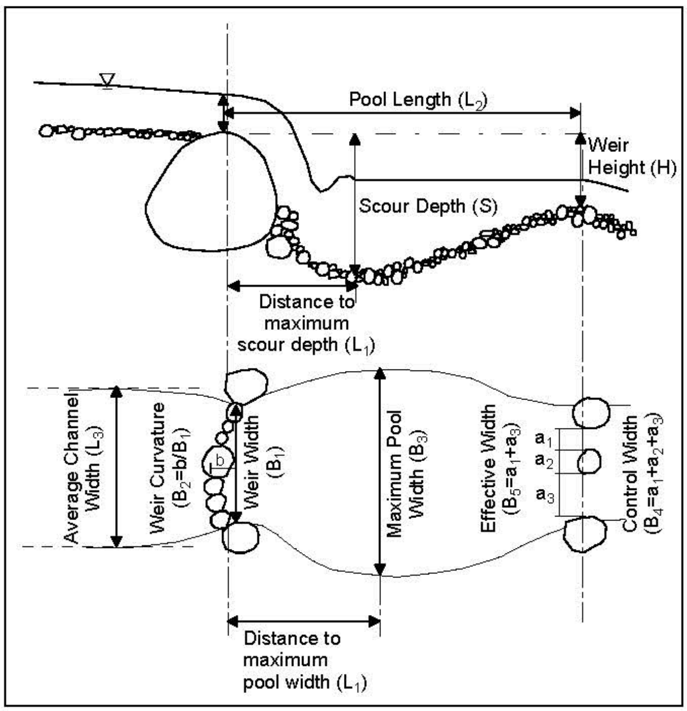


Figure 8.1 Plan and Profile of Step-Pool

Once the structure dimensions have been determined, the required stone size is established by comparing the results from three different stone sizing equations. The first formulation is the Corps' steep-slope equation, described in EM 1110-2-1601. This equation is defined as

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}}$$

where S is the bed slope and q is the unit discharge.

The second formulation relates velocity to the stable stone diameter and is found on Chart 712-1 in Hydraulic Design Criteria (USACE 1970). This equation is defined as

$$V = C\left[2g\left(\frac{\gamma_S - \gamma_w}{\gamma_W}\right)\right]^{0.5} D_{50}^{0.5}$$

The third formulation is the basic equation for stone sizing found in EM 1110-2-1601:

$$D_{30} = S_f C_S C_V C_T d\left[\left(\frac{\gamma_W}{\gamma_S - \gamma_W}\right)^{0.5} \frac{V}{\sqrt{K_1 g d}}\right]^{2.5}$$

Combinations of depth and velocity for each of the 1% chance exceedance were compared to determine which created the limiting stone size. The results from each of these three formulations were compared and the maximum stone size established as the controlling value.

The required thickness of the structures was specified as the greater of $1xD_{100(max)}$ and $1.5xD_{50(max)}$.

When constructing the step-pools, the larger stones in the gradation will be used to construct the weir and step, while the smaller stones will be used to construct the pool. The weir will be constructed with a slight depressed v-shape to ensure that the flow is directed towards the centerline of the pool and channel.

8.2. Riffle Design

The typical design methodology for riffles adopted for this project is detailed in EMSR 4-XX. In this method, the riffle spacing is based on a limiting slope, S_L. Similar to step-pools, the amount of drop removed from a reach required to be removed from a reach can be computed as follows:

$$H = (S_O - S_L)x$$

The total drop removed in the reach, H, is divided by the drop across each structure to determine the required number of structures. This number is typically established by minimizing the number of riffles required without exceeding 12”.

The configuration of the riffles consists of a series of stones constructed with an upstream slope of 1V:4H and a downstream slope of 1V:10H. The design documentation recommends an interior angle of approximately 120 degrees, but there can be significant variability to this estimate. A narrow channel, which many of the reaches could be characterized as being, may prohibit any shape other than a line of stones perpendicular to the channel. The angle of the stones will deflect flows from the banks and thus provide a measure of local bank protection. To reduce the possibility of flanking, the entire structure should be keyed into the banks.

CHAPTER 9. MEASURE IDENTIFICATION

9.1. SMC – Stream Morphology & Connectivity

This measure seeks to maintain connectivity to the lake while addressing the downcutting within the channel. Riffle complexes, the final location of which would be determined later in the design process, would be constructed to ultimately decrease the reach slopes to 0.005 ft/ft.

A minimum of twenty step-pools would be constructed in the North Reach using D_{30} = 1.2 foot stone. An elevation drop of 9” would be achieved across each structure. A minimum of eight step-pools would be constructed in the South reach using D_{30} = 0.9 foot stone. An elevation drop of 6” would be achieved across each structure. Additional structures may be added at the discretion of Corps ecologists.

REFERENCES

Biedenharn, D. S. and Hubbard, L. C. *Design Considerations for Siting Grade Control Structures.* CHETN-VII-3. U.S. Army Corps of Engineers, Washington, D.C. December 2001.

Cowan, W.L. *Estimating Hydraulic Roughness Coefficients.* Agricultural Engineering, 37(7), pp. 473-475, 1956.

Chow, V.T., et al. *Applied Hydrology*. New York: McGraw-Hill, 1988.

Fripp, J., Fischenich, C., and Biedenharn, D. *Low Head Stone Weirs*. Technical Note EMSR 4-XX. United States Department of Agriculture. Date Unknown

Huff, F.A. *Time Distributions of Heavy Rainstorms in Illinois (Circular 173)*. Illinois State Water Survey, 1990.

Lindeburg, Michael, R. *Civil Engineering Reference Manual*. Belmont, California: Professional Publications, Inc., 2008.

Natural Resources Conservation Service (Formerly Soil Conservation Service). *Urban Hydrology for Small Watersheds: Technical Release 55*. Second Edition, June 1986.

Sturm, T.W. *Open Channel Hydraulics*. New York: McGraw-Hill, 2001.

U.S. Army Corps of Engineers. *Hydraulic Design of Flood Control Channels*. EM 1110-2-1601. June 1994.

U.S. Department of Agriculture – Soil Conservation Service. *Technical Release 55 – Urban Hydrology for Small Watersheds*. 210-VI-TR-55. U.S. Department of Agriculture. June 1986.

Appendix A.

REFERENCES FOR DETERMINING SYNTHETIC PRECIPITATION

Frequency Distributions of Heavy Precipitation in Illinois: Updated Bulletin 70
LCSMC Watershed Development Ordinance, Huff Quartile Distributions

Updated Bulletin 70. Illinois State Water Survey. 2019.

