



## MEMORANDUM

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**Subject:** Analysis and Mapping Procedures for Non-Accredited Levees - Initial Data Analysis  
Village of Moravia, Cayuga County, NY

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### Purpose

This memorandum summarizes the application of Natural Valley (NV) and Structure Based Inundation (SBI) procedures for developing flood hazard data for the Dry Creek Right Bank Levee system adjacent to Dry Creek in the Village of Moravia, Cayuga County, NY (Figure 1). The Dry Creek Right Bank Levee system is part of the Moravia Flood Damage Reduction Project.

The hydrologic and hydraulic assumptions, approaches, and methodology applied to develop NV and SBI floodplains are summarized in the sections that follow. Details on the general guidance for these procedures are available in "[Analysis and Mapping Procedures for Non-Accredited Levee Systems](#)". Details regarding specific attributes of the levee system and available data will be included in the Analyses and Mapping Plan.

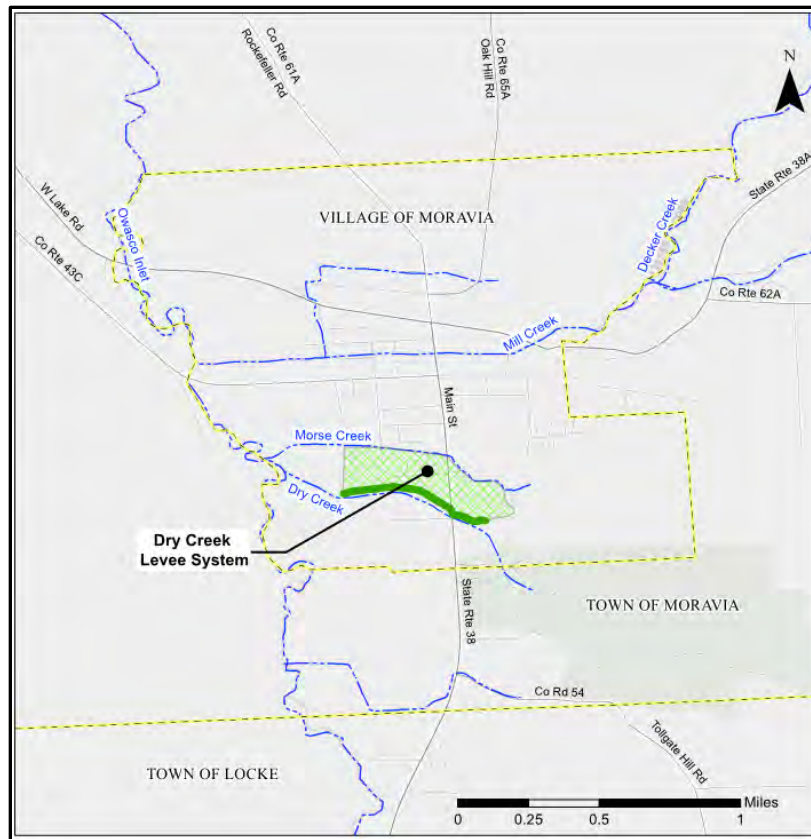


Figure 1: Levee System at the Village of Moravia, NY

## Hydrology: Methodology

This section summarizes methods and source data used for the development of 1-percent-annual-chance flow hydrographs used in the two-dimensional (2-D) unsteady-state model of the NV and SBI Procedures. The effective Cayuga County, New York Flood Insurance Study (FIS) report flow rates were used for the one-dimensional (1-D) steady-state models leveraged to develop the 2-D unsteady-state model of Dry Creek.

An inflow hydrograph was required for 2-D unsteady-state flow modeling of the NV and SBI Procedures; however, stream gage data was not available in the vicinity of the Dry Creek study. To develop a hydrograph for the 2-D unsteady-state analysis, the dimensionless unit hydrograph generation approach by USDA-NRCS (2007) was utilized. The time of concentration ( $t_c$ ) was estimated based on Kirpich (1940) equation:

$$t_c = 0.0078 L^{0.77} (L/H)^{0.385}$$

$L$  – maximum flow path (ft),  $H$  – elevation difference (ft)

The flow path and elevation were determined for the Dry Creek watershed from the U.S. Geological Survey StreamStats application. The peak discharge used for the unsteady-state

discharge hydrograph was from the effective FIS 1-percent-annual-chance flood. Table 1 below summarizes hydrologic inputs used to generate the discharge hydrograph.

