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Modeling the Impact of Future Climate on Drainage Infrastructures

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MODELING THE IMPACT OF FUTURE CLIMATE ON DRAINAGE INFRASTRUCTURES

BY

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MODELING THE IMPACT OF FUTURE CLIMATE ON DRAINAGE INFRASTRUCTURES

This thesis is approved as a creditable and independent investigation by a candidate for the Master of Engineering degree and is acceptable for meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

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LIST OF ABBREVIATIONS

Abbreviation	**Definition**
AASHTO	American Association of State Highway and Transportation Officials
ADD	Annual-Duration-Depth
ADT	Average Daily Traffic
AutoCAD	Auto Computer-Aided Design
ArcGIS	Arc Geographic Information System
BIA	Bureau of Indian Affairs
cfs	Cubic Feet per Second
CMP	Corrugated Metal Pipe
CN	Curve Number
DDF	Depth-Duration-Frequency
DENR	Department of Environmental and Natural Resources
DEM	Digital Elevation Model
FHWA	Federal Highway Administration
ft	Feet
GPS	Global Positioning System
GeoHMS	Geospatial Hydrologic Modeling System
HDSC	Hydrometeorological Design Studies Center
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
HGL	Hydraulic Grade Line
HUC	Hydrologic Unit Code
HW	Headwater
HYDRO	Office of Hydrology
IDF	Intensity-Duration-Frequency
IPCC	Intergovernmental Panel on Climate Change
LiDAR	Light Detection and Ranging
mi	Miles
MPU	Master Plan Update
n	Manning’s Coefficient

NE	Nebraska
NHD	National Hydrography Dataset
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
NYSDOT	New York State Department of Transportation
OLC	Oglala Lakota College
OSSPEEC	Oglala Lakota-South Dakota State University-South Dakota School of Mines and Technology-PreEngineering Educational Collaborative
OST DOT	Office of the Secretary Department of Transportation
psf	Pounds per Square Foot
Q	Flow
s	Seconds
SCS	Soil Conservation Service
SD	South Dakota
SDDOT	South Dakota Department of Transportation
SDSU	South Dakota State University
SSURGO	Soil Survey Geographic Database
TR-55	Technical Release - 55
TW	Tailwater
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USGS	United States Geological Survey
3D	Three-Dimensional

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ABSTRACT

MODELING THE IMPACT OF FUTURE CLIMATE ON DRAINAGE INFRASTRUCTURES

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Research has shown a potential 20% increase in future heavy and extreme precipitation events over the Midwestern States. Drainage infrastructures designed using current design conditions may not be able to convey projected runoffs resulting in flooding and damage to infrastructure. The objective of this paper is to determine the effects of future climate variability on culvert selections in a southwest South Dakota watershed.

The scope of the study was defined through a comprehensive literature review. Future climate events were based on a 20% increase in current annual precipitation over the Upper White River Subbasin Watershed. A portion of the White River was modeled to obtain simulated current and future peak discharges for a 10, 25, 50, and 100 year return period using ArcGIS and HEC-HMS. A previously washed out 12 foot CMP culvert on BIA-route 32 was analyzed under each specified return period, using HY-8 and Hydraflow Express, to verify culvert performance. This was compared to the capacity of the current 12 foot x 12 foot – side by side – box culvert following the same procedure.

Results indicated the 12 foot CMP culvert was underdesigned for the current 25 year return period; intuitively was also not able to convey the future 25 year return period. The 25 year return period was the main focus of the study because BIA-Route 32 is classified as local and street road (ADT > 100) with a minimum design return period of 25 year precipitation event (SDDOT, 2013). Compared to the 12 foot x 12 foot –side by side

–box culvert which was able to convey the current 25 and 50 year return periods, but was unable to convey the projected future 25 year return period. The 12 foot x 12 foot – side by side – box culvert being able to convey the current but not the future peak discharges was an indication of future climate having a possible effect on culvert design.

1. INTRODUCTION

1.1 Background

"The Earth holds more than 300 million mi^3 of water beneath the land surface, on the surface, and in the atmosphere. This vast amount of water is in constant motion, known as the hydrologic cycle" (Ward and Trimble pp. 4, 2004). One element in the hydrologic cycle is precipitation. Precipitation can take on the form of rain, snow, sleet, or hail as it falls to the Earth (Ward and Trimble pp. 5, 2004). The amount of rain was one parameter of interest for this study. Once the rain falls to the Earth a portion is converted to runoff which flows to tributaries, streams, rivers, ponds, lakes and oceans. How the water flows across the land, into tributaries, and to the rivers can cause problems for civil engineers.

Civil engineers solve problems of a state, city, town, or single person. One problem is how to construct safe roads for travel. A common problem arises when the road must cross a moving body of water like a river. The solution is to design a drainage appurtenances (e.g. bridge or culvert). A drainage appurtenance must be designed to convey a preselected probability rain event. The specific probability rain event is referred to as a return period. Most culvert crossings have to be designed to convey a 10, 25, or 50 year return period based on current rain events.

Drainage appurtenances are designed from old return period data and have the potential of being undersized or underdesigned for future return periods. Research shows a potential increase in annual rain depth over the Midwestern United States. This could result in undersigned culvert crossings on Midwestern roadways.

1.2 Objective and Scope

The objective of this paper is to determine the effects of future climate variability on culvert selections in a southwest South Dakota watershed. Narrowing the scope to an amount of climate increase, type of culvert, and location of watershed was conducted through a literature review. Oglala Lakota-South Dakota State University-South Dakota School of Mines and Technology-PreEngineering Educational Collaborative (OSSPEEC) interests were factored into the ultimate selection of the watershed location.

The scope of this study is limited to a southwest South Dakota watershed. The Upper White River Subbasin Watershed was selected because of its geographical location and OSSPEEC interest in the Pine Ridge Reservation. Figure 1.1 shows the location of the Upper White River Subbasin Watershed in reference to the South Dakota, SD and Nebraska, NE state boundaries. Through a literature review, the amount of climate increase was defined as very heavy and extreme rainfall events for current and future design conditions. Current design conditions came from the National Oceanic and Atmospheric Administration (NOAA) (HDSC webmaster, 2014). Estimated future design conditions came from the studies performed by Karl and Knight (1998); Kunkel, Andsager, and Easterling (1999); and Groisman, et al. (2004). An Upper White River Subbasin Watershed model was created using computer programs. Simulated current and future design condition rain events were also modeled and applied to the watershed model to estimate potential peak discharges of the White River at the culvert. The potential peak discharges were analyzed through Hydraulics-8 and Hydraflow Express on a culvert crossing; which was recently replaced because of structure damage.

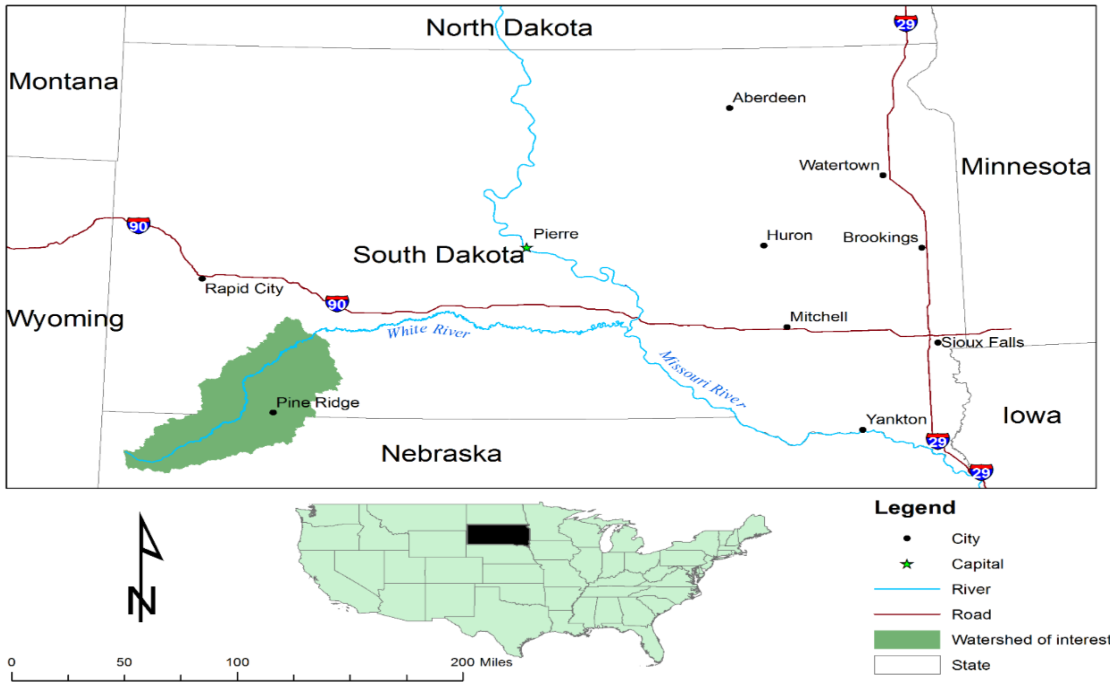


Figure 1.1: Upper White River Subbasin Watershed

2. LITERATURE REVIEW

Chapter 2 consists of an overview of all research which pertains to this paper's objective statement. Topics included are a section on climate: importance of climate, current climate conditions for the project location, and future projecting of climate conditions. Some specific watershed studies which related to how precipitation has concomitantly caused rising of river levels across the United States are discussed. The last section reviewed the effects of raising river levels on current infrastructure and the current design criteria for current infrastructure.

2.1 Climate

Climate impacts how people, animals, and plants live on Earth. Climate consists of historical patterns in temperature, precipitation, humidity, wind, and seasons for different regions across the United States. On a state and global scale, climate has witnessed a change in historical trends over the centuries (Washington State Department of Ecology, 2014). In this report, precipitation would be the only aspect of climate subjected to the watershed study.

Precipitation characteristics of storms including: total precipitation, duration, and intensity are important for understanding the impacts of precipitation on the society and environment (Palecki, Angel, and Hollinger, 2005). The amount of precipitation either large or small affects the amount of runoff, infiltration, and soil erosion; along with potential flooding, agricultural production, and aquifer recharge. All can have a tremendous impact on society's safety and production; as with the environment's quantity and quality of water resources (Santoso, Idinoba, and Imbach, 2008).

2.1.1 Current Design Conditions

Studies of trends in precipitation are usually conducted by government agencies such as: National Oceanic and Atmospheric Administration (NOAA), National Weather Service (NWS), United States Geological Survey (USGS), Department of Environmental and Natural Resources (DENR), United States Department of Agriculture (USDA), and Intergovernmental Panel on Climate Change (IPCC). The data is then analyzed and compiled into tables and atlases which are available to the public (Yarnell, 1935; Hershfield, 1961; Frederick, Myers, and Auciello, 1977). Figure 2.1 shows an Intensity Duration-Frequency (IDF) curve of interpolated rainfall intensity-frequency data analyzed by Yarnell for time durations from 5 to 120 minutes. Figure 2.2 shows a continuation of the same IDF curve as in Figure 2.1, but for time durations from 2 to 24 hours. The raw data used to create the IDF curves can be found in Appendix A.

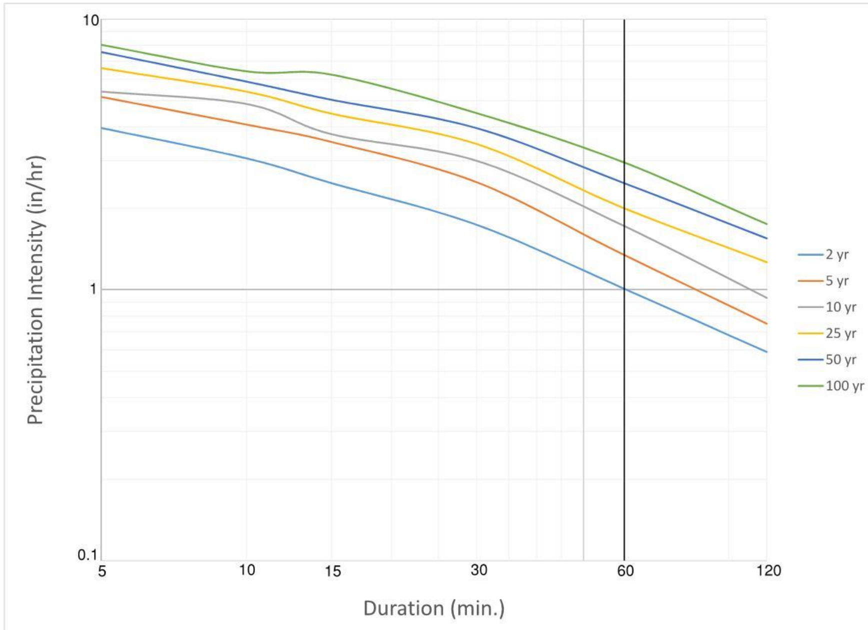


Figure 2.1: IDF curve for storm durations from 5 to 120 minutes (Yarnell, 1935)

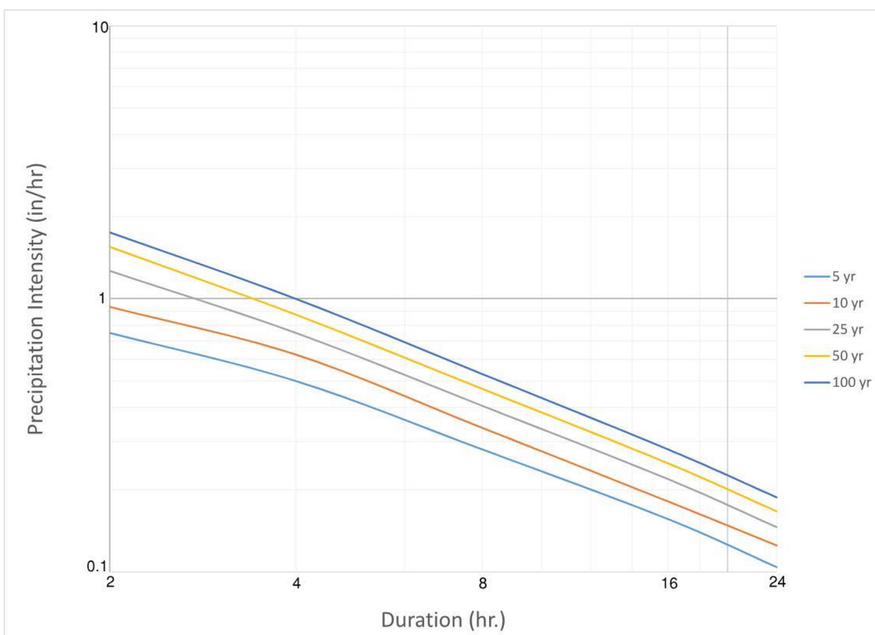


Figure 2.2: IDF curve for storm durations from 2 to 24 hours (Yarnell, 1935)

Figure 2.1 and Figure 2.2 have been used for any economic and engineering design up to the year of 1953 requiring rainfall frequency or intensity data (Hershfield, 1961). Yarnell (1935) classified storms into two different classes; (1) rains of great intensity and short duration, and (2) rains of moderate intensity and long duration. For the purpose of Yarnell's report, he used the first classification due to their destructive nature. Yarnell (1935) produced the intensity-frequency diagrams using 211 automatic rain gages that were ranging from 33-year to 20-year records.

HDSC webmaster (2014) provided precipitation frequency estimated IDF curves for 11 Midwestern states: Colorado, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Oklahoma, South Dakota, and Wisconsin. These atlases would be used in determining design limits of engineered infrastructures or other projects with the potential of being affected by precipitation. Figure 2.3 shows an IDF curve of interpolated rainfall intensity-frequency data analyzed by NOAA's National Weather Service for time durations from 5 to 120 minutes. Figure 2.4 shows a continuation of the same IDF curve as in Figure 2.3, but for time durations from 2 to 24 hours. The raw data used to create the IDF curves can be found in Appendix A.

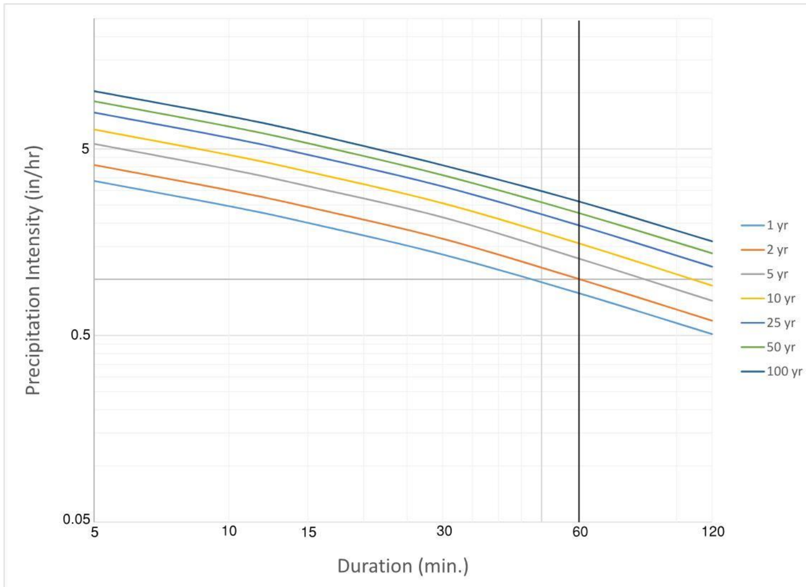


Figure 2.3: IDF curve for storm durations from 5 to 120 minutes (HDSC webmaster, 2014)

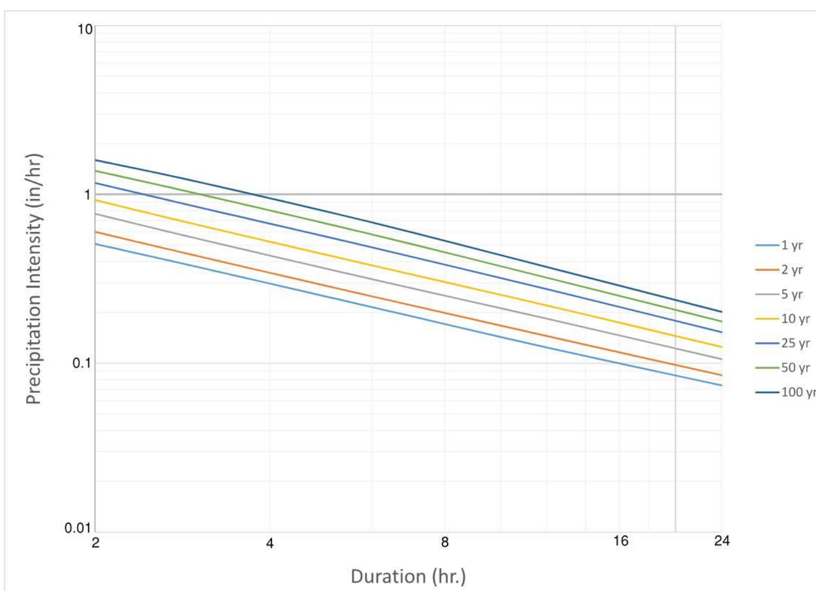


Figure 2.4: IDF curve for storm durations from 2 to 24 hours (HDSC webmaster, 2014)

Figure 2.3 and Figure 2.4 have been used for any economic and engineering design from the years of 1977 to 2015 requiring rainfall frequency or intensity data (SDDOT, 2013). For the years in between 1953 to 1977 different numerical Weather Bureau's technical papers and NOAA technical memorandum NWS HYDRO's were used. The current publications referenced by the South Dakota Drainage Manual under the hydrology chapter, which consist of guidelines to producing IDF curves for a particular location in South Dakota, are Hershfield (1961) and Frederick, Myers, and Auciello (1977).

2.1.2 Future Design Conditions

"Over the contiguous United States, precipitation, temperature, streamflow, heavy and very heavy precipitations have increased during the twentieth century." (Groisman, et al., 2004). Hershfield (1961) was the last national analysis of precipitation events, but large changes in frequency have been occurring since 1960. Such changes suggest a need for updated data. If engineers continued to design infrastructures based on Hershfield (1961) results, the subsequent infrastructures could be drastically underdesigned for future streamflows and runoff (Kunkel, Andsager, and Easterling, 1999). Several studies have documented an increase in precipitation over the past century, but not due to just one factor (Karl and Knight, 1998). Three significant studies have shown a 20% increasing trend of annual precipitation (Karl and Knight, 1998; Kunkel, Andsager, and Easterling, 1999; and Groisman, et al., 2004).

Karl and Knight (1998) credited "a change in precipitation amount as a change in frequency of precipitation events, intensity of precipitation per event, or any combination thereof." The study, on how trends of precipitation has changed or varied, was done from the years of 1948 to 1995 on heavy or extreme daily precipitation events. For the study, "[t]he upper 10 percentile was defined as a very heavy precipitation event." During the 47 years study, a statistically significant linear trend of a 19.5% increase in annual national precipitation was concluded for the very heavy precipitation event.

Kunkel, Andsager, and Easterling (1999) analyzed trends in changes in extreme precipitation events which could have been linked to flooding. "The particular measured of extreme precipitation events were used because a previous study (Changnon and Kunkel 1995) determined a positive correlation between such events and hydrologic flood events on small to medium-sized rivers in the Midwest." The results found could have been relevant to engineering design criteria for infrastructures pertaining to runoff. Extreme precipitation events were analyzed from 1931 to 1996 for nine (9) regions of the contiguous United States. "The trend in total annual precipitation for U.S. climate divisions, base only on the long-term stations used in the study, suggests upward trends of 20% or more in the Southwest, Great Plains, and parts of the upper Mississippi River Valley and Great Lakes Basins."

One aspect Groisman, et al. (2004) studied was annual precipitation with records from 1908 to 2002. The study analyzed heavy, very heavy, and extreme precipitation events in nine (9) regions across the contiguous United States. Precipitation events were based on three (3) percentile ranges: 90^{th}-95^{th}, 99^{th}-99.7^{th}, and 99.9^{th} for the study respectively. A statistically significant trend was defined for the heavy and very heavy precipitation events, but not for the extreme precipitation event. The heavy and very heavy precipitation events had a linear trend of a 14% and 20% increase in annual national precipitation respectively.

2.1.3 Climate Conclusion

Climate impacts how people, animals, and plants live on earth (Washington State Department of Ecology, 2014). The one aspect of climate focused on was precipitation and more specifically heavy and extreme precipitation events. Hershfield (1961) and Frederick, Myers, and Auciello (1977) used the extreme precipitation events to produce IDF curves, which are currently consulted for any hydrologic engineering design. Due to the outdated data, studies have been conducted estimating a 20% increasing trend of annual precipitation (Karl and Knight, 1998; Kunkel, Andsager, and Easterling, 1999; Groisman, et al., 2004). With the projected future precipitation increase, watersheds' characteristics and features can be evaluated for future performance.

2.2 Watershed Study

There are multiple reasons why watershed studies are conducted in the United States. Some of these reasons are to evaluate: climate change on river flow; non-point and/or point source pollution; alternative watershed management practices; water quality of surface and/or ground waters; erosion control and/or perdition; better land use practices; and flood risk management (Migliaccio and Srivastava, 2007; Halmstad, Reza Najafi, and Moradkhani, 2013). For each watershed study, certain watershed characteristics must be analyzed, quantified, and evaluated in order to obtain measureable results. These characteristics include but are not limited to the following spatial and temporal data: topography, land use/cover, soils, rainfall, and flow monitoring data (Chu and Steinman, 2009).

The scope of this paper will focus on only one of the listed watershed studies, evaluating climate change on river flow conditions. The following are some watershed studies that do not pertain to this paper's scope, but are good references: Jha and Gassman (2014); Duliére, Zhang and Salathé Jr. (2013); Tanaka, et al. (2006); Grassotti, et al. (2003); Miller and Friedman (2009); Shultz and Kjelland (2002); and Halmstad, Reza Najafi, and Moradkhani (2013). Two watersheds pertaining to this paper's scope are Acharya, Lamb, and Piechota (2013) and Rosenberg, et al. (2010).

2.2.1 Acharya, Lamb, and Piechota (2013)

Acharya, Lamb, and Piechota (2013) evaluated how an extreme rainfall event could affect an urban watershed in Las Vegas, Nevada. For the purpose of the study, an extreme rainfall event was classified as a 100-year return period storm. "The increase in frequency and intensity of extreme rainfall events may cause serious impacts on both natural and engineered systems in terms of increased frequency and severity of floods." The U.S. Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling Software (HEC-HMS) and 2008 Flood Control Master Plan Update (MPU) were analyzed to determine how extreme rainfall events could have impacted future stormwater management. The data period for the study consisted of average monthly temperature and precipitation from 1950 to 2099. During these periods, precipitation patterns and streamflow projections increases and decreases were analyzed. Acharya, Lamb, and Piechota (2013) concluded that future extreme storms are going to be more intense, resulting in an increase in peak streamflow and total runoff volume. The predicted peak streamflow for extreme rainfall events would assist in evaluating existing flood control facilities.

2.2.2 Rosenberg, et al. (2010)

Rosenberg, et al. (2010) conducted two small urban watershed studies in Washington State, to determine the effect runoff has on streams and drainage infrastructures. Future precipitation predictions used to calculate the runoff came from reports done by Karl and Knight (1998) and Kunkel, Andsager, and Easterling (1999). The watershed studies focused on two ranges of return periods; 2- to 5-year return periods (roadside swales, gutters, and sewers) and 50- to 100-year return periods (flood control structures) for urban areas. Rosenberg, et al. (2010) concluded drainage infrastructures designed in the mid-20th century rainfall data could be subjected to rain events unlike current design standards.

2.2.3 Watershed Study Conclusion

There are multiple reasons why watershed studies are conducted in the United States. The scope of this paper will focus on only one of the listed watershed studies, evaluating climate change on streamflow conditions. Two different watersheds which do pertain to this paper's scope are Acharya, Lamb, and Piechota (2013) and Rosenberg, et al. (2010). Acharya, Lamb, and Piechota (2013) concluded future extreme storms are going to be more intense resulting in an increase in peak streamflow and total runoff volume. The predicted peak streamflow for extreme rainfall events would help to evaluate existing flood control facilities. Rosenberg, et al. (2010) concluded that drainage infrastructures designed in the mid-20th century rainfall data could be subjected to rain events different from current design standards. Both watershed studies emphasized with an increase in extreme rainfall events, there could be a destructive impact on current infrastructures.

2.3 Infrastructure

Drainage infrastructures are defined, for this paper, as any structure which transvers or allows water to move from one side of the road to the other side. (NYSDOT, 2013; SDDOT, 2013) For more information on standards of drainage infrastructure, the New York State DOT suggested the following agencies websites: the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), the U.S. Army Corps of Engineers (USACE), the National Resource Conservation Service (NRCS), and the U.S. Geological Survey (USGS) (NYSDOT, 2013). In this paper, drainage infrastructure criteria for South Dakota with govern any design criteria.

2.3.1 SDDOT

SDDOT (2013) defined infrastructures as drainage appurtenances, which included: bridge waterway openings, roadway cross culverts, storm drainage systems and roadside ditches. Table 2.1 consists of a table from the South Dakota Drainage Manual which was used for sizing drainage appurtenances based on the return period of a precipitation event. Referring back to the scope of the paper, roadway cross culverts was the only drainage appurtenance tested.

Table 2.1: Design return periods for drainage appurtenances in South Dakota modified from the SDDOT (2013), for more information see Appendix B

Highway Classifications	Return Period (Years) for Drainage Appurtenances					
	Bridge Waterway Openings		Roadway Cross Culverts	Storm Drainage Systems	Roadside Ditches	
	Design Headwater	Scour	Design Headwater	Inlet Spacing & Trunk Line	Design Headwater	Permanent Erosion Protection
Interstate	50	100	50	10	50	50
US & State Highways	25	100	25	10	25	25
Local Roads & Streets (ADT > 100)	25	100	25	10	25	25
Local Roads & Streets (ADT < 100)	10	100	10	10	10	10

Note: ADT = average daily traffic

As stated by Kunkel, Andsager, and Easterling (1999) if engineers continued to design infrastructures based on Hershfield (1961) results, those infrastructures could be drastically underdesigned for future streamflows and runoff. Two examples of underdesigned infrastructures, based on Hershfield (1961), are two separate culvert washouts. The first culvert washout resulted in a three year detour for the citizens while the road was under construction on the Pine Ridge Reservation. The second culvert washout resulted in two deaths on the Lower Brule Reservation.