Table 4 Storm and Sectional Codes for Table 5

Storm Code		Sectional Code	
1	240 hours	1	Northwest
2	120 hours	2	Northeast
3	72 hours	3	West
4	48 hours	4	Central
5	24 hours	5	East
6	18 hours	6	West Southwest
7	12 hours	7	Southeast
8	6 hours	8	Southwest
9	3 hours	9	Southeast
10	2 hours	10	South
11	1 hour		

		Rainfall (inches) for given recurrence interval						
Storm code	*Section code*	*2-year*	*5-year*	*10-year*	*25-year*	*50-year*	*100-year*	*500-year*
1	1	5.48	6.86	7.98	9.55	10.84	12.14	15.65
1	2	5.60	7.09	8.25	9.90	11.26	12.65	16.00
1	3	5.62	7.00	8.10	9.60	10.65	11.64	13.99
1	4	5.46	6.87	8.04	9.53	10.55	11.50	13.65
1	5	5.50	6.84	7.90	9.35	10.45	11.55	13.96
1	6	6.00	7.38	8.47	9.95	10.99	11.95	14.08
1	7	6.57	7.86	8.90	10.20	11.20	12.06	13.95
1	8	6.75	8.18	9.30	10.80	11.95	13.10	15.95
1	9	7.06	8.30	9.22	10.37	11.21	11.96	13.75
1	10	6.36	7.65	8.76	10.40	11.66	12.96	16.20
2	1	4.35	5.51	6.46	7.88	8.96	10.20	13.33
2	2	4.42	5.63	6.68	8.16	9.39	10.66	13.81
2	3	4.51	5.66	6.62	7.94	8.93	9.83	11.99
2	4	4.27	5.42	6.42	7.75	8.72	9.60	11.54
2	5	4.34	5.43	6.41	7.73	8.79	9.80	11.93
2	6	4.49	5.60	6.49	7.77	8.69	9.57	11.53
2	7	5.00	6.11	7.01	8.23	9.11	9.95	11.71
2	8	5.31	6.51	7.47	8.79	9.81	10.84	13.45
2	9	5.73	6.78	7.60	8.64	9.47	10.20	11.97
2	10	5.18	6.30	7.29	8.69	9.78	10.91	13.84
3	1	3.90	4.95	5.87	7.21	8.30	9.45	12.30
3	2	3.97	5.08	6.05	7.49	8.64	9.85	12.81
3	3	4.11	5.18	6.08	7.34	8.31	9.18	11.27
3	4	3.88	4.96	5.90	7.17	8.09	8.98	10.81
3	5	3.88	4.90	5.78	7.04	8.01	8.93	11.00
3	6	4.00	5.00	5.83	7.01	7.91	8.73	10.61
3	7	4.35	5.37	6.19	7.34	8.19	8.97	10.57
3	8	4.74	5.82	6.71	7.96	8.89	9.86	12.32
3	9	5.13	6.09	6.86	7.87	8.63	9.34	10.93
3	10	4.54	5.61	6.50	7.78	8.79	9.86	12.55

		Rainfall (inches) for given recurrence interval						
Storm code	Section code	2-year	5-year	10-year	25-year	50-year	100-year	500-year
4	1	3.61	4.59	5.43	6.72	7.73	8.83	11.53
4	2	3.66	4.71	5.62	6.99	8.13	9.28	12.10
4	3	3.76	4.76	5.62	6.81	7.72	8.60	10.58
4	4	3.59	4.61	5.47	6.65	7.55	8.40	10.21
4	5	3.54	4.49	5.32	6.48	7.38	8.27	10.26
4	6	3.66	4.61	5.38	6.48	7.33	8.11	9.93
4	7	3.92	4.85	5.61	6.67	7.46	8.21	9.76
4	8	4.28	5.29	6.10	7.25	8.15	9.08	11.40
4	9	4.64	5.54	6.27	7.24	7.94	8.58	10.06
4	10	4.06	5.02	5.86	7.04	8.01	9.02	11.56
5	1	3.34	4.22	5.03	6.20	7.20	8.25	10.84
5	2	3.34	4.30	5.15	6.45	7.50	8.57	11.24
5	3	3.48	4.45	5.24	6.38	7.25	8.06	9.91
5	4	3.32	4.30	5.10	6.20	7.05	7.85	9.53
5	5	3.12	3.97	4.71	5.78	6.62	7.43	9.32
5	6	3.23	4.07	4.76	5.79	6.56	7.31	9.04
5	7	3.49	4.33	5.00	5.98	6.71	7.40	8.84
5	8	3.69	4.56	5.27	6.30	7.14	7.96	10.06
5	9	4.07	4.89	5.55	6.42	7.06	7.68	8.99
5	10	3.63	4.52	5.28	6.38	7.29	8.23	10.57
6	1	3.14	3.97	4.73	5.83	6.77	7.75	10.19
6	2	3.14	4.04	4.84	6.06	7.05	8.06	10.57
6	3	3.27	4.18	4.93	6.00	6.82	7.58	9.32
6	4	3.12	4.04	4.79	5.83	6.63	7.38	8.96
6	5	2.93	3.73	4.43	5.43	6.22	6.98	8.76
6	6	3.04	3.83	4.47	5.44	6.17	6.87	8.50
6	7	3.28	4.07	4.70	5.62	6.31	6.96	8.31
6	8	3.47	4.29	4.95	5.92	6.71	7.48	9.45
6	9	3.83	4.60	5.22	6.03	6.64	7.22	8.45
6	10	3.41	4.25	4.96	6.00	6.85	7.73	9.93
7	1	2.91	3.67	4.38	5.40	6.26	7.18	9.43
7	2	2.91	3.74	4.48	5.61	6.53	7.46	9.78
7	3	3.03	3.87	4.56	5.55	6.31	7.01	8.62
7	4	2.89	3.74	4.44	5.39	6.13	6.83	8.29
7	5	2.71	3.45	4.10	5.03	5.76	6.46	8.11
7	6	2.81	3.54	4.14	5.04	5.71	6.36	7.86
7	7	3.04	3.77	4.35	5.20	5.84	6.44	7.69
7	8	3.21	3.97	4.58	5.48	6.21	6.93	8.75
7	9	3.54	4.25	4.83	5.59	6.14	6.69	7.82
7	10	3.16	3.93	4.59	5.55	6.34	7.16	9.19

Storm code	Section code	Rainfall (inches) for given recurrence interval 2-year	5-year	10-year	25-year	50-year	100-year	500-year
8	1	2.51	3.17	3.77	4.65	5.40	6.19	8.13
8	2	2.51	3.23	3.86	4.84	5.63	6.43	8.43
8	3	2.61	3.34	3.93	4.79	5.44	6.05	7.43
8	4	2.49	3.23	3.83	4.65	5.29	5.89	7.15
8	5	2.34	2.98	3.53	4.34	4.97	5.57	6.99
8	6	2.42	3.05	3.57	4.34	4.92	5.48	6.78
8	7	2.62	3.25	3.75	4.49	5.03	5.55	6.63
8	8	2.77	3.42	3.95	4.73	5.36	5.97	7.54
8	9	3.05	3.67	4.16	4.82	5.30	5.76	6.74
8	10	2.72	3.39	3.96	4.79	5.47	6.17	7.92
9	1	2.14	2.70	3.22	3.97	4.61	5.28	6.94
9	2	2.14	2.75	3.30	4.13	4.80	5.49	7.20
9	3	2.23	2.85	3.35	4.08	4.64	5.16	6.34
9	4	2.12	2.75	3.26	3.97	4.51	5.02	6.10
9	5	2.00	2.54	3.01	3.70	4.24	4.76	5.97
9	6	2.07	2.60	3.05	3.71	4.20	4.68	5.79
9	7	2.23	2.77	3.20	3.83	4.29	4.74	5.66
9	8	2.36	2.92	3.37	4.03	4.57	5.09	6.44
9	9	2.60	3.13	3.55	4.11	4.52	4.92	5.75
9	10	2.32	2.89	3.38	4.09	4.66	5.26	6.76
10	1	1.94	2.45	2.92	3.60	4.17	4.78	6.29
10	2	1.94	2.49	2.99	3.74	4.35	4.97	6.52
10	3	2.02	2.58	3.04	3.70	4.21	4.67	5.75
10	4	1.93	2.49	2.96	3.60	4.09	4.55	5.53
10	5	1.81	2.30	2.73	3.35	3.84	4.31	5.41
10	6	1.87	2.36	2.76	3.36	3.80	4.24	5.24
10	7	2.02	2.51	2.90	3.47	3.89	4.29	5.13
10	8	2.14	2.64	3.06	3.65	4.14	4.62	5.83
10	9	2.36	2.84	3.22	3.72	4.09	4.46	5.21
10	10	2.10	2.62	3.06	3.70	4.23	4.77	6.13
11	1	1.57	1.98	2.36	2.92	3.38	3.88	5.09
11	2	1.57	2.02	2.42	3.03	3.53	4.03	5.28
11	3	1.64	2.09	2.46	3.00	3.41	3.79	4.66
11	4	1.56	2.02	2.40	2.91	3.31	3.69	4.48
11	5	1.47	1.87	2.21	2.72	3.11	3.49	4.38
11	6	1.52	1.91	2.24	2.72	3.08	3.44	4.25
11	7	1.64	2.04	2.35	2.81	3.15	3.48	4.15
11	8	1.73	2.14	2.48	2.96	3.36	3.74	4.73
11	9	1.91	2.30	2.61	3.02	3.32	3.61	4.23
11	10	1.71	2.12	2.48	3.00	3.43	3.87	4.97

Watershed Development Ordinance. Lake County Stormwater Management Commission. 2008.