Flooding Source	Flow Length (feet)	Elevation Change (feet)	Time of Concentration (minutes)	Time to Peak (hours)	Peak Discharge (cfs)
Dry Creek	34,807	986	96.6	1.07	2,020

Table 1 - Hydrograph Development Parameters

Figure 2 shows the computed discharge hydrograph for Dry Creek.

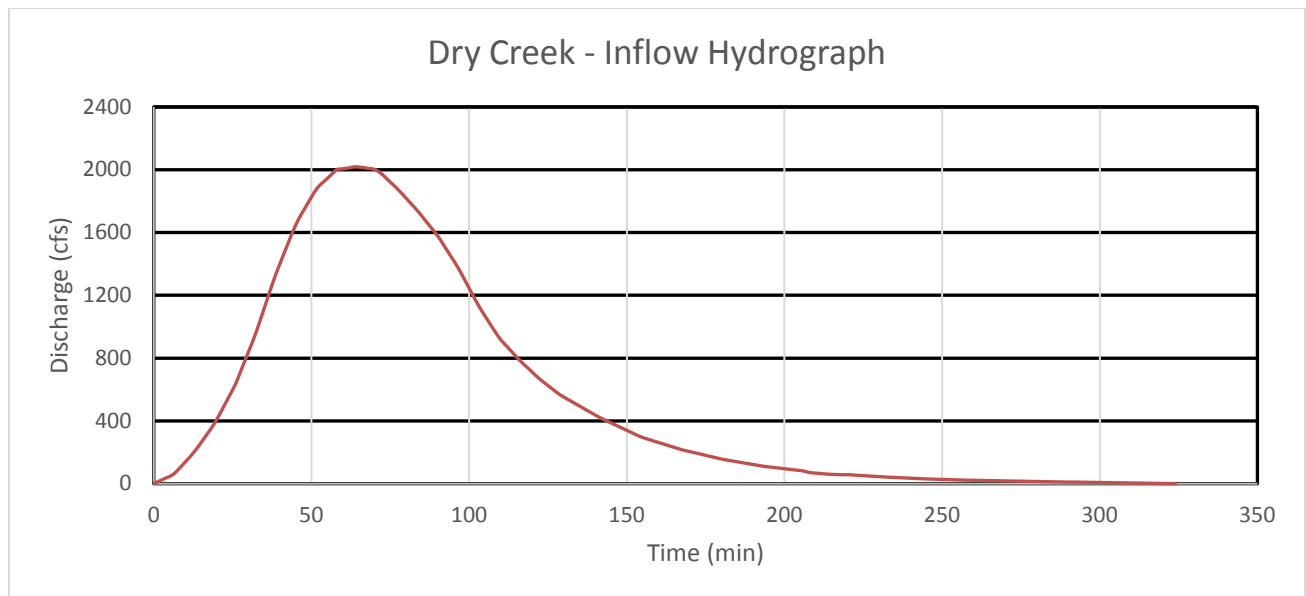


Figure 2 – Dry Creek Discharge Hydrograph

## Hydraulics: Methodology

This section summarizes hydraulic methods and assumptions used to model NV and SBI Procedures for the Dry Creek Right Bank Levee system. Models used for this analysis were adapted from the effective HEC-RAS model for Dry Creek.

### Natural Valley Procedure

Due to the sloping topography away from Dry Creek in the leveed area, it was determined that the inundations extents and depth of flooding for the study area could be better represented through a 2-D unsteady-state analysis. A 2-D unsteady-state HEC-RAS model was developed to perform the NV and SBI Procedures.