2.3.2 Pine Ridge Reservation

On the Pine Ridge Reservation, a 12 foot corrugated metal pipe (CMP) washed out due to a flash flood back in Spring of 2010. The 20 year old CMP was located on BIA Route-32 in the Slim Buttes area west of Pine Ridge, SD. Due to the washout, an alternate route was put in place which lasted for roughly three years (Testerman, 2010-2012; Crash, 2011). The Oglala Sioux Tribe Department of Transportation (OST DOT) installed a new 12 foot x 12 foot – side by side - box culvert in the Spring on 2013 (Office of Federal Lands Highway, 2013; SDDOT, 2011). Figure 2.5 shows a modified map produced by Joanita Kant of the Pine Ridge Reservation showing the washed out culvert on BIA Route-32.

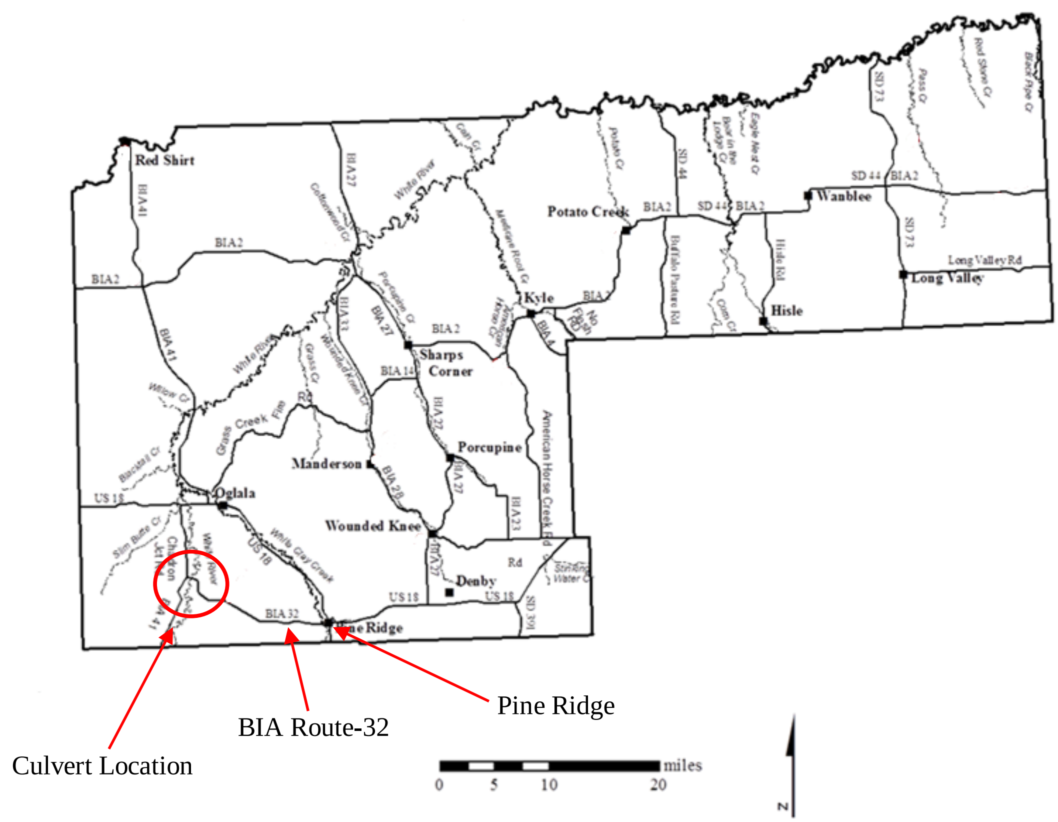


Figure 2.5: Modified map of Pine Ridge Reservation in South Dakota, originally produced by Joanita Kant

2.3.3 Lower Brule Reservation

Halverson (2011) reported on a flash flood which caused a culvert to washout on June 21, 2011. This incident took place near the Lower Brule Reservation, an Indian reservation to the northeast of the Pine Ridge Reservation. Both the Pine Ridge and Lower Brule Reservation washouts happened under similar conditions. Halverson (2011) article:

> *Creeks and dams overflowing from three days of heavy rain caused flash flooding throughout Lyman County last Tuesday and Wednesday. Two people died Tuesday morning, June 21 around 10:00 AM when their vehicles drove into a washout on BIA 10, also known as 329th Street, just north of the intersection with SD Highway 47 about 9.9 miles north of Reliance.Gwen L. Michalek, a 56-year-old Chamberlain woman was found in her car about 100 yards downstream from the crash site. The other victim, Ellen E. Wright, age 61, of Lower Brule, who was driving a van was found about four miles downstream from the crash site. Neither victim was wearing a seatbelt. Heavy rainfall washed out a culvert taking a section of the roadway with it leaving a 30 foot deep gap in the road The vehicles separately drove into the washout and were carried downstream in the rushing water. Lyman County Sheriff, rescue, and ambulance along with Presho, Reliance, and Chamberlain fire departments and Lower Brule law enforcement, fire and ambulance responded to the accident site and assisted in the research and recovery. The South Dakota Highway Patrol is investigating the accident. Businesses, farms and residences along the creek from Vivian to Oacoma were inundated by flood waters. Lyman County Sheriff and Emergency Management coordinator Steve Manger said that in addition to heavy rain showers, a dam north of Draper reportedly washed out and helped fuel the flooding of Stony Butte and Medicine Creek. See this week's issue of the Lyman County Herald for more story.*

As the article reads, two separate deaths happened on the BIA Route-10 culvert washout. The flood waters created a “30 foot deep gap in the road” where the culvert used to be. Unlike the BIA Route-10 culvert washout, BIA Route-32 culvert washout did not result in any deaths of residents. There are similarities between the BIA Route-10 and BIA Route-32 culvert washouts; both were caused by heavy precipitation events and were designed from past IDF curves.

2.3.4 Infrastructure Conclusion

South Dakota Department of Transportation are designing infrastructures from data analyzed by Hershfield (1961) and Frederick, Myers, and Auciello (1977). Kunkel, Andsager, and Easterling (1999) stated if engineers continued to design infrastructures based on Hershfield (1961) results, those subsequent infrastructures could be drastically underdesigned for future streamflows and runoff. The BIA Route-32 and BIA Route-10 culverts were examples of underdesigned infrastructures in South Dakota. With updated design criteria on drainage appurtenances the deadly washout on BIA-Route 10 could have been prevented.

3. DATA & PROCESSING

Chapter 3 consists of an overview of data and processes used to model the Upper White River Subbasin Watershed. A flowchart outlining the sections covered in this chapter is presented in Figure 3.1. All data used to develop and analyze the watershed model is provided with data sources if required. Refer to Baumbach (2015) for the detailed procedure for how the watershed was developed and analyzed. The culverts were analyzed using Hydraflow and HY-8. The last section covers the SDDOT guidelines used to design a culvert.

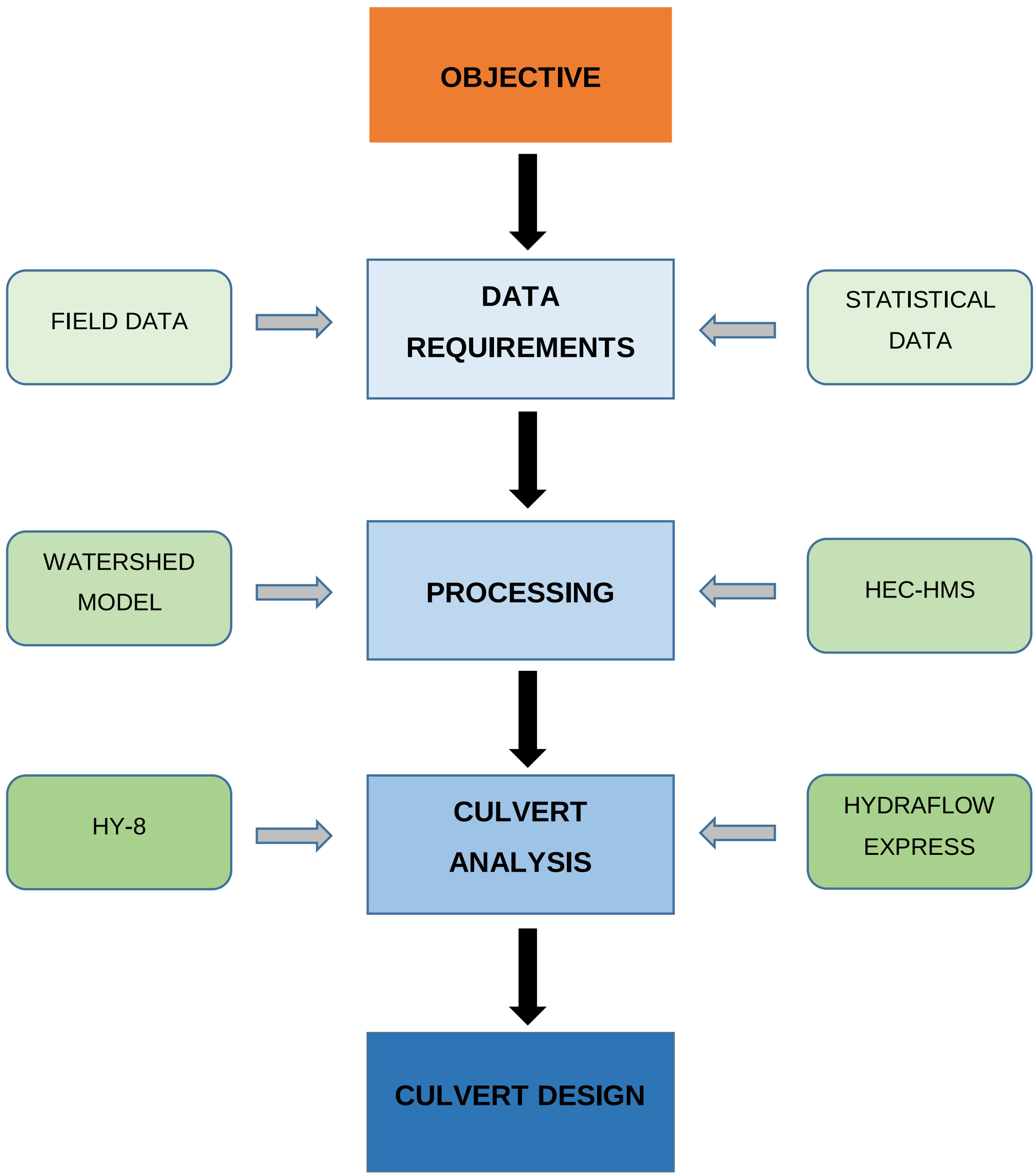


Figure 3.1: Flow chart of data & processing steps for analyzing a culvert design

3.1 Objective

When starting a project the objective(s) should be realistic and clearly stated. Most projects have general guidelines and limitations which must addressed before any data collecting and processing can be performed. Some general guidelines and limitations are but not limited to the needs/wants of the client, timeline, budget, funding, location, procedures, and documentation. The Upper White River Subbasin Watershed study contains some of these guidelines and limitations. The following guidelines and limitations were considered:

- Client's wants/needs

  Client – Oglala Lakota College

  Wants/Needs – A study performed to analyze the 100 year flood level for the Pine Ridge Indian Reservation.

- Timeline

  The timeline was based on an average graduate student's master program of one and a half years. This gave adequate time for researching, program training, data collection, analyzing results, and report construction.

- Budget/Funding

  Funding was provided by the NSF OSSPEEC grant to cover the cost of labor.

- Location

  The Pine Ridge Indiana Reservation in South Dakota was selected for the study area because Oglala Lakota College's (OLC) has a strong connection with the people and land. Oglala Lakota College conducted a number of research projects for the reservation on soil conditions, water quality, plant and animal life along with construction projects. OLC conducted a research project which entailed installing stream gauges at a number of location on or near the White River. These stream gauges had the potential of continuously gathering tributary stream level data, but only one stream gauge data was sent to be analyzed. Stream level data was continuously collected on a 15 minute time interval for the months June through October of 2013.

Unfortunately, the data for the particular stream gauge was not able to be utilized for the Upper White Water Subbasin Watershed study. The decision to abandon this particular dataset for the study was because (1) small sample size (2) small tributary stream (3) secluded location (4) low to no potential impact on infrastructure and (5) limited statistical data available. For these reasons a river crossing on the White River was selected. The BIA Route 32 12 foot CMP culvert washout in the Slim Buttes area west of Pine Ridge on the Pine Ridge Reservation was chosen for the study; refer to Figure 2.5 back in section 2.3.2 for the location map.

- Procedures

Watershed models are analyzed with the aid of computer programs specially designed to handle large amounts of spatial datasets. Standard processing procedures were created for these computer programs and can be usually be downloaded or purchased with the programs. The Upper White Water Subbasin Watershed was developed and analyzed using ArcGIS and HEC-HMS software. The watershed was both developed and analyzed by following standard processing procedures. Details on standard processing procedures used can be found in section 3.4.

- Documentation

  Documentation can entail but not be limited to government reports, state and/or federal grants, past studies, instrument data logs, and legal forms. Other forms of documentation are proposals, memos, or journal articles pertaining to either the study area, standard processing procedures, laboratory trials, or other significant information. Taken into consideration with the budget, there may be fees associated with obtaining some of these forms of documentation.

  In the case of the Upper White River Subbasin Watershed documentation for research was obtained through SDSU's databases. A majority of the documentation research was journal articles, user manuals, past studies, and books on computer programs. Other documentation was accessed through the appropriate government websites.

3.2 Data Requirements

Data specifications should be thoroughly reviewed and stated. General specifications include but are not limited to data: format, resolution, use, quality, accuracy, and price (e.g. time to survey, database fee, etc.). The user's discretion is used to select acceptable data in the dataset. This entails, but is not limited to, considering data storage size, resolution, and accuracy; and timeline of the overall project.

3.2.1 Field Data

In this paper, field data consists of data which was or could be physically reordered. This would include but is not limited to GPS topographical surveys, LiDAR, stream gauges, and precipitation gauges. GPS topographical surveys and LiDAR data consist of geographical coordinate points. Each point has an X, Y, and Z coordinate which represent latitude, longitude, and elevation respectively. The points are used to create surfaces in the model. Stream gauges measure the depth or level of the flowing water. Combining the topographical survey of the cross section of the stream and water depth the stream's flow and velocity can be computed. Precipitation gauges measure the amount of precipitation, usually rain, on a specific area of land. The stream level is directly affected by precipitation, soil classification, topography, land cover, and temperature.

The Upper White River Subbasin Watershed study used LiDAR and precipitation databases to develop models. Table 3.1 consists of the forms of LiDAR data and location of the datasets. The LiDAR is stored as DEM files and is formatted for a variety of software programs. National hydrography dataset (NHD) is a form of LiDAR that consists of streams, rivers, and lakes for a particular region. This data was used to process the White River Subbasin Watershed model which will be covered in a later section.

Table 3.1: Field data sources (USGS, 2014)

Data Type	Data Source	Data Sets
DEM (ArcGIS)	USGS TNM 2.0 Viewer	Subbasin 10140201 Subbasin 10140202 Subbasin 10140203 Subbasin 10140204
NHD (ArcGIS)	USGS TNM 2.0 Viewer	Subbasin 10140201 Subbasin 10140202 Subbasin 10140203 Subbasin 10140204

The current design precipitation data was obtained from NOAA Atlas 14, Volume 8, Version 2 for the City of Pine Ridge, SD (HDSC webmaster, 2014). The City of Pine Ridge, SD was chosen because of its geographical proximity to the culvert of interest. Return periods were selected based on the SDDOT return periods for roadways referenced in section 2.3.1. BIA-Route 32 which crosses over the culvert of interest was assumed to be classified as a local road or street (ADT > 100) with a return period of 25 years. This assumption is consistent with U.S. and state highway culverts are designed using an identical return period. Current design precipitation depths in inches for a particular intensity duration expected for each return period can be found in Table 3.2. Refer back to section 2.3.1 for more information on current design conditions.

Table 3.2: Current design annual precipitation depth for the Pine Ridge area

Duration	Return Period			
	10 Years	25 Year	50 Years	100 Years
5 Minutes	0.529	0.658	0.759	0.863
15 Minutes	0.945	1.18	1.36	1.54
60 Minutes	1.54	1.91	2.20	2.51

Note: Precipitation depth are recorded as an average for the area in inches, full table in Appendix C. (HDSC webmaster, 2014)

The current design conditions have been concluded to be outdated for future culvert designs. Due to the outdated data, studies have been conducted estimating a 20% increasing trend of annual precipitation (Karl and Knight, 1998; Kunkel, Andsager, and Easterling, 1999; Groisman, et al., 2004). For full details on these articles refer back to section 2.1.2. Table 3.3 consists of the estimated future design precipitation depths in inches for a particular intensity duration expected for each return period. Future design precipitations were calculated by increasing the current design precipitations depths by 20%.

Table 3.3: Future design annual precipitation depth for the Pine Ridge area

Duration	Return Period			
	10 Years	25 Year	50 Years	100 Years
5 Minutes	0.635	0.790	0.911	1.04
15 Minutes	1.13	1.42	1.63	1.85
60 Minutes	1.85	2.29	2.64	3.01

Note: Precipitation depth are recorded as an average for the area in inches.

No GPS topographical survey or stream gauge data was used in the Upper White River Subbasin Watershed study. The reason for not using GPS topographical surveys was because the area was roughly 1,426 mi^2 (912,640 acres). A GPS topographical survey could have been done upstream and downstream of the culvert for more accurate elevations. The reason for not using stream gauge data was because there were no stream gauges within reasonable distance of the culvert. If stream gauge data was used an average baseflow conditions could have been gathered for later processes.

3.2.2 Statistical Data

In this paper, statistical data consists of data interpolated from scarce samples gathered in the field or from history data. This would include but not limited to soil classification, land cover and use, curve number (CN) values, and StreamStats. Soils are classified by their components of water storage, flooding frequency, hydrologic group, ratings for building applications, and erosion hazard Esri, 2014). The type of land cover and use can range from impervious urban areas to grassland for ranchers and crop land for farmers to mining for minerals. The hydrologic group of the soil and main land use must be estimated for the region to select an appropriate CN value. The CN value is a dimensionless numerical value associated with how much of the rain will be converted to runoff. If more than one region is within the boundaries of the study area a weighted area average must be calculated. A weighted area average was taking for the Upper White River Subbasin Watershed model as shown in Table 3.4. Refer to Baumbach (2015) for the produce on how to calculate each subbasins' CN value.

Table 3.4: CN values for the Upper White River Subbasin Watershed model

Region	HEC-HMS Subbasin	Weighted Curve Numbers	Region	HEC-HMS Subbasin	Weighted Curve Numbers
1	W260	84	14	W390	74
2	W270	87	15	W400	87
3	W280	81	16	W410	84
4	W290	82	17	W420	84
5	W300	78	18	W430	81
6	W310	88	19	W440	84
7	W320	76	20	W450	82
8	W330	88	21	W460	84
9	W340	80	22	W470	87
10	W350	88	23	W480	84
11	W360	85	24	W490	77
12	W370	71	25	W500	81
13	W380	85			

Note: Region represents the label used in ArcGIS. Land use, hydrologic condition, and curve numbers (A, B, C, and D) were kept the same for all subbasins; herbaceous, fair, and 62, 71, 81, and 89 respectively.

StreamStats is a Web-based Geographic Information Systems (GIS) which makes processing un-gauged streams and rivers statistics faster, more accurate and consistent than past methods (USGS, 2015). Statistical data from StreamStats for peak stream flows were not used to verify simulated river levels because the majority of the Upper White River Subbasin Watershed Model presides in Nebraska. StreamStats did not have un-gauged river levels calculated for Nebraska and using South Dakota would result in inaccurate comparison river levels. Therefore, regional regression equations from "Nationwide Summary of U.S. Geological Survey Regional Regression Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites, 1993" were used to verify the calculated peak flows (USGS pp. 104-109, 1994). Table 3.5 compares the regional regression equations peak discharges to the simulated river peak discharges for each return year.

There are assumptions made and parameters which cannot be accurately accounted for in both the regional regression equations and simulated river levels. Most of the error comes from the sheer size of the study area and estimating the amount of precipitation that occurs during the return year. The regional regression equations used to estimate the Upper White River Subbasin Watershed Model river level upper limits had a standard error of 98 – 102 % (USGS pp. 104, 1994). Equations correlated with Region 1 were used and solutions are found in Appendix D.

Region 1:

$$Q10 = 67.19Ac^{0.737}(P-13)^{1.149}L^{-0.608}$$

$$Q25 = 222.93Ac^{0.690}(P-13)^{0.905}L^{-0.573}$$

$$Q50 = 490.86Ac^{0.656}(P-13)^{0.742}L^{-0.543}$$

$$Q100 = 996.78Ac^{0.624}(P-13)^{0.588}L^{-0.512}$$

Variables:

Q = Estimated peak discharge at given return period

Ac = contributing drainage area, mi^2

P = mean annual precipitation, inches

L = main stream length, mi

Table 3.5: Region 1 regression equation vs simulated current peak discharges

Return Period (Years)	Regression Equation (CFS)	Simulated (CFS)	Percent Error
10	2,960	2,265	23%
25	5,980	3,595	40%
50	9,589	4,748	50%
100	14,645	6,063	59%

Note: Regression equation peak discharges are only for current design precipitation values.

Results from the simulations are lower than estimated peak discharges calculated using regional regression equations. The comparison provides evidence of a correlation between predicting a lager return period and degree of uncertain that corresponds to the prediction. This illustrates the assumptions made for the Upper White River Subbasin Watershed Model are reasonable.

3.3 Processing

Processing entailed using raw data to develop a working watershed model using computer programs. The two computer programs used in processing the raw data were ArcGIS and HEC-HMS. ArcGIS was used to create the working watershed model by utilizing terrain preprocessing and GeoHMS functions. The outputs from ArcGIS were converted to HEC-HMS compatible format before being uploaded into HEC-HMS. In HEC-HMS meteorological models were developed and applied to the watershed resulting in unit hydrographs of estimated river levels under specific precipitation conditions. Each unit hydrograph peak flow was extracted from a parameter used to analyze the 12 foot CMP culvert. For the complete processing procedure of the Upper White River Subbasin Watershed study refer to “Watershed Modeling Using Arc Hydro Tools, GeoHMS, and HEC-HMS" (Baumbach, 2015). The following sections are an overview of the paper referenced and what data was used.

3.3.1 Watershed Model

The Upper White River Subbasin Watershed was built with the raw field data, but first the raw data was processed into a working watershed model. Terrain preprocessing and GeoHMS functions in ArcGIS were used to develop the working watershed model. Terrain preprocessing involves defining the surface and river channels of the watershed. This was done by reconditioning the DEM and NHD data files and filling in sinks or low spots from the recondition process. From reconditioning flow direction and accumulation; stream definition and segmentation; and subbasin delineation layers were determined. The determination of stream segments and subbasin allowed for drainage processing to calculate drainage points for each stream and subbasin. A slope calculating process was applied to the working watershed to develop a slope layer for future applications. All processes had to be sequentially completed before applying the GeoHMS function to further develop the working watershed model. Figure 3.2 is a screen shot of the Upper White River Subbasin Watershed after terrain preprocessing was applied. The shaded region is the Upper White River Subbasin Watershed overlaid with a USA counties map.

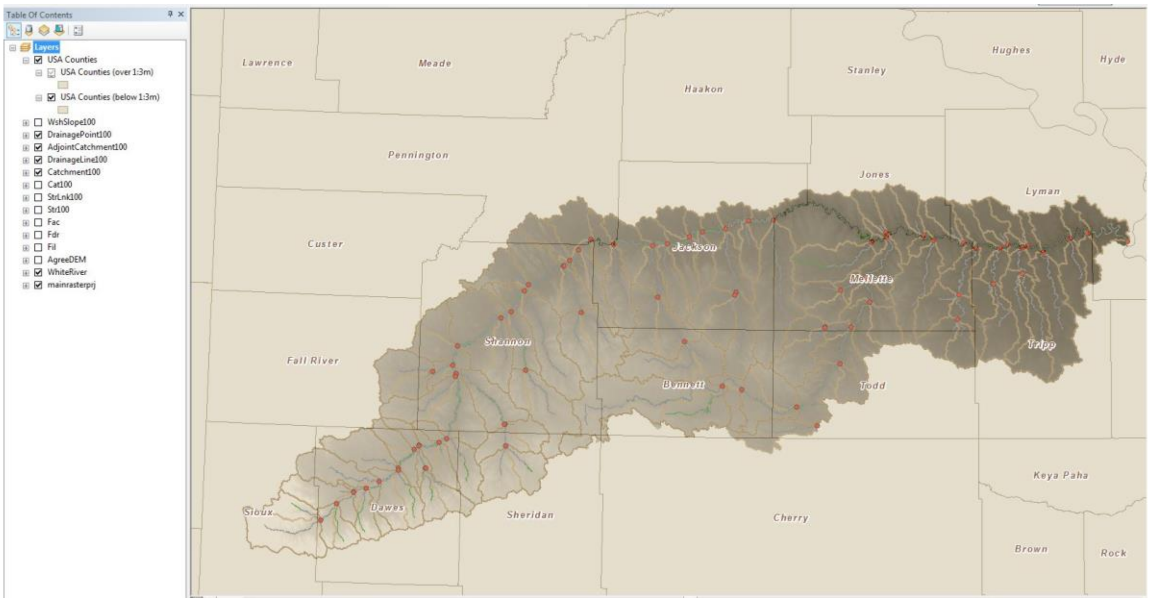


Figure 3.2: Upper White River Subbasin Watershed after terrain preprocessing

Legend:

- Blue Lines = river or stream
- Tan Lines = subbasin boundaries
- Red Dotes = drainage points

HEC-GeoHMS is the intermediate step between terrain preprocessing watershed model and a HEC-HMS project. "HEC-GeoHMS can be used to refine the subbasin and stream delineations, extract physical characteristics of subbasins and streams, estimate model parameters, and prepare input files for HEC-HMS" (US Army Corps of Engineers(a), 2013). First, a project area (Upper White River Subbasin Watershed portion upstream of the culvert) and point (12 foot CMP culvert location) was selected from the terrain preprocessed inputs. Second, modifying subbasin and river delineations were not performed on project area. Modified delineations would include merging or dividing existing subbasins or rivers. Third, physical characteristics calculated for the river were length, slope, and longest flow path. Subbasin physical characteristics included slope, subbasin centroid, and centroidal flow path. Fourth, model parameters came from statistical data which include CN values and TR-55 time of concentration estimates. Fifth, files from HEC-GeoHMS were converted into compatible HEC-HMS files. These include the basin model, background shapefile, unit conversions, and the optional meteorological model. Figure 3.3 is a screen shot of the basin model output file converted into HEC-HMS units. The meteorological models were chosen to be created in HEC-HMS and will be covered in a later section.