HUFF QUARTILE DISTRIBUTIONS												
CUMUL.	AREA < 10 SM				AREA > 10 & AREA < 50				AREA > 50 & AREA < 400			
STORM	HUFF QUARTILE				HUFF QUARTILE				HUFF QUARTILE			
PERCENT	1st	2nd	3rd	4th	1st	2nd	3rd	4th	1st	2nd	3rd	4th
05	16	03	03	02	12	03	02	02	08	02	02	02
10	33	08	06	05	25	06	05	04	17	04	04	03
15	43	12	09	08	38	10	08	07	34	08	07	05
20	52	16	12	10	51	14	12	09	50	12	10	07
25	60	22	15	13	62	21	14	11	63	21	12	09
30	66	29	19	16	69	30	17	13	71	31	14	10
35	71	39	23	19	74	40	20	15	76	42	16	12
40	75	51	27	22	78	52	23	18	80	53	19	14
45	79	62	32	25	81	63	27	21	83	64	22	16
50	82	70	38	28	84	72	33	24	86	73	29	19
55	84	76	45	32	86	78	42	27	88	80	39	21
60	86	81	57	35	88	83	55	30	90	86	54	25
65	88	85	70	39	90	87	69	34	92	89	68	29
70	90	88	79	45	92	90	79	40	93	92	79	35
75	92	91	85	51	94	92	86	47	95	94	87	43
80	94	93	89	59	95	94	91	57	96	96	92	54
85	96	95	92	72	96	96	94	74	97	97	95	75
90	97	97	95	84	97	97	96	88	98	98	97	92
95	98	98	97	92	98	98	98	95	99	99	99	97

References: *Floyd A. Huff and James R. Angel, 1989 "Frequency Distributions and Hydroclimatic Characteristics of Heavy Rainstorms in Illinois', Illinois State Water Survey, Bulletin 70.*

Appendix B.

