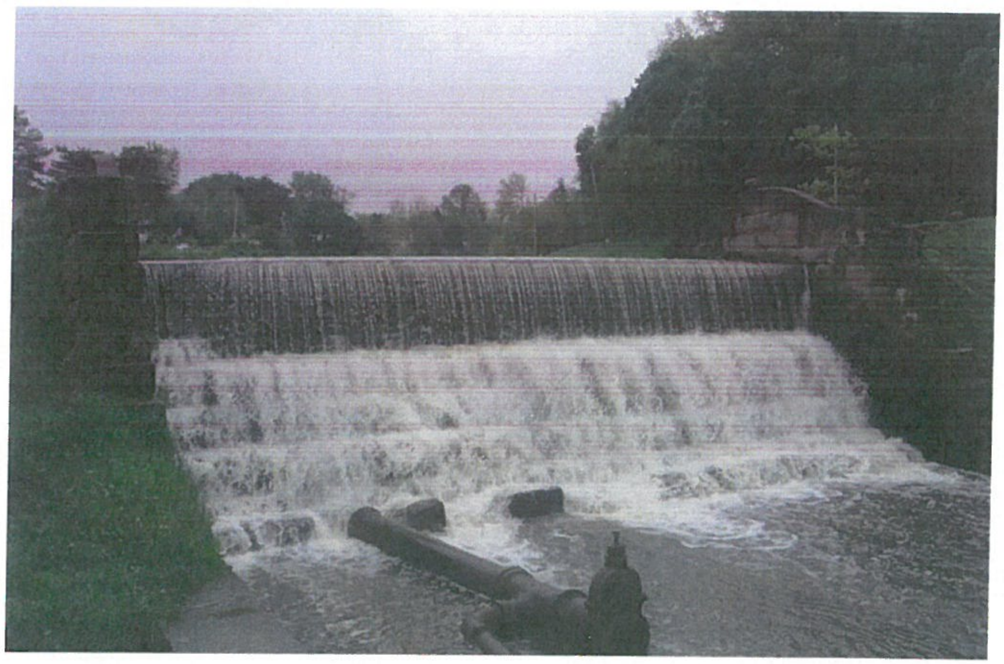
FINAL REPORT

HYDROLOGIC AND HYDRAULIC ANALYSIS OF THE SIPPO CREEK RESERVOIR DAM WATERSHED ODNR FILE NO. 0614-012

CITY OF MASSILLON, OHIO



Prepared for the
City of Massillon
Stark County, Ohio

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The following report describes the results of URS Corporation's (URS's) hydrologic and hydraulic analyses of the contributing watershed for Sippo Creek Reservoir located in the City of Massillon, Stark County, Ohio. URS performed the work as authorized by the City of Massillon (City) in April, 2011. These analyses were performed to determine the Probable Maximum Flood (PMF) inflow into Sippo Creek Reservoir and to assist in the investigation of potential improvements. In addition, a failure analysis of the dam was performed to determine the appropriate hazard category for the dam. The Sippo Creek Reservoir watershed consists of rural and subdivided areas draining a portion of Stark County as shown on Figure 1. The 14.8 square mile (9,459 acres) watershed of Sippo Creek Reservoir consists mostly of residential lots, with some wooded grassland, and wooded areas. A PMF discharge of 31,970 cubic feet per second (cfs) into Sippo Creek Reservoir was determined using HydroCAD's TR-20 model.

The information derived from our site visit and analyses are assembled in this report, including: descriptions of the general physiographic and geologic setting of the area and the results of the hydrologic and hydraulic analyses.

1.1 EXISTING SITE CONDITIONS AND DAM HISTORY

Sippo Creek Reservoir (Reservoir) is located in the City of Massillon, in west central Stark County, Ohio, east of the Tuscarawas River and State Route 21 and west of Interstate 77, as shown on Figure 1. Sippo Creek Reservoir Dam (Dam) is an earthen dam approximately 19 feet high and 265 feet long, and was built between 1875 and 1896 by the Massillon Water Works Supply Company to supply water to the City. The Massillon Water Works Supply Company is defunct and no longer supplies water to the City, who now owns the dam and the surrounding park. The old pumping house is still intact and exists on the right abutment of the Dam (looking downstream). There is no current or planned access across the dam, as there is a small pedestrian bridge just downstream of the dam. The Ohio Department of Natural Resources (ODNR), which regulates dams in the State of Ohio, has assigned the Dam ODNR File No. 0614-012.

Sippo Creek Reservoir Dam is currently classified as a Class I High Hazard Dam by ODNR because its failure would cause flooding to a residential neighborhood (approximately 3,300 feet downstream of it) with a possible loss of human life. A Class I dam is required by ODNR to safely store or discharge the Probable Maximum Flood (PMF) or a Critical Flood without overtopping. The analyses in this report are based on the PMF. Determining the magnitude of a Critical Flood may be available for Sippo Creek Reservoir Dam, but determining its magnitude is outside of the scope of this study.

In February 2002, MS Consultants (MSC) submitted a hydrology report to the City to support design plans for a roller compacted concrete (RCC) overlay to prevent overtopping failure of the

Dam. The design plans and the hydrologic report were not approved by ODNR. URS reviewed the MS Consultants hydrologic report and the associated analyses and a subsequent analysis performed by ODNR.

Both the MS Consultants report and the ODNR analysis show that in its present condition, Sippo Creek Reservoir Dam will overtop and possibly fail during to floods considerably smaller than the PMF.

The Reservoir is at the lower end of the drainage area of Sippo Creek, which is a tributary of the Tuscarawas River. The Reservoir is used for recreational purposes and has a surface area of approximately 7.0 acre in size at normal pool (1001.64 ft.) and impounds 82.2 acre-feet at the crest of the dam (1004.2 ft.). The dam has a 50 foot wide stone block spillway overflow weir which serves as the principal spillway. The weir discharges onto a series of small stone steps, which subsequently discharge onto a stone pad, which acts as an energy dissipater. A lake drain pipe emerges from the stone steps and lies on the floor of the stone pad. The channel then returns to normal width about 150 feet downstream of the Dam, where a small pedestrian bridge crosses the channel. There is currently no emergency spillway.

There are two major controls for the hydraulics of the Sippo Creek downstream of the dam. The main hydraulic control is the large road embankment for Lincoln Way (SR 142) approximately 1,500 feet downstream of the Dam which acts as a constriction point during very large flows (Figure 2). The maximum capacity of the Lincoln Way culvert, prior to overtopping the road embankment, is approximately 3,500 cubic feet per second. For comparison, the 500-year flood, as determined by the Federal Emergency Management Agency (FEMA), is 2,650 cfs. The top of the Lincoln Way embankment (el. 1008± ft.) is about 4 feet higher than the crest of the Sippo Reservoir Dam. Failure of the Lincoln Way embankment during a large flood would cause considerable flood damage downstream. A failure analysis of the Lincoln Way embankment is outside of the scope of this report.

The other major hydraulic control downstream of the Dam is a large storm sewer called the Sippo Pressure Conduit (SPC) located approximately 3,800 feet downstream of the Dam. The SPC is a large, drop structure and storm drain that acts as a siphon to transport the majority of flows, smaller than the 100-year flow, below the City and into the Tuscarawas River. Flows that are not much larger than the 100-year flood will exceed the capacity of the SPC and affect most residences along Tremont Avenue SE and flood portions of the City to the west of 3rd Street SE. The majority of the flow, in very large flows, will travel above the SPC and flood many of the residences on Tremont and most buildings to the west of its intersection with 3rd Street SE. The location of the Lincoln Way embankment and the SPC is shown on Figure 2.

The current owner of the Dam is the City of Massillon, who plans to construct dam modifications to increase the discharge/storage capacity, in excess of the primary spillway's capacity, in order to comply with ODNR dam safety regulations. This hydrologic and hydraulic analysis will support an alternatives study which is currently being performed by URS for the City.

In April 2011 the City authorized URS Corporation to proceed with the development of hydrology and hydraulics to determine the design flood in conjunction with the preparation of final design plans and specifications for the dam modifications to bring the Dam into compliance with the ODNR regulations. This hydrology and hydraulics study presents the results of the analysis of the design flood and the hazard classification for the Dam. If approved, this report will form the basis of design to facilitate development of final design plans, drawings and technical specifications. All other pertinent engineering analyses, site investigations, references, standards and backup calculations will be included in the Basis of Design Report. This document was prepared for submittal to ODNR as the initial step in the permitting process for modifying the Dam.

1.1.1 Site Setting

The dam site consists of the Reservoir, the abandoned pump house, a recreational park, and surrounding residences with large areas of open space and some mature trees. Surface topography of the site vicinity consists of gently rolling hills with ground surface elevations varying between approximately 985 and 1085 feet (NAVD 88).