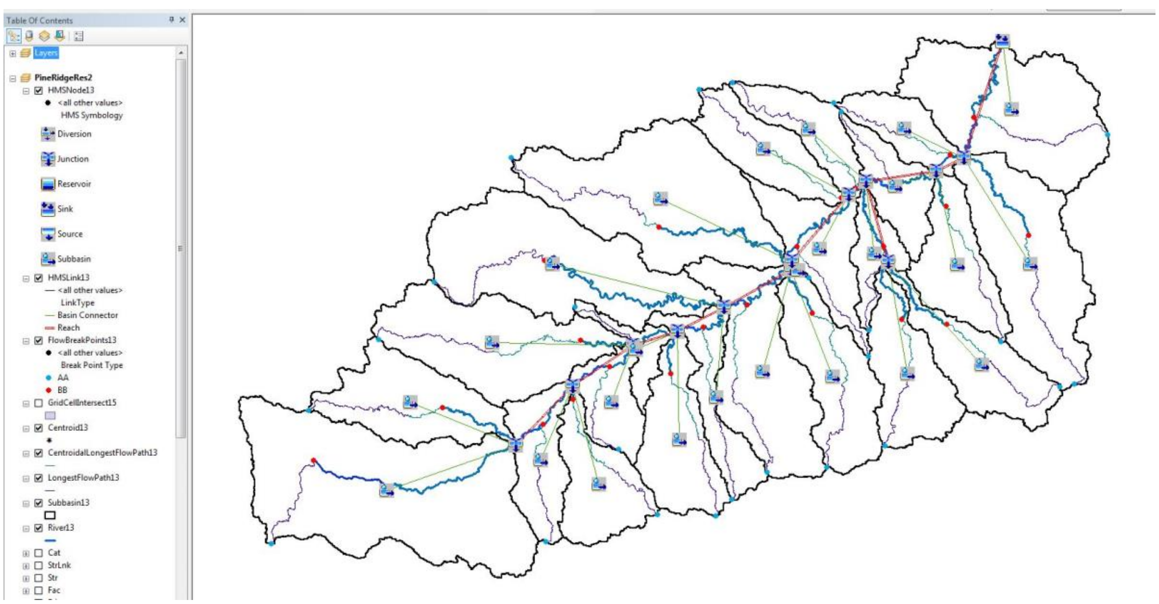


Figure 3.3: Upper White River Subbasin Watershed schematic with HEC-HMS symbols

3.3.2 Hydrologic Engineering Center–Hydrologic Modeling System

"The Hydrologic Modeling System is designed to simulate the precipitation-runoff processes of dendritic watershed systems" (US Army Corps of Engineers(b), 2013). The HEC-HMS was selected because it could be used in conjunction with ArcGIS for analyzing a watersheds wide range of parameters. The parameters include but are not limited to water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation (US Army Corps of Engineers(b), 2013). Flow forecasting was the parameter focused on for the Upper White River Subbasin Watershed study.

A HEC-HMS model has six components categorizing the data in the project; (1) Basin Models (2) Meteorological Models (3) Control Specifications (4) Time-Series Data (5) Paired Data (6) Grid Data. This study utilized components (1) through (3) due to the data available for the study area. As stated earlier ArcGIS was used to create the basin model and uploaded into HEC-HMS. Further parameters had to be defined for the subbasin, sink, junction, and reach elements which made up the basin model. Some parameters were left blank or set to a default setting assigned by HEC-HMS. Multiple meteorological models were created using Table 3.2 and Table 3.3 precipitation depth enters. Frequency storm was set as the type of precipitation and U.S. Customary for the unit system. The control specifications purpose was to set the model's time limits. The Upper White River Subbasin Watershed model control specifications were arbitrary dates and times. The only stipulation was the time period needed to contain all of the runoff effects on the river. Results from each trial will be discussed in Chapter 4.

(1) Basin Model

Subbasin: (Labeled with a W followed by an identification number)

- Canopy Method – None
- Surface Method – None
- Loss Method – SCS Curve Number
    - Initial Abstraction (IN) – Blank
    - Curve Number – Refer to Table 3.4
    - Impervious (%) – 0.0
- Transform Method – SCS Unit Hydrograph
    - Graph Type – Standard
    - Lag Time (MIN) – Lag time, T_L was manually computed using subbasin time of travel from the TR-55 Excel file (Baumbach, 2015). "Studies by the SCS found that in general the lag time can be approximated by taking 60% of the time of concentration" (US Army Corps of Engineers(b), 2013). Refer to Table 3.6 for enters.
- Baseflow Method – None

Table 3.6: Time of concentration and lag time for each subbasin

Watershed	T_c (hr)	T_L (hr)	T_L (min)	Watershed	T_c (hr)	T_L (hr)	T_L (min)
W260	24.20	14.52	871	W390	20.51	12.31	738
W270	15.78	9.47	568	W400	16.96	10.18	611
W280	21.34	12.80	768	W410	21.69	13.01	781
W290	14.59	8.75	525	W420	16.73	10.04	602
W300	12.90	7.74	464	W430	10.41	6.25	375
W310	22.58	13.55	813	W440	13.50	8.10	486
W320	10.42	6.25	375	W450	21.64	12.98	779
W330	21.44	12.86	772	W460	7.95	4.77	286
W340	15.48	9.29	557	W470	10.34	6.20	372
W350	36.52	21.91	1315	W480	12.15	7.29	437
W360	32.18	19.31	1158	W490	16.36	9.82	589
W370	1.78	1.07	64	W500	17.96	10.78	647
W380	14.19	8.51	511				

Note: T_C = Time of concentration
T_L = Lag time

Sink: (Culvert)

The sink is represented by the project point which was the 12 foot CMP culvert location. All enters were left blank or set to default.

Junction: (Labeled with a J followed by an identification number)

The junction represents the location where all subbasin runoff outlets into the river. All enters were left blank or set to default.

Reach: (Labeled with a R followed by an identification number)

- Routing Method – Lag routing; this had to be calculated for each reach. Lag time was calculated using equation 3.43 and 3.45 (McCuen pp. 148, 2005) and Table 5.4 for the *k* function of the landcover (Ward and Trimble pp.138, 2004). Refer to Table 3.7 for enters.
- Loss/Gain Method – None

Table 3.7: Lag routing variables and lag time for each reach

Reach	Slope (ft/ft)	Velocity, V (ft/s)	Length, L (ft)	Lag Time, T_L (min)
R10	0.035	0.94	87319	1548
R20	0.002	0.22	17850	1352
R30	0.001	0.16	51784	5394
R40	0.002	0.22	13249	1004
R80	0.001	0.16	55181	5784
R90	0.003	0.27	45589	2814
R110	0.003	0.27	3273	202
R120	0.001	0.16	58533	6097
R170	0.002	0.22	40281	3052
R190	0.003	0.27	26505	1636
R210	0.004	0.32	38974	2030
R240	0.004	0.32	40403	2104

NOTE: Assumed river bed was composed mainly of sand and gravel, therefore, k = 5

$$T_L = \frac{L}{60V} \qquad Equation\ (3.43)$$

$$V = kS^{0.5} \qquad Equation\ (3.46)$$

<u>Varaibales:</u>

L = Length of flow (Appendix E Table 7.4), ft

V = Velocity, ft/s

k = function of landcover with the effect measeare by the value Manning's number and hydrologic radius

S = Slope (Appendix E Table 7.4), ft/ft

(2) Meteorological Models

There was a current and future design meteorological model created and applied to the basin model. Each meteorological model had four (4) different return periods tested with three (3) intensity duration; totaling 12 trials per meteorological model. Table 3.8 and Table 3.9 list the combinations of return period and intensity durations analyzed. Note: each trial is color coded based on the intensity duration for each return period.

Frequency Storm:

- Probability – Varies; refer to Table 3.8 or Table 3.9
- Input Type – Annual Duration
- Output Type – Annual Duration
- Intesnity Duration – Varies; refer to Table 3.8 or Table 3.9
- Storm Duration – 1 hour (assumption made for an average storm)
- Intensity Position – 50% (default)
- Storm Area – Left blank (will use the whole watershed)
- Curve – Uniform For All Subbasins

Table 3.8: Current design annual precipitation depth meteorological models

Probability (%)	Intensity Duration (min)	Duration (min)	Annual-Duration Depth (in)
1	5	5	0.863
		15	1.54
		60	2.51
	15	15	1.54
		60	2.51
	60	60	2.51
2	5	5	0.759
		15	1.36
		60	2.20
	15	15	1.36
		60	2.20
	60	60	2.20
4	5	5	0.658
		15	1.18
		60	1.91
	15	15	1.18
		60	1.91
	60	60	1.91
10	5	5	0.529
		15	0.945
		60	1.54
	15	15	0.945
		60	1.54
	60	60	1.54

Table 3.9: Future design annual precipitation depth meteorological models

Probability (%)	Intensity Duration (min)	Duration (min)	Annual-Duration Depth (in)
1	5	5	1.04
		15	1.85
		60	3.01
	15	15	1.85
		60	3.01
	60	60	3.01
2	5	5	0.911
		15	1.63
		60	2.64
	15	15	1.63
		60	2.64
	60	60	2.64
4	5	5	0.790
		15	1.42
		60	2.29
	15	15	1.42
		60	2.29
	60	60	2.29
10	5	5	0.635
		15	1.13
		60	1.85
	15	15	1.13
		60	1.85
	60	60	1.85

(3) Control Specifications

Figure 3.4 is a screen shot of the control specifications used for each of the 24 trials. Iterations were ran to select the time limit for the control specifications. The date and time were arbitrarily selected because the meteorological models were theoretical values with no comparable storms time intervals.

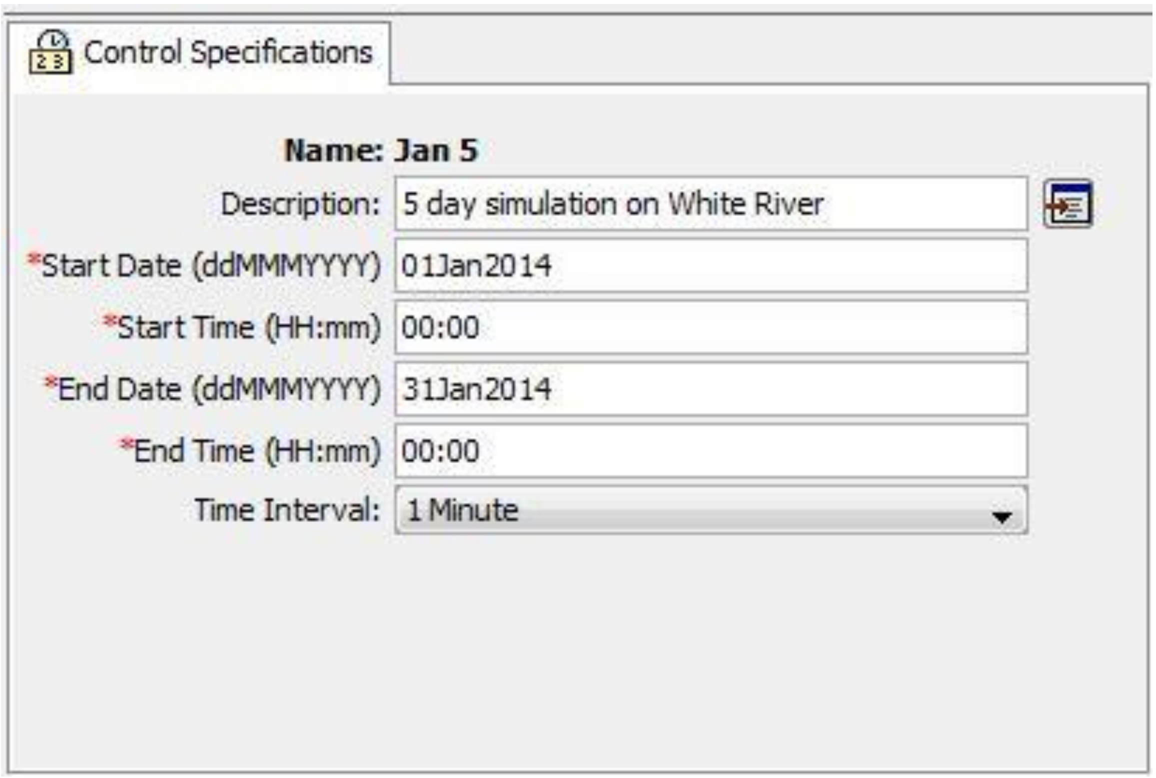


Figure 3.4: Control Specifications set at one month limit measured every 1 minute

<u>Results:</u>

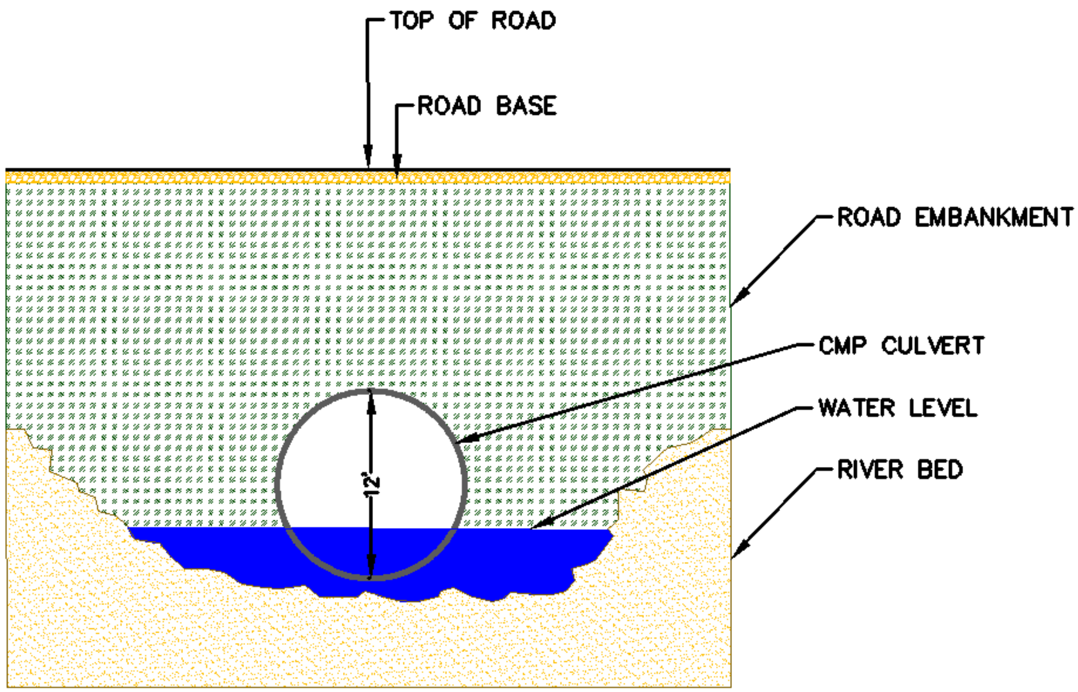
Unit hydrographs are produced from running simulations through HEC-HMS for every subbasin, junction, reach, and culvert. The values of interest are the peak discharges for the culvert from each trial. Table 3.10 lists the peak discharge results for both the current design and future design flow conditions; these values were needed for culvert analysis. A more detailed analysis of the HEC-HMS results is in Chapter 4.

Table 3.10: Current and future design peak flow conditions

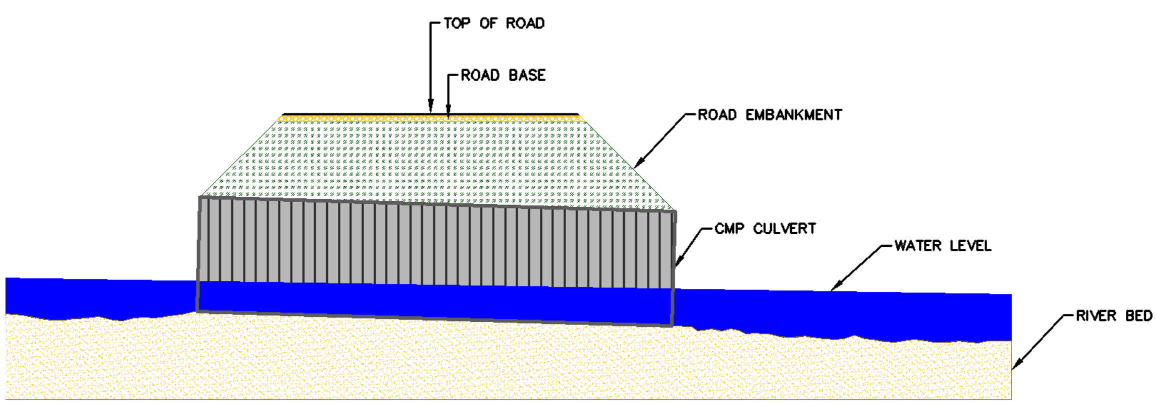
Current Design Precipitation			Future Design Precipitation		
Probability (%)	Intensity Duration (min)	Flow (CFS)	Probability (%)	Intensity Duration (min)	Flow (CFS)
1	5	6062.7	1	5	8331.4
	15	6062.8		15	8331.5
	60	6061.7		60	8329.6
2	5	4747.6	2	5	6635.4
	15	4747.7		15	6635.5
	60	4747.0		60	6634.2
4	5	3594.8	4	5	5129.8
	15	3594.9		15	5129.9
	60	3594.5		60	5129.1
10	5	2264.9	10	5	3359.8
	15	2265.0		15	3359.9
	60	2264.9		60	3359.6

3.4 Culvert Analysis

Previously, the watershed boundaries, physical characteristics, and parameters were established in ArcGIS and HEC-HMS. A meteorological model was also created and applied to the watershed resulting in unit hydrographs for each subbasin, junction, reach, and outlet for all 24 trials in HEC-HMS. Using the culvert unit hydrographs both the 12 foot CMP culvert and 12 foot x 12 foot – side by side – box culvert were analyzed for capacity performance. Figure 3.5 and Figure 3.6 are typical culvert designs with dimensions drawn in AutoCAD.

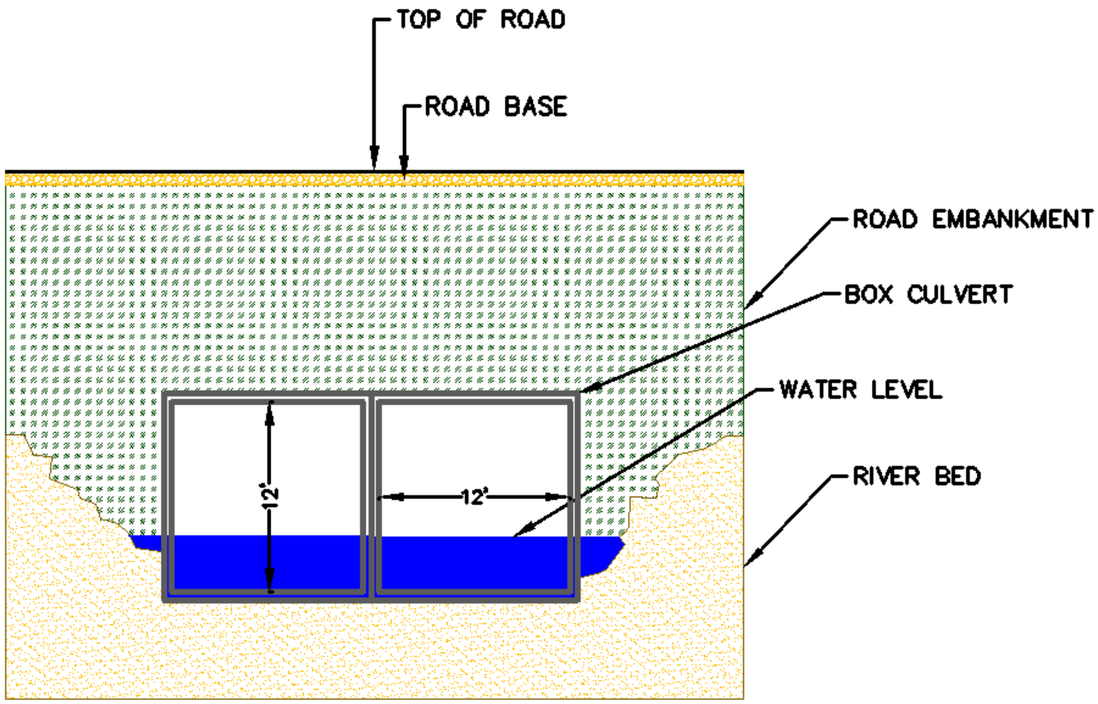


(a)

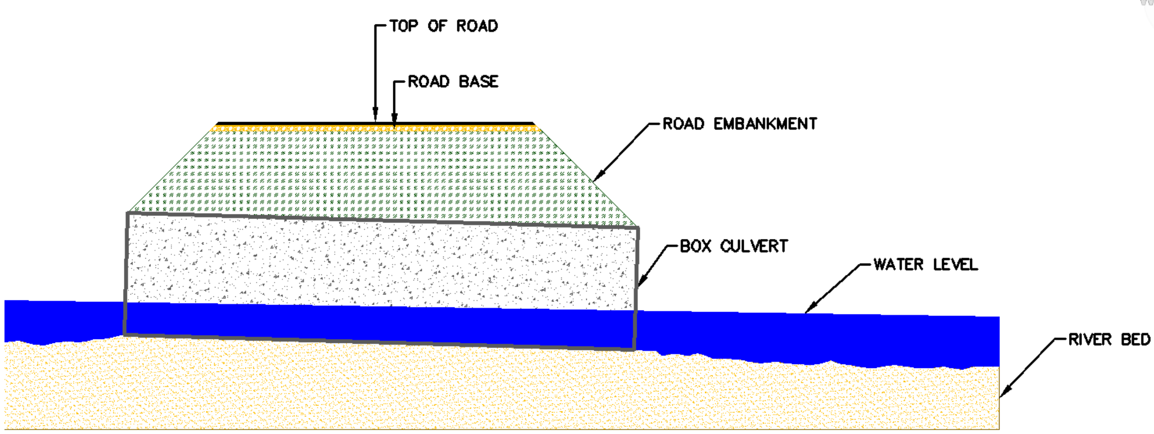


(b)

Figure 3.5: 12 foot CMP culvert not to scale (a) cross-section (b) profile view



(a)



(b)

Figure 3.6: 12 foot x 12 foot – side by side – box culvert not to scale (a) cross-section (b) profile view

There have been computer programs developed to analyze culvert designs. In order to use the computer software river and culvert parameters must be known or calculated. The major river parameters are channel characteristics, peak flows, water levels both upstream and downstream of the culvert, and embankment characteristics. Some major culvert parameters are: type, material, dimensions and inlet and outlet elevations. The Upper White River Subbasin Watershed study used two different culvert analysis computer programs to analyze the existing 12 foot CMP culvert and the current 12 foot x 12 foot – side by side – box culvert. The first culvert analyzing program used was HY-8 Version 7.3 (HY-8); second was AutoCAD Civil 3D Hydraflow Express Extension (Hydraflow Express).

Figure 3.7 is a screen shot of the 12 foot CMP culvert washout site with estimated dimensions from Google Earth (2015). Table 3.11 lists what the dimensions on Figure 3.7 represent. These values were used as parameter enters for HY-8 and Hydraflow Express.

Table 3.11: List of estimated elevations and dimensions of the White River and embankment

Embankment		Culvert		River	
Variables	Values	Variables	Values	Variables	Values
Top Elevation	2980.00 ft	Inlet Elevation	2958.00 ft	Surface Width	50.00 ft
Top Width	50.00 ft	Outlet Elevation	2956.00 ft		
Crest Length	125.00 ft	Length	110.00 ft		

Note: Top elevation for embankment was agreed upon after talking with Joanita Kant a research who has visited the site and SDSU professor Dr. Burckhard, P.E.

Figure 3.7: 12 foot CMP culvert with estimated dimensions (Google Earth, 2015)

3.4.1 HY-8

"An HY-8 project involves the design and analysis of single or multiple culverts at one or more crossings" (HY-8 Culvert Analysis Program). For more information of specific details and/or the procedure refer to HY-8 User Manual v7.3 (HY-8 Culvert Analysis Program). The Upper White River Subbasin Watershed study used the analysis of a single crossing application of the HY-8.

Figure 3.8 is a screen shot of the 100 year peak flow conditions entered into the crossing data table used to analyze the 12 foot CMP culvert. River and culvert parameters were kept constant throughout the 12 foot CMP culvert analysis and the peak flow varied for each trial. Refer back to Table 3.10 for peak flow values. The same process was followed to analyze the 12 foot x 12 foot – side by side – box culvert. Figure 3.9 is a screen shot of the 100 year peak flow conditions entered into the crossing data table used to analyze the 12 foot x 12 foot – side by side – box culvert. Again, the river and culvert parameters were kept constant thought the 12 foot x 12 foot – side by side – box culvert analysis and the peak flow varied for each trial. Refer back to Table 3.10 for peak flow values.

Parameter Enters:

- Discharge Data – Used calculated peak flow discharges from Table 3.10.
- Tailwater Data – Assumed a trapezoidal channel type with a side slope of 3:1. Calculated the bottom width using the side slope and surface width. Channel slope was assumed to be equal to culvert slope with a Manning's value, n = 0.03 (McCuen 2005, 135-137). The n value was based on a combination of short grass, fine sand and gravels, and condition of the

channel's bed and embankment. The channel invert elevation was equal to the outlet elevation of the culvert.

- Roadway Data – Used values from Table 3.11 and assumed a constant roadway elevation with no stationing 10 feet above the culvert.
- Culvert Data – Varied depending on if analyzing the 12 foot CMP culvert or 12 foot x 12 foot – side by side – box culvert. The Manning's values varied depending on the material used to construct the culvert; CMP = 0.022 and concrete = 0.013 (McCuen 2005, 118).
- Site Data – Used values from Table 3.11.