DETERMINING SCS CURVE NUMBERS

NIPC Land-use for Fort Sheridan Ravines

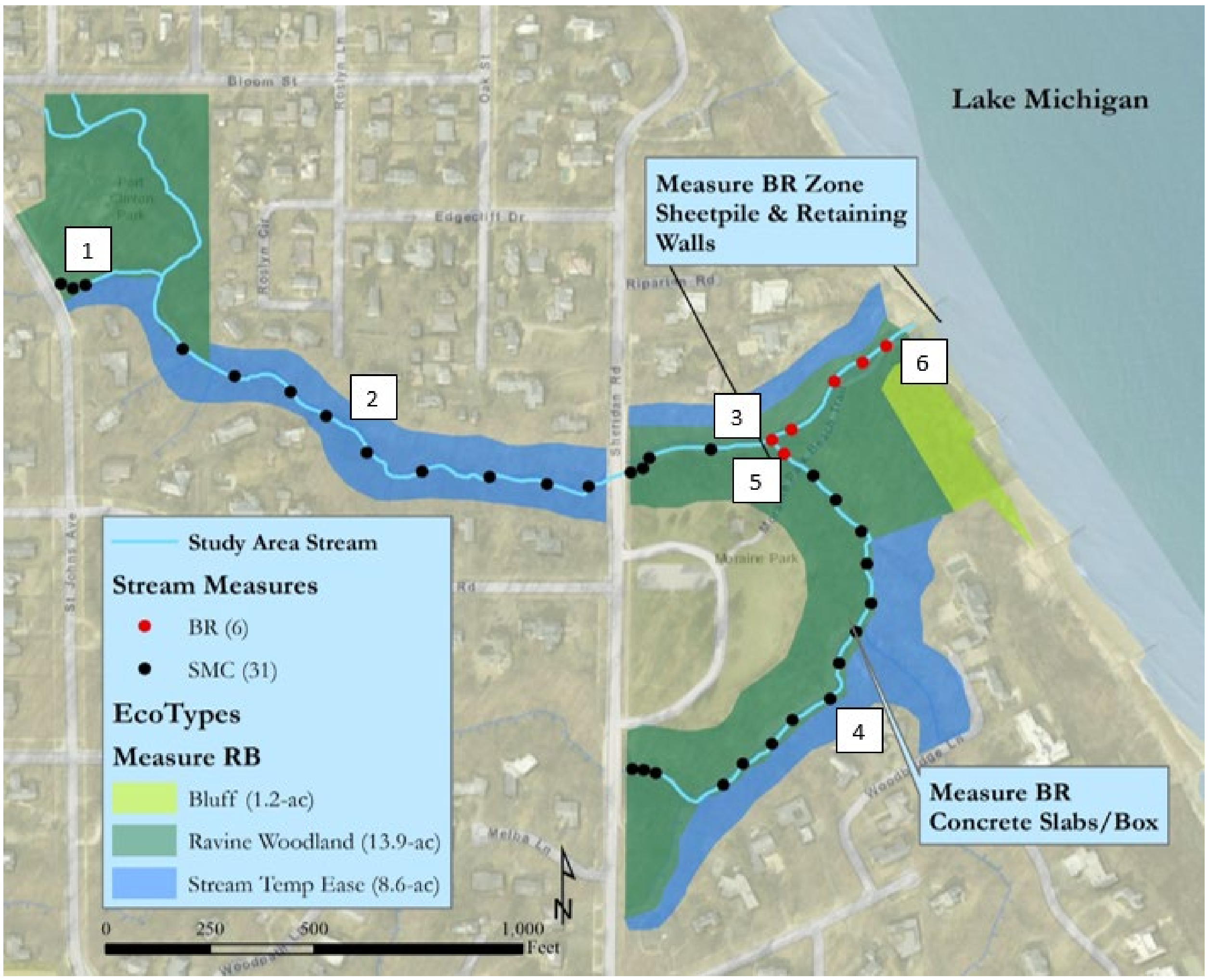
SCS CURVE NUMBERS BY LAND-USE AND SOIL CLASSIFICATION GROUP

NIPC Landuse for Fort Sheridan Ravines		Curve Numbers for Hydrologic Soil Groups				
		A	B	C	D	NRCS Cover Type
	URBAN & BUILT-UP LAND					
1100	RESIDENTIAL	67	78	86	89	
1110	Single Duplex and Townhouse Units	61	75	83	87	Residential: 1/4 acre avg. lot size (100' x 100')
1120	Farmhouse	51	68	79	84	Residential: 1 acre avg. lot size
1130	Multi-Family	77	85	90	92	Residential: 1/8 acre avg. lot size (75' x 75')
1140	Mobile Home Parks and Trailer Courts	77	85	90	92	Residential: 1/8 acre avg. lot size (75' x 75')
1200	COMMERCIAL AND SERVICES	86	90	93	94	
1210	Primary Retail/Service	89	92	94	95	Urban Districts: Commercial & Business
1211	Shopping Malls	89	92	94	95	Urban Districts: Commercial & Business
1212	Retail Centers	89	92	94	95	Urban Districts: Commercial & Business
1220	Primarily Office/Professional	89	92	94	95	Urban Districts: Commercial & Business
1221	Office Campus/Research Park	77	84	89	91	75% Commercial & Business, 25% Open Space
1222	Single-Structure Office Building	77	84	89	91	75% Commercial & Business, 25% Open Space
1223	Business Park	77	84	89	91	Urban Districts: Commercial & Business
1230	Urban Mix	89	92	94	95	Urban Districts: Commercial & Business
1231	Urban Mix With Dedicated Parking	89	92	94	95	Urban Districts: Commercial & Business
1232	Urban Mix , No Dedicated Parking	89	92	94	95	Urban Districts: Commercial & Business
1240	Cultural/Entertainment	89	92	94	95	Urban Districts: Commercial & Business
1250	Hotel/Motel	89	92	94	95	Urban Districts: Commercial & Business
1300	INSTITUTIONAL	71	81	87	90	
1310	Medical and Health Care Facilities	64	77	84	88	50% Commercial & Business, 50% Open Space
1320	Educational	64	77	84	88	50% Commercial & Business, 50% Open Space
1330	Governmental administration and services	89	92	94	95	Urban Districts: Commercial & Business
1340	Prisons and Correctional Facilities	89	92	94	95	Urban Districts: Commercial & Business
1350	Religious Facilities	77	84	89	91	75% Commercial & Business, 25% Open Space
1360	Cemeteries	39	61	74	80	Open Space: Good
1370	Other Institutional	77	84	89	91	75% Commercial & Business, 25% Open Space
1400	INDUSTRIAL AND WAREHOUSING AND WHOLESALE TRADE	81	88	91	93	
1410	Mineral Extraction	81	88	91	93	Urban Districts: Industrial
1420	Manufacturing and Processing	81	88	91	93	Urban Districts: Industrial
1430	Warehousing/Distribution Center	81	88	91	93	Urban Districts: Industrial
1440	Industrial Park	81	88	91	93	Urban Districts: Industrial
1500	TRANSPORTATION, COMMUNICATION, AND UTILITIES	75	84	89	91	
1510	Automotive Transportation	83	89	92	93	Streets & Road: Paved-Open Ditches (Including ROW)
1511	Interstate and Tollway	83	89	92	93	Streets & Road: Paved-Open Ditches (Including ROW)
1512	Other Roadway	83	89	92	93	Streets & Road: Paved-Open Ditches (Including ROW)

		Curve Numbers for Hydrologic Soil Groups				
		A	B	C	D	NRCS Cover Type
1520	Other Linear Transportation w/ Assoc. Facilities	83	89	92	93	Streets & Road: Paved-Open Ditches (Including ROW)
1530	Aircraft Transportation	83	89	92	93	Streets & Road: Paved-Open Ditches (Including ROW)
1540	Independent Parking	98	98	98	98	Impervious Areas: Paved Parking Lots
1550	Communication	49	69	79	84	Open Space: Fair Condition
1560	Utilities & Waste Facilities	39	61	74	80	Open Space: Good
	AGRICULTURAL LAND					
2100	Row Crops, Grains, & Grazing	64	75	83	86	Open Space: Good
2200	Nurseries, Greenhouses, Orchards, Tree Farms, & Sod Farm	64	75	83	86	Open Space: Good
2300	Agricultural , Other	64	75	83	86	Open Space: Good
	OPEN SPACE					
3100	Open Space, Primarily Recreational	39	61	74	80	Open Space: Good
3200	Golf Courses	39	61	74	80	Open Space: Good
3300	Open Space, Primarily Conservation (Forest & Natural Preserves	39	61	74	80	Open Space: Good
3400	Hunting Clubs, Scout Camps, & Private Campgrounds	39	61	74	80	Open Space: Good
3500	Linear Open-Space Corridors	39	61	74	80	Open Space: Good
3600	Other Open Space	39	61	74	80	Open Space: Good
	VACANT AND WETLANDS					
4100	VACANT LAND	35	62	76	83	Open Space: Good
4110	Forest and Grassland	30	55	70	77	Woods: Good
4120	Wetlands	40	68	81	88	Meadow (+10)
4200	UNDER DEVELOPMENT OR CONSTRUCTION	50	68	79	84	Open Space: Fair Condition
4210	Residential	61	75	83	87	Residential: 1/4 acre avg. lot size (100' x 100')
4220	Non-Residential Includes Unidentifiable Lots	39	61	74	80	Open Space: Good
4300	OTHER VACANT	39	61	74	80	Open Space: Fair Condition
	WATER					
5100	Rivers, Streams & Canals	98	98	98	98	Water (98)
5200	Lakes, Reservoirs & Lagoons	78	89	96	98	Pasture, Grassland: Poor (+10)
5300	Lake Michigan	98	98	98	98	

Appendix C.

COMPUTATION OF TIME OF CONCENTRATION


Lake Michigan
Measure BR Zone
Sheetpile & Retaining
Walls
Measure BR
Concrete Slabs/Box
1
2
3
4
5
6
Bloom St
Edgecliff Dr
Riparian Rd
Sheridan Rd
St Johns Ave
Moraine Park
Melba Ln
Study Area Stream
Stream Measures
BR (6)
SMC (31)
EcoTypes
Measure RB
Bluff (1.2-ac)
Ravine Woodland (13.9-ac)
Stream Temp Ease (8.6-ac)
0
250
500
1,000
Feet
N

	Computation Point #1	Computation Point #2	Computation Point #3	Computation Point #4	Computation Point #5	Computation Point #6
Sheet Flow						
Manning's Roughness	0.011	0.011	0.011	0.03	0.03	0.011
Flow Length	102.8	102.8	102.8	87.9	99.8	102.8
Two-year 24-Hour Storm	3.3	3.3	3.3	3.3	3.34	3.3
High Elevation	689.3	689.3	689.3	672.9	669.5	689.3
Low Elevation	688.0	688.0	688.0	670.9	668	688.0
Land Slope (ft/ft)	0.0	0.0	0.0	0.0	0.02	0.0
Travel Time (hr)	*0.02*	*0.02*	*0.02*	*0.04*	*0.05*	*0.02*
Shallow Concentrated Flow						
Flow Length (ft)	857.0	857.0	857.0	437.9	166.3	857.0
Low Elevation (ft)	680.3	680.3	680.3	658.3	658.2	680.3
Watercourse Slope (ft/ft)	0.0	0.0	0.0	0.0	0.06	0.0
Average Velocity (ft/s)	1.6	1.6	1.6	2.7	3.99	1.6
Travel Time (hr)	*0.15*	*0.15*	*0.15*	*0.04*	*0.01*	*0.15*
Pipe Flow						
Travel Time (hr)	*0.03*	*0.03*	*0.03*	*0.00*	*0.00*	*0.03*
Ravine Slope						
Flow Length (ft)				89.6	12.3	
Low Elevation (ft)				617.3	653.5	
Watercourse Slope (ft/ft)				0.5	0.38	
Average Velocity (ft/s)				11.4	10.10	
Travel Time (hr)				*0.002*	*0.000*	
Channel Flow						
Channel width (ft), b	10.5	10.5	10.5	13.5	13.5	10.5
Side slopes, m	1.0	1.0	1.0	1.0	1.0	1.0
Depth to bank full flow (ft), y	1.0	1.0	1.0	1.5	1.0	1.0
Area, A	11.5	11.5	11.5	22.5	14.5	11.5
Wetted Perimeter, P	13.3	13.3	13.3	17.7	16.3	13.3
Hydraulic Radius, R_h	0.9	0.9	0.9	1.3	0.9	0.9
High Elevation (ft)	646.0	646.0	646.0	617.3	653.5	646.0
Low Elevation (ft)	620.3	605.9	583.1	591.9	588.4	579.7
Channel Slope	0.0	0.0	0.0	0.0	0.0	0.0
Flow Length (ft)	1389.0	1964.5	3404.5	1755.4	2437.3	3804.4
Computed Average Velocity (ft/s)	4.6	4.8	4.6	5.2	5.6	4.5
Travel Time (hr)	*0.08*	*0.11*	*0.21*	*0.09*	*0.12*	*0.24*
Estimated Tc (hr)	**0.29**	**0.32**	**0.41**	**0.18**	**0.18**	**0.44**

Appendix D.