The site is situated in the west central part of Stark County, and is within the watershed of the Tuscarawas River. The site is in the glaciated part of Ohio, in the Akron-Canton Interlobate Plateau, which is a part of the Appalachian Plateaus. This physiographic region is characterized as a hummocky area between two converging glacial lobes dominated by kames, kame terraces, eskers, kettles, kettle lakes and bogs/fens. This region contains deranged drainage with many natural and man-made lakes.

There is currently no subsurface information available specifically for the site. A subsurface exploration program will be initiated in support of the final Dam improvements. Bedrock is not anticipated to be encountered within 30 feet of the ground surface.

1.2 PROJECT OBJECTIVES

The objectives of this study are to determine the design flood discharge for design dam modifications that will safely pass the design flood and satisfy ODNR regulations; and to provide backup calculations to support the analysis. In addition, the requirements of Stark County, the

City Engineer, and the Stark Soil and Water Conservation District need to be satisfied. The intent is to determine the design flood to facilitate the design of an economically feasible dam improvement plan that satisfies all parties; and to determine the dam's appropriate hazard classification.

1.3 DESIGN CONSTRAINT IDENTIFICATION

To construct Dam improvements, acceptable to ODNR, several issues are required to be resolved. The following is a summary of the design considerations, constraints, and issues that will need to be considered based on the current configuration of the Dam.

- The selected solution to passing or storing the design flood requires a system to direct the floodwater over the Dam and into the downstream channel, in a controlled manner, to protect the Dam from overtopping failure; and to protect the outlet channel from erosion.
- The existing normal pool elevation is surveyed at an elevation of 1001.64 ft. (NAVD 88). The proposed lake water surface elevation may be lowered, but is preferred by the City to remain at its current level.
- The existing lake water surface elevation during the design flood will not increase due to the dam modifications.
- Any change to the existing lake outlet structure will need to be designed to enable floodwater to flow out of the lake and over the stone steps and onto the stone pad in a controlled manner.
- Overtopping protection will be necessary on the upstream and downstream face of the dam due to the large flows anticipated to flow over the dam crest.
- The flow out of the Dam must have its energy dissipated prior to discharging to the outlet channel downstream of the dam to prevent erosion damage.
- The pedestrian bridge immediately downstream of the Dam must be protected from erosion damage during large flows out of the Dam.
- A drainage easement will not be needed for the modifications, as the property is currently wholly owned by the City.
- The Stark County Engineer requires that the flooding conditions downstream of the dam not be worsened due to the dam modifications.

- The Stark County Engineer requires that the storage of the lake be consistent with current conditions.

These considerations and other issues will be incorporated into the final design to pass the design flood by storage, discharge, or a combination of both without threatening the Dam with an uncontrolled overtopping.Phase I and Phase II Property Assessment

2.1 HYDROLOGY

The following sections describe the determination of the design discharge that will be used as the basis for any design modifications.

2.2 HYDROLOGIC MODELS

2.2.1 Previously Developed HEC-1 Models

HEC-1 is a watershed runoff model originally developed by the US Army Corps of Engineers Hydrologic Engineering Center to determine peak discharges due to rainfall events. The HEC-1 model takes into consideration many watershed parameters to determine the runoff hydrographs. Complex basins can be analyzed based on their characteristics and spatial relationships to other basins. HEC-1 uses hydrology techniques and methodology developed by the US Department of Agriculture, Soil Conservation Service (SCS) (since renamed the Natural Resource Conservation Service (NRCS)).

Two HEC-1 models have been previously developed for the Sippo Creek Reservoir watershed. The initial model was developed in January 1998 by MSC in support of their hydrologic and hydraulic study as the basis for the design plans submitted to ODNR at that time. The MSC hydrologic and hydraulic study and subsequent design were not approved by ODNR. MSC used a 72-hour PMP value of 34.50 inches to determine a design flood discharge of 12,485 cfs. MSC determined that the dam was not capable of storing and/or discharging this design flood without overtopping.

The second HEC-1 model was developed by ODNR in support of their ODNR Dam Safety Investigation (May 2001) to determine the capability of the dam to store and/or discharge the design flood without overtopping. The ODNR HEC-1 was based on the MSC HEC-1 model with several modifications to the geometry and a different rainfall distribution. ODNR used a 6-hour PMP to determine a design flood discharge of 42,442 cfs. ODNR determined that the dam was not capable of storing and/or discharging this design flood without overtopping. ODNR also used a 24-hour PMP to compare with the 6-hour PMP. The 24-hour PMP resulted in a flood discharge of 20,180 cfs. Since the 6-hour PMP results in a 100 percent larger flood, it is more appropriate to use the more conservative flood as the design flood. The current accepted PMF discharge for the Dam is 42,442 cfs.

2.2.2 HydroCAD Analysis

A subsequent hydrologic model was developed by URS using HydroCAD (See Appendix A) to determine the actual existing condition design discharge for the Dam, the correct hazard classification, and if a possible Critical Flood exists for the Dam. URS used a combination of the parameters used in the HEC-1 models for Sippo Creek Reservoir from ODNR and MS Consultants to refine the outflow calculations for Sippo Creek Reservoir Dam. URS revised the outflow rating curve for the upstream lakes and the Dam based on field measurements, corrected and added storage to the model based on available topography, and incorporated the downstream tailwater conditions for the Dam to the model. Based upon these revisions, URS determined that the peak discharge of the 6-hour PMF at the dam is 31,970 cfs, the 6-hour 50 percent PMF is 11,457 cfs, the 6-hour 40 percent PMF is 8,226 cfs, and the 6-hour 25 percent PMF is 3,881 cfs.

The HydroCAD modeling system was used due to its ease of use when modeling multiple storm events and its ability to give similar results to HEC-1. Like HEC-1, it is based largely on hydrology methodology and equations developed by the SCS. HydroCAD uses the SCS TR-20 unit hydrograph and routing method, combined with other hydrology and hydraulics calculations to determine stormwater runoff from rainfall events. Similar to HEC-1, for a given rainfall event, these techniques are used to generate hydrographs throughout a watershed.

The model used the broad-crested weir equation to determine the ability of the Dam's existing stone block weir spillway and the earthen dam crest to pass flows at various headwater and tailwater elevations. Furthermore, the flows out of the Dam were checked against a rating curve developed using the Bentley's "FlowMaster" program. A combined rating curve for the existing stone block weir and the dam crest was developed to check against the HydroCAD results.

Flows through the Lincoln Way culvert and over the road were checked against a rating curve developed using Bentley's "CulvertMaster" program.

The 6-hour PMP incremental rainfall distribution described in the NRCS TR-60 Manual was used in the URS Sippo Creek Reservoir HydroCAD for storms larger than the 500-year flood. The SCS Type II distribution was used for the 100-year and 500-year floods.

2.2.2.1 Drainage Area

The drainage area for Sippo Creek Reservoir was determined by MSC to be 14.8 square miles (9,459 acres) and confirmed by ODNR and URS. The Sippo Creek Reservoir watershed consists of rural and subdivided areas draining a portion of Stark County as shown on Figure 1. The watershed of Sippo Creek Reservoir consists mostly of residential lots, with some wooded grassland, and wooded areas. The URS total drainage area was determined in AutoCAD using

USGS quadrangles of the watershed. The total drainage area determined by URS was divided into 11 sub-basins that were used in the subsequent HydroCAD model. MSC subdivided the drainage area into 5 sub-basins. The ODNR Sippo Creek Reservoir Dam HEC-1 further subdivided the drainage area into 11 sub-basins. URS used the same 11 sub-basins as used by ODNR.

Upstream of Sippo Creek Reservoir there are several large lakes and other storage areas that were included in the URS HydroCAD model. The series of man-made lakes to the northeast of the Dam is a private development built in the 1920's and is used for recreational purposes. These lakes include Cable, O'Springs and Slagle/Eric. The largest of the lakes is Lake Cable and has a surface area of 150 acres.

Sippo Lake is a large (107 acres) man-made lake to south and east of the Reservoir. MSC and ODNR also modeled these lakes using HEC-1 to determine their effect on the design storm. Downstream of the Sippo Lake is a large wetland area behind Genoa Road that was part of an old abandoned lake bed. URS added this storage area to the model which was not included in either the MSC or ODNR HEC-1 models. In smaller storms, these storage areas significantly attenuate flood peaks. During larger storms these areas do not attenuate the peak discharge as much, as the majority of their storage is filled to capacity, and inflow equals outflow, with little additional storage.

In addition, downstream of the Dam, is the North Sippo Park, which also acts as a storage area, due to the large road embankment for Lincoln Way. It was determined that the embankment attenuates large floods, but does little to attenuate floods as large as the PMF. This embankment was added to the URS HydroCAD model to determine its effect on the design flood and lesser floods. The North Sippo Park has sufficient capacity to store/discharge floods larger than the 500-year flood without overtopping the Lincoln Way embankment. The MSC and ODNR HEC-1 models did not include the storage area of the park to determine tailwater conditions at the Dam. The channel downstream of the Lincoln Way culvert was also added to the model to determine if the culvert was inlet or outlet controlled throughout a particular flood.