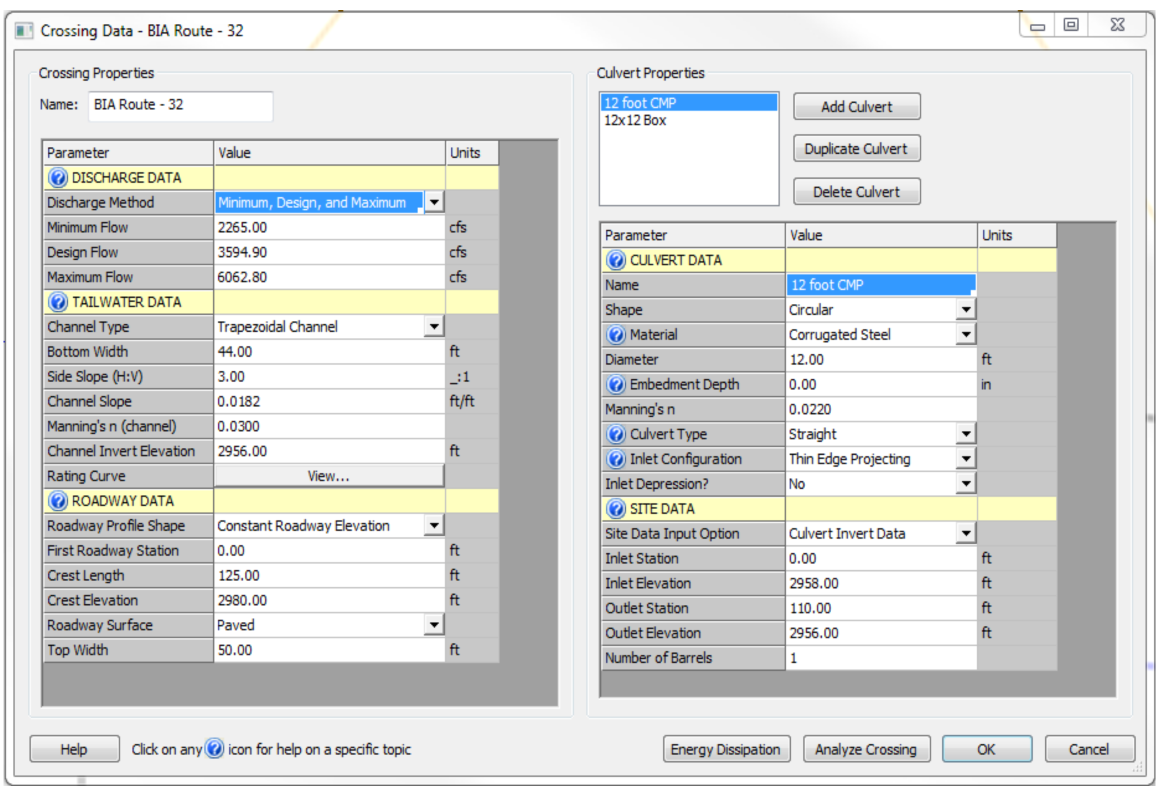


Figure 3.8: 12 foot CMP culvert crossing data enters

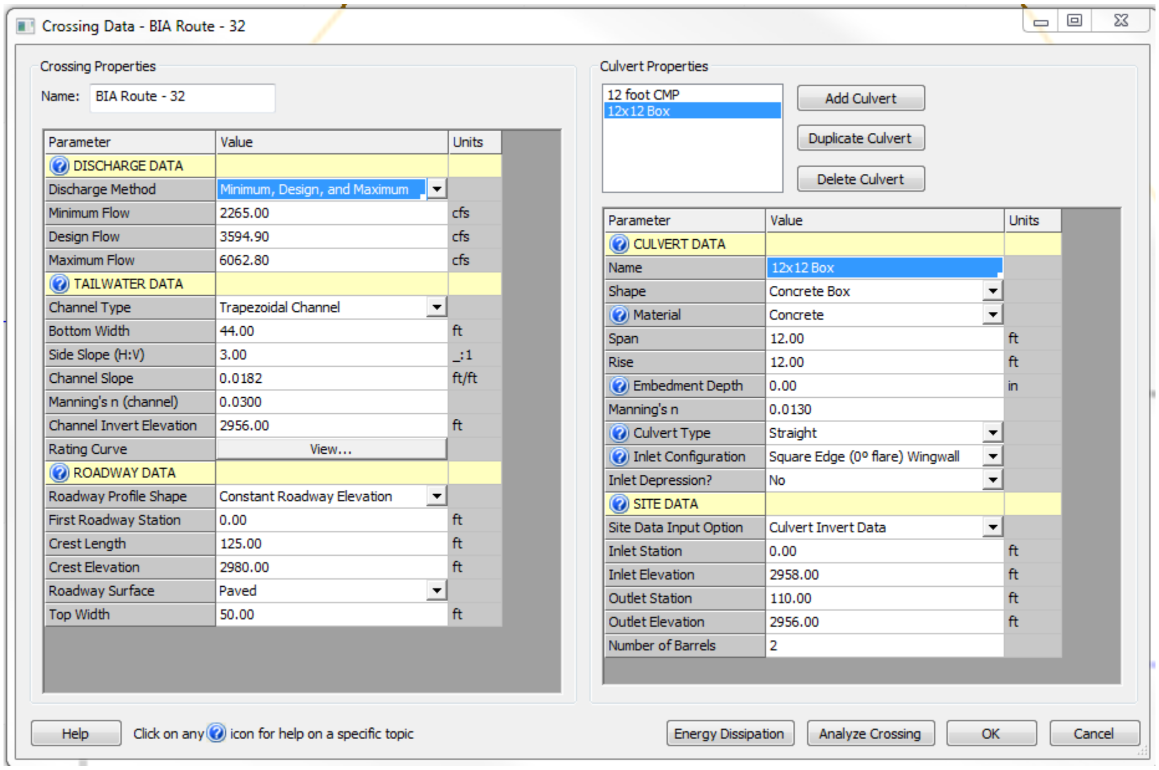


Figure 3.9: 12 foot x 12 foot – side by side – box culvert crossing data enters

3.4.2 Hydraflow Express

"Hydraflow Express is an application for solving typical hydraulics and hydrology problems. It addresses a wide variety of tasks, including culverts, open channels, inlets, hydrology and weirs, using a unique user interface" (Autodesk, 2010). For more information on specific details and/or the procedure refer to AutoCAD Civil 3D Hydraflow Express Extension User's Guide (Autodesk, 2010). AutoCAD Civil 3D has an extension called Hydraflow Hydrographs which allows the user to build a watershed. This extension was not used because the Upper White River Subbasin Watershed was built in ArcGIS. The culvert analysis option in Hydraflow Express was used to analyze both the 12 foot CMP culvert and 12 foot x 12 foot – side by side – box culvert while exposed current design and future design river flows.

Figure 3.10 is a screen shot of the 100 year peak flow conditions entered into the window used to analyze the 12 foot CMP culvert. River and culvert parameters were kept constant throughout the 12 foot CMP culvert analysis and the peak flow varied for each trial. Refer back to Table 3.10 for peak flow values. The same process was followed to analyze the 12 foot x 12 foot – side by side – box culvert. Figure 3.11 is a screen shot of the 100 year peak flow conditions entered into the window used to analyze the 12 foot x 12 foot – side by side – box culvert. Again, the river and culvert parameters were kept constant throughout the 12 foot x 12 foot – side by side – box culvert analysis and the peak flow varied for each trial. Refer back to Table 3.10 for peak flow values.

Parameter Enters:

- Pipe – Used values from Table 3.11 for: invert elevation down river (Inv Elev Dn), length, and invert elevation up river (Inv Elev Up). Culvert enters varied depending on if analyzing the 12 foot CMP culvert or 12 foot x 12 foot – side by side – box culvert. The Manning's values varied depending on the material used to construct the culvert; CMP = 0.022 and concrete = 0.013 (McCuen 2005, 118).
- Embank – Used values from Table 3.11 and assumed a constant roadway elevation with no stationing 10 feet above the culvert.
- Calcs - Used calculated peak flow discharges from Table 3.10.

Section	Item	Input
Pipe	Inv Elev Dn =	2956.00
	Length (ft) =	110.00
	Slope (%) =	1.82
	Inv Elev Up =	2958.00
	Rise (in) =	144.0
	Shape =	Circular
	Span (in) =	144.0
	No. Barrels =	1
	n-value =	0.022
	Culvert Type =	Circular Corrugate Metal Pipe
	Culvert Entrance =	Projecting
Embank	Top Elev =	2980.00
	Top Width (ft) =	50.00
	Crest Len (ft) =	125.00
Calcs	Q Min (cfs) =	2264.90
	Q Max (cfs) =	6062.80
	Q Incr (cfs) =	200.00
	Tailwater (ft) =	(dc+D)/2

Clear　　　Run

Figure 3.10: 12 foot CMP culvert window

Section	Item	Input
Pipe	Inv Elev Dn =	2956.00
	Length (ft) =	110.00
	Slope (%) =	1.82
	Inv Elev Up =	2958.00
	Rise (in) =	144.0
	Shape =	Box
	Span (in) =	144.0
	No. Barrels =	2
	n-value =	0.013
	Culvert Type =	Flared Wingwalls
	Culvert Entrance =	0D wingwall flares
Embank	Top Elev =	2980.00
	Top Width (ft) =	50.00
	Crest Len (ft) =	125.00
Calcs	Q Min (cfs) =	2264.90
	Q Max (cfs) =	6062.8
	Q Incr (cfs) =	200.00
	Tailwater (ft) =	(dc+D)/2

Clear Run

Figure 3.11: 12 foot x 12 foot – side by side – box culvert window

3.5 Culvert Design

Culvert design was beyond the scope of this project because there was an existing culvert at the crossing. The 12 foot CMP culvert washed out in the Spring of 2010 and a 12 foot x 12 foot – side by side – box culvert was constructed in the crossing. If the reader is interested in learning more about culvert design please refer to SDDOT Drainage Manual Section 10.4 Culvert Hydraulic Design (SDDOT, 2013).

4. RESULTS AND ANALYSIS

Chapter 4 presents the simulated results and analysis of the Upper White River Subbasin Watershed. The current and future design simulated results from HEC-HMS, HY-8, and Hydraflow Express are discussed for the 25 year design return period for both the 12 foot CMP culvert and 12 foot x 12 foot – side by side – box culvert. A comparison of how the 12 foot CMP culvert vs the 12 foot x 12 foot – side by side – box culvert performed under current and future design condition was conducted for capacity performance. An engineering recommendation on which subbasin should also be considered for water retention management practices under simulated design criteria.

4.1 Simulated HEC-HMS Hydrographs

As stated previously, HEC-HMS was used to conduct precipitation modeling on the Upper White River Subbasin Watershed model. HEC-HMS calculated each subbasins' runoff as water traveled to stream channels and eventually to the White River. The White River was broken up into reaches and junctions automatically in HEC-HMS. Each reach and junction had a hydrograph calculated for it by HEC-HMS. A hydrograph was also calculated for the culvert location which composed of the sum of reaches and junctions hydrographs.

A total of 24 trials were conducted resulting in 24 culvert hydrographs; 12 using the current design precipitations and 12 using the estimated future design precipitations. The culvert hydrographs were used to obtain the peak discharges of the river flows due to direct runoff only for each scenario. This means the baseflow conditions were neglected in the study. Figure 4.1 and Figure 4.2 show the culvert hydrographs resulting from a 25 year return period precipitation event using an intensity duration of 15 minutes for the current and future design precipitations respectively. The 25 year return period precipitation culvert hydrographs were selected for discussion because BIA-Route 32 crosses over the culvert of interest was assumed to be classified as a local road or street (ADT > 100) with a return period of 25 years. An intensity duration of 15 minutes was selected because it corresponded to the highest peak discharge flows for each scenario. The simulated peak discharge flows were used to analyze the 12 foot CMP culvert and 12 foot x 12 foot – side by side – box culvert capacities. The 10, 50, and 100 year return period precipitation 'culvert' hydrographs are located in Appendix F.

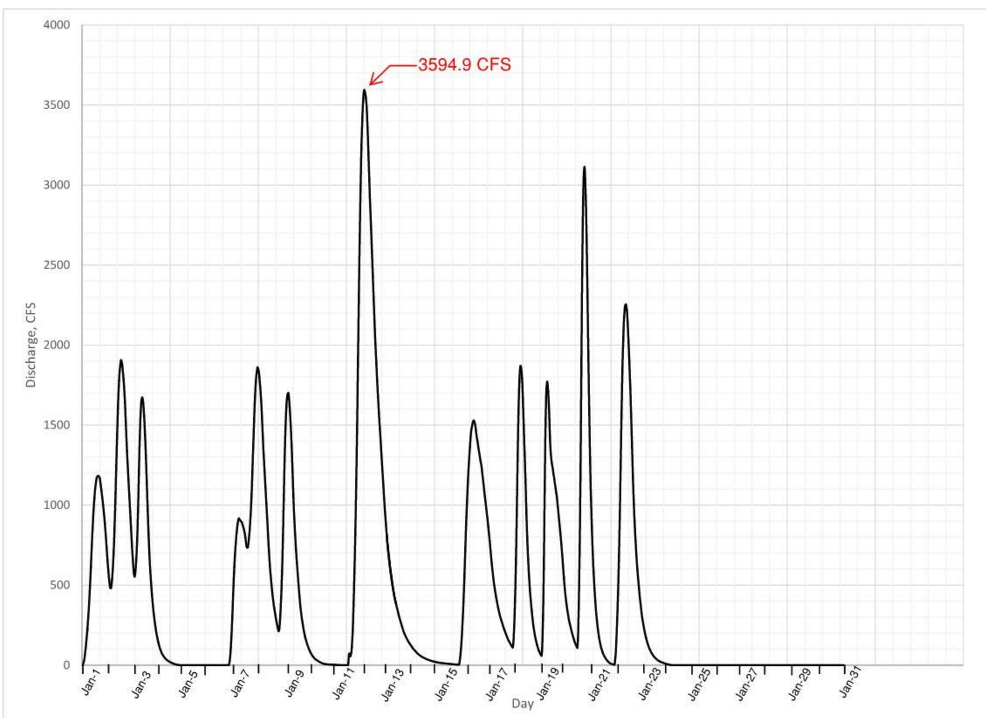


Figure 4.1: Current 25 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

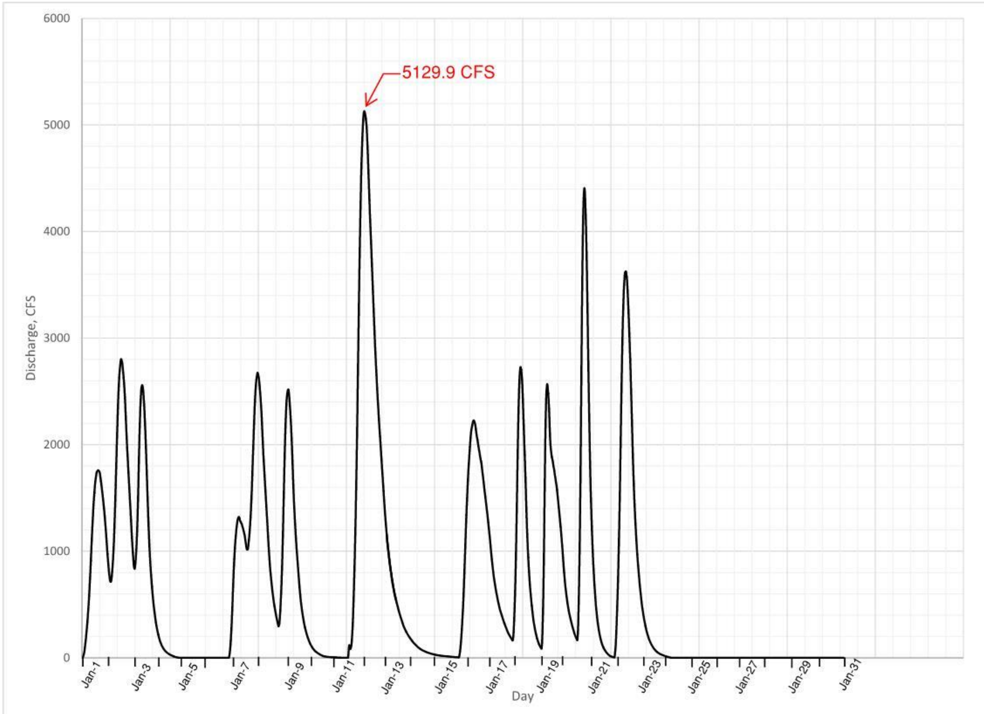


Figure 4.2: Future 25 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

There was an increase in peak discharges from the current 25 year return period to future 25 year return period. By increasing the current design annual precipitation depth by 20% correlated to a 30% potential peak discharge increase. The potential peak discharges increase for the 10, 50, and 100 year return periods are presented in Table 4.1.

Table 4.1: Potential peak discharge increases

Return Period (years)	Increase (%)
10	33
25	30
50	28
100	27

4.2 12 foot CMP Culvert

A 12 foot CMP culvert was previously in-place on BIA-Route 32 as a crossing over the White River up to a couple of years ago when it was washed out by a storm. As stated previously, BIA-Route 32 is currently classified as a local road or street (ADT > 100) with a return period of 25 years. The 12 foot CMP culvert was analyzed using two separate computer programs under a 10, 25, 50, and 100 year return period with intensity duration of 5, 15, and 60 minutes. The following analysis is for a 25 year return period with a 15 minute intensity duration. Culvert analysis for return periods 10, 50, and 100 years with a 15 minute intensity duration are located in Appendix G for both current and future design conditions.

The 12 foot CMP culvert was analyzed as if it was still in place on BIA-Route 32 under current and future design conditions. In the Lakota Times Road Construction report Tom Crash stated "a 20 year old metal culvert on BIA-Route 32 in Slim Buttes west of Pine Ridge finally gave in to the acidic soil prevalent to the area and collapsed in the Spring of 2010." One cause for the collapse could have resulted from the soil condition, which is outside the scope of this paper, or it could have been due to inefficient culvert capacity.

4.2.1 Current Design Conditions

The 12 foot CMP culvert was analyzed following section 3.4 and using the simulated current 25 year return period peak discharge of 3,594.9 cfs. HY-8 performed iterations to calculate the culvert discharge and overtopping discharge if it occurred. Results from the iterations graphically showed overtopping of the 12 foot CMP culvert under the current 25 year return period with 15 minute intensity duration in Figure 4.3.

Overtopping was the indication of inefficient culvert capacity for the simulated scenario. With additional baseflow the water level would be elevated beyond the headwater depth, and the 12 foot CMP culvert would continue to fail. Table 4.2 provides the list generated by HY-8 of calculated culvert and downriver channel parameters.

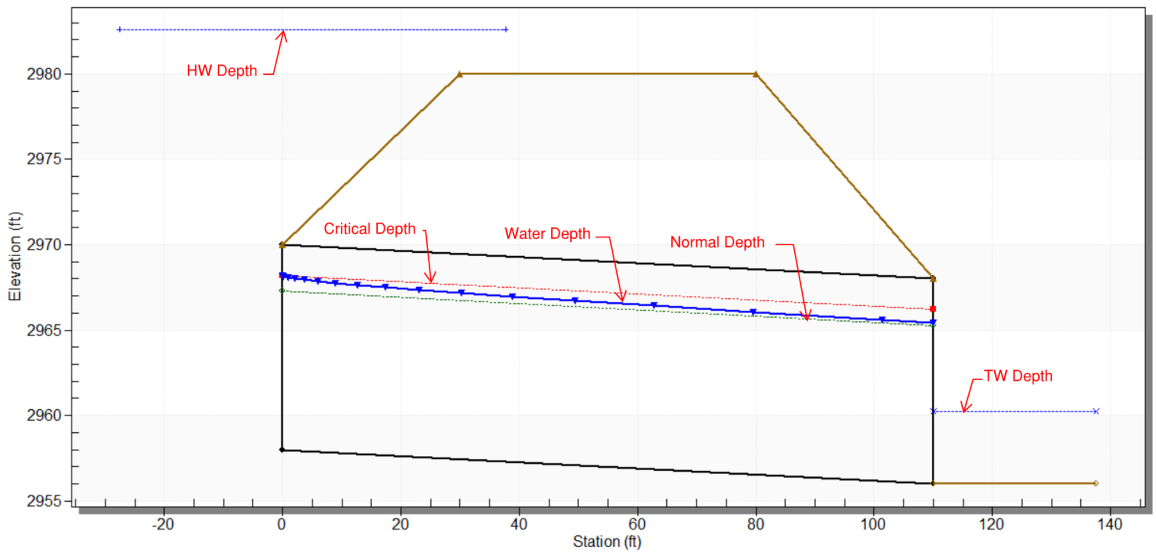


Figure 4.3: HY-8 analyzed 12 foot CMP culvert under current design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.2: Current 12 foot CMP culvert and river parameters generated by HY-8

Culvert		Units	Downriver		Units
Total Discharge	3594.90	cfs	Flow	3594.90	cfs
Culvert Discharge	2017.41	cfs	Tailwater Elevation	2960.22	ft
Overtopping Discharge	1577.44	cfs	Depth	4.22	ft
Headwater Elevation	2982.59	ft	Velocity	15.05	ft/s
Inlet Control Depth	24.59	ft	Shear	4.79	psf
Flow Type	5-S2n		Froude Number	1.43	
Normal Depth	9.28	ft			
Critical Depth	10.21	ft			
Outlet Depth	9.45	ft			
Outlet Velocity	21.16	ft/s			

Note: Flow type is broken into three parts; flow type, flow profile, and outlet type.
Flow type = 5 meaning headwater depth is greater than diameter (HW > D)
Flow profile = S2 meaning steep channel, supercritical flow, and water depth is in between the critical and normal depths or type 2 (Osman Akan 2008, 99-101).
Outlet type = n meaning normal outlet depth

The parameter of concern was the inlet control depth also referred to as the headwater depth, HW. Under the current design condition scenario the culvert would experience a HW depth of 24.59 ft; which resulted in overtopping the 10 foot embankment stated early. With a 24.59 ft HW depth the culvert was subjected to 44% of the total discharge converted to overtopping discharge.

The outlet depth gradually subsided to meet the tailwater depth, TW of 4.22 ft some distance downriver of the culvert. Downriver shear corresponds to the potential shearing force of the water relating to the potential destruction force of the river. The Froude number is a dimensionless number relating to the flow state of the water profile as it flows through the culvert. If the Froude number is greater than 1.0 then the flow type is said to be supercritical (Osman Akan 2008, 11). Flow types classify water profiles and culverts as either inlet or outlet controlled.

Hydraflow Express was used to verify the result obtained from HY-8. Following section 3.4.2 and entering 3,954.9 cfs for the discharge parameters reiterated the HY-8 result; which was the culvert was undersized for the current 25 year return period. The graphical representation of the overtopping is showing in Figure 4.4 with the water flowing from left to right. Additionally Hydraflow Express classified the culvert as inlet control flow which agrees with the flow type 5-S2n. "Inlet control flow generally occurs in steep, smooth culverts. The culvert will flow partially full under supercritical conditions" (Osman Akan 2008, 214). Figure 4.4 shows a submerged inlet, pipe flowing almost full under supercritical flow, and an unsubmerged outlet with no hydraulic jump occurring. A list of calculated Hydraflow Express parameters are shown in Table 4.3.

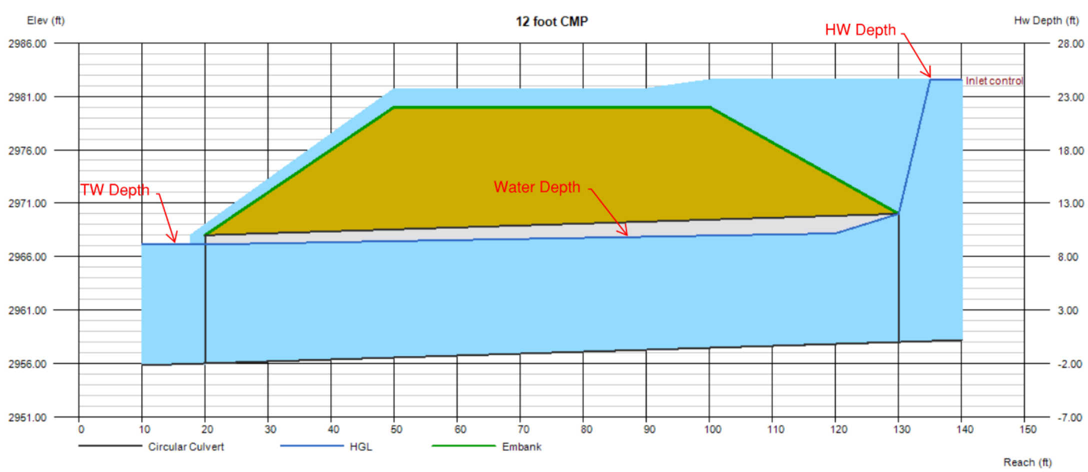


Figure 4.4: Hydraflow Express analyzed 12 foot CMP culvert under current design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.3: Current 12 foot CMP culvert parameters generated by Hydraflow Express

Category	Parameter	Value	Units
Flow	Total	3594.90	cfs
	Culvert	2048.84	cfs
	Overtopping	1546.06	cfs
Velocity	Outlet	18.72	ft/s
	Inlet	19.88	ft/s
Depth	Outlet	11.14	ft
	Inlet	10.27	ft
Hydraulic Grade Line	Outlet Elevation	2967.14	ft
	Inlet Elevation	2968.27	ft
	Headwater Elevation	2982.52	ft
	HW/D	2.04	

The hydraulic grade line, HGL represents the water profile as the water flows through the culvert. The HGL also measured the ratio of headwater depth to culvert diameter, HW/D. A HW/D of 2.04 corresponds to a HW depth twice the diameter of the culvert. The embankment height was set at 10 ft which means for this scenario the HW is 2 ft above the roadway resulting in overtopping. Hydraflow Express calculated 43% of the total discharge was converted to overtopping discharge.

There are some differences between what was calculated by Hydraflow Express and HY-8. Hydraflow Express does not include river channel input parameters, therefore did not calculate any of the downriver parameters. Also, Hydra flow Express also did not classify the flow type. Table 4.4 provides the list of consistence between the computer programs on four (4) major parameters: HW elevation, culvert discharge, overtopping discharge, and inlet control depth or HW/D. The percent differences are small, therefore both computer programs are consistence with one another.

Table 4.4: Current 12 foot CMP culvert percent difference between HY-8 and Hydraflow Express parameters

Parameters	Percent Difference (%)
Culvert Discharge (cfs)	-1.6
Overtopping Discharge (cfs)	2.0
Headwater Elevation (ft)	0.0
HW/D	0.5

Note: A negative percent meant HY-8 was lower than Hydraflow Express calculated parameter.

4.2.2 Future Design Conditions

The future design conditions were analyzed based on the section 2.1.2 journal articles stating the potential of increase precipitation events in the future. Kunkel, Andsager, and Easterling (1999) stated if engineers continued to design infrastructures based on Hershfield (1961) results, the subsequent infrastructures could be drastically underdesigned for future streamflows and runoff. Based on these statements, the 12 foot CMP culvert was analyzed using the simulated future 25 year return period peak discharge of 5129.9 cfs. Following the same procedure used to analyze the culvert under current design condition in section 3.4.1, but increasing the peak discharge to 5,129.9 cfs.

Since future design conditions are higher than current design condition peak discharges the culvert by default would have inefficient culvert capacity. HY-8 provided evidence of inefficient capacity through iterations of culvert and overtopping discharges. Overtopping occurred under future 25 year return period with 15 minute intensity duration is displayed in Figure 4.5. The overtopping confirmed inefficient capacity of the 12 foot CMP culvert. Again, the water level would be elevated if the baseflow condition were considered. Table 4.5 provides the list generated by HY-8 of calculated culvert and downriver channel parameters.

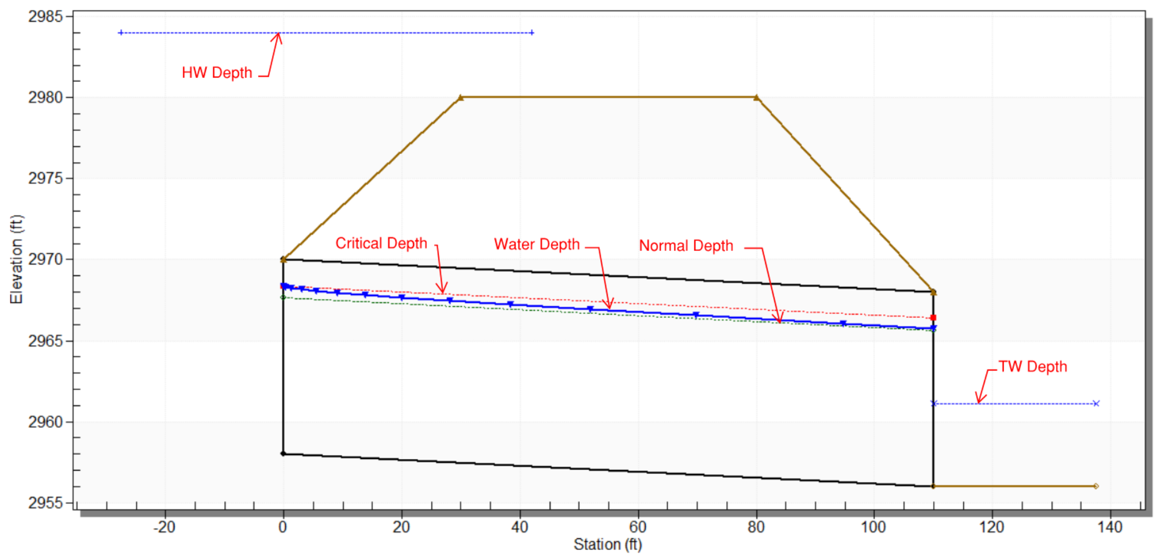


Figure 4.5: HY-8 analyzed 12 foot CMP culvert under future design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.5: Future 12 foot CMP culvert and river parameters generated by HY-8

Culvert		Units	Downriver		Units
Total Discharge	5129.90	cfs	Flow	5129.90	cfs
Culvert Discharge	2098.25	cfs	Tailwater Elevation	2961.14	ft
Overtopping Discharge	3031.59	cfs	Depth	5.14	ft
Headwater Elevation	2983.99	ft	Velocity	16.81	ft/s
Inlet Control Depth	25.99	ft	Shear	5.83	psf
Flow Type	5-S2n		Froude Number	1.47	
Normal Depth	9.63	ft			
Critical Depth	10.38	ft			
Outlet Depth	9.76	ft			
Outlet Velocity	21.32	ft/s			

The HW depth increased from 24.59 ft to 25.99 ft which relates to an additional 1.4 ft of water flowing over the roadway. An increase in 1.4 ft of HW depth corresponded to 49% of the total discharge converted to overtopping discharge. This was a 16% increase compared to the current design conditions. Consequently the increase caused a degree of increase to the other calculated parameters as expected.