RECOMMENDED PLAN DESIGN CALCULATION SHEETS

Design Requirements

- Maintain connectivity to the shoreline for fish access to the ravine
- Provide adequate cover for the active interceptor line running down the centerline of the channel
- Mitigate for downcutting from excessive channel velocities
- D_{100} stone sizes can not exceed the maximum sized substrate currently present in the channel

Design Assumptions

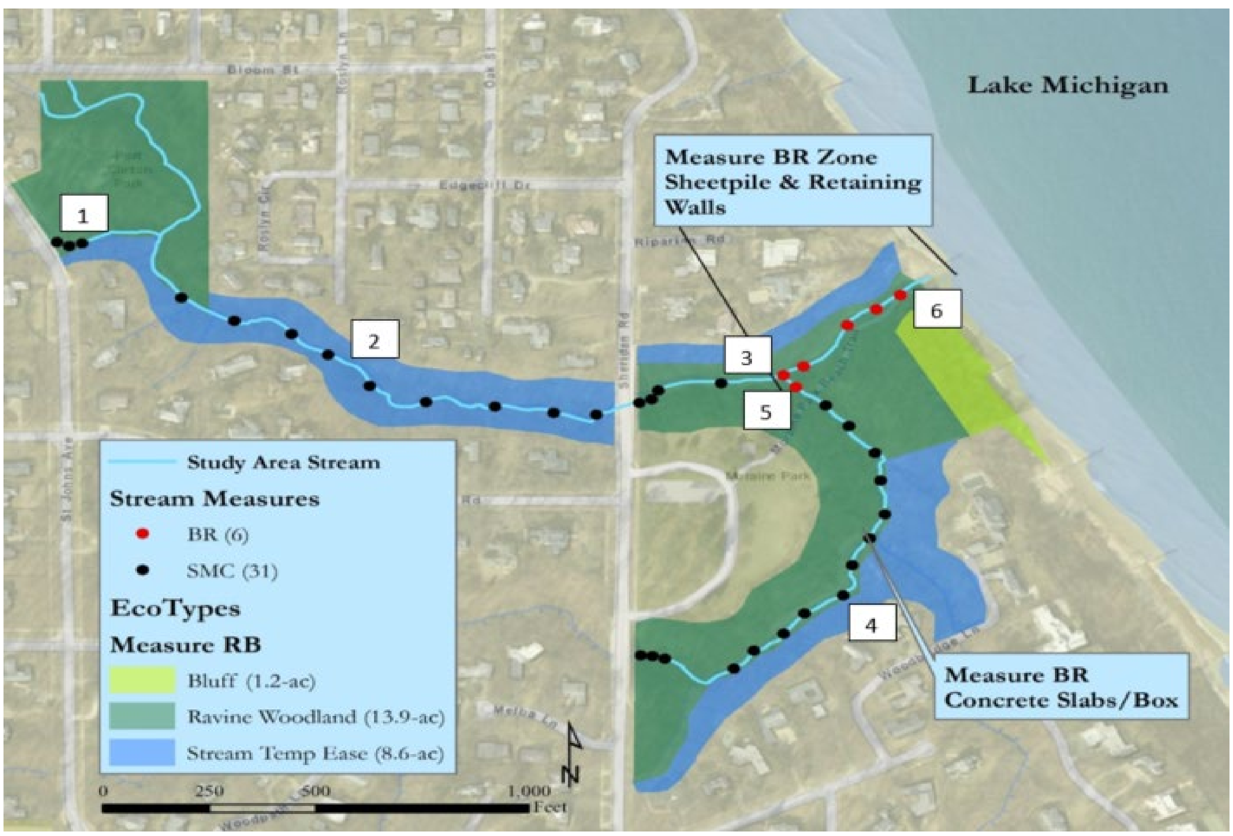
- Flow from the existing sewer outfalls is not attenuated
- Existing elevation drops across features is currently ignored
- Prescribed "typical" channel cross-section is applicable throughout the channel
- In computations which use smaller return periods (e.g. 25-year), a 100-year return will be temporarily used.

Low-flow channels will be estimated based on observed field conditions.

Design Flow Rates

The design flow rate for the feasibility-level design was estimated by combining a peak flow rate developed using the Rational Method and the peak discharge from the storm sewer network currently modeled in SWMM. It was assumed that the peak discharges coincided, regardless of the computation point (i.e. travel time was ignored). Only the storm duration for the 100-year events used for the Rational Method were considered.

The cumulative peak outflows at the noted computation points are tabulated below.



	CP #1	CP #2	CP #3	CP #4	CP #5	CP #6
Rtnl Mthd	174.2	238	302.1	93.2	216.8	519
SWMM	-	-	-	55.7	55.7	55.7
TOTAL	174.2	238	302.1	148.9	272.5	574.7

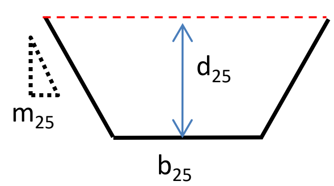
Design of Typical Ravine Channel, North Reach (current condition)

For the feasibility-level design, the conveyance of the existing channel will be estimated based on a typical cross-section and the channel slope developed from the available DEM. It must be noted that the channel slope is based on upstream and downstream elevations and does not account for features currently functioning as ad hoc grade control structures; the actual channel slope is shallower than this estimate.

Define the design conveyance capacity of the existing channel

Manning's n, n =	0.035		
Base Width, b_{25} =	3.0	ft	
Side Slope, m_{25} =	2.0	ft/ft	
Dsgn Chnl Slope, S_o =	0.017	ft/ft	
Normal Flow Depth, $y_{n,25}$ =	0.62	ft	
Chnl Depth, d_{25} =	0.7	ft	
$Q_{25,des}$ =	8.50	cfs	NO!
V_{des} =	3.24	ft/s	OKAY!

Area, A =	2.62	ft^2
Wetted Permiter, P =	5.7677	ft
Hydraulic Radius, R_h =	0.4547	ft



Define the design conveyance capacity of the overbank area

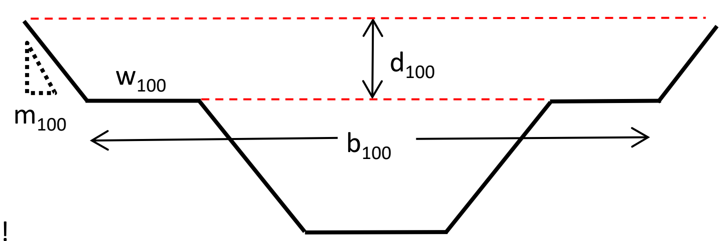
The design conveyance of the overbank area is the peak 100-year discharge at the mouth of the tributary.