2.2.2.2 Curve Number

MSC performed a detailed curve number analysis for their hydrologic study of the basin in support of their design improvements for the Dam. MSC determined SCS curve numbers for the individual sub-basins of the Lake's watershed, which takes many watershed and ODNR characteristics (such as slope, cover type and soils) into consideration. The SCS curve number for each basin was determined from the US Department of Agriculture "Soil Survey of Stark County". The soil survey showed the boundaries between the curve numbers values for residential, wooded, meadow and agricultural crop areas. MSC determined the curve numbers averaged 75, which was used for their 5 sub-basins.

The ODNR reviewed the MSC curve numbers and made revisions to them, in their HEC-1 analysis. ODNR determined that the curve numbers ranging from 80, for subdivisions, to 67 used for woodlands/grasslands. The average curve number of the 11 sub-basins used by ODNR, was 72 for the entire watershed. URS also reviewed the ODNR curve numbers, and an aerial photo (dated 2007), and a field survey was conducted to verify the aerial photo. URS determined that the ODNR curve numbers were appropriate for the sub-basins, and did not modify their curve numbers. The average SCS curve number in the URS HydroCAD model is 72 for the entire basin. The average SCS curve number of 72 used for the entire basin in the ODNR HEC-1 appears to be reasonable and conservative.

2.2.2.3 Storage

The storage for the watershed was calculated by the HydroCAD model from the SCS Storage equation. The basin storage is derived from the SCS Storage equation that is dependent on the SCS curve number. The variable SCS curve numbers were used for all the sub-basins storage calculations to remain conservative.

There appears to be more storage in the watershed then is being accounted for in the HydroCAD model for smaller storms, hence the large discharges for the Type II distribution. There are several sub-basins that will have more storage than calculated using the simplified SCS Storage equation. In several sub-basins there are culverts, road embankments, small ponds and unconnected low-lying areas that were not included in the modeling. These and other low-lying areas store more water in the sub-basins than is accounted for in the modeling and generally decrease the run-off from the watershed. The net effect on the peak discharge will be to decrease it. These storage areas will tend to decrease floods peaks smaller than the 100-year event, but would not significantly decrease discharges due to larger rainfall events.

An average storage depth of 3.89 inches was estimated using the SCS Storage equation and a basin average SCS curve number of 72. This rainfall amount is stored in the basin and not applied as direct runoff.

The storage volume for the lakes (Slagle, O'Springs, Cable, Sippo Lake, and Sippo Reservoir) used in the ODNR and MSC HEC-1 models were calculated with AutoCAD and slightly modified where appropriate. The modified storage volumes were based on the Stark County Engineer's topography and input into the URS HydroCAD model. In addition, the area downstream of Sippo Lake and upstream of Genoa Road was input into the URS HydroCAD model to determine its effect on the outflow hydrograph. The rating curves for flows out of Lake

Slagle and Sippo Lake were slightly modified based on the Stark County Engineer's topography and observations and measurements taken during the site visit to the lakes.

The storage area downstream of Sippo Creek Reservoir, the culvert under Lincoln Way, and the channel downstream of Lincoln Way were also added to the URS HydroCAD model to determine the tailwater conditions affecting the outflow from Sippo Creek Reservoir during various floods.

2.2.2.4 Initial Abstraction

The initial abstraction is the amount of water that is initially stored in the basin prior to initiation of runoff. The initial abstraction for the entire watershed was calculated from the SCS Initial Abstraction equation using the storage calculated from the SCS curve numbers. The initial abstraction for a sub-basin is calculated using the SCS Initial Abstraction equation that states that the initial abstraction for a basin is 20 percent of the storage.

The average initial abstraction for the watershed was approximated to be 0.78 inches based on an average SCS curve number of 72 and a storage depth of 3.89 inches. Each basin had its individual storage and initial abstraction calculated for input into HydroCAD. The average initial abstraction number is based on the entire watershed having an SCS curve number of approximately 72. The actual initial abstraction for the watershed is probably larger than that estimated by the SCS equation, which was used to remain conservative.

2.2.2.5 Lag Time

Lag time by definition is the time differential between the centroid of the rainfall excess to the centroid of the discharge. It can be interpreted as the time it takes water to flow from the center of the basin to the outlet point of the basin or the time to peak at the outlet of the basin. In essence, the longer the rainfall lags in the basin, the lower the peak discharge and conversely, the quicker it leaves the basin the higher the peak discharge.

The MSC HEC-1 used varying lag times ranging from of 1.78 hours for smaller basins, to as much as 16.4 hours for the majority of the watershed. Although these lag times might be reasonable for a larger basin with a 72-hour rainfall, they are outside of the practical range of lag times for a basin of this size and rainfall of the required duration. The lag times determined by MSC appear to be overestimated for the sub-basin size and length to width ratios.

The lag time for each sub-basin was determined by ODNR, and confirmed by URS, using the SCS TR-55 time of concentration and using the SCS lag time equation (LT=0.6Tc). The various lag times for the sub-basins ranged from 2.26 hours, to as little as 0.44 hours, using variable SCS curve numbers for each sub-basin. The average lag time for the sub-basins is about 1.2 hours, which is a reasonable estimate for the basin configuration. Almost one-half of the watershed has a lag time longer than 1.5 hours. URS determined that the ODNR lag times used were reasonable and used them in the URS HydroCAD modeling. Since the peaks of the hydrographs generated by the varying lag times are not coincident, the sub-basin hydrographs are add not additive. Therefore, due to the difference in lag times for the various sub-basins, it is apparent that the peak flows of the individual sub-basin hydrographs do not coincide, which reduces the peak flow at the Dam.

2.2.2.6 Impermeability

The United States Geological Service (USGS) 7.5 Minute Quadrangles for Massillon and West Canton, Ohio with a scale of 1:24,000 and a 10' contour interval were input into AutoCAD and used to check the MSC and ODNR drainage areas, stream lengths, and slopes. In addition, the areas upstream of the Dam identified as lakes and ponds and other impervious areas such as roads, buildings and parking lots were used to estimate the percentage of connected impervious area.

In both the ODNR HEC-1 and the MSC Sippo Creek Reservoir HEC-1, only some of the impermeability of the basin was accounted for. The impermeability of a watershed greatly affects how much water is absorbed by the basin and offsets some of the storage and initial abstraction of the basin depending how much impermeable surface area there is and if it is connected. The SCS curve number takes larger amounts of the impermeable area of the watershed into account (tract houses, commercial real estate, etc.) but not the smaller percentages that can become significant.

In addition, the large number of lakes, ponds, and low-lying areas that contribute to the direct runoff from the basin will also add some storage volume during smaller floods. The percentage of impermeable area of the various sub-basins was determined by taking the surface area of the connected lakes and ponds as shown on the USGS quadrangle. It was presumed that all of the small lakes and ponds were connected, at capacity and had little storage. The percentage of connected impermeable area calculated for the sub-basins was used in the HydroCAD model.

2.2.2.7 Survey

A survey of the spillway and the downstream channel was performed in August 2011 by the City of Massillon and the information was incorporated into the URS basemap and models. There are currently no other available surveys of the dam and surrounding area. Ground surveys will be performed to support the hydrologic and hydraulic analyses for the final design. The future survey will include the topography of the Dam and Sippo Creek, surface drainage features, utility boxes, the upstream and downstream slopes and downstream foot bridge. In addition, the finished floor elevations of the low-lying houses on Tremont Avenue SE will be determined. The vertical datum for the survey is the National Geodetic Vertical Datum of 1988 (NAVD 88) and the horizontal control is the Ohio State Plane Coordinate System NAD 1983 (NAD 83), North Zone, U.S. Foot.

2.2.2.8 Probable Maximum Precipitation

The Probable Maximum Flood that would flow into Sippo Creek Reservoir would be a direct result of the Probable Maximum Precipitation falling on the watershed during a given time period. The PMP is the largest expected storm event based on historical rainfall and predicted meteorological data. The PMP as described by the National Oceanic and Atmospheric Administration (NOAA) is "Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year."

PMP rainfall totals were determined by charts available in NOAA's HMR-51 (1978) All Season Probable Maximum Precipitation for 6 and 24-Hour Duration, 10 Square Mile Area and are 26.15 inches and 32.00 inches, respectively. The 6-hour PMP chart for Ohio is also shown on the ODNR Division of Water Fact Sheet 95-37 "Dam Safety-Probable Maximum Flood".