Hydraflow Express was used to verify the result obtained from HY-8. Following section 3.4.2 and entering 5,129.9 cfs for the discharge parameters reiterated the HY-8 result; which was the culvert was undersized for the current 25 year return period. The graphical representation of the overtopping is showing in Figure 4.6 with the water flowing from left to right. The increase in flow did not change the classification of the culvert as inlet controlled with a water profile of 5-S2n. Figure 4.6 shows an increased in HW depth resulting in the culvert remaining submerged, pipe flowing almost full under supercritical flow, and an unsubmerged outlet with no hydraulic jump. Table 4.6 provides the list generated by Hydraflow Express of calculated culvert parameters.

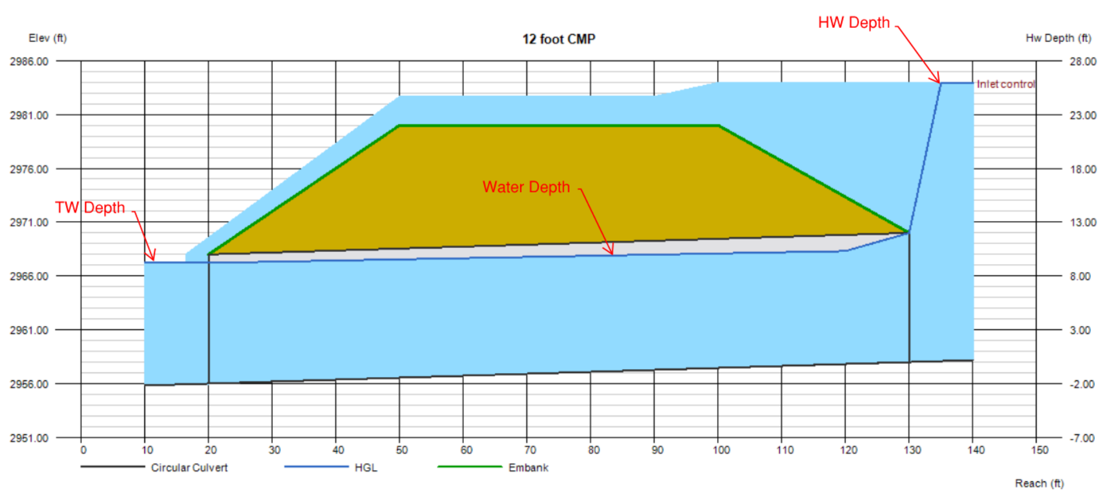


Figure 4.6: Hydraflow Express analyzed 12 foot CMP culvert under future design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.6: Future 12 foot CMP culvert parameters generated by Hydraflow Express

Category	Parameter	Value	Units
Flow	Total	5129.90	cfs
	Culvert	2127.18	cfs
	Overtopping	3002.72	cfs
Velocity	Outlet	19.35	ft/s
	Inlet	20.39	ft/s
Depth	Outlet	11.21	ft
	Inlet	10.42	ft
Hydraulic Grade Line	Outlet Elevation	2967.21	ft
	Inlet Elevation	2968.42	ft
	Headwater Elevation	2983.93	ft
	HW/D	2.16	

The HGL increase was directly related to the increase in total discharge. As a result of this increase the HW/D increased to 2.16. An increase of 0.12 corresponds to an increase of 1.92 ft of water flowing over the road compared to current design condition. Hydraflow Express calculated 59% of the total discharge was converted to overtopping discharge. The differences between what was calculated by HY-8 and Hydraflow Express were the same as the current design conditions of the 12 foot CMP culvert. Table 4.7 provides the list of consistence between the computer programs on four (4) major parameters: HW elevation, culvert discharge, overtopping discharge, and HW/D. The percent differences are small, therefore both computer programs are consistence with one another.

Table 4.7: Future 12 foot CMP culvert percent difference between HY-8 and Hydraflow Express parameters

Parameters	Percent Difference (%)
Culvert Discharge (cfs)	-1.4
Overtopping Discharge (cfs)	1.0
Headwater Elevation (ft)	0.0
HW/D	0.5

Note: A negative percent meant HY-8 was lower than Hydraflow Express calculated parameter.

4.2.3 Compare and Contrast

A comparison was conducted to analyze the increases the 12 foot CMP culvert potentially could have experienced due to the 30% in peak discharge. The only variable which was changed between the current and future design conditions was the peak discharges. By keeping all other variables constant the HW depth was directly correlated to the increase in peak discharge. Table 4.8 shows percent increase from current to future design conditions which the 12 foot CMP culvert and river experienced.

Table 4.8: 12 foot CMP culvert comparison of future vs. current design conditions

HY-8			
Culvert		River	
Total Discharge	30%	Flow	30%
Culvert Discharge	4%	Tailwater Elevation	0.03%
Overtopping Discharge	48%	Depth	18%
Headwater Elevation	0.05%	Velocity	10%
Inlet Control Depth	5%	Shear	18%
Flow Type	N.A.	Froude Number	3%
Normal Depth	4%		
Critical Depth	2%		
Outlet Depth	3%		
Outlet Velocity	1%		

Hydraflow Express	
Total Discharge	30%
Culvert Discharge	4%
Overtopping Discharge	49%
Outlet Velocity	3%
Inlet Velocity	3%
Outlet Depth	1%
Inlet Depth	1%
Outlet Elevation	0.00%
Inlet Elevation	0.01%
Headwater Elevation	0.05%
HW/D	6%

As was expected all the parameters increased or remained the same as the peak discharge increased from current to future design conditions. There were a range of increase from 0% to 49%. The results corresponded the highest percent increase were total discharge (30%) and overtopping discharge (49%). One interpretation of a resulting 49% increase in overtopping discharge was due to the culvert discharge only increased by 4%. The reason for a small increase in culvert discharge was because the culvert was approaching critical depth, represented by a small percent increase, in the culvert under current design conditions. Critical depth directly relates to the maximum capacity of the culvert.

The parameters with a small degree of increase: culvert discharge, HW depth, critical depth, normal depth, and velocities meant the 12 foot CMP culvert was at its maximum capacity. The additional peak discharge flows were converted to overtopping discharge. The HW elevation is highlighted in yellow because the increase is very small compared to the HW depth and overtopping discharge. A practical reason for such a small increase is because HW elevation was spread over the whole crest length of 125 ft. There was a difference of 1.4 ft in the HW elevation resulting in an additional 175 square ft for the overtopping discharge to flow through. Therefore, results confirmed the 12 foot CMP culvert was not able to convey current design conditions nor future design conditions peak discharges without overtopping the roadway.

4.3 12 foot x 12 foot – Side by Side - Box Culvert

A 12 foot x 12 foot – side by side – box culvert is currently in-place on BIA-Route 32 as a crossing over the White River. As stated previously, BIA-Route 32 is currently classified as a local road or street (ADT > 100) with a return period of 25 years. The 12 foot x 12 foot – side by side - box culvert was analyzed using two separate computer programs under current and future design conditions of a 10, 25, 50, and 100 year return period with intensity duration of 5, 15, and 60 minutes. The following analysis is for a 25 year return period with a 15 minute intensity duration. Culvert analysis for return periods 10, 50, and 100 years with a 15 minute intensity duration are located in Appendix H for both current and future design conditions.

4.3.1 Current Design Conditions

The 12 foot x 12 foot – side by side - box culvert was analyzed following section 3.4.1 and using the simulated current 25 year return period peak discharge of 3,594.9 cfs. HY-8 performed iterations to calculate the culvert discharge and overtopping discharge if it occurred. Results from the iterations graphically showed no overtopping of the 12 foot x 12 foot – side by side - box culvert under the current 25 year return period with 15 minute intensity duration in Figure 4.7. No overtopping was the indication of efficient culvert capacity for the simulated scenario, therefore the culver met SDDOT design criteria. The water level would be elevated if the baseflow condition were considered, but visually should not overtop the crossing. Table 4.9 provides the list generated by HY-8 of calculated culvert and downriver channel parameters.

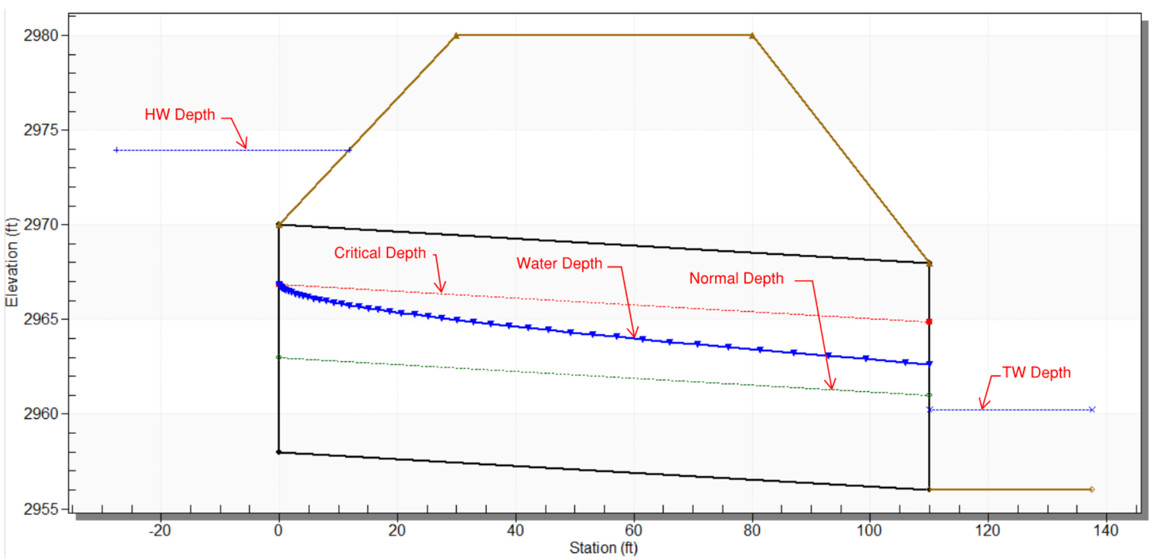


Figure 4.7: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under current design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.9: Current 12 foot x 12 foot – side by side – box culvert and river parameters generated by HY-8

Culvert		Units	Downriver		Units
Total Discharge	3594.90	cfs	Flow	3594.90	cfs
Culvert Discharge	3594.90	cfs	Tailwater Elevation	2960.22	ft
Overtopping Discharge	0.00	cfs	Depth	4.22	ft
Headwater Elevation	2973.94	ft	Velocity	15.05	ft/s
Inlet Control Depth	15.94	ft	Shear	4.79	psf
Flow Type	5-S2n		Froude Number	1.43	
Normal Depth	4.97	ft			
Critical Depth	8.87	ft			
Outlet Depth	6.64	ft			
Outlet Velocity	22.57	ft/s			

The parameter of concern for this study was the inlet control depth also referred to as the HW depth. Under the current design condition scenario the culvert would experience a HW depth of 15.94 ft; which is significantly lower than the 10 foot embankment above the 12 foot x 12 foot – side by side – box culvert. All of the discharge was conveyed by the 12 foot x 12 foot – side by side – box culvert for the current design conditions peak flows Downriver conditions were consist between the 12 foot CMP culvert and 12 foot x 12 foot – side by side – box culvert under current design conditions. The only difference occurred at the inlet of each culvert.

Hydraflow Express was used to verify the result obtained from HY-8. Following section 3.4.2 and entering 3,954.9 cfs for the discharge parameters reiterated the HY-8 result; which was the culvert was undersized for the current 25 year return period. Results from the iterations graphically shown in Figure 4.8 of the 12 foot x 12 foot – side by side - box culvert under the current 25 year return period with 15 minute intensity duration has the appropriate capacity as the water flows from left to right. The water level would be elevated if the baseflow condition were considered, but visually should not overtop the crossing. Additional Hydraflow Express classified the culvert as inlet control flow with a submerged inlet, pipe flow not completely full, and an unsubmerged outlet verifying the water profile as 5-S2n. Table 4.10 provides the list generated by Hydraflow Express of calculated culvert parameters.

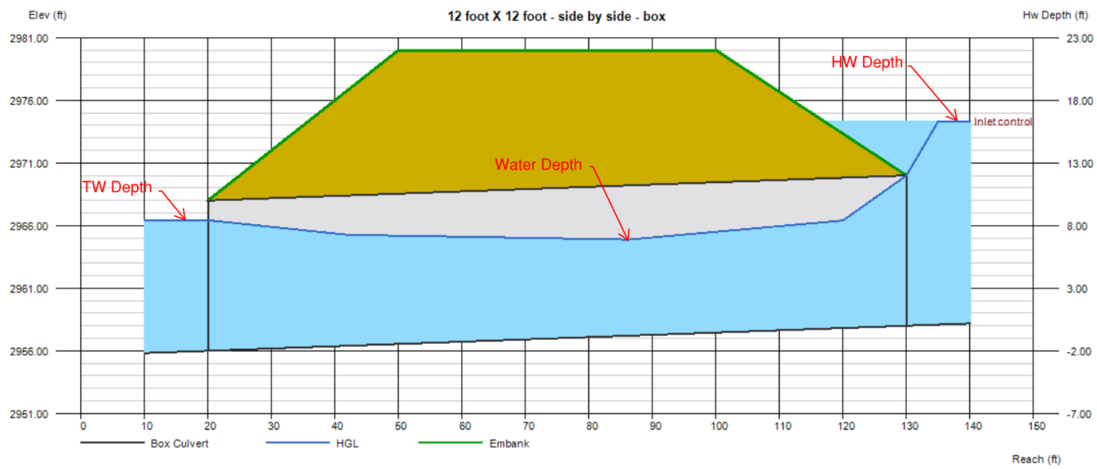


Figure 4.8: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under current design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.10: Current 12 foot x 12 foot – side by side – box culvert parameters generated by Hydraflow Express

Category	Parameter	Value	Units
Flow	Total	3594.90	cfs
	Culvert	3594.90	cfs
	Overtopping	0.00	cfs
Velocity	Outlet	14.37	ft/s
	Inlet	16.93	ft/s
Depth	Outlet	10.43	ft
	Inlet	8.85	ft
Hydraulic Grade Line	Outlet Elevation	2966.43	ft
	Inlet Elevation	2966.85	ft
	Headwater Elevation	2974.32	ft
	HW/D	1.36	

The HGL calculated HW/D to be 1.36 corresponds to a HW depth of 16.32 ft. The embankment height was set at 10 ft which means for this scenario the HW is 5.68 ft below the roadway. Hydraflow Express calculated 100% of the total discharge was converted to culvert discharge. The differences between what was calculated by HY-8 and Hydraflow Express were the same as the current design conditions of the 12 foot CMP culvert. Table 4.11 provides the list of consistence between the computer programs on four (4) major parameters: HW elevation, culvert discharge, overtopping discharge, and HW/D. The percent differences are small, therefore both computer programs are consistence with one another.

Table 4.11: Current 12 foot x 12 foot – side by side – box culvert percent difference between HY-8 and Hydraflow Express parameters

Parameters	Percent Difference (%)
Culvert Discharge (cfs)	0.0
Overtopping Discharge (cfs)	0.0
Headwater Elevation (ft)	0.0
HW/D	2.21

Note: A negative percent meant HY-8 was lower than Hydraflow Express calculated parameter.

4.3.2 Future Design Conditions

The future design conditions were analyzed based section 2.1.2 journal articles stating the potential of increase precipitation events in the future and the impacts on infrastructure design conditions. Based on these statements, the 12 foot x 12 foot – side by side - box culvert was analyzed using the simulated future 25 year return period peak discharge of 5129.9 cfs. Following the same procedure used to analyze the culvert under current design condition in section 3.4.1, but increasing the peak discharge to 5,129.9 cfs.

HY-8 provided evidence of inefficient capacity through iterations of culvert and overtopping discharges. Overtopping occurred due to future 25 year return period with 15 minute intensity duration is displayed in Figure 4.9. The overtopping confirmed inefficient capacity of the 12 foot x 12 foot – side by side- box culvert. Again, the water level would be elevated if the baseflow condition were considered. Table 4.12 provides the list generated by HY-8 of calculated culvert and downriver channel parameters.

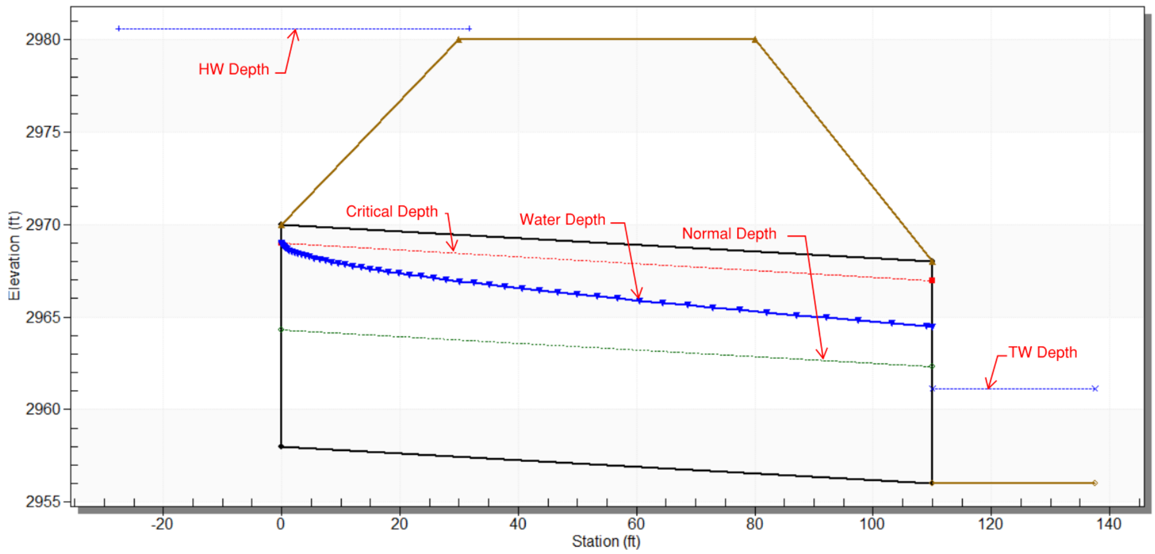


Figure 4.9: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under future design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.12: Future 12 foot x 12 foot – side by side – box culvert and river parameters generated by HY-8

Culvert		Units	Downriver		Units
Total Discharge	5129.90	cfs	Flow	5129.90	cfs
Culvert Discharge	4956.89	cfs	Tailwater Elevation	2961.14	ft
Overtopping Discharge	172.93	cfs	Depth	5.14	ft
Headwater Elevation	2980.59	ft	Velocity	16.81	ft/s
Inlet Control Depth	22.59	ft	Shear	5.83	psf
Flow Type	5-S2n		Froude Number	1.47	
Normal Depth	6.32	ft			
Critical Depth	10.98	ft			
Outlet Depth	8.48	ft			
Outlet Velocity	24.67	ft/s			

The HW depth increased from 15.94 ft to 22.59 ft exceeding the embankment elevation resulting in 0.59 ft of water flowing over the roadway. An increase in 6.65 ft of HW depth corresponded to 3.4% of the total discharge converted to overtopping discharge. Consequently the increase caused a degree of increase to the other calculated parameters as expected.

Hydraflow Express was used to verify the result obtained from HY-8. Following section 3.4.2 and entering 5,129.9 cfs for the discharge parameters reiterated the HY-8 result; which was the culvert was undersized for the current 25 year return period. The graphical representation of the overtopping is showing in Figure 4.10 with the water flowing from left to right. The increase in flow did not change the classification of the culvert as inlet controlled with a water profile of 5-S2n. Figure 4.10 shows an increased in HW depth resulting in the culvert remaining submerged, pipe flowing almost full under supercritical flow, and an unsubmerged outlet with no hydraulic jump. Table 4.13 provides the list generated by Hydraflow Express of calculated culvert parameters.

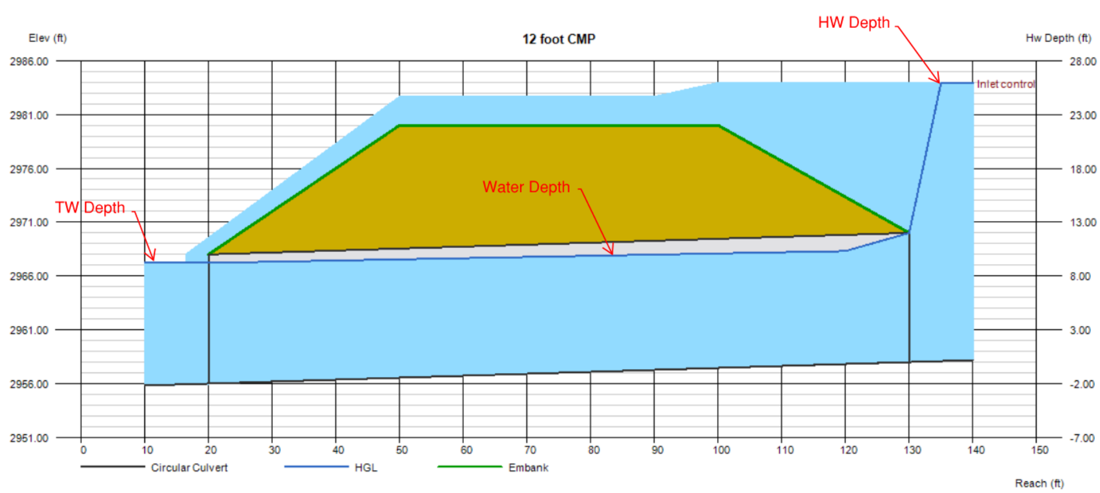


Figure 4.10: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under future design, 25 year return period 15 min intensity duration simulated flow conditions

Table 4.13: Future 12 foot x 12 foot – side by side – box culvert parameters generated by Hydraflow Express

Category	Parameter	Value	Units
Flow	Total	5129.90	cfs
	Culvert	4999.69	cfs
	Overtopping	130.21	cfs
Velocity	Outlet	18.10	ft/s
	Inlet	18.90	ft/s
Depth	Outlet	11.52	ft
	Inlet	11.02	ft
Hydraulic Grade Line	Outlet Elevation	2967.51	ft
	Inlet Elevation	2969.02	ft
	Headwater Elevation	2980.48	ft
	HW/D	1.87	

The HGL increased is directly related to the increase in total discharge. As a result the HW/D increased to 1.87. An increase of 0.51 corresponds to an increase of 0.44 ft of water flowing over the roadway. Hydraflow Express calculated 2.5% of the total discharge was converted to overtopping discharge. The differences between what was calculated by HY-8 and Hydraflow Express were the same as the current design conditions of the 12 foot CMP culvert. Table 4.14 provides the list of consistence between the computer programs on four (4) major parameters: headwater elevation, culvert discharge, overtopping discharge, and HW/D. The percent differences are small for all parameters expect for overtopping discharge. Difference between smaller values have a more significant effect on the percent difference than larger values skewing the interpretation.

Table 4.14: Future 12 foot x 12 foot – side by side – box culvert percent difference between HY-8 and Hydraflow Express parameters

Parameters	Percent Difference (%)
Culvert Discharge (cfs)	-0.86
Overtopping Discharge (cfs)	24.7
Headwater Elevation (ft)	0.0
HW/D	0.53

Note: A negative percent meant HY-8 was lower than Hydraflow Express calculated parameter.

4.3.3 Compare and Contrast

A comparison was conducted to analyze the increases the 12 foot x 12 foot – side by side – box culvert potentially could have experienced due to the 30% in peak discharge. The only variable which was changed between the current and future design conditions was the peak discharges. By keeping all other variables constant the HW depth was directly correlated to the increase in peak discharge. Table 4.15 shows percent increase from current to future design conditions which the 12 foot x 12 foot – side by side – box culvert and river experienced.

Table 4.15: 12 foot x 12 foot – side by side – box culvert comparison of future vs. current design conditions

HY-8			
Culvert		River	
Total Discharge	30%	Flow	30%
Culvert Discharge	27%	Tailwater Elevation	0.03%
Overtopping Discharge	100%	Depth	18%
Headwater Elevation	0.22%	Velocity	10%
Inlet Control Depth	29%	Shear	18%
Flow Type	N.A.	Froude Number	3%
Normal Depth	21%		
Critical Depth	19%		
Outlet Depth	22%		
Outlet Velocity	9%		

Hydraflow Express	
Total Discharge	30%
Culvert Discharge	28%
Overtopping Discharge	100%
Outlet Velocity	21%
Inlet Velocity	10%
Outlet Depth	9%
Inlet Depth	20%
Outlet Elevation	0.04%
Inlet Elevation	0.07%
Headwater Elevation	0.21%
HW/D	27%

As was expected, all the parameters increased or remained the same as the peak discharge increased from current to future design conditions. There were a range of increase from 0% to 100%. The result corresponding to the highest percent increase was overtopping discharge (100%). An increase of 100% was because the 12 foot x 12 foot – side by side – box culvert was able to convey the total. A significant increase of normal, critical, and outlet depths were delineated from the comparison. One explanation is the culvert did not reach full capacity for the current design condition, but approach capacity limits under future design conditions.

The major difference between the current and future design conditions was the 12 foot x 12 foot – side by side – box culvert's performance. Under current design conditions the culvert was safely able to convey the simulated peak discharge flow of 3,594.90 cfs without overtopping. The HW were roughly 6 ft below road elevation ensuring additional culvert capacity for baseflow conditions.

4.4 12 foot CMP vs. 12 foot x 12 foot – Side by Side – Box Culvert

As stated previously, the 12 foot CMP culvert failed the current and future 25 year return period, 15 minute intensity duration design conditions. During the simulated design conditions BIA-Rout 32 would have experienced 3 and 4 foot water depths respectively. The Lakota Times Road Construction report contributed culvert failure to "acidic soils" whereas results indicated capacity failure from an underdesigned culvert. A complete list of the 12 foot CMP culvert performance is presented in Table 4.16 emphasizing the conclusion of the 12 foot CMP culvert was underdesigned for the White River crossing.

Table 4.16: 12 foot CMP culvert results for all tested return periods

Return Period	Peak Discharge (cfs)	Culvert Discharge (cfs)	Overtopping Discharge (cfs)	Headwater Depth (ft)	Road Water Depth (ft)	Pass/Fail
Current 10 year	2265	1935	330	23	1	FAIL
Current 25 year	3595	2033	1562	25	3	FAIL
Current 50 year	4748	2094	2654	25	3	FAIL
Current 100 year	6063	2153	3910	27	5	FAIL
Future 10 year	3360	2019	2341	24	2	FAIL
Future 25 year	5130	2113	3017	26	4	FAIL
Future 50 year	6636	2177	4459	27	5	FAIL
Future 100 year	8332	2239	6093	28	6	FAIL

Note: Discharges and depths are presented as an average between computed results. A red depth stands for water depth above the road.

Presently a 12 foot x 12 foot – side by side – box culvert is constructed for the White River crossing on BIA-Route 32 west of Pine Ridge, SD. The culvert met current design condition for the 25 year return period which were the minimum requirements for a local road or street (ADT > 100) (SDDOT, 2013). Design condition results shown in Table 4.17 illustrate the 12 foot x 12 foot – side by side – box culvert starting to overtop at the future 25 return period. The culvert conveying the current 25 year return period peak discharge but not the future 25 year return period peak discharge demonstrates the effect of future climate variability on culvert selection.