$Q_{100,tar}$ = 302 cfs

The design channel configuration assumes the typical post-project channel slope and a fully vegetated channel. Iterate the parameters to achieve the required design discharge.

Manning's n, n =	0.035		
Shelf Width, w_{100} =	4.50	ft	
Base Width, b_{100} =	14.8	ft	
Side Slope, m_{100} =	2.0	ft/ft	
Dsgn Chnl Slope, S_o =	0.017	ft/ft	
Normal Flow Depth, $y_{n,100}$ =	3.32	ft	
Chnl Depth, d_{100} =	3.5	ft	
Total Channel Depth =	4.2	ft	
$Q_{100,des}$ =	334	cfs	OKAY!
V_{des} =	6	ft/s	NO!

Area, A =	55.70	ft^2
Wetted Permiter, P =	26.867	ft
Hydraulic Radius, R_h =	2.0731	ft

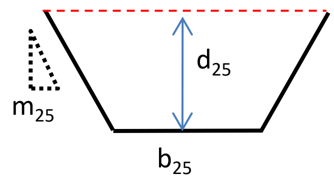


Design of Typical Ravine Channel, North Reach (proposed)

For the feasibility-level design, it is assumed that the channel must be raised by several feet across the reach to provide adequate cover for the existing sewer running through the channel. This will be coupled with the

Define the design conveyance capacity of the channel

Manning's n, n =	0.035			Area, A =	3.38	ft^2
Base Width, b_{25} =	3.0	ft		Wetted Permiter, P =	6.3541	ft
Side Slope, m_{25} =	2.0	ft/ft		Hydraulic Radius, R_h =	0.5312	ft
Dsgn Chnl Slope, S_o =	0.005	ft/ft				
Normal Flow Depth, $y_{n,25}$ =	0.75	ft				
Chnl Depth, d_{25} =	1	ft				
$Q_{25,des}$ =	6.65	cfs	OKAY!			
V_{des} =	1.97	ft/s	OKAY!			



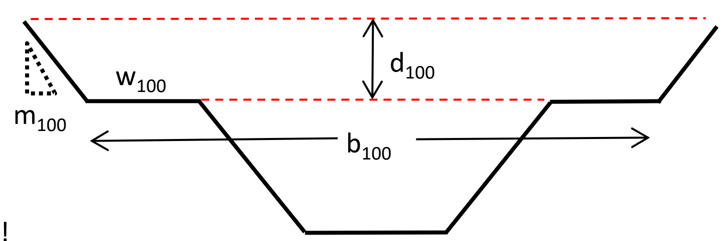
Define the design conveyance capacity of the overbank area

The design conveyance of the overbank area is the peak 100-year discharge at the mouth of the

$Q_{100,tar}$ = 302 cfs

The design channel configuration assumes the typical post-project channel slope and a fully

Manning's n, n =	0.035			Area, A =	80.16	ft^2
Shelf Width, w_{100} =	4.50	ft		Wetted Permiter, P =	31.32	ft
Base Width, b_{100} =	16.0	ft		Hydraulic Radius, R_h =	2.56	ft
Side Slope, m_{100} =	2.0	ft/ft				
Dsgn Chnl Slope, S_o =	0.005	ft/ft				
Normal Flow Depth, $y_{n,100}$ =	4.32	ft				
Chnl Depth, d_{100} =	4.5	ft				
Total Channel Depth =	5.5	ft				
$Q_{100,des}$ =	303	cfs	OKAY!			
V_{des} =	3.78	ft/s	OKAY!			

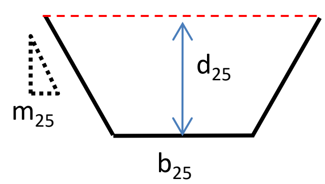


Design of Typical Ravine Channel, South Reach (current condition)

For the feasibility-level design, the conveyance of the existing channel will be estimated based on a typical cross-section and the channel slope developed from the available DEM. It must be noted that the channel slope is based on upstream and downstream elevations and does not account for features currently functioning as ad hoc grade control structures; the actual channel slope is shallower than this estimate.

Define the design conveyance capacity of the channel

Manning's n, n =	0.035			Area, A =	3.39	ft^2
Base Width, b_{25} =	3.0	ft		Wetted Permiter, P =	6.363	ft
Side Slope, m_{25} =	2.0	ft/ft		Hydraulic Radius, R_h =	0.532	ft
Dsgn Chnl Slope, S_o =	0.011	ft/ft				
Normal Flow Depth, $y_{n,25}$ =	0.75	ft				
Chnl Depth, d_{25} =	0.75	ft				
$Q_{25,des}$ =	10.00	cfs	OKAY!			
V_{des} =	2.95	ft/s	OKAY!			



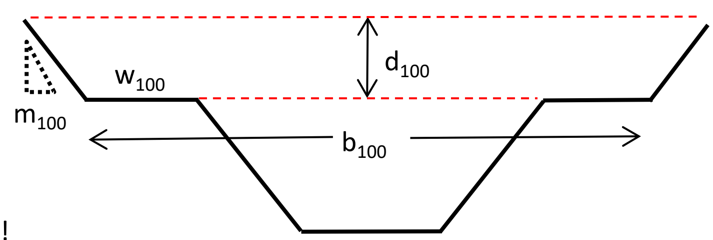
Define the design conveyance capacity of the overbank area

The design conveyance of the overbank area is the peak 100-year discharge at the mouth of the tributary.

$Q_{100,tar}$ = 385 cfs

The design channel configuration assumes the typical post-project channel slope and a fully vegetated channel. Iterate the parameters to achieve the required design discharge.

Manning's n, n =	0.035			Area, A =	71.63	ft^2
Shelf Width, w_{100} =	5.00	ft		Wetted Permiter, P =	30.13	ft
Base Width, b_{100} =	16.0	ft		Hydraulic Radius, R_h =	2.377	ft
Side Slope, m_{100} =	2.0	ft/ft				
Dsgn Chnl Slope, S_o =	0.011	ft/ft				
Normal Flow Depth, $y_{n,100}$ =	3.83	ft				
Chnl Depth, d_{100} =	3.85	ft				
Total Channel Depth =	4.6	ft				
$Q_{100,des}$ =	386	cfs	OKAY!			
V_{des} =	5.39	ft/s	NO!			



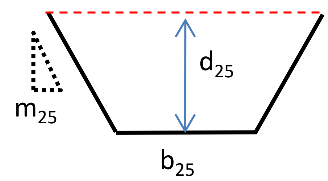
Design of Typical Ravine Channel, South Reach (proposed)

For the feasibility-level design, it is assumed that the channel must be raised by several feet across the reach to provide adequate cover for the existing sewer running through the channel. This will be coupled with the

Define the design conveyance capacity of the channel

Manning's n, n =	0.035		
Base Width, b_{25} =	3.0	ft	
Side Slope, m_{25} =	2.0	ft/ft	
Dsgn Chnl Slope, S_o =	0.005	ft/ft	
Normal Flow Depth, $y_{n,25}$ =	1	ft	
Chnl Depth, d_{25} =	0.5	ft	
$Q_{25,des}$ =	10.89	cfs	OKAY!
V_{des} =	2.18	ft/s	OKAY!