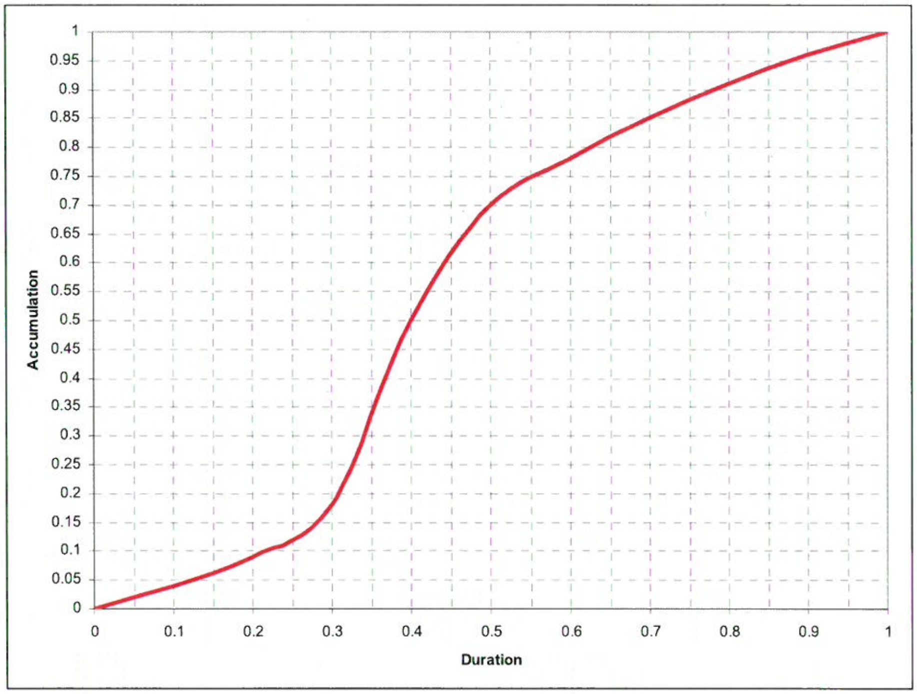
2.2.2.9 Rainfall Distribution

Rainfall for storms larger than the 500-year flood was distributed in the HydroCAD model according to the Natural Resources Conservation Service (NRCS) Technical Release 60 (TR-60, dated 2005). The NRCS utilizes a standard Emergency Spillway and Freeboard (ESFB) design hydrograph rainfall distribution as shown in graph below. This distribution curve was applied to the PMP, and fractions thereof, for the Sippo Creek Reservoir watershed over both a 6-hour and 24-hour period to determine the worst case scenario peak inflow into the lake.

The SCS Type II rainfall distribution was applied to the 24-hour/100-year rainfall depth determined from the Midwest Climate Center's (MCC) Rainfall Frequency Atlas of the Midwest (Huff and Angel, 1992). The discharge from the MCC depth was compared to the FEMA estimated discharge. A rainfall depth was estimated to give a respective discharge based on the FEMA Flood Insurance Study (FIS). Since the FEMA estimated discharge of 1,980 cfs is higher than the MCC determined discharge of 1,763 cfs, the FEMA values were used. There are no MCC values estimated for the 500-year flood.

The Soil Conservation Service (SCS) Type II distribution was also used to estimate the 500-year flood rainfall depth to result in the FEMA 500-year discharge of 2,650 cfs.

The 6-hour PMF estimated inflow discharge of 31,970 cfs is considerably higher than the estimated 24-hr PMF inflow of 23,172 cfs. Therefore, the peak discharge of the Probable Maximum Flood into the Sippo Creek Reservoir is determined to be 31,970 cfs and is considered the PMF design discharge.



ESFB Dimensionless Design Storm Distribution Graph

2.2.2.10 Probable Maximum Flood

The design discharge for a Class I dam by ODNR rule is the Probable Maximum Flood (PMF) or the Critical Flood. The PMF is the expected discharge that is caused by the Probable Maximum Precipitation (PMP). The PMF is the flood that is a result of the PMP on a watershed basin. The PMP for the watershed that was used in the modeling to determine the design discharge is 26.15 inches in a 6-hour period. The URS determined PMF, using this rainfall value on the Sippo Creek Reservoir watershed, is 31,970 cfs.

In its present condition, Sippo Creek Reservoir Dam will overtop and possibly fail due to floods considerably smaller than the estimated Critical Flood.

2.2.2.11 Critical Flood

A Class I dam is required by ODNR to safely store and/or discharge the Probable Maximum Flood (PMF) or a Critical Flood without overtopping. If approved by ODNR, this Critical Flood can be used as the design flood for the future modifications planned for the Dam.

The critical flood is the flood discharge at which no further damages downstream would occur whether or not the dam fails. Preliminary results from the URS dam breach parameter analysis show that a Critical Flood probably exists for the Dam. The smallest Critical Flood for a Class I dam is 40 percent of the PMF and the smallest Critical Flood for a Class II dam is 20 percent of the PMF. Determination of the acceptable critical discharge requires approval by ODNR. A Critical Flood for Sippo Creek Reservoir Dam may be approved by ODNR once the analysis is performed.

The URS dam failure analysis estimates that that a Critical Flood exists for the Sippo Creek Reservoir Dam due to the submergence of the Dam and the overtopping of the Lincoln Way embankment as shown on Chart 1.

A Critical Flood inflow discharge as low as 4,500 cubic feet per second (cfs) is estimated by URS, which is approximately 30 percent of the PMF discharge. The 20 percent PMF is determined to be 2,672 cfs, which approximates the FEMA 500-year flood discharge of 2,650 cfs.

2.3 HYDRAULICS

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) v4.1.0 was used to model the hydraulics of the Sippo Creek Reservoir Dam, the Lincoln Way culvert, the Sippo Pressure Conduit and the downstream channel to just past the SPC. The limits of the HEC-RAS model extend from just upstream of Hankins Street, to just downstream of the intersection of Tremont Avenue SE and 2nd Street SE as shown on Figure 2. The area downstream of the Sippo Pressure Conduit, was modeled to determine how larger discharges might flood the area when the conduit capacity is exceeded. The HEC-GeoRAS extension of ESRI's ArcMap was used to develop a HEC-RAS model to determine how the Lincoln Way embankment and culvert affect flooding during storms ranging from the 100-year flood, up to and including the Probable Maximum Flood. HEC-RAS output was compared with the HydroCAD output to ensure that it gave reliable results and supported the conclusions. Table 1 shows the results of the comparison. The following is a description of the HEC-RAS modeling effort.

2.3.1 HEC-RAS Model Development and Methodology

A site basemap was developed in AutoCAD Civil-3D 2011 using a topographic map with two-foot contour intervals that was obtained from the Stark County Engineer. The topography along the crest of the dam and the spillway were adjusted slightly based on the August 2011 survey performed by the City. The basemap was augmented with hydrology, buildings and culverts based on existing as-built plans, GIS information, the 2011 survey, aerial photographs and a site visit. The topographic basemap and a color aerial photograph were used with the GIS program ArcMap 9.3 to determine the geometry of the channel and floodplain; and to develop cross sections for input into the HEC-RAS model. A HEC-RAS cross section location map is shown on Figure 2. Site features maps are shown on Figures 3 and 4 of the areas south and north of Lincoln Way, respectively. ArcMap was also used to determine hydraulic parameters such as slope, distance between cross sections, channel limits, and top of bank elevations. In addition, a site visit and extensive review of field photographs was used to verify the model data input.

There are three main parts of the HEC-RAS modeling effort. The upstream portion of the Sippo Creek HEC-RAS model runs from Hankins Road NE to the Sippo Creek Reservoir Dam. The middle portion of the model extends downstream form the Dam to the Lincoln Way embankment and culvert and incorporated the North Sippo Park. The lower portion of the model extends downstream from Lincoln Way, past the Sippo Pressure Conduit, to just beyond the intersection of Tremont Avenue SE and 2nd Avenue SE, in the City of Massillon.

The critical structures on Tremont Avenue SE, downstream of the Lincoln Way embankment, were added to the model to determine actual depths of flooding in the area. There are some critical structures downstream of the Sippo Creek Pressure Conduit. Critical structures are defined as habitable structures where a possible loss of life can be expected during large floods or a possible dam/embankment failure. The low-lying houses on Tremont Avenue SW, downstream of South Sippo Park, are subject to flooding depths as much as 3 feet during the 100-year flood, and to depths in excess of 15 feet during the Probable Maximum Flood. Depths in excess of three feet are considered dangerous as cars, and other large floating objects, can be mobilized and carried in the flood. In addition, due to the hydraulics of the Sippo Creek Pressure Conduit, floating objects close to the inlet to the conduit are subject to being drawn into the pressure pipe.

The HEC-RAS model was run in steady state mode over a range of flows to determine the water surface elevations, velocities and flow regimes; and to determine the limits of flooding through the studied reach.

AutoCAD Civil-3D 2011 was used in conjunction with the topographic basemap to determine the limits of flooding as a result of the various floods.

2.3.2 Expansion/Contraction Coefficients

The expansion and contraction coefficients significantly affect the head losses at a culvert or bridge, as water is forced through an opening smaller than the upstream and downstream channel. The expansion and contraction coefficients for the culvert under Lincoln Way and the Sippo Pressure Conduit were a set at 0.5 and 0.3, respectively to remain conservative. In addition, the expansion and contraction coefficients were also increased, in the same manner, for the roads that cross the creek, upstream of the Dam. The expansion and contraction coefficients for all other sections were set at 0.3 and 0.1, respectively.