Table 4.17: 12 foot x 12 foot – side by side – box culvert results for all tested return periods

Return Period	Peak Discharge (cfs)	Culvert Discharge (cfs)	Overtopping Discharge (cfs)	Headwater Depth (ft)	Road Water Depth (ft)	Pass/Fail
Current 10 year	2265	2265	0	11	11	PASS
Current 25 year	3595	3595	0	16	6	PASS
Current 50 year	4748	4748	0	21	1	PASS
Current 100 year	6063	5195	868	24	2	FAIL
Future 10 year	3360	3360	0	15	7	PASS
Future 25 year	5130	4978	152	23	1	FAIL
Future 50 year	6636	5298	1338	24	2	FAIL
Future 100 year	8332	5547	2785	26	4	FAIL

In conclusion, the simulated hydrographs used to analyze the culverts were developed without baseflow conditions. Under the simulated conditions the 12 foot x 12 foot – side by side – box culvert outperformed the 12 foot CMP culvert for both current and future design conditions. One reason for an increased culvert discharge capacity was because the 12 foot x 12 foot – side by side – box culvert had 60% more area for flows. The 12 foot x 12 foot – side by side –box culvert meets the current SDDOT design criteria for drainage infrastructures. Research indicates future precipitation events increasing by 20% resulting in this culvert being underdesigned for the White River crossing.

4.5 Engineering Recommendations

More than just river levels and culvert capacities can be gathered from the Upper White River Subbasin Watershed model created in this study for solving engineering problems. The following two suggestions are outside the scope of this paper but have real-world applications. As stated these are suggestions and if pursued should be further studied.

The first engineering application would be water management practices on controlling peak discharges from the subbasins. Refer to Figure 4.11 for subbasin location on the Upper White River Subbasin Watershed. Figure 4.12 is the future simulated 100 year return period with the peak discharges labeled according to which subbasin contributes to the peak. The peaks could be further broken down into individual contributing flows to determine which subbasin would water management considerations. An example of a consideration would be a retention pond on the subbasin. The retention pond would increase the time of concentration of the water directly decreasing the peak discharge experienced at the culvert.

The second engineering application would be to conduct a study to find the 100 year flood elevation for a city on the Pine Ridge Reservation. The 100 year flood elevation for the city would aid engineers and contractors on construction boundaries near the White River. A 100 year flood elevation would be useful for home owners when it comes to deciding if flood insurance is a feasible option.

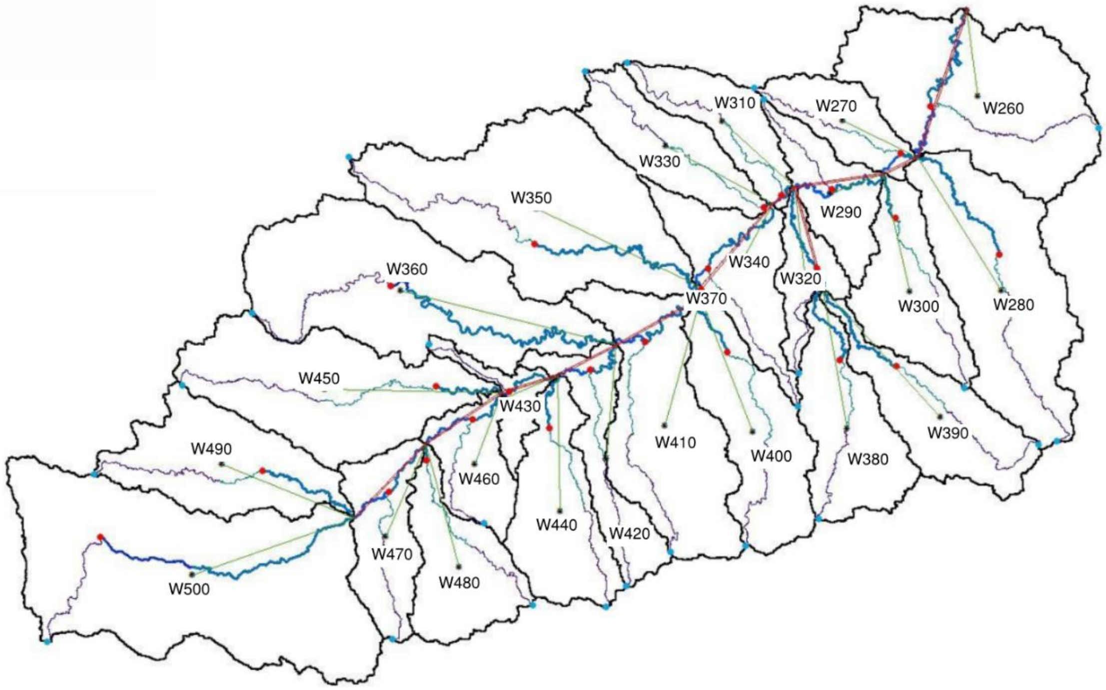


Figure 4.11: Labeled subbasins corresponding to Figure 4.11 peak discharges

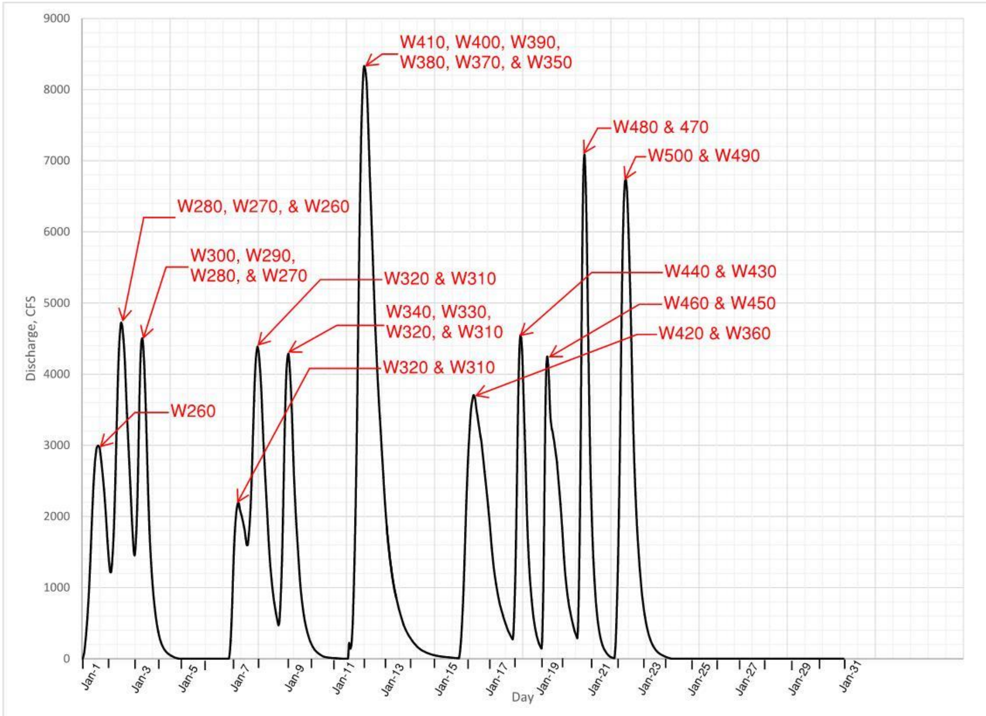


Figure 4.12: Future 100 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

5. CONCLUSIONS

The purpose of the watershed study was to evaluate the impact of future precipitation on infrastructure designed and constructed using current design condition. The objective was completed by developing simulated current and future design condition precipitation and applying the precipitation to a culvert crossing on the Upper White River Subbasin Watershed. Culverts analyzed were the past 12 foot CMP culvert which washed out in the Spring of 2010, and the currently in-place 12 foot x 12 foot – side by side – box culvert on BIA-Route 32 in SD. The following conclusions were made based on presented research:

- TR-55 was designed for small subbasins of 4,000 acres or less, by breaking the 25 subbasins in the Upper White River Subbasin Watershed it would increase the accuracy of time of concentration.
- ArcGIS and HEC-HMS are capable of accuracy processing large spatial datasets and have other applications not discussed in this study.
- Research indicates future precipitation events are expected to increase by 20% and current design conditions used to size culverts for South Dakota do not consider the increase. If not addressed this will lead to under designing future infrastructure.
- On average, an increase in 20% of annual precipitation depth results in an increase of 30% in expected future peak discharges for the 10, 25, 50, 100 year return periods on the Upper White River Watershed excluding fluctuations in baseflow at the culvert's location.

- A total of six subbasins developed in ArcGIS contributed to the peak discharge which was applied to the culvert analysis. The reason for multiple subbasins contributing over a short time period was because of a fork in the simulated drainage path of the longest flow paths.
- HY-8 and Hydraflow Express require a couple different parameters enters in order to analyze a culvert but results are consistence when compared. HY-8 requires the addition of river parameters of the tailwater conditions to perform an analysis. While Hydraflow Express for a culvert only analyzes the culvert under flow conditions.
- Simulated current peak discharges were within acceptable ranges when compared to the regression equations for Region 1 in Nebraska. Nebraska was used for comparison because the major of the Upper White River Subbasin Watershed is located in Nebraska. Region 1 regression equations used to estimate the Upper White River Subbasin Watershed Model river level upper limits had a standard error of 98 – 102 % (USGS 1994, 104). Percent error were 23%, 40%, 50%, and 59% for the return periods of 10, 25, 50, and 100 years respectively; therefore were acceptable for Region 1.
- Analysis of the 12 foot CMP culvert failed under current and future design conditions for tested return periods. The overtopping ranged from 1 to 5 feet for current design conditions and 2 to 6 feet for future design conditions. The 12 foot CMP culvert failed to convey the current 25 year return period stated by the SDDOT; raising the question if the culvert was ever properly designed?

- The 12 foot CMP culvert washout could have been a result of the culvert being underdesigned for the river crossing in addition to the acidic soils. A more extensive study would need to be conducted for definitive conclusion.
- Analysis of the 12 foot x 12 foot – side by side – box culvert conveyed the current 25 year return design condition when tested. Overtopping occurred for the future 25 year return design condition by 0.6 feet indicating the SDDOT design criteria need to be updated to meet future demands.
- The 12 foot x 12 foot – side by side – box culvert conveying peak discharges of the current 25 year return design condition and not the future 25 year return design condition indicates future climate effects culvert selection and sizing.

6. RECOMMENDATIONS

Despite the fact of an extensive study of impacts of future precipitation on infrastructure designed and constructed using current design condition further research of some parameters is recommended. Further research recommendations are to be conducted independently or in conjunction with one another on the Upper White River Subbasin Watershed model. Additional research topics include but are not limited to:

- To increase accuracy of runoff time of concentration the subbasins need to be smaller than or equal to 4,000 acres.
- Stream gauges could be used to calibrate the meteorological datasets used in HEC-HMS if stream gauges are within the study area or near. Stream gauges would allow the user to model precipitation events based on site specific data inside of interpolated statistical datasets. Site specific data would result give more accurate results.
- A survey should be conducted of the culvert of interest prior to modeling. The survey would compose of but not limited to: cross sections upstream and downstream of the culvert, inlet and outlet elevations, crest length and width, visual inspection of river conditions for Manning's number estimation, and determining culvert type and dimensions.
- Model verification should be conducted for future projects by running known discharges and precipitation events over the Upper White River Subbasin Watershed model. The verification would require finite modification to but not limited to: CN values, soil properties, precipitation data, time of concentration, river lag times, channel slope and shape, and water storage capabilities. Due to

the size of the watershed, amount of unknown variables, and time constraints this verification was unable to be performed and a worst case scenario was modeled.

7. APPENDIX

7.1 Appendix A

Table 7.1: Interpolated IDF data (Yarnell, 1935)

X-axis	Y-axis						
Duration(min)	Precipitation intensity (in/hr)						
	1 yr	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
5		3.96	5.16	5.40	6.60	7.56	8.04
10		3.06	4.08	4.86	5.40	5.88	6.42
15		2.48	3.52	3.76	4.48	5.04	6.24
30		1.74	2.50	3.00	3.46	3.96	4.50
60		1.02	1.36	1.74	2.02	2.50	2.98
120		0.59	0.75	0.94	1.27	1.55	1.75
Duration(hrs)							
2			0.750	0.935	1.265	1.550	1.750
4			0.500	0.625	0.750	0.875	1.000
8			0.281	0.338	0.406	0.469	0.531
16			0.156	0.181	0.219	0.250	0.281
24			0.104	0.125	0.146	0.167	0.188

NOAA Atlas 14, Volume 8, Version 2
Location name: Pine Ridge, South Dakota, US*
Latitude: 43.0259°, Longitude: -102.5563°
Elevation: 3238 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

AMS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Annual exceedance probability (1/years)								
	1/2	1/5	1/10	1/25	1/50	1/100	1/200	1/500	1/1000
5-min	3.76 (2.90-4.87)	5.21 (4.01-6.77)	6.35 (4.86-8.27)	7.90 (5.87-10.7)	9.11 (6.64-12.6)	10.4 (7.32-14.5)	11.6 (7.93-16.7)	13.4 (8.80-19.7)	14.7 (9.46-22.0)
10-min	2.75 (2.12-3.56)	3.82 (2.94-4.95)	4.65 (3.56-6.06)	5.78 (4.30-7.80)	6.67 (4.86-9.12)	7.58 (5.36-10.6)	8.51 (5.80-12.2)	9.79 (6.44-14.4)	10.8 (6.92-16.1)
15-min	2.24 (1.73-2.90)	3.10 (2.39-4.03)	3.78 (2.90-4.92)	4.70 (3.50-6.34)	5.42 (3.95-7.42)	6.16 (4.36-8.62)	6.92 (4.72-9.95)	7.96 (5.24-11.7)	8.76 (5.63-13.1)
30-min	1.54 (1.19-1.99)	2.11 (1.63-2.74)	2.56 (1.96-3.33)	3.17 (2.35-4.27)	3.64 (2.65-4.98)	4.13 (2.92-5.77)	4.63 (3.15-6.65)	5.30 (3.49-7.83)	5.83 (3.75-8.72)
60-min	0.958 (0.740-1.24)	1.28 (0.985-1.66)	1.54 (1.18-2.01)	1.91 (1.43-2.58)	2.20 (1.61-3.02)	2.51 (1.78-3.50)	2.84 (1.94-4.09)	3.29 (2.17-4.88)	3.65 (2.35-5.47)
2-hr	0.574 (0.448-0.734)	0.752 (0.584-0.966)	0.900 (0.697-1.16)	1.12 (0.844-1.50)	1.29 (0.956-1.76)	1.48 (1.06-2.06)	1.68 (1.16-2.41)	1.97 (1.31-2.89)	2.20 (1.42-3.25)
3-hr	0.414 (0.325-0.527)	0.534 (0.418-0.682)	0.638 (0.497-0.818)	0.792 (0.606-1.07)	0.922 (0.688-1.25)	1.06 (0.768-1.48)	1.22 (0.846-1.73)	1.43 (0.962-2.10)	1.61 (1.05-2.37)
6-hr	0.237 (0.188-0.299)	0.305 (0.241-0.385)	0.364 (0.287-0.462)	0.454 (0.351-0.606)	0.530 (0.399-0.713)	0.613 (0.447-0.844)	0.703 (0.494-0.993)	0.832 (0.564-1.21)	0.939 (0.618-1.37)
12-hr	0.135 (0.109-0.169)	0.178 (0.142-0.222)	0.214 (0.170-0.268)	0.266 (0.207-0.349)	0.309 (0.235-0.410)	0.355 (0.261-0.482)	0.405 (0.287-0.564)	0.475 (0.324-0.680)	0.531 (0.353-0.767)
24-hr	0.079 (0.064-0.097)	0.104 (0.084-0.128)	0.124 (0.100-0.154)	0.153 (0.120-0.198)	0.177 (0.135-0.232)	0.202 (0.150-0.271)	0.228 (0.163-0.314)	0.265 (0.183-0.375)	0.295 (0.198-0.421)
2-day	0.046 (0.038-0.056)	0.058 (0.048-0.071)	0.069 (0.056-0.084)	0.084 (0.066-0.107)	0.096 (0.074-0.124)	0.109 (0.082-0.144)	0.123 (0.088-0.167)	0.142 (0.099-0.199)	0.158 (0.107-0.223)
3-day	0.033 (0.028-0.041)	0.042 (0.034-0.051)	0.049 (0.040-0.060)	0.059 (0.047-0.075)	0.067 (0.052-0.086)	0.076 (0.057-0.100)	0.085 (0.062-0.115)	0.098 (0.069-0.137)	0.109 (0.074-0.153)
4-day	0.027 (0.022-0.032)	0.033 (0.027-0.040)	0.039 (0.032-0.047)	0.046 (0.037-0.058)	0.052 (0.041-0.067)	0.059 (0.045-0.077)	0.066 (0.048-0.088)	0.076 (0.053-0.105)	0.084 (0.057-0.117)
7-day	0.018 (0.015-0.021)	0.022 (0.018-0.026)	0.025 (0.021-0.031)	0.030 (0.024-0.037)	0.034 (0.027-0.043)	0.038 (0.029-0.049)	0.042 (0.031-0.055)	0.047 (0.033-0.065)	0.052 (0.035-0.071)
10-day	0.014 (0.012-0.017)	0.017 (0.015-0.021)	0.020 (0.017-0.024)	0.024 (0.019-0.029)	0.026 (0.021-0.033)	0.029 (0.022-0.038)	0.032 (0.024-0.043)	0.036 (0.026-0.049)	0.040 (0.027-0.054)
20-day	0.009 (0.008-0.011)	0.012 (0.010-0.014)	0.013 (0.011-0.016)	0.016 (0.013-0.019)	0.018 (0.014-0.022)	0.020 (0.015-0.025)	0.021 (0.016-0.028)	0.024 (0.017-0.032)	0.026 (0.018-0.035)
30-day	0.008 (0.006-0.009)	0.009 (0.008-0.011)	0.011 (0.009-0.013)	0.013 (0.010-0.015)	0.014 (0.011-0.017)	0.016 (0.012-0.020)	0.017 (0.013-0.022)	0.019 (0.014-0.025)	0.020 (0.014-0.027)
45-day	0.006 (0.005-0.007)	0.008 (0.007-0.009)	0.009 (0.008-0.010)	0.010 (0.008-0.012)	0.011 (0.009-0.014)	0.012 (0.010-0.015)	0.013 (0.010-0.017)	0.015 (0.011-0.019)	0.015 (0.011-0.021)
60-day	0.005 (0.005-0.006)	0.007 (0.006-0.008)	0.008 (0.007-0.009)	0.009 (0.007-0.010)	0.010 (0.008-0.012)	0.010 (0.008-0.013)	0.011 (0.008-0.014)	0.012 (0.009-0.016)	0.012 (0.009-0.017)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of annual maxima series (AMS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and annual exceedance probability) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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Figure 7.1: Current annual precipitation intensity estimates for Pine Ridge, SD (HDSC webmaster, 2014)

7.2 Appendix B

Table 7.2: Design years for drainage appurtenances (SDDOT 2013, 7-10)

Highway Classifications	Return Period (Years) for Drainage Appurtenances					
	Bridge Waterway Openings		Roadway Cross Culverts	Storm Drainage Systems	Roadside Ditches	
	Design Headwater	Scour	Design Headwater	Inlet Spacing & Trunk Line	Design Headwater	Permanent Erosion Protection
Interstate	50	100	50	10	50	50
US & State Highways	25	100	25	10	25	25
Local Roads & Streets(7) (ADT > 100)	25	100	25	10	25	25
Local Roads & Streets (ADT < 100)	10	100	10	10	10	10

Notes:
1(a) The allowable design headwater elevation should not exceed 1 ft below the low subgrade shoulder at the lowest point of the roadway within the drainage basin. For NFIP-mapped floodplains, see Section 7.6.2.2.
1(b) The review flood frequency should be 100 years. The headwater elevation for the review flood should not overtop the highway for Interstate and NHS highways. In addition, bridges should not raise backwater more than 1 ft above existing conditions and should provide for at least 2 ft of free board below the lowest superstructure element. For smaller crossings (typically smaller than 1000 acres) on non-NHS highways, review the impacts of the 100-year flood, but overtopping is allowed; however, sensitive sites (urban areas, nearby homes or farmsteads, etc.) may require further analysis regarding the effects of overtopping. Where there is development near a cross culvert, the 100-year event should be reviewed to evaluate the potential impacts.
1(c) Ramps should be designed for the lower design frequency of the two intersecting highways, but reviewed with the higher review frequency of the two intersecting highways.
1(d) Approaches should be designed for the 10-year flood and reviewed with the review frequency of the highway. The design headwater elevation should not overtop the approach. Approaches should be designed to meet the roadside ditch criteria for the highway. Approaches that do not serve a house may be designed for less than the 10-year flood.
2 In addition to the 100-year or worst case up to the 100-year flood, bridge foundations should be evaluated for scour at the super flood (500-year event or that which produces the maximum scour up to the 500-year event) so that the resulting ratio of ultimate to applied loads is greater than 1.0.
3 If a storm drain provides the outlet for a cross drain, then the design frequency of the cross drain should be used for the storm drain system downstream from the cross drain inlet.
4 If local drainage facilities and practices have provided storm drains of lesser standard, to which the highway system should connect, provide special consideration to whether it is realistic to design the highway system to a higher standard than available outlets.
5 Roadway inlets should be designed to meet the spread criteria in Section 12.7.3. Bridge deck inlets should be designed to meet the spread criteria in Section 14.7.3.6.
6 For certain sag points, the design frequency should be 50 years. See Section 12.5.2.
7 Local roads and streets are considered to be facilities that are not Interstate, US or State highways. For local facilities not eligible for Federal-aid funds, use the design frequencies for ADT < 100.

7.3 Appendix C

Precipitation Frequency Data Server http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_printpage.html?lat=43.0259...

NOAA Atlas 14, Volume 8, Version 2
Location name: Pine Ridge, South Dakota, US*
Latitude: 43.0259°, Longitude: -102.5563°
Elevation: 3238 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffrey Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

AMS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Annual exceedance probability (1/years)								
	1/2	1/5	1/10	1/25	1/50	1/100	1/200	1/500	1/1000
5-min	0.313 (0.242-0.406)	0.434 (0.334-0.564)	0.529 (0.405-0.689)	0.658 (0.489-0.888)	0.759 (0.553-1.04)	0.863 (0.610-1.21)	0.969 (0.661-1.39)	1.11 (0.733-1.65)	1.23 (0.788-1.83)
10-min	0.459 (0.354-0.594)	0.636 (0.490-0.825)	0.775 (0.593-1.01)	0.964 (0.717-1.30)	1.11 (0.810-1.52)	1.26 (0.893-1.77)	1.42 (0.967-2.04)	1.63 (1.07-2.41)	1.80 (1.15-2.69)
15-min	0.560 (0.432-0.725)	0.776 (0.597-1.01)	0.945 (0.724-1.23)	1.18 (0.874-1.59)	1.36 (0.988-1.85)	1.54 (1.09-2.16)	1.73 (1.18-2.49)	1.99 (1.31-2.94)	2.19 (1.41-3.28)
30-min	0.770 (0.595-0.997)	1.06 (0.813-1.37)	1.28 (0.980-1.67)	1.58 (1.18-2.13)	1.82 (1.33-2.49)	2.06 (1.46-2.89)	2.31 (1.58-3.33)	2.65 (1.75-3.92)	2.92 (1.87-4.36)
60-min	0.958 (0.740-1.24)	1.28 (0.985-1.66)	1.54 (1.18-2.01)	1.91 (1.43-2.59)	2.20 (1.61-3.02)	2.51 (1.78-3.53)	2.84 (1.94-4.09)	3.29 (2.17-4.88)	3.65 (2.35-5.47)
2-hr	1.15 (0.895-1.47)	1.50 (1.17-1.93)	1.80 (1.39-2.32)	2.23 (1.69-3.01)	2.58 (1.91-3.52)	2.96 (2.12-4.13)	3.37 (2.33-4.82)	3.93 (2.62-5.78)	4.39 (2.85-6.50)
3-hr	1.24 (0.976-1.58)	1.60 (1.25-2.05)	1.92 (1.49-2.46)	2.38 (1.82-3.20)	2.77 (2.06-3.76)	3.19 (2.31-4.44)	3.65 (2.54-5.21)	4.30 (2.89-6.30)	4.84 (3.15-7.13)
6-hr	1.42 (1.13-1.79)	1.83 (1.44-2.31)	2.18 (1.72-2.77)	2.72 (2.10-3.62)	3.17 (2.39-4.27)	3.67 (2.68-5.05)	4.21 (2.96-5.95)	4.98 (3.38-7.23)	5.62 (3.70-8.19)
12-hr	1.63 (1.31-2.03)	2.14 (1.71-2.68)	2.57 (2.05-3.23)	3.20 (2.49-4.20)	3.72 (2.83-4.94)	4.28 (3.15-5.81)	4.87 (3.45-6.80)	5.72 (3.91-8.19)	6.40 (4.25-9.24)
24-hr	1.89 (1.53-2.33)	2.49 (2.01-3.08)	2.98 (2.39-3.70)	3.68 (2.88-4.76)	4.24 (3.25-5.56)	4.84 (3.59-6.49)	5.48 (3.91-7.54)	6.36 (4.38-9.00)	7.08 (4.74-10.1)
2-day	2.21 (1.81-2.69)	2.81 (2.29-3.43)	3.30 (2.68-4.05)	4.01 (3.18-5.14)	4.60 (3.56-5.96)	5.22 (3.91-6.93)	5.88 (4.25-8.02)	6.82 (4.74-9.54)	7.57 (5.12-10.7)
3-day	2.41 (1.99-2.92)	3.01 (2.47-3.66)	3.51 (2.87-4.29)	4.23 (3.38-5.38)	4.83 (3.76-6.21)	5.46 (4.11-7.19)	6.12 (4.45-8.29)	7.07 (4.95-9.83)	7.83 (5.32-11.0)
4-day	2.57 (2.13-3.10)	3.19 (2.63-3.86)	3.70 (3.04-4.50)	4.44 (3.55-5.61)	5.03 (3.94-6.44)	5.67 (4.29-7.43)	6.34 (4.62-8.54)	7.28 (5.11-10.1)	8.03 (5.49-11.2)
7-day	2.99 (2.49-3.57)	3.69 (3.08-4.43)	4.26 (3.53-5.13)	5.05 (4.06-6.29)	5.67 (4.46-7.17)	6.32 (4.81-8.18)	6.99 (5.13-9.31)	7.92 (5.60-10.8)	8.65 (5.96-12.0)
10-day	3.36 (2.83-4.00)	4.17 (3.49-4.97)	4.81 (4.00-5.75)	5.67 (4.58-7.01)	6.35 (5.02-7.96)	7.04 (5.39-9.05)	7.76 (5.71-10.3)	8.73 (6.19-11.9)	9.48 (6.56-13.1)
20-day	4.48 (3.81-5.26)	5.61 (4.75-6.61)	6.48 (5.45-7.66)	7.63 (6.22-9.30)	8.52 (6.79-10.5)	9.41 (7.26-11.9)	10.3 (7.66-13.4)	11.5 (8.24-15.4)	12.4 (8.68-17.0)
30-day	5.46 (4.67-6.38)	6.84 (5.83-8.01)	7.88 (6.68-9.26)	9.23 (7.56-11.2)	10.3 (8.22-12.6)	11.3 (8.74-14.2)	12.3 (9.16-15.9)	13.6 (9.78-18.1)	14.6 (10.2-19.8)
45-day	6.76 (5.82-7.83)	8.42 (7.22-9.78)	9.63 (8.21-11.2)	11.2 (9.17-13.3)	12.3 (9.90-14.9)	13.4 (10.4-16.6)	14.4 (10.8-18.5)	15.7 (11.4-20.7)	16.7 (11.8-22.5)
60-day	7.90 (6.83-9.11)	9.77 (8.42-11.3)	11.1 (9.52-12.9)	12.7 (10.5-15.1)	13.9 (11.2-16.7)	15.0 (11.7-18.5)	16.0 (12.0-20.3)	17.2 (12.4-22.5)	18.0 (12.8-24.1)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of annual maxima series (AMS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and annual exceedance probability) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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Figure 7.2: Current annual precipitation depths estimates for Pine Ridge, SD (HDSC webmaster, 2014)