The reach upstream boundary condition was set to the 1-percent-annual-chance inflow hydrograph. The downstream channel boundary condition was set as normal depth with friction slope of 0.005 ft/ft to match the effective HEC-RAS model.

The 2-D HEC-RAS model utilizes a mesh (based on a DEM downloaded from NYSGIS Clearinghouse), that controls the movement of water through the 2-D flow area, to evaluate and plot the inundation area resulting from a breach. A Manning's "n" land cover layer was generated based on aerial imagery to simulate the approximate roughness coefficients experienced by overland flow. The northern extent of the mesh is Morse Creek.

The 2-D mesh contains an outflow boundary along Morse Creek and the Owasco Inlet floodplain. The boundary condition was set to normal depth with a friction slope of 0.01 ft/ft for Morse Creek, and 0.004 ft/ft for Owasco Inlet Tailwater from Owasco Inlet was assumed to have no effect the Dry Creek water surface elevation as the peak flows are not expected to be coincident. The 1-percent-annual-chance flood of Morse Creek has little overlap with the Dry Creek leveed area and was not considered to have an effect on the Dry Creek water surface elevations.

The Natural Valley Procedure was modeled for the Dry Creek Right Bank Levee system by connecting the 1-D cross sections to the 2-D mesh and allowing the discharge to flow from Dry Creek naturally as if the levee was not in place.

## **Structural-Based Inundation Procedure**

The georeferenced, steady-state, HEC-RAS model was also used to develop an unsteady-state, 2-D model for the SBI Procedure. For the SBI Procedure, hypothetical breaches of the levee system were simulated at three locations to evaluate the potential flood risk to the area north of the levee within the NV inundation area for the 1-percent-annual-chance flood. No locations of levee impairment or historic breaches were reported along the levee system to assist in the selection of the modeled breach locations.

The Dry Creek earthen embankment levee extends from approximately 470 feet upstream of North Main Street to approximately 900 feet upstream of its mouth at Owasco Inlet. The total levee length is approximately 2,200 feet. The Structural-Based Inundation Procedure was performed by breaching the levee on the right bank at three locations. Beach locations were not selected between lateral structure station 0+00 and 7+00 because the ground elevation landward of the levee is above the effective 1-percent annual-chance flood.

Breach parameters used at each location are summarized in Table 2. Because the levee does not overtop, each breach assumed a piping failure triggered when the water surface elevation reached the approximately landside levee toe elevation. Each breach shape was assumed to be a trapezoid with a 1:1 side slopes, a breach weir coefficient of 2.6, and piping coefficient of 0.5. Breach formation times were limited to a maximum of 1.0 hour because of the short duration of the peak

hydrograph. Maximum breach widths were limited to 300 feet because the duration of maximum pressure on the levee during peak flow is relatively short.

<b>Breach ID</b>	<b>Lateral Structure Station</b>	<b>Final Bottom Width (feet)</b>	<b>Initial Piping /Final Bottom Elevation (feet)</b>	<b>Breach Formation Time (hours)</b>
1A	2025	300	738.6	1.0
1B	2025	100	737.2	0.16 (10 mins.)
1C*	2025	300	738.6	0.16
2A	1475	300	742.6	1.0
2B	1475	100	742.3	0.16
2C*	1475	300	742.6	0.16
3A	1200	300	744.7	1.0
3B	1200	100	744.7	0.16
3C*	1200	300	744.7	0.16

\*Selected for composite inundation area / depth grid mapping

Table 2 –Breach Parameters

The location of the breach resulted in minor variations in the inundation areas and depth of flooding. Changes in breach parameters (width, formation time) had a minor effect on the overall inundation areas and depth of flooding. Structural-Based Inundation Procedure results can be found in the attached maps, Appendices B and C. The final inundation limits were determined by creating a composite inundation area from the three breach scenarios.

## Results

Results for all modelling scenarios were presented at the Local Levee Partnership Team (LLPT) 2 meeting and follow-up touchpoint call. Summary results are available in the presentation slides, and will be included in Analysis and Mapping Plan.