2.3.3 Weirs and Weir Coefficients

There are three separate weirs included in the HEC-RAS model, which were input as broad crested weirs, but with different weir coefficients. The three weirs modeled are the Dam, the Lincoln Way roadway, and the flow above the Sippo Pressure Conduit.

The Dam was modeled in HEC-RAS as an in-line structure with an average weir coefficient of 2.71, due to the large depth of flow over the Dam during extreme flood events, and the existence of the concrete spillway. HEC-RAS does not have the capability of modeling an inline structure with multiple weirs. The entire weir in the HEC-RAS model was modeled with an average breadth (length in flow direction) of 14.9 feet. In comparison, the primary spillway was modeled separately in the HydroCAD model, and a weir coefficient was assigned a value of 3.32 which is the upper limit for an inefficient broad crested weir. The top of the earthen dam, in the HydroCAD model, was assigned a value of 2.63, which is the lower limit of a broad crested weir under the expected flow conditions. The average of the weir coefficients in both models is approximately 2.71. Although the average weir coefficient is variable and dependent on the depth of flow over the weir, and the breadth of the weir, it is probably close to the value assigned during larger flows. This average weir coefficient assigned for the Dam is a practical lower limit, to minimize the flow over the dam, and to remain conservative.

The actual weir coefficient for the primary concrete spillway of the Dam is probably higher than 3.3, at the depths of flooding analyzed, while the actual weir coefficient for the weir flow over the dam would probably approach 2.8 at the depths analyzed. In addition, in the model calculations, since the weir is submerged during large floods, the actual weir coefficient is minimized by the submergence factor.

The weir coefficient for Lincoln Way roadway was determined to be 2.63, with a broad crested weir breadth of 60 feet. The roadway is overlaid with asphalt, but has curbs and guardrails which limit the actual flow over the roadway. The weir coefficient may be slightly higher than that used in the model, but the value was used to remain conservative. The actual weir coefficient is dependent on depths of flow over the roadway. In the HEC-RAS and HydroCAD models, the roadway weir coefficient was assigned the lower limit of the expected values.

The weir coefficient for the flow above the Sippo Pressure Conduit was input into the HEC-RAS model to determine flows above the culvert. A broad crested weir coefficient of 2.63 was used to approximate flows in this area. This average weir coefficient assigned for the Sippo Creek Pressure Conduit is the practical lower limit, to minimize the flow over the pipe, and to remain conservative.

All weir coefficients were estimated from the HydroCAD technical reference guide, which is reprinted from "Practical Hydraulics" by Andrew Simon (1981).

2.3.4 Manning's n Roughness Coefficients

The existing Manning's roughness coefficients were developed from information derived from site visits, aerial photographs and site photographs. Manning's roughness coefficients were also estimated from those used in the FEMA Flood Insurance Study for Stark County, Incorporated Areas. In addition, the United States Geological Survey (USGS) (1984) Water Supply Paper 2339 "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains" was used as a reference.

Manning's roughness coefficients for the channel ranged from 0.022 to 0.035, with an average of 0.03 for the majority of the creek. The areas that had a rough weedy channel were assigned a value of 0.035 and the lake bottom was assigned a value of 0.022 to remain conservative. The overbanks were given an average value of 0.045. Roads in the floodplain were assigned a value of 0.020 and large trees and higher/thicker vegetation areas were assigned a value of 0.10.

2.3.5 Ineffective Flow Areas

Ineffective flow areas are areas that are not used in the conveyance calculations in HEC-RAS, but are used to calculate storage between cross sections. The most significant areas that are considered ineffective in the HEC-RAS model are the areas just outside of the opening of the modeled culverts. It is recommended by the Hydrologic Engineering Center that these areas be designated ineffective to ensure that head losses through the culvert are calculated correctly.

A small area upstream of 17th Street NE was also included as ineffective as it is protected by a small levee along the creek during smaller floods. This area does not affect the flooding conditions downstream of the Dam.

2.3.6 Obstructions

Obstructions such as existing buildings and structures were added to the HEC-RAS model, where appropriate, to ensure that blocked areas that can't convey flood water were not included in the conveyance calculations. The abandoned pump house building on the right embankment of the Dam was included, as it blocks flow out of the Reservoir during large floods. In addition, the houses and large out-buildings (detached garages/ storage sheds, etc.) on Tremont Avenue SE were included to determine the depths of flooding at these structures during various flood events. These buildings also block flow in the floodplain and take up storage area.

In addition, the upstream buildings and out-buildings were included in the HEC-RAS modeling to determine how the backwater from the dam affects these structures. Several of the houses and commercial buildings upstream of the dam are subject to flooding during the 100-year flood. Several of these structures between the Reservoir and Hankins Road are severely affected during flood events larger than the 500-year flood.

2.3.7 Bridges/Culverts

The HEC-RAS modeling includes two main culverts to determine flooding depths downstream of the dam. The culvert under Lincoln Way and the Sippo Creek Pressure Conduit were included as they affect the hydraulics of the creek. The minor culverts and bridges upstream and downstream of the dam were not included in the HEC-RAS modeling as they are too small to affect the hydraulics during larger floods. The ones upstream do not affect the hydraulics of the downstream area and the bridges downstream would be submerged during large floods. It should be notes that the culvert under Genoa Road and its associated storage area were added to the HydroCAD model as this area affects the peak flow for various storm events in the watershed.

2.3.8 Existing Primary Spillway

The existing primary spillway of the Dam consists of a concrete block weir that is 50 feet in length and a surveyed elevation of 1001.64 ft. (NAVD 88). The weir outlets onto a set of concrete blocks, which act as an energy dissipater, with a concrete slab below that to further dissipate the energy of flow. Downstream of the primary spillway is a 50 foot wide manmade channel with little or no erosion protection. Further downstream of the spillway pad is a small pedestrian bridge, with 45 degree wingwalls, and concrete abutments. The channel returns to a natural 20 foot wide meandering channel downstream of the footbridge as shown on Figure 4.

The top of the Dam is earthen with a concrete block cutoff wall that is exposed near the primary spillway. The top of the cutoff wall on the left abutment is surveyed to be at an elevation of 1004.2 ft. and the embankment has a playground downstream of it. The top of the cut-off wall on the right abutment has a surveyed elevation of 1005.0 ft. and terminates at an abandoned water supply building. During large floods, water may flow around the abandoned water supply building.

The primary spillway is not capably of passing large floods without overtopping the crest of the dam. There is evidence that the Dam has overtopped during large floods and eroded the embankment on both sides of the primary spillway downs to the concrete block cutoff wall. The primary spillway can pass approximately 600 cfs without overtopping the left embankment and 900 cfs without overtopping the right embankment. By comparison, FEMA determined that the 100-year flood on the Sippo Creek is approximately 1,980 cfs.

2.3.9 North Sippo Park

The wide valley below the Sippo Creek Reservoir Dam is part of the North Sippo Park, which is maintained as part of the Massillon Park System. The park in general is surrounded by high hills and consists of the floodplain terrace of the Sippo Creek. The valley between the Dam and Lincoln Way is subject to extreme flooding. The 100-year flood would have a channel depth of approximately 12 feet. Larger floods could fill the entire valley and overtop the Lincoln Way embankment, with depths over 30 feet in the North Sippo Park.

In addition, since the North Sippo Park has a large storage area, large floods downstream of Lincoln Way are attenuated. Since the park acts as a defacto dry detention basin, it is an integral part of flood prevention during very large floods. It should be noted that the storage area of North Sippo Park is larger than the storage of the entire Sippo Creek Reservoir. In fact, flooding in the park during very large floods would back water into the Sippo Creek Reservoir, creating one large lake that would submerge the Dam.

The North Sippo Park was not analyzed by MSC or ODNR in their respective HEC-1 models. Since the tailwater elevations below the Dam controls the actual discharge out of the Dam during large floods, the analysis of the park area is integral to understanding the flooding condition downstream of the Dam.

The storage area of the park was estimated from the URS topographic basemap and calculated in the HydroCAD model for irregular prismatic shapes. The volumes calculated were checked against the total volume calculated in AutoCAD Civil 3-D. The total volume of the park between the Dam and the Lincoln Way embankment (el. 1108) is approximately 197 acre-feet and has a surface area of approximately 13.5 acres. By comparison, the Sippo Creek Reservoir has a total volume of approximately 143 acre-feet and a surface area of 21.5 acres at the same elevation (1008 ft.). The difference in volume attenuates large floods and minimizes any dam break flooding in the North Sippo Park. Table 3 shows that peak discharges downstream of the North Sippo Park slightly increases due to dam failures during larger floods.