7.4 Appendix D

Region 1:

$$Q10 = 67.19Ac^{0.737}(P-13)^{1.149}L^{-0.608}$$

$$Q25 = 222.93Ac^{0.690}(P-13)^{0.905}L^{-0.573}$$

$$Q50 = 490.86Ac^{0.656}(P-13)^{0.742}L^{-0.543}$$

$$Q100 = 996.78Ac^{0.624}(P-13)^{0.588}L^{-0.512}$$

Variables:

Q = Estimated peak discharge at given return period, CFS

Ac = contributing drainage area, mi^2

P = mean annual precipitation, inches

L = main stream length, mi

Constant Values:

Ac = 1424 mi^2 (Table 7.3)

P = 17 inches (USGS 1994, 106)

L = 180 miles (Table 7.4)

$$L = W500 + R240 + R210 + R190 + R170 + R120 + R110 + R80 + R40 + R30 + R20 + R10$$

$$L = 97 + 8 + 7 + 5 + 8 + 11 + 1 + 10 + 3 + 10 + 3 + 17 = 180\ miles$$

$$Q10 = 67.19(1424)^{0.737}(17-13)^{1.149}(180)^{-0.608} = 2{,}960\ CFS$$

$$Q25 = 222.93(1424)^{0.690}(17-13)^{0.905}(180)^{-0.573} = 5{,}980\ CFS$$

$$Q50 = 490.86(1424)^{0.656}(17-13)^{0.742}(180)^{-0.543} = 9{,}589\ CFS$$

$$Q100 = 996.78(1424)^{0.624}(17-13)^{0.588}(180)^{-0.512} = 14{,}645\ CFS$$

7.5 Appendix E

Table 7.3: Modified attribute table of ArcGIS Subbasins

OBJECTID	Shape_Length (m)	Shape_Area (m^2)	Name	Slope (%)	BasinCN	Tc (hrs)	Area_HMS (mi^2)
1	112690	266647120	W260	8	84	24	103
2	58454	67638566	W270	6	87	16	26
3	113014	237237904	W280	11	81	21	92
4	69809	78437491	W290	5	82	15	30
5	78893	115513338	W300	9	78	13	45
6	66619	79520089	W310	6	88	23	31
7	64834	50370374	W320	7	76	10	19
8	74405	101810918	W330	5	88	21	39
9	91168	111753122	W340	6	80	15	43
10	130804	317434159	W350	5	88	37	123
11	138807	302056308	W360	4	85	32	117
12	10761	1386689	W370	2	71	2	1
13	86031	127693835	W380	14	85	14	49
14	93818	124993556	W390	11	74	21	48
15	90195	148385988	W400	11	87	17	57
16	114961	211186128	W410	9	84	22	82
17	96197	85164067	W420	9	84	17	33
18	68025	36551726	W430	3	81	10	14
19	83003	130506688	W440	13	84	13	50
20	115447	223571302	W450	10	82	22	86
21	59103	70798639	W460	6	84	8	27
22	73594	93407449	W470	10	87	10	36
23	78191	110694648	W480	13	84	12	43
24	100307	153878652	W490	15	77	16	59
25	156273	440444537	W500	14	81	18	170

Table 7.4: Modified attribute table of ArcGIS River

OBJECTID	Shape_ Length (m)	Slope (m/m)	ElevUP (m)	ElevDS (m)	RivLen (m)	Name	ElevUP_ HMS (ft)	ElevDS_ HMS (ft)	RivLen_ HMS (ft)
1	26615	0.035	935.87	0.00	26615	R10	3070.44	0.00	87319
2	5441	0.002	945.09	935.87	5441	R20	3100.68	3070.44	17850
3	15784	0.001	957.83	945.09	15784	R30	3142.49	3100.68	51784
4	4038	0.002	965.19	957.83	4038	R40	3166.64	3142.49	13249
5	996	0.005	969.79	965.19	996	R50	3181.74	3166.64	3269
6	5962	0.003	964.31	945.09	5962	R60	3163.73	3100.68	19559
7	18366	0.004	1010.09	935.87	18366	R70	3313.96	3070.44	60256
8	16819	0.001	985.59	965.19	16819	R80	3233.57	3166.64	55181
9	13895	0.003	999.88	957.83	13895	R90	3280.45	3142.49	45589
10	27810	0.002	1033.81	985.59	27810	R100	3391.75	3233.57	91239
11	998	0.003	988.10	985.59	998	R110	3241.79	3233.57	3273
12	17841	0.001	1005.18	988.10	17841	R120	3297.83	3241.79	58533
13	40350	0.002	1075.38	1005.18	40350	R130	3528.14	3297.83	132383
14	7756	0.005	1023.07	988.10	7756	R140	3356.52	3241.79	25448
15	11612	0.005	1055.15	999.88	11612	R150	3461.77	3280.45	38096
16	14232	0.005	1077.18	999.88	14232	R160	3534.06	3280.45	46694
17	12277	0.002	1027.73	1005.18	12277	R170	3371.83	3297.83	40281
18	9169	0.003	1076.03	1047.94	9169	R180	3530.27	3438.12	30083
19	8079	0.003	1047.94	1027.73	8079	R190	3438.12	3371.83	26505
20	7823	0.004	1061.02	1027.73	7823	R200	3481.05	3371.83	25665
21	11879	0.004	1091.78	1047.94	11879	R210	3581.96	3438.12	38974
22	1793	0.006	1102.68	1091.78	1793	R220	3617.71	3581.96	5884
23	13841	0.007	1240.46	1146.69	13841	R230	4069.76	3762.12	45409
24	12315	0.004	1146.69	1091.78	12315	R240	3762.12	3581.96	40403
25	32924	0.006	1351.66	1146.69	32924	R250	4434.57	3762.12	108019

7.6 Appendix F

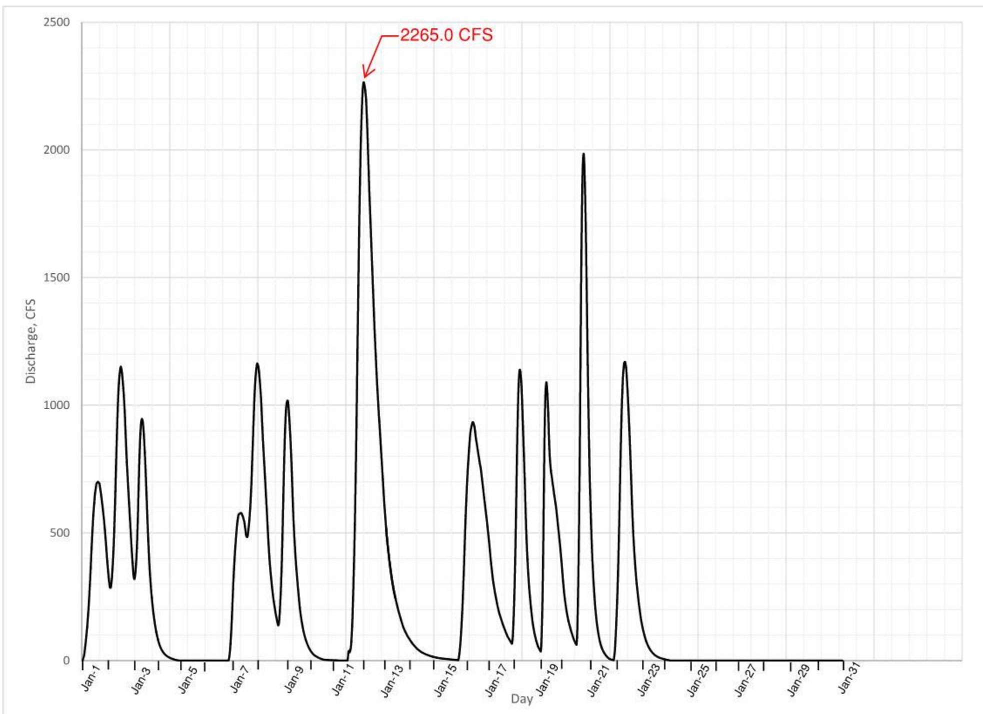

Figure 7.3: Current 10 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

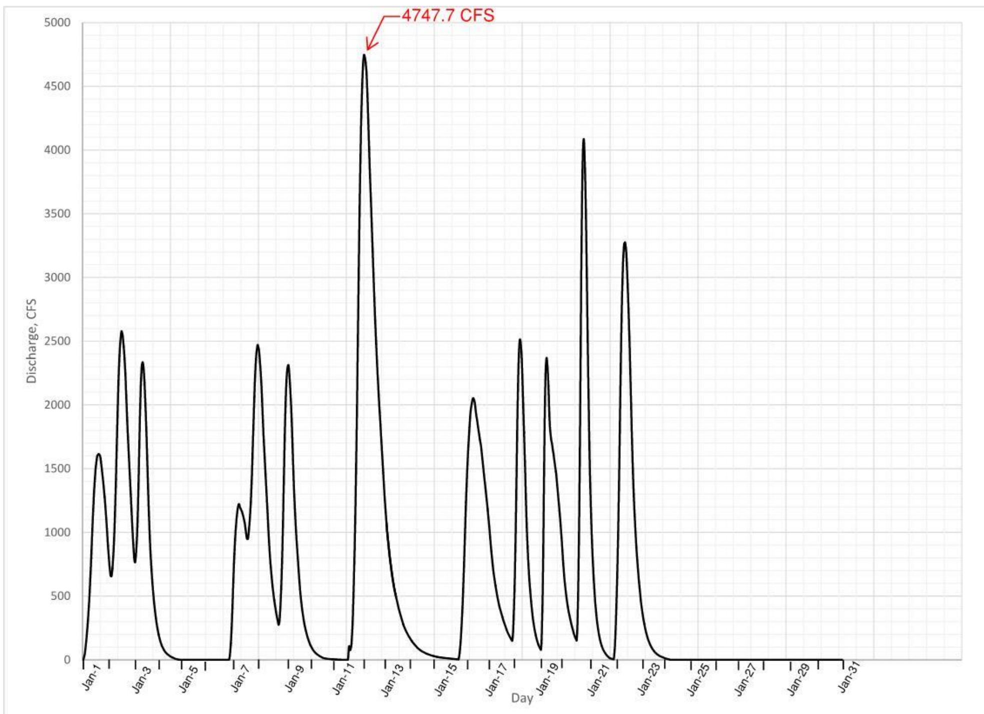

Figure 7.4: Current 50 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

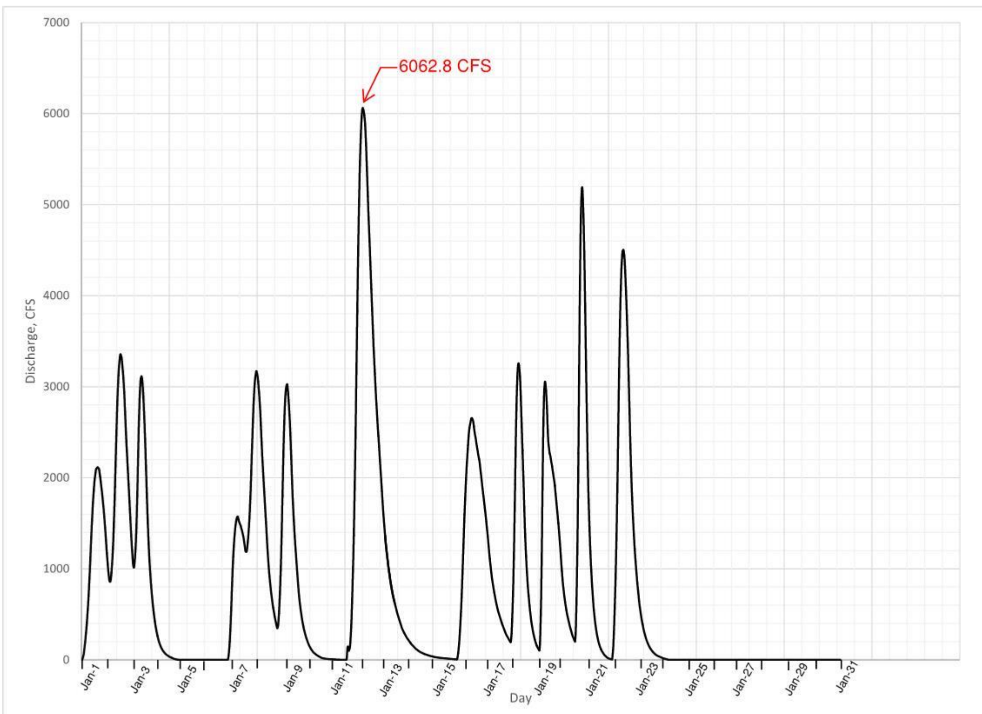

Figure 7.5: Current 100 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

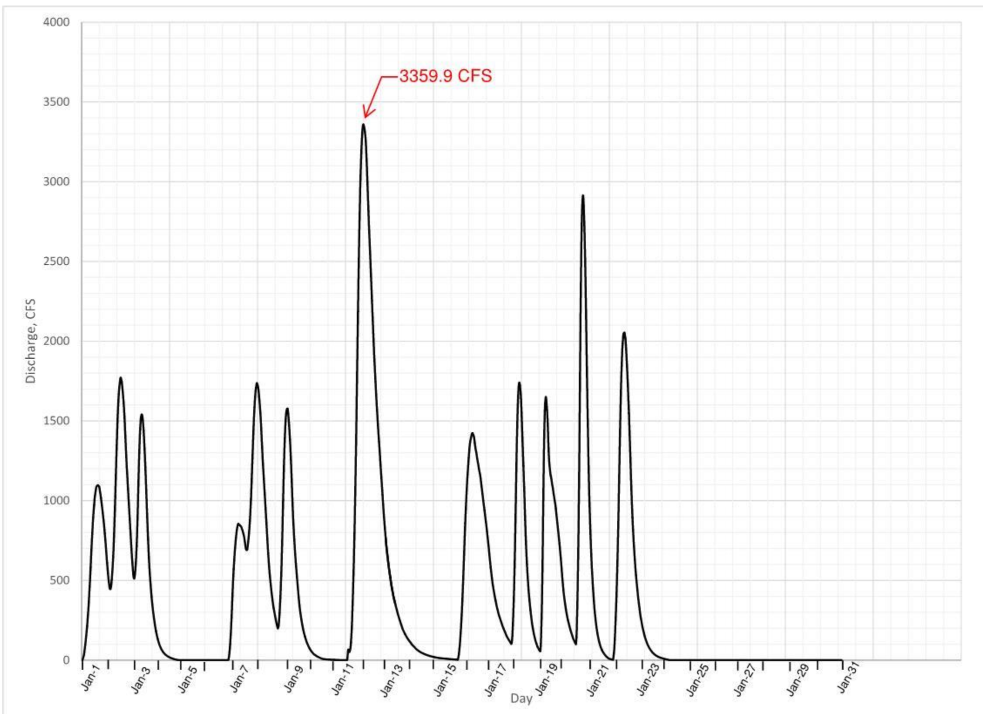

Figure 7.6: Future 10 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

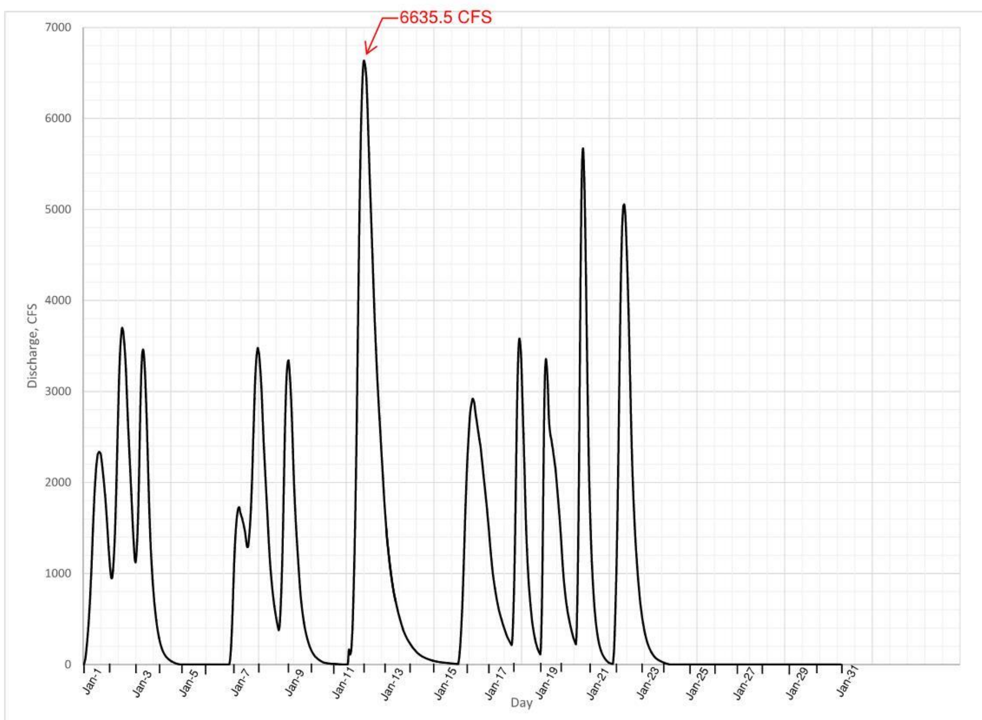

Figure 7.7: Future 50 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

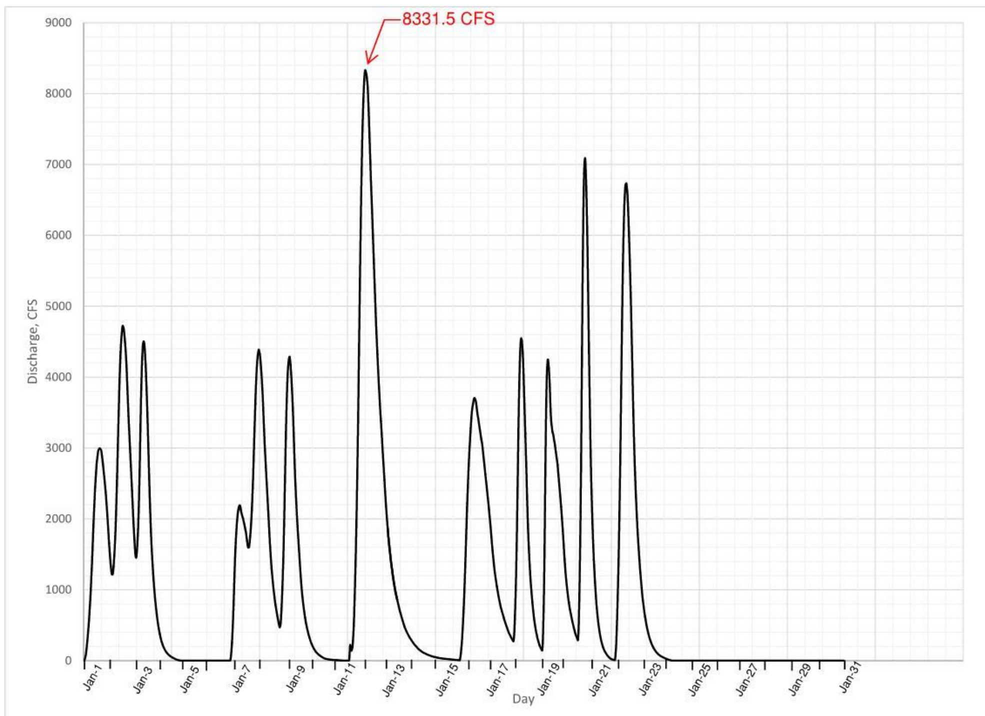

Figure 7.8: Future 100 year return period, 15 min intensity duration simulated hydrograph excluding baseflow

7.7 Appendix G

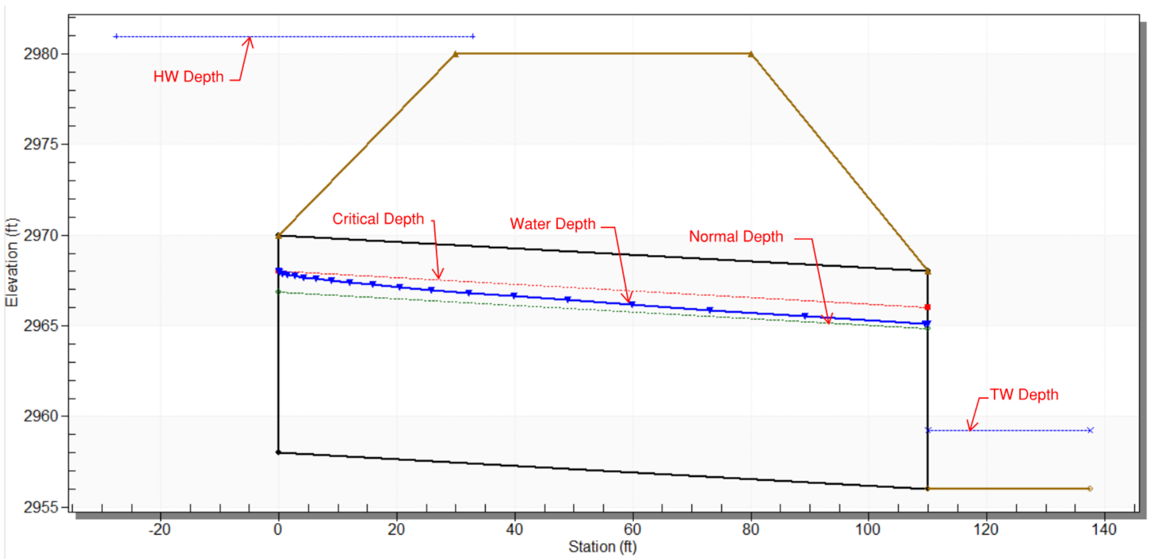

Figure 7.9: HY-8 analyzed 12 foot CMP culvert under current design, 10 year return period 15 min intensity duration simulated flow conditions

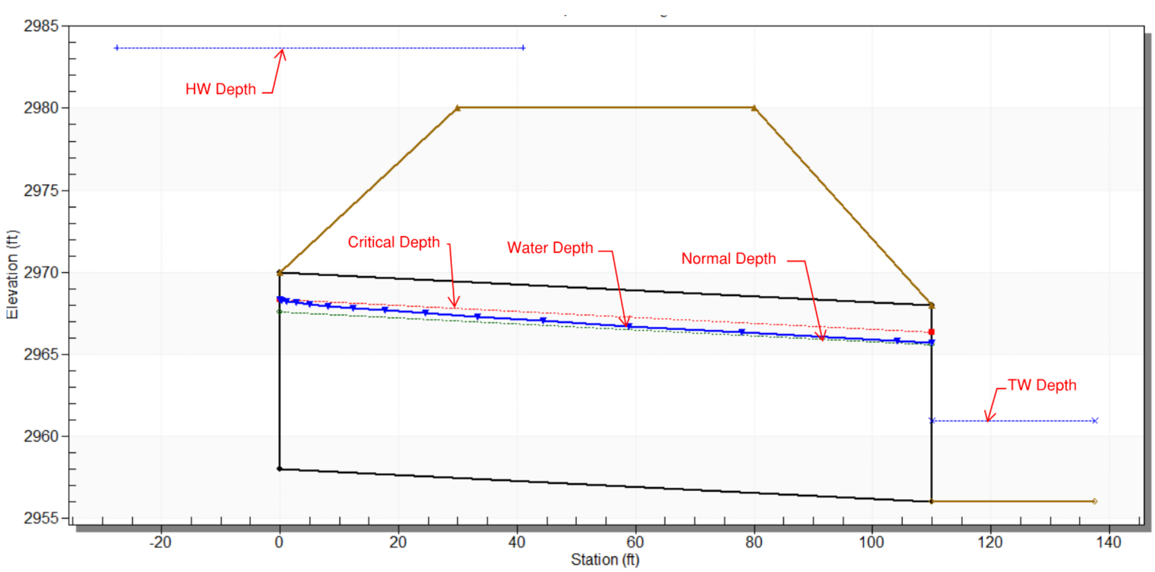

Figure 7.10: HY-8 analyzed 12 foot CMP culvert under current design, 50 year return period 15 min intensity duration simulated flow conditions

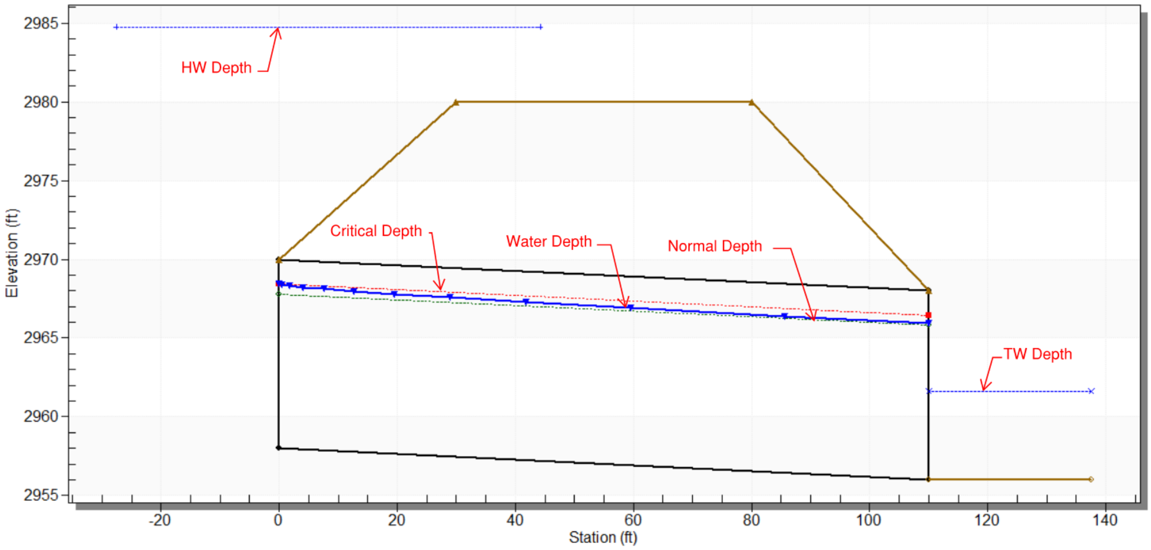

Figure 7.11: HY-8 analyzed 12 foot CMP culvert under current design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.5: Current 12 foot CMP culvert and river parameters generated by HY-8 Part 2

10 year return period					
Culvert		Units	Downriver		Units
Total Discharge	2265.00	cfs	Flow	2265.00	cfs
Culvert Discharge	1917.71	cfs	Tailwater Elevation	2959.25	ft
Overtopping Discharge	347.13	cfs	Depth	3.25	ft
Headwater Elevation	2980.94	ft	Velocity	12.97	ft/s
Inlet Control Depth	22.94	ft	Shear	3.69	psf
Flow Type	5-S2n		Froude Number	1.38	
Normal Depth	8.86	ft			
Critical Depth	10.00	ft			
Outlet Depth	9.10	ft			
Outlet Velocity	20.88	ft/s			
50 year return period					
Culvert		Units	Downriver		Units
Total Discharge	4747.7	cfs	Flow	4747.7	cfs
Culvert Discharge	2079.89	cfs	Tailwater Elevation	2960.92	ft
Overtopping Discharge	2667.57	cfs	Depth	4.92	ft
Headwater Elevation	2983.67	ft	Velocity	16.41	ft/s
Inlet Control Depth	25.02	ft	Shear	5.59	psf
Flow Type	5-S2n		Froude Number	1.46	
Normal Depth	9.55	ft			
Critical Depth	10.34	ft			
Outlet Depth	9.68	ft			
Outlet Velocity	21.29	ft/s			
100 year return period					
Culvert		Units	Downriver		Units
Total Discharge	6062.8	cfs	Flow	6062.8	cfs
Culvert Discharge	2139.73	cfs	Tailwater Elevation	2961.63	ft
Overtopping Discharge	3922.82	cfs	Depth	5.63	ft
Headwater Elevation	2984.74	ft	Velocity	17.69	ft/s
Inlet Control Depth	26.74	ft	Shear	6.39	psf
Flow Type	5-S2n		Froude Number	1.48	
Normal Depth	9.80	ft			
Critical Depth	10.45	ft			
Outlet Depth	9.92	ft			
Outlet Velocity	21.41	ft/s			