Area, A =	5.00	ft^2
Wetted Permiter, P =	7.472	ft
Hydraulic Radius, R_h =	0.669	ft



Define the design conveyance capacity of the overbank area

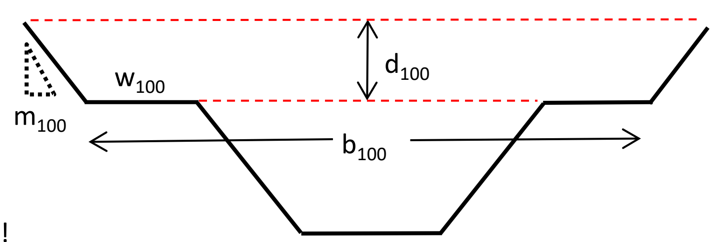
The design conveyance of the overbank area is the peak 100-year discharge at the mouth of the

$Q_{100,tar}$ = 273 cfs

The design channel configuration assumes the typical post-project channel slope and a fully vegetated channel. Iterate the parameters to achieve the required design discharge.

Manning's n, n =	0.035		
Shelf Width, w_{100} =	5.00	ft	
Base Width, b_{100} =	15.0	ft	
Side Slope, m_{100} =	2.0	ft/ft	
Dsgn Chnl Slope, S_o =	0.005	ft/ft	
Normal Flow Depth, $y_{n,100}$ =	3.94	ft	
Chnl Depth, d_{100} =	4	ft	
Total Channel Depth =	4.5	ft	
$Q_{100,des}$ =	274	cfs	OKAY!
V_{des} =	3.55	ft/s	OKAY!

Area, A =	77.3	ft^2
Wetted Permiter, P =	30.62	ft
Hydraulic Radius, R_h =	2.5	ft



Reinforcement of North Reach

Estimate the required step height for the step-pools:

Initial Slope, S_o =	0.017	ft/ft	Required Drop, H =	23.7	ft	Req. # of Steps =	31.6	
Final Slope, S_f =	0.005	ft/ft	Allowable Step, h =	0.75	ft	Spec # of Steps =	20	
Distance, L =	1976	ft						

Calculate the active channel width for the reach:

The active channel width represents the average of the cross-section topwidths at the 25-year discharge for each local site.

ACW, L_3 =	14	ft	ACW, L_3 =	4.1	m

Estimate the pool length and maximum scour depth:

$$\frac{L_2}{L_3} = 0.409 + 4.211\frac{H}{L_3} + 87.341\frac{S_o q_{25}}{\sqrt{g(L_3)^{3/2}}}$$

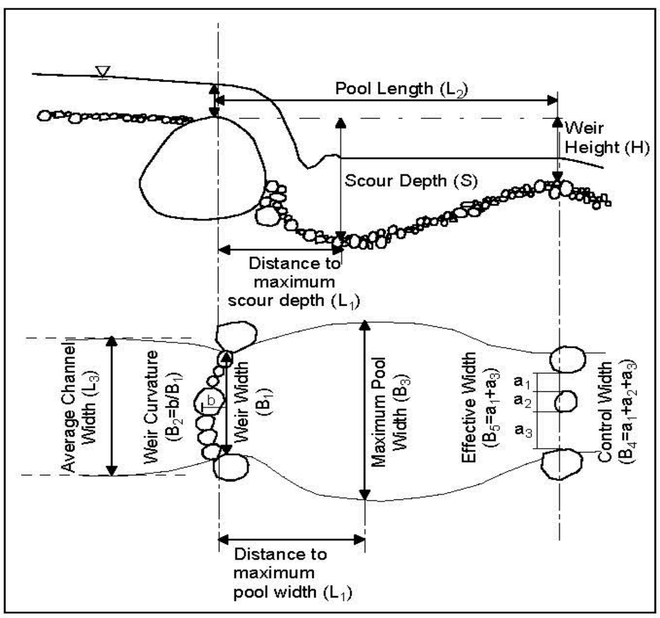
$$\frac{S}{L_3} = -0.0118 + 1.394\frac{H}{L_3} + 5.514\frac{S_o q_{25}}{\sqrt{g(L_3)^{3/2}}}$$

$$B_3 = 1.2 \cdot B_1$$

$$B_5 = 0.92 \cdot B_1$$

$$D_{30} = \frac{1.95 \cdot S^{0.555} \cdot q^{2/3}}{g^{1/3}}$$

H = Weir Height
S = Scour Depth
L_1 = Distance to maximum scour depth
L_2 = Pool Length
L_3 = Average Channel Width
B_1 = Weir Width
B_2 = Weir Curvature
B_3 = Maximum Pool Width
B_4 = Control Width



H =	0.75	ft	H (metric) =	0.2	m
Q_{25} =	302	cfs	Q_{25} (metric) =	8.6	cms
1.2*q25 =	30.2	cfs/ft	1.2*q_{25} (metric)=	3.0	cms/m
g =	32.2	ft/s^2	g =	9.8	m/s^2
Avg Chnl Width, L_3 =	13.5	ft	Avg Chnl Width (metric) =	4.1	m
S_f =	0.005	ft/ft	S_f =	0.01	m/m
Weir Width, B_1 =	12.0	ft	Weir Width (metric) =	3.7	m

Max Pool Wdth, B_3 =	14.4	ft	Max Pool Width (metric) =	4.4	m
Effective Wdth, B_5 =	11.04	ft	Effective Width (metric) =	3.4	m
Pool Length, L_2 =	10.9	ft	Pool Length (metric) =	3.3	m
Max Scour Dpth, S =	1.0	ft	Max Scour Depth (metric) =	0.3	m

Does the design meeting the spacing and drop requirement?

Required Drop =	23.7	ft	Total Structure Length =	218.2	ft
Step-Pool Drop =	15.0	ft	Length Less Structures =	1757.8	ft
Drop from Slope =	8.8	ft	Crest Spacing =	98.8	ft
Total Drop =	23.8	ft	End-to-End Spacing =	87.9	ft
	OKAY!				

Determine the required stone size for step-pool:

Check using the formulation for steep-slopes, Equation 3-5 from EM 1110-2-1601

Q_{100} =	302	cfs	S_f =	0.069	ft/ft
$1.2*Q_{100}$ =	362.4	cfs	D_{30} =	1.11	ft
q_{100} =	22.65	cfs/ft	$1.0*D_{30}$ =	1.11	ft

Check using formulation relating velocity and stable stone diameter found on Hydraulic Design Chart 712-1, found in Hydraulic Design Critera (USACE 1970)

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)^{0.5}\right](D_{50})^{0.5}$$

V_{100} =	3.78	ft/s	D_{50} =	0.27	ft
$1.2*V_{100}$ =	4.536	ft/s	D_{85}/D_{15} =	2.0	(assumed)
C =	0.86	(High Turbulence Flow)	D_{30} =	0.21	ft
γ_S =	165	lb/ft^3	$1.0*D_{30}$ =	0.21	ft
γ_W =	62.5	lb/ft^3			

Check using formulation for velocity and depth, Equation 3-3 from EM 1110-2-1601

$$D_{30} = S_f C_S C_V C_T d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\frac{V}{\sqrt{K_1 g d}}\right]^{2.5}$$

S_f =	1.2
C_S =	0.375
C_V =	1
C_T =	1
K_1 =	0.72

Storm Event	V	1.2*V	d	D_{30}
100yr Rational Meth	3.78	4.5	4.32	0.15
~~100yr-12hr~~	~~5.41~~	~~7.0~~	~~1.35~~	~~0.58~~
~~100yr-24hr~~	~~5.09~~	~~6.6~~	~~1.17~~	~~0.52~~
~~100yr-48hr~~	~~4.62~~	~~6.0~~	~~0.94~~	~~0.43~~
~~100yr-72hr~~	~~4.39~~	~~5.7~~	~~0.83~~	~~0.39~~

Design stone size will be taken as the maximum of the three formulations

	D_{30}	
Steep-Slope Formulation =	1.11	ft
Chart 712-1 Formulation =	0.21	ft
Velocity & Depth Formulation =	0.17	ft
Adopted Design D_{30} =	1.11	ft

Use EM 1110-2-1601 Table 3-1 to establish an acceptable gradation

D_{100}	max	2.5	ft	W_{100}	max	1350	lbs
	min	1.8	ft		min	540	lbs
D_{50}	max	1.7	ft	W_{50}	max	400	lbs
	min	1.5	ft		min	270	lbs
D_{15}	max	1.3	ft	W_{15}	max	200	lbs
	min	1.0	ft		min	84	lbs

Using EM 1110-2-1601, determine the required thickness

$1*D_{100(max)}$ =	2.5	ft
$1.5*D_{50(max)}$ =	2.50	ft

Use a thickness of 2.5 ft

Reinforcement of South Reach

Estimate the required step height for the step-pools:

Initial Slope, S_o =	0.011 ft/ft	Required Drop, H =	8.02 ft	Req. # of Steps =	16
Final Slope, S_f =	0.005 ft/ft	Allowable Step, h =	0.5 ft	Spec # of Steps =	8
Distance, L =	1336.5 ft				

Calculate the active channel width for the reach:

The active channel width represents the average of the cross-section topwidths at the 25-year discharge for each local site.