2.3.10 Lincoln Way Culvert

The Lincoln Way culvert (Stark County Bridge No. PE-8-49) is a 14 foot wide by 10 foot tall, 121'-10" long, semi-circular concrete box. The inlet to the culvert has wingwalls and a headwall to minimize head losses into the structure. The top of the Lincoln Way embankment is approximately 29'-9" above the culvert flowline. The top of the roadway is estimated to be at an elevation of 1008.0 ft., which was used in both the HEC-RAS and HydroCAD models. The maximum flow through the culvert is approximately 3,500 cfs, which is larger than the estimated 500-year flood discharge. Larger flows would overtop the embankment, possibly causing its failure. A failure of this embankment would cause severe flooding downstream. Damage from a failure of this embankment could possibly have a large economic impact and a possible loss of life. The information sheet from the Stark County Engineer is included in Appendix 4.

In the HEC-RAS and HydroCAD models the box culvert was modeled as an equivalent area box culvert with a width of 14.0' and a height of 8'-2" (area = 114.33 sf). The length is 121.83' with a slope of 0.1 percent. A Manning's roughness coefficient of 0.015 was assigned to the culvert, since only half of the culvert is concrete, and the other half is brick.

2.3.11 Sippo Pressure Conduit

The Sippo Pressure Conduit (SPC) is a 13.25 foot wide by 10 foot tall reinforced concrete box culvert siphon that starts approximately 850 feet east of the intersection of Tremont Avenue SE and 3rd Street SE as shown on Figures 2 and 3. The inlet to the SPC is a drop structure which is approximately 8.5 feet below the invert of the creek, as it enters the siphon. The invert of the inlet to the SPC is at elevation 935.38 ft. The flow enters the SPC by way of a steeply sloped concrete drop structure that allows large flows to be conveyed underground along Tremont Avenue SE and into the Tuscarawas River. Flows in the pipe are under pressure the entire length of the culvert. Backwater gates along the pipe prevent surcharges into the areas along the pipe. The pipe is capable of handling the majority of flow during the 100-year flood, but cannot pass larger floods. Any flows in excess of the 100-year flow are conveyed along Tremont Road SE above the pipe and cause flooding to Downtown Massillon.

The SPC was modeled in HEC-RAS as a long 9.5' tall by 13.25' wide concrete culvert to match the actual open area of the culvert, since the corners are mitered and the invert is an inverted crown. The culvert was assigned a Manning's roughness coefficient of 0.012 and a slope of 0.5

percent. The SPC was not added to the HydroCAD model since only discharges downstream of the Lincoln Way culvert were required to determine actual depths of flooding.

The plan and profile design sheets from the United States Army Corps of Engineers are included in Appendix 4.

2.4 DAM FAILURE ANALYSIS

The parameters for the hydraulic calculations used in the hydraulic analysis were taken from various sources. The following is a synopsis of the determined parameters used for input into the HydroCAD dam failure models. The existing conditions HydroCAD model was developed to determine various floods inflow and outflow discharges for the Reservoir and the resulting downstream flood conditions. Subsequent HydroCAD models were developed by URS to determine approximate outflow conditions should the dam fail during different flooding scenarios. This analysis is the basis for reclassifying the dam from its current hazard classification.

2.4.1 Methodology

A dam failure analysis spreadsheet using industry standard dam failure equations and estimated parameters was developed to estimate the outflow dam failure discharge, the average width of breach, and the timing of the breach during different dam failure scenarios. The dam failure spreadsheets are included in Appendix 3. The dam failure equations used in the analysis spreadsheet are described in detail in Section 2.4.2 below. The determined peak inflow discharge from the URS existing conditions HydroCAD model was used as the starting baseflow condition for the dam failure analysis. This flow was added to the outflow discharge from the Reservoir estimated from the spreadsheet. In this manner, the peak flow from the dam failure is added to the baseflow to give reasonable estimates of the total outflow discharge and the storage area inundated downstream. There were seven dam failure scenarios run for the analysis. The runs were for the "Sunny Day" condition {where the dam fails with minimal flow and no overtopping), the 100-year and 500-year floods (where slight overtopping occurs), and the 0.22 PMF, the 0.24 PMF, the 0.25 PMF, the 0.5 PMF, and the full PMF (where significant overtopping occurs). The URS existing conditions HydroCAD model was modified to develop a series of dam failure HydroCAD models.

The dam failure HydroCAD model was developed using the known peak inflow during each event from the existing conditions HydroCAD model. The input parameters were the increase in flow due to the dam failure estimated from the dam failure parameter spreadsheet, and the peak headwater and tailwater condition for the selected scenario from the existing conditions

HydroCAD model. In this way, the total outflow from the dam is modeled as a combination of the peak flow without the failure and the increase in flow due to the failure, and the maximum water surface elevation during flood conditions upstream and downstream of the dam, at the time the dam fails.

The outflow discharge estimates from the dam failure analysis spreadsheet were input into the dam failure HydroCAD models. The peak discharge of each modeled flood was added to the additional discharge from the corresponding dam failure. The two discharges were assumed to be concurrent to assure that the maximum downstream discharge was calculated. The outflow from the dam failure was modeled as a broad crested rectangular weir with a head at the full height of the dam. The width of the breach was adjusted to give the expected dam failure discharge at the peak of the flood event.

Each breach scenario was initiated using the maximum elevation in the lake, corresponding to the flood scenario water surface elevation determined in the existing conditions HydroCAD model. The model assumed the breach failure was initiated at the start of the modeling and was an instantaneous dam failure. The breach model calculates the displacement of storage, but not the movement of the actual dam failure wave. The topographic basemap and HEC-RAS were used to determine the actual depths of flooding downstream of the dam due to each failure. The dam breach analysis in this manner is more conservative than a progressive failure due to the timing of the lake volume release and the changing downstream conditions.

The starting water surface elevation of the Reservoir and the downstream areas of North Sippo Park matched the peak discharge elevations from the existing conditions HydroCAD model for each scenario. In this manner, the model was started with the most conservative initial conditions for each failure scenario. The resultant maximum peak flow out of the Reservoir occurs with the maximum volume in the Reservoir and the least storage available in the North Sippo Park during the dam failure. The upstream and downstream storage areas were those used in the existing conditions HydroCAD model; as was the geometry of Lincoln Way culvert and the downstream channel.

The actual inflow hydrograph for the watershed was not used due to the difficulty with matching the peak of the flood with the timing of the dam failure in HydroCAD. However, the dam failure HydroCAD model starting with the peak condition during the flood gives reliable answers in determining the changes in storage as the Reservoir drains and fills the North Sippo Park; while estimating the attenuation of the peak discharge in the downstream channel due to the Lincoln Way embankment culvert. Therefore, the total outflow from the dam failure is added to the peak flow of the flood while the storage is maximized upstream and minimized downstream, to keep conservative. In actuality, the dam could fail much earlier than the peak of any given storm event.

A full dam break analysis using an unsteady-state HEC-RAS model would be necessary to perform a Critical Flood Analysis, which is outside the scope of this study. However, expected peak dam failure flows were input into the existing conditions steady-state HEC-RAS model to assist in determining estimated flood depths and velocities expected during each failure scenario.

Water elevations in the Reservoir and the North Sippo Park were set to their peak elevations for each scenario from the results of the existing conditions HydroCAD model. The "Sunny Day" failure scenario modeled a piping failure mechanism since the Dam is not overtopped. For all other scenarios, overtopping was used as the breach mechanism for the modeling, as it is the more likely, and conservative, cause of failure for the Dam.

2.4.2 Breach Parameters and Assumptions

Common practice for breach parameters have been compiled by Bill Irwin in Workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analysis and were followed in this analysis. The breach parameters and assumptions for the embankment failure analysis were taken from the Bureau of Reclamation (BOR) "Guidelines for Estimating Dam Breach Parameters", the journal of Hydraulic Engineering "Breach Characteristics of Dam Failures", and others. The two major input parameters for breach modeling are average breach width and breach development time. Earth embankment dams historically have been found to have average breach widths of 1 to 5 times the hydraulic height of the dam and breach development times between 6 and 60 minutes. Average breach width and breach development times were calculated for each failure scenario based on equations derived by MacDonald & Langridge-Monopolis reported in Washington State Dept of Ecology's Dam Safety Guidelines Technical Note 1 as well as equations derived by Von Thun & Gillette as seen in Prediction of Embankment Dam Breach Parameters and are included in Appendix 3. The calculated values that correlated best with widely accepted common practice values were reviewed to estimate actual peak discharges due to a dam failure for each scenario. URS used the more conservative values for the failure peak discharge, average breach width, and breach development time. Breach side slopes were set at 1H:1V since cohesive fill dikes typically fail at slopes this steep or steeper, and the hydraulic embankment height was taken as the difference between the reservoir starting water surface elevation and the tailwater surface elevation at the base of the embankment at the peak of the flood scenario.

Table A below shows the parameters URS used for the modeling efforts.