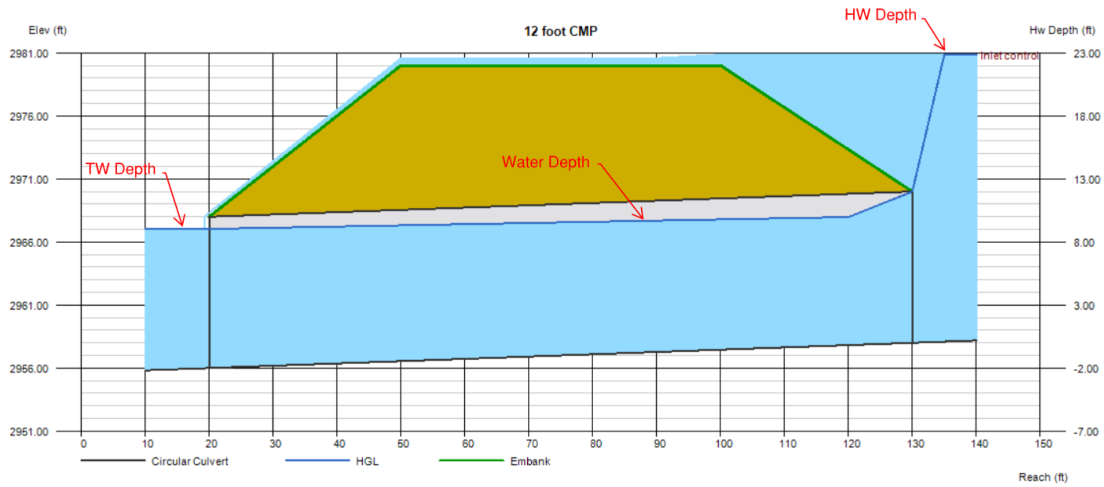

Figure 7.12: Hydraflow Express analyzed 12 foot CMP culvert under current design, 10 year return period 15 min intensity duration simulated flow conditions

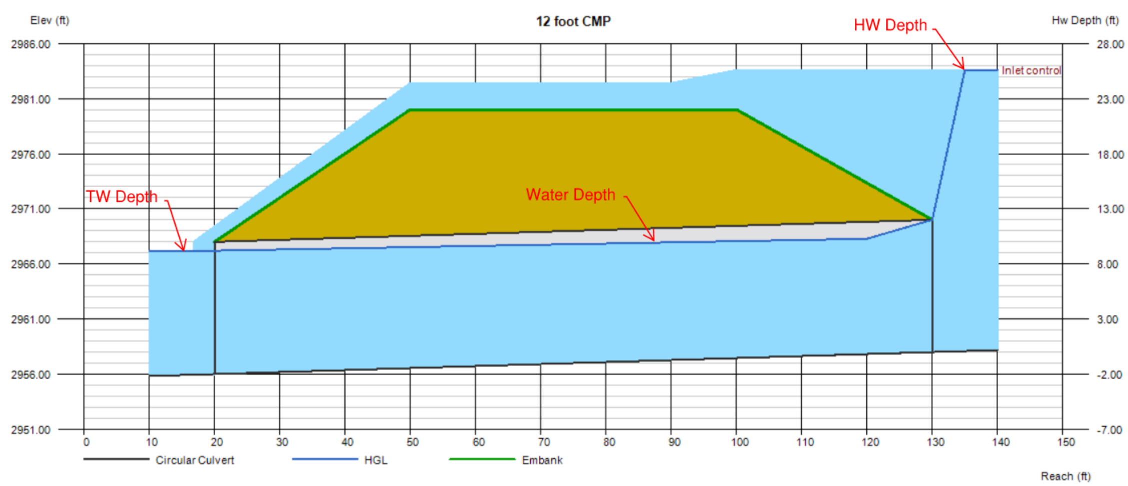

Figure 7.13: Hydraflow Express analyzed 12 foot CMP culvert under current design, 50 year return period 15 min intensity duration simulated flow conditions

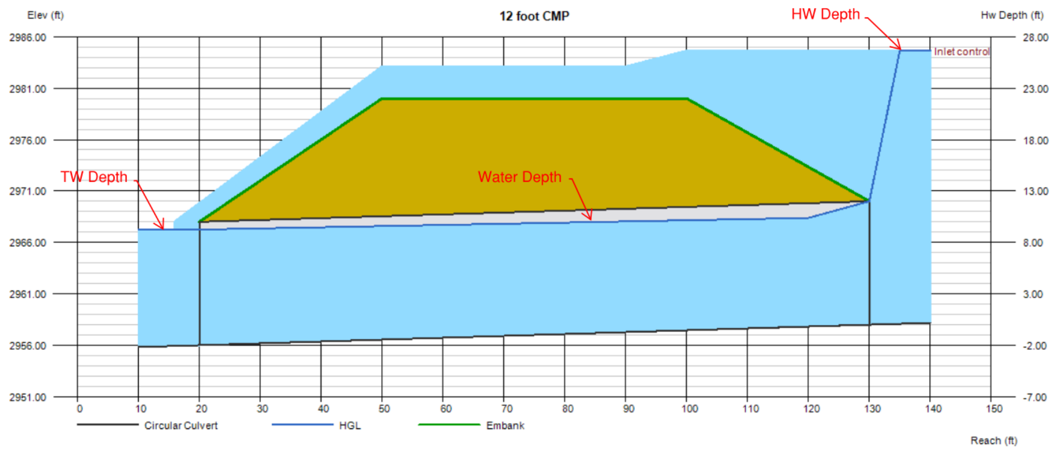

Figure 7.14: Hydraflow Express analyzed 12 foot CMP culvert under current design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.6: Current 12 foot CMP culvert parameters generated by Hydraflow Express Part 2

10 year return period				50 year return period			
Category	Parameter	Value	Units	Category	Parameter	Value	Units
Flow	Total	2265.00	cfs	Flow	Total	4747.70	cfs
	Culvert	1952.95	cfs		Culvert	2108.77	cfs
	Overtopping	312.05	cfs		Overtopping	2638.93	cfs
Velocity	Outlet	17.95	ft/s	Velocity	Outlet	19.20	ft/s
	Inlet	19.27	ft/s		Inlet	20.27	ft/s
Depth	Outlet	11.04	ft	Depth	Outlet	11.20	ft
	Inlet	10.07	ft		Inlet	10.39	ft
Hydraulic Grade Line	Outlet Elevation	2967.04	ft	Hydraulic Grade Line	Outlet Elevation	2967.20	ft
	Inlet Elevation	2968.07	ft		Inlet Elevation	2968.39	ft
	Headwater Elevation	2950.86	ft		Headwater Elevation	2983.60	ft
	HW/D	1.91			HW/D	2.13	

100 year return period			
Category	Parameter	Value	Units
Flow	Total	6062.80	cfs
	Culvert	2166.41	cfs
	Overtopping	3896.39	cfs
Velocity	Outlet	19.67	ft/s
	Inlet	20.65	ft/s
Depth	Outlet	11.25	ft
	Inlet	10.50	ft
Hydraulic Grade Line	Outlet Elevation	2967.25	ft
	Inlet Elevation	2968.50	ft
	Headwater Elevation	2984.56	ft
	HW/D	2.22	

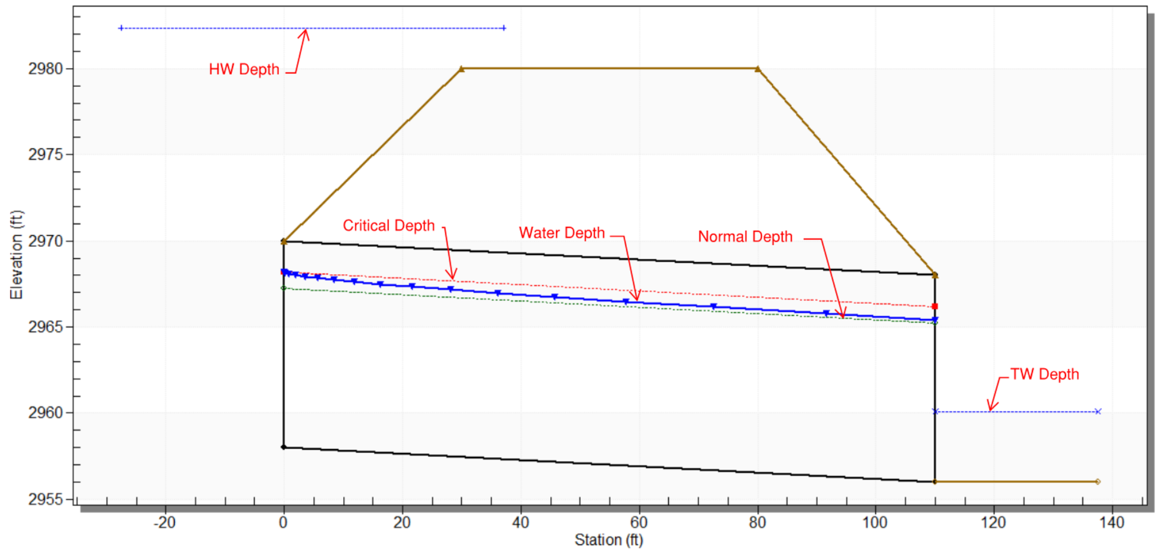

Figure 7.15: HY-8 analyzed 12 foot CMP culvert under future design, 10 year return period 15 min intensity duration simulated flow conditions

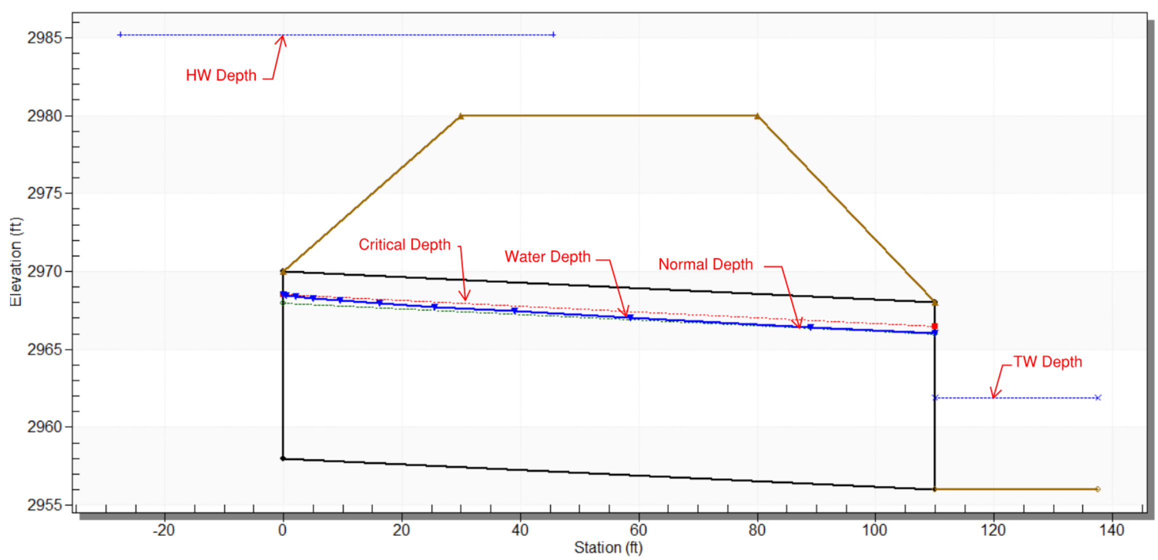

Figure 7.16: HY-8 analyzed 12 foot CMP culvert under future design, 50 year return period 15 min intensity duration simulated flow conditions

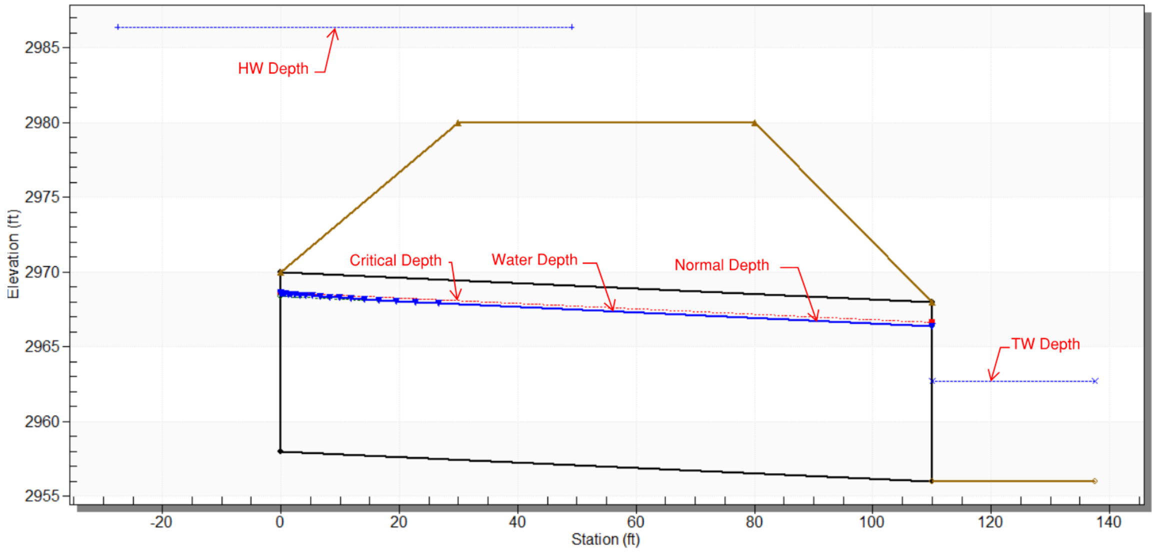

Figure 7.17: HY-8 analyzed 12 foot CMP culvert under future design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.7: Future 12 foot CMP culvert and river parameters generated by HY-8 Part 2

10 year return period					
Culvert		Units	Downriver		Units
Total Discharge	3359.90	cfs	Flow	3359.90	cfs
Culvert Discharge	2002.82	cfs	Tailwater Elevation	2960.06	ft
Overtopping Discharge	1357.01	cfs	Depth	4.06	ft
Headwater Elevation	2982.34	ft	Velocity	14.73	ft/s
Inlet Control Depth	24.38	ft	Shear	4.61	psf
Flow Type	5-S2n		Froude Number	1.42	
Normal Depth	9.22	ft			
Critical Depth	10.18	ft			
Outlet Depth	9.40	ft			
Outlet Velocity	21.12	ft/s			
50 year return period					
Culvert		Units	Downriver		Units
Total Discharge	6635.50	cfs	Flow	6635.50	cfs
Culvert Discharge	2163.24	cfs	Tailwater Elevation	2961.91	ft
Overtopping Discharge	4471.89	cfs	Depth	5.91	ft
Headwater Elevation	2985.17	ft	Velocity	18.17	ft/s
Inlet Control Depth	27.17	ft	Shear	6.72	psf
Flow Type	5-S2n		Froude Number	1.49	
Normal Depth	9.96	ft			
Critical Depth	10.50	ft			
Outlet Depth	10.03	ft			
Outlet Velocity	21.47	ft/s			
100 year return period					
Culvert		Units	Downriver		Units
Total Discharge	8331.50	cfs	Flow	8331.50	cfs
Culvert Discharge	2226.51	cfs	Tailwater Elevation	2962.69	ft
Overtopping Discharge	6104.84	cfs	Depth	6.69	ft
Headwater Elevation	2986.37	ft	Velocity	19.45	ft/s
Inlet Control Depth	28.27	ft	Shear	7.59	psf
Flow Type	5-S2n		Froude Number	1.52	
Normal Depth	10.41	ft			
Critical Depth	10.61	ft			
Outlet Depth	10.41	ft			
Outlet Velocity	21.43	ft/s			

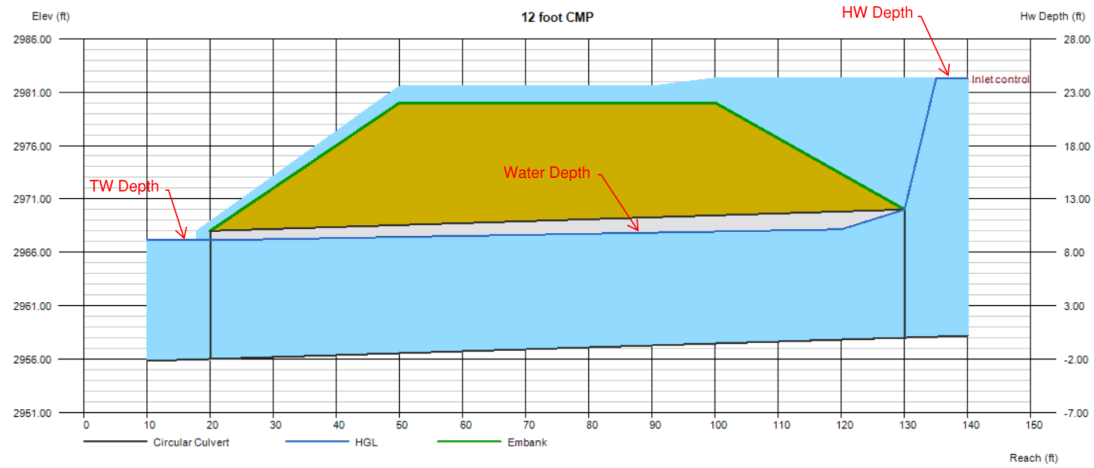

Figure 7.18: Hydraflow Express analyzed 12 foot CMP culvert under future design, 10 year return period 15 min intensity duration simulated flow conditions

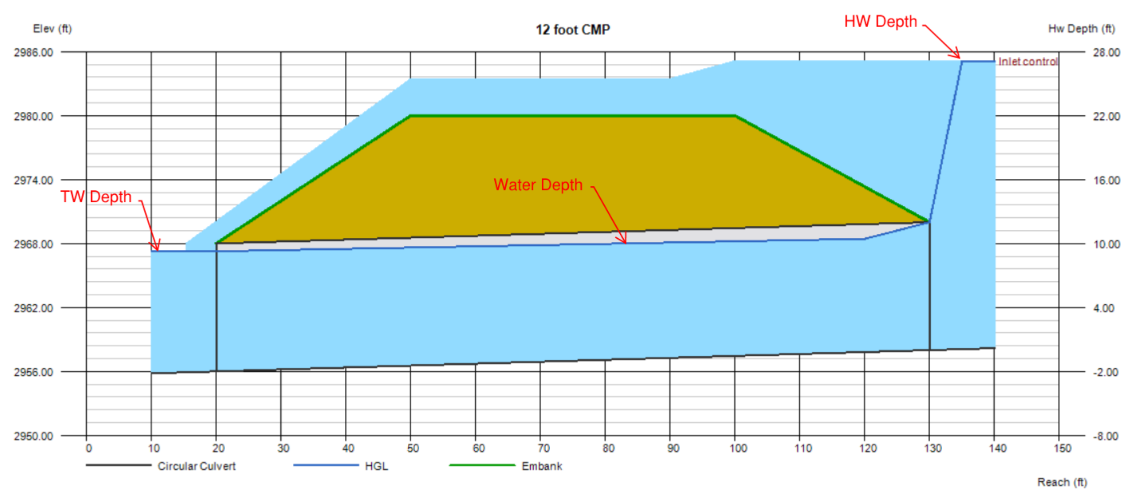

Figure 7.19: Hydraflow Express analyzed 12 foot CMP culvert under future design, 50 year return period 15 min intensity duration simulated flow conditions

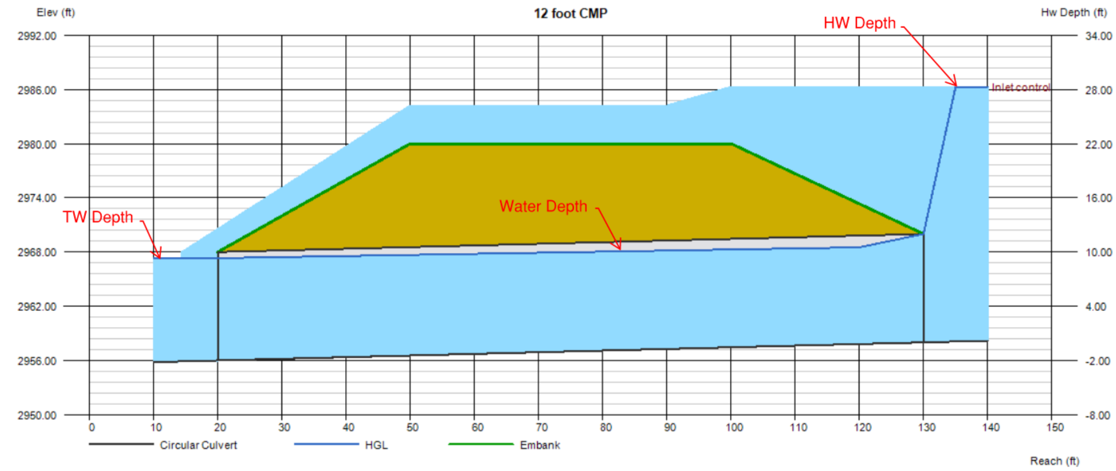

Figure 7.20: Hydraflow Express analyzed 12 foot CMP culvert under future design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.8: Future 12 foot CMP culvert parameters generated by Hydraflow Express Part 2

10 year return period				50 year return period			
Category	Parameter	Value	Units	Category	Parameter	Value	Units
2Flow	Total	3359.90	cfs	Flow	Total	6635.50	cfs
	Culvert	2034.61	cfs		Culvert	2189.96	cfs
	Overtopping	1325.30	cfs		Overtopping	4445.54	cfs
Velocity	Outlet	18.60	ft/s	Velocity	Outlet	19.86	ft/s
	Inlet	19.79	ft/s		Inlet	20.78	ft/s
Depth	Outlet	11.12	ft	Depth	Outlet	11.27	ft
	Inlet	10.24	ft		Inlet	10.56	ft
Hydraulic Grade Line	Outlet Elevation	2967.12	ft	Hydraulic Grade Line	Outlet Elevation	2967.27	ft
	Inlet Elevation	2968.24	ft		Inlet Elevation	2968.56	ft
	Headwater Elevation	2982.27	ft		Headwater Elevation	2985.11	ft
	HW/D	2.02			HW/D	2.26	

100 year return period			
Category	Parameter	Value	Units
Flow	Total	8331.50	cfs
	Culvert	2251.02	cfs
	Overtopping	6080.48	cfs
Velocity	Outlet	20.36	ft/s
	Inlet	21.23	ft/s
Depth	Outlet	11.32	ft
	Inlet	10.64	ft
Hydraulic Grade Line	Outlet Elevation	2967.32	ft
	Inlet Elevation	2968.64	ft
	Headwater Elevation	2986.28	ft
	HW/D	2.36	

7.8 Appendix H

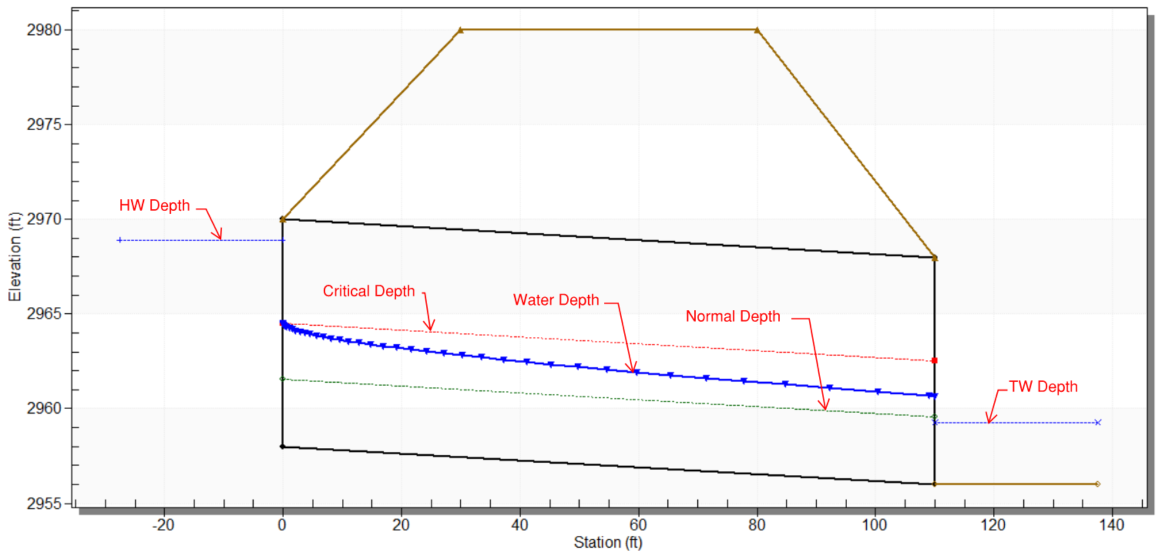

Figure 7.21: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under current design, 10 year return period 15 min intensity duration simulated flow conditions

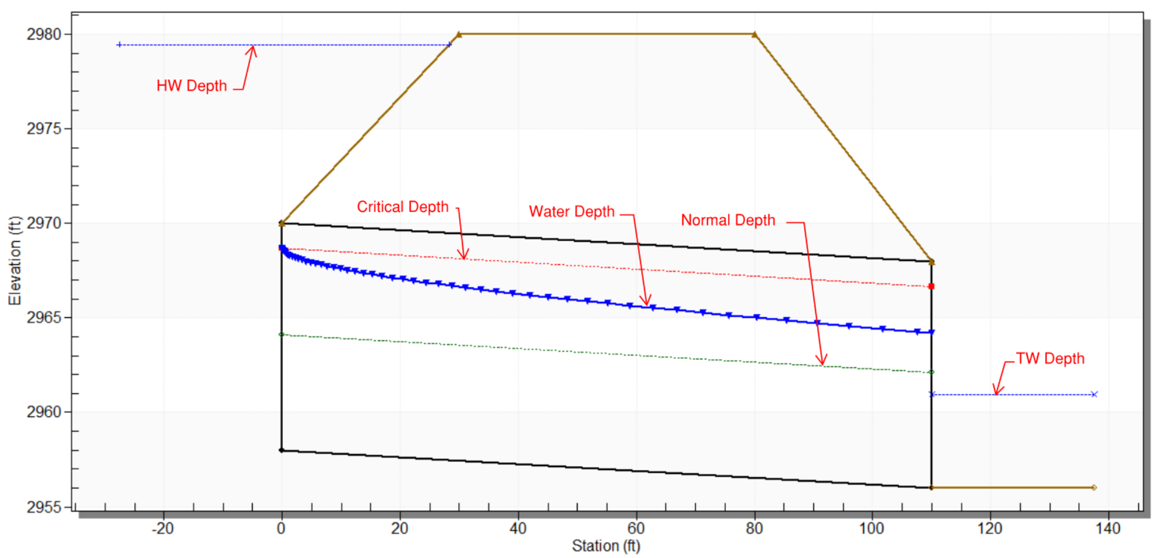

Figure 7.22: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under current design, 50 year return period 15 min intensity duration simulated flow conditions