ACW, L_3 = 14 ft   ACW, L_3 = 4.1 m

Estimate the pool length and maximum scour depth:

$$\frac{L_2}{L_3} = 0.409 + 4.211\frac{H}{L_3} + 87.341\frac{S_o q_{25}}{\sqrt{g(L_3)^{3/2}}}$$

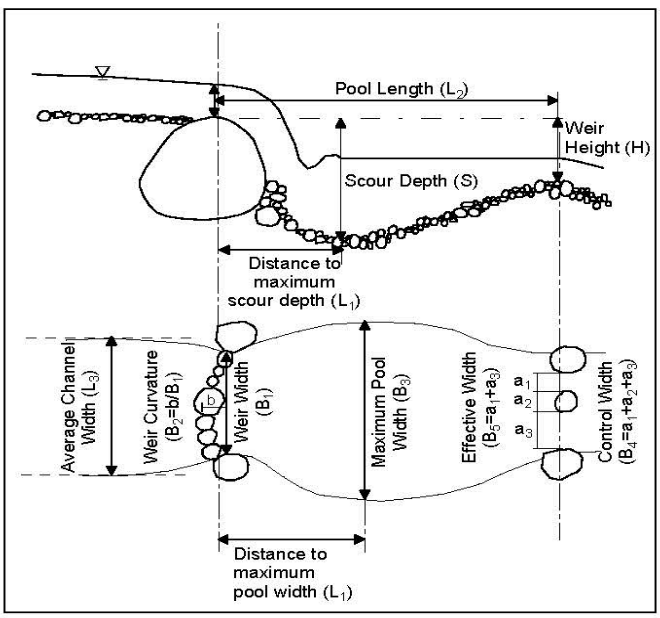
$$\frac{S}{L_3} = -0.0118 + 1.394\frac{H}{L_3} + 5.514\frac{S_o q_{25}}{\sqrt{g(L_3)^{3/2}}}$$

$$B_3 = 1.2 \cdot B_1$$

$$B_5 = 0.92 \cdot B_1$$

$$D_{30} = \frac{1.95 \cdot S^{0.555} \cdot q^{2/3}}{g^{1/3}}$$

H = Weir Height
S = Scour Depth
L_1 = Distance to maximum scour depth
L_2 = Pool Length
L_3 = Average Channel Width
B_1 = Weir Width
B_2 = Weir Curvature
B_3 = Maximum Pool Width
B_4 = Control Width



H =	0.5 ft	H (metric) =	0.2 m
Q_{25} =	273 cfs	Q_{25} (metric) =	7.7 cms
1.2*q25 =	25.2 cfs/ft	1.2*q_{25} (metric)=	2.5 cms/m
g =	32.2 ft/s^2	g =	9.8 m/s^2
Avg Chnl Width, L_3 =	13.5 ft	Avg Chnl Width (metric) =	4.1 m
S_f =	0.005 ft/ft	S_f =	0.01 m/m
Weir Width, B_1 =	13.0 ft	Weir Width (metric) =	4.0 m

Max Pool Wdth, B_3 =	15.6	ft	Max Pool Width (metric) =	4.8	m
Effective Wdth, B_5 =	11.96	ft	Effective Width (metric) =	3.6	m
Pool Length, L_2 =	9.5	ft	Pool Length (metric) =	2.9	m
Max Scour Dpth, S =	0.7	ft	Max Scour Depth (metric) =	0.2	m

Does the design meeting the spacing and drop requirement?

Required Drop =	8.0	ft	Total Structure Length =	75.9	ft
Step-Pool Drop =	4.0	ft	Length Less Structures =	1260.6	ft
Drop from Slope =	6.3	ft	Crest Spacing =	167.1	ft
Total Drop =	10.3	ft	End-to-End Spacing =	157.6	ft
	OKAY!				

Determine the required stone size for step-pool:

Check using the formulation for steep-slopes, Equation 3-5 from EM 1110-2-1601

Q_{100} =	273	cfs	S_f =	0.053	ft/ft
1.2*Q_{100} =	327.6	cfs	D_{30} =	0.90	ft
q_{100} =	20.48	cfs/ft	1.0*D_{30} =	0.90	ft

Check using formulation relating velocity and stable stone diameter found on Hydraulic Design Chart 712-1, found in Hydraulic Design Critera (USACE 1970)

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)^{0.5}\right](D_{50})^{0.5}$$

V_{100} =	3.55	ft/s	D_{50} =	0.23	ft
1.2*V_{100} =	4.26	ft/s	D_{85}/D_{15} =	2.0	(assumed)
C =	0.86	(High Turbulence Flow)	D_{30} =	0.18	ft
γ_S =	165	lb/ft^3	1.0*D_{30} =	0.18	ft
γ_W =	62.5	lb/ft^3			

Check using formulation for velocity and depth, Equation 3-3 from EM 1110-2-1601

$$D_{30} = S_f C_S C_V C_T d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5} \frac{V}{\sqrt{K_1 g d}}\right]^{2.5}$$

S_f =	1.2
C_S =	0.375
C_V =	1
C_T =	1
K_1 =	0.72

Storm Event	V	1.2*V	d	D_{30}
100yr Rational Meth	3.55	4.3	3.94	0.13
~~100yr-12hr~~	~~5.41~~	~~7.0~~	~~1.35~~	~~0.58~~
~~100yr-24hr~~	~~5.09~~	~~6.6~~	~~1.17~~	~~0.52~~
~~100yr-48hr~~	~~4.62~~	~~6.0~~	~~0.94~~	~~0.43~~
~~100yr-72hr~~	~~4.39~~	~~5.7~~	~~0.83~~	~~0.39~~

Design stone size will be taken as the maximum of the three formulations

	D_{30}	
Steep-Slope Formulation =	0.90	ft
Chart 712-1 Formulation =	0.18	ft
Velocity & Depth Formulation =	0.15	ft
Adopted Design D_{30} =	0.90	ft

Use EM 1110-2-1601 Table 3-1 to establish an acceptable gradation

D_{100}	max	2.0	ft	W_{100}	max	691	lbs
	min	1.5	ft		min	276	lbs
D_{50}	max	1.3	ft	W_{50}	max	205	lbs
	min	1.2	ft		min	138	lbs
D_{15}	max	1.1	ft	W_{15}	max	102	lbs
	min	0.8	ft		min	43	lbs

Using EM 1110-2-1601, determine the required thickness

$1*D_{100(max)}$ = 2.0 ft

$1.5*D_{50(max)}$ = 2.00 ft

Use a thickness of 2.0 ft