Dam Breach Input Parameters

Scenario	Dam Breach Input Parameters*							
	Height of Water (ft)	Crest Width (ft)	Storage Volume (ac-ft)	Upstream Slope (Z_1:1)	Downstream Slope (Z_2:1)	Breach Sideslope	Surface Area of Reservoir (ac)	Failure Mode
Sunny Day**	15	15.0	61.0	3.0	2.5	1.0	7.1	Piping
100-yr	13.61	15.0	104.1	3.0	2.5	1.0	18.4	Overtopping
500-yr	9.35	15.0	83.9	3.0	2.5	1.0	18.5	Overtopping
0.22 PMF	5.56	15.0	66.7	3.0	2.5	1.0	19..4	Overtopping
0.24 PMF	1.97	15.0	35.8	3.0	2.5	1.0	20.8	Overtopping
0.25 PMF	0.93	15.0	20.1	3.0	2.5	1.0	22.6	Overtopping
0.50 PMF	1.2	15.0	48.9	3.0	2.5	1.0	47.4	Overtopping
PMF***	2.14	15.0	149.3	3.0	2.5	1.0	72.9	Overtopping

*Dam Safety Guidelines - Dam Break Inundation Analysis and Downstream Hazard Classification - Technical Note 1 – Washington

**Sunny Day dam failure with water surface elevation in the lake at 1001.64 at time of breach.

*** 24-hour PMF using TR-60 Distribution

The table below shows the average breach width expected, the time of breach development, downstream slope used and the estimated breach discharge increase. As shown, the increase in discharge due to the breach is diminished, up to the 0.25 PMF, as the difference in headwater to tailwater depth is reduced. After the 0.30 PMF, the difference in head starts to increase, up to the full PMF, due to overtopping of the Lincoln Way embankment.

Dam Breach Output

Scenario	Dam Breach Results			Estimated Outflow Q increase (cfs)
	Average Breach Width (ft.)	Breach Development Time (hr)	Downstream Channel Slope (ft/ft)	
Sunny Day**	57.5	0.55	0.004	3,500
100-yr	54.1	0.52	0.004	4,400
500-yr	43.4	0.44	0.004	2,820
0.22 PMF	33.9	0.36	0.004	1,820
0.24 PMF	24.9	0.29	0.004	412
0.25 PMF	22.3	0.27	0.004	115
0.50 PMF	23.0	0.27	0.004	303
PMF***	25.4	0.29	0.004	1,350

2.4.3 Breach Modeling and Analysis

Modeling efforts began by assigning known variables, and outputs from the dam failure analysis spreadsheet and the existing conditions HydroCAD model into dam failure HydroCAD models to simulate each scenario as accurately as possible. Peak discharges, which were taken from the existing conditions HydroCAD model, were used for input into the dam failure HydroCAD model for each breach event that would occur at the Dam. The additional breach discharge, taken from the dam failure spreadsheet, was added to the peak discharge to ensure the correct total discharge was modeled. Embankment failures were modeled using an unsteady state analysis of the flood discharge out of the Reservoir into the downstream area.

Individual HydroCAD models were set up to accurately model the behavior of each breach scenario. The models generated the total outflow hydrograph of the Reservoir breach, to determine the depth of water downstream and to determine if the Lincoln Way embankment would be overtopped. The outflow discharge from the Reservoir was compared to the discharge downstream of the Lincoln Way embankment to determine the increase in flooding at the critical structures as shown on Table 3. The depth of flooding at the critical structures was determined using the existing conditions HEC-RAS, based on the increased flow due to failure of the Dam.

During the Sunny Day scenario, the water surface elevation was set at the Dam's normal pool elevation of 1001.64 ft., and a tailwater depth of 1 foot. For the 100-year flood scenario, the water surface elevation was set at an elevation of 1006.27 ft., which is the maximum water surface in the Reservoir during the 100-year flood. This elevation is about 2 feet higher than the lowest elevation on the crest of the dam. Overtopping at this depth would be expected to cause considerable overtopping damage and possible failure of the dam. During successively larger floods, the water surface elevations in the Reservoir and the downstream area were set according to headwater and tailwater elevations shown on Table 2. Modeling the Reservoir failures in this manner gives the most conservative output.

2.4.4 Results

The results of the dam break analyses show that the Dam is subject to backwater/tailwater conditions, due to the Lincoln Way embankment, that submerge the Dam during large floods. Impacts from a failure of the Dam are reduced due to these tailwater conditions. The difference in elevation between the downstream water surface elevation of the Dam and the headwater over the Dam decrease as the inflow discharge increases, as shown on Table 2. Due to this condition, should the Dam fail during large floods, the expected increase in flooding downstream of the Dam, is minimized as shown on Table 3.

In addition, due to the large storage area between Lincoln Way and the Dam, in North Sippo Park, increased discharges caused by the Dam failing, during smaller floods, are minimized. In addition, the Lincoln Way meters the flow to the downstream area, minimizing flooding during floods smaller than the 0.3 percent PMF.

As shown on Table 3, the "Sunny Day" and the 100-year failure scenarios are the only floods that dramatically increase the flooding discharge downstream of Lincoln Way, without overtopping the embankment. Floods larger than the 0.25 percent PMF would overtop the Lincoln Way Embankment and discharges downstream are no longer attenuated by the culvert.

The "Sunny Day" failure scenario starts with a minimum discharge, and the Dam failure increases the discharge to 3,500 cfs, which is attenuated to 2,211 cfs by the Lincoln Way culvert. During the 100-year failure the inflow discharge peaks at 1,980 cfs and increases to 6,380 cfs due to the dam failure, and is attenuated to 3,737 cfs by the Lincoln Way culvert. During larger failure scenarios, the increase in discharge due to the failure decreases, even while the inflow discharge increases. During the 0.25 percent PMF failure, the inflow discharge peaks at 3,881 cfs and increases only to 3,996 cfs, due to a dam failure, since the difference in elevation between the headwater over the dam and the tailwater is less than a foot. The total discharge is attenuated to 3,400 cfs by the Lincoln Way culvert. Floods much larger than the 30 percent PMF would overtop, and probably fail the Lincoln Way Embankment. Failure of the embankment would cause worse flooding conditions, than a Sippo Creek Reservoir Dam failure. Failure of the Sippo Creek Dam, during floods larger than the 50 percent PMF, would not cause any additional damage downstream, as long as the Lincoln Way embankment hadn't already failed. If the Lincoln Way embankment is overtopped, and fails, during floods larger than the 30 percent, failure of the Sippo Creek Reservoir Dam would not make matters worse.

The table below illustrates the results for the Sippo Creek Reservoir Dam Break Analysis. As shown, except for the "Sunny Day" scenario, the additional discharge from a dam failure decreases, while the total discharge at the habitable structures downstream of the Lincoln Way Embankment only slightly increase. The depth of flooding also only slightly increases for floods up to the 0.25 PMF. The floods at the habitable structures downstream of the Lincoln Way embankment are attenuated by the storage area of North Sippo Park and the metering of the flow by the Lincoln Way culvert.

URS Dam Breach Analysis Results

Scenario	Sippo Creek Reservoir Dam Breach Analysis Results			
	Total Maximum Dam Breach Outflow (cfs)	Total Discharge Outflow downstream of Lincoln Way (cfs)	Approximate Lowest Floor Elevation of Critical Structures on Tremont Avenue SW* (ft)	Water Surface Elevation at Critical Section (ft)
Sunny Day	3,500	1,289	962.0	963.96
100-yr	6,320	2,643	962.0	965.23
500-yr	5,470	2,924	962.0	965.43
0.22 PMF	4,965	3,116	962.0	965.57
0.24 PMF	4,051	3,640	962.0	965.92
0.25 PMF	4,000	3,400	962.0	965.77
0.50 PMF	11,760	11,760	962.0	970.18
PMF	24,522	24,229	962.0	975.46

*HEC-RAS cross section 2823.359

There are multiple low-lying habitable structures, in the 100-year floodplain of Sippo Creek on Tremont Avenue SE that would be subject to some flooding during any Dam failure. These structures should be evacuated during any event that overtops the Dam.

2.5 DAM HAZARD CATEGORY

The following is a summation of the results of the Dam failure analysis as it pertains to the hazard category of the Dam. The total discharge from the dam is a function of the peak flow at the dam, and the resultant increase due to a dam failure. Using the dam failure discharge spreadsheet and the HydroCAD dam failure modeling, it can be shown that failure of the Dam would not be expected to cause a loss of life downstream due to the presence of the Lincoln Way embankment and culvert, and the large storage area of North Sippo Park.

2.5.1 Dam Failure Analysis

A dam failure analysis was performed to determine the increased flows downstream as a result of a failure during various flood events. The dam failure analysis determined that as the peak discharge out of the dam increased, the total discharge at the critical structures only slightly increased as shown on Table 3. As the discharge increased into the Reservoir, the tailwater, created by the Lincoln Way embankment and culvert, submerged the Dam. Because of this occurrence, the difference in headwater of the Reservoir to tailwater in North Sippo Park decreased, which lowered the additional discharge caused by failure of the Dam.

The total flow capacity of the existing primary spillway, without overtopping the left embankment is approximately 600 cfs. The storage capacity of North Sippo Park, downstream of the Dam at this discharge is larger than the volume of the lake during this flood. Therefore, any failure of the Dam during this condition would not threaten the integrity of the Lincoln Way embankment and would only slightly increase flooding to the downstream residences.

During larger floods, prior to the Lincoln Way embankment overtopping, the total volume of the park between the Dam and the Lincoln Way embankment (el. 1108 ft.) is approximately 197 acre-feet and has a surface area of approximately 13.5 acres. By comparison, the Sippo Creek Reservoir has a total volume of approximately 143 acre-feet and a surface area of 21.5 acres at the same elevation (el. 1008 ft.). The difference in volume attenuates large floods and minimizes any dam break flooding in the North Sippo Park. Table 3 shows that flooding downstream of the North Sippo Park only increases slightly due to dam failures during larger floods.

2.5.2 Hazard Classification

The Sippo Creek Reservoir Dam is considered a High Hazard Class I Dam due to the expected loss of life from a failure of the embankment. It was expected that due to the extreme depths of flooding over the Dam, it would fail and cause a large flood wave to travel downstream. As shown by the modeling, the tailwater caused by the Lincoln Way embankment and culvert, and the large storage area of the North Sippo Park, would minimize increases in flooding downstream of the Dam due to its failure. In addition, floods larger than the 0.25 PMF would overtop the Lincoln Way embankment. During the 0.25 PMF, the difference between the tailwater and the headwater over the Dam is only about 1 foot and the increase in flooding due to a failure of the Dam, would be nominal. Any dam failure during floods greater than the 0.25 PMF would not be likely to cause additional damage downstream. In the existing conditions, failure of the Lincoln Way embankment would be worse than the Sippo Creek Reservoir failing during the same flood. In essence, the Lincoln Way embankment minimizes flooding downstream for floods up to the 0.25 PMF, while larger floods would be likely to overtop the embankment, whether the Sippo Reservoir Dam failed, or not.

Due to this condition, it appears that the Sippo Reservoir Dam should be more appropriately classified as a Hazard Class II Dam, whose failure would cause damage to state and interstate highways, and cause floodwater damage to homes, businesses, industrial structures, with no loss of life expected.

3.1 SUMMARY OF ANALYSES

The following is a summary of the URS hydrologic and hydraulic analyses.

3.1.1 Hydrologic Analysis

The drainage area for the Reservoir is 14.8 square miles with an average SCS curve number of 72. There are several large lakes upstream of the Reservoir that store some floodwater and attenuate flooding downstream during smaller storms. However, these lakes and other storage areas do little to attenuate flooding downstream during extreme flooding events.

The results of the URS HydroCAD hydrologic model show that the peak discharge into the Reservoir for the 6-hour PMF is 31,970 cfs and for the 24-hour PMF discharge is 23,172 cfs. The 100-year and the 500-year floods determined by FEMA for the area are 1,950 cfs and 2640 cfs respectively. The twenty-five percent (0.25) PMF discharge is 3,881 cfs and the 50-percent (0.5) PMF discharge into the Reservoir is 11,457 cfs.

It does not appear that the Reservoir can store or pass the 100-percent PMF design flood without severely overtopping. The Sippo Reservoir Dam will overtop during the 100-year flood by approximately 2 feet over the left embankment. However, discharges up to the 0.25 PMF, downstream of the Lincoln Way embankment, are attenuated by the storage in North Sippo Park and metered by the Lincoln Way culvert. Once the Lincoln Way embankment is overtopped during floods larger than the 0.25 PMF, the discharges downstream of the embankment are not attenuated and the structures downstream will be affected by the entire discharge out of the Reservoir.

It appears that a Critical Flood of approximately 4,100 cfs (0.26 PMF) exists for the Dam, as at this discharge the Lincoln Way embankment would be overtopped by over 0.5 feet and possibly fail, whether the Sippo Reservoir Dam failed or not.

Drawings of the any proposed improvements to the Dam will be developed in a Feasibility Report, once the design discharge has been approved by ODNR.

3.1.2 Hydraulic Analysis

There are approximately 40 low-lying habitable structures along Tremont Avenue SE, east of 3rd Street SE, below the Dam and the Lincoln Way embankment. These structures are subject to minor flooding during large floods and to dangerous flooding levels during extreme storms. Although the Sippo Pressure Conduit offers some protection to these structures during floods smaller than the 100-year flood, it does not offer them much protection from larger floods. It appears that the maximum discharge for the Lincoln Way culvert is approximately 3,300 cfs. The maximum discharge of the Sippo Pressure Conduit is approximately 1,300 cfs flowing full and 1,800 cfs under pressure.

The low-lying structures on Tremont Avenue Se will be expected to flood to depths ranging from 3 feet during the 100-year flood up to 16 feet deep during the full PMF. Several of these structures are built along the creek and would also be subject to erosion damage and possible damage from floating objects.

It should be further noted that during large floods, the Lincoln Way embankment backs water over the Sippo Creek Reservoir submerging the Dam. This is due to the embankment being almost 4 feet higher (el. 1008.0 ft.) than the top of the Dam (el. 1004.2 ft.). Once the embankment backs waster into the reservoir, failure of the Dam is less likely to cause damage due to increased discharges downstream.

3.1.3 Dam Failure Analysis

Sippo Creek Reservoir Dam is subject to overtopping during large floods. However, due to the large storage area below the dam, portions of smaller floods are stored in North Sippo Park should the dam fail. In addition, the presence of the Lincoln Way embankment, and culvert, backs up water, which submerges the Dam during large floods. Although large floods would cause flooding to occur downstream prior to any dam failure, the failure of the Dam does not exacerbate the downstream flooding, as in other similar dams of its size and storage capacity.

3.1.4 Dam Hazard Category

The Sippo Creek Reservoir Dam is considered a High Hazard Class I Dam due to the expected loss of life from a failure of the embankment. As shown by the modeling, the tailwater caused by the Lincoln Way embankment and culvert, and the large storage area of the North Sippo Park, would minimize increases in flooding downstream of the Dam due to its failure.

Any failure of the Dam during floods greater than the 0.25 PMF would not be likely to cause additional damage downstream. In the existing conditions, failure of the Lincoln Way embankment would be worse than the Sippo Creek Reservoir Dam failing during the same flood.

The Lincoln Way embankment minimizes flooding downstream for floods up to the 0.25 PMF, while larger floods would be likely to overtop the embankment, whether the Sippo Reservoir Dam failed, or not. Since a failure of the Sippo Creek Reservoir Dam would not cause a probable loss of life, it would be more appropriate to classify it as a Class II High Hazard Dam, whose failure would cause damage to state and interstate highways, and cause floodwater damage to homes, businesses, industrial structures, with no loss of life expected.

This section provides the recommendation by URS for the appropriate hazard category of the Dam and the required design flood.

Currently, the Dam is considered in Hazard Category I, where a loss of life would be expected should the dam fail. However, based on our hydrologic and hydraulic analyses, and our current understanding of the project, due to the submergence of the dam during large floods, a failure of the dam is not likely to cause a loss of life downstream. It is recommended that the current Hazard Classification be revised to a Hazard Category II, where only flooding of structures, and/or damage to major roads would occur due to failure of the dam.

URS determined that the peak inflow discharge of the PMF at the Dam is 31,970 cfs. A storm using 50 percent of the PMP rainfall depth yields a discharge of 11,457 cfs. It is recommended that since failure of the Dam would not result in a probable loss of life, that the design flood be be 11,457 cfs. All calculations and subsequent improvement designs, permits, and the annual fee should be based on the Dam being able to store or pass the 50 percent PMF discharge without uncontrolled overtopping.

In conclusion, URS believes that the appropriate hazard category for the Dam is Hazard Class II and not Class I as it currently categorized. The Dam should have a design flood no greater than the 50 percent PMF. Any proposed improvements should be designed to pass the 50 percent PMF design flood. The current design flood was determined to be 31,970 cfs, and the 50-percent design flood has been determined to be 11,457 cfs.

The conclusions and analyses presented in this report are based on the best available information, and are based on our current understanding of the existing site conditions and the scope of the project, and the existing conditions found by URS at the site and information provided in other reports for the area. This detailed hydrologic and hydraulic analysis includes development of the design flood, hazard category determination and additional engineering as required so that the project can proceed to the design phase.

In the event that changes are made to the site or upstream conditions, the conclusions and recommendations presented herein should not be considered valid, unless URS has reviewed the changes, and incorporated their impact in the analyses provided.

The conclusions and recommendations presented in this report are based on our analysis of the information collected for this project. The recommendations presented in this report should not be used for other projects or purposes. Conclusions or recommendations made from these data by others are their responsibility.

Our services were provided in a manner consistent with the level of care and skill ordinarily exercised by other professional consultants under similar circumstances. No other representation is intended.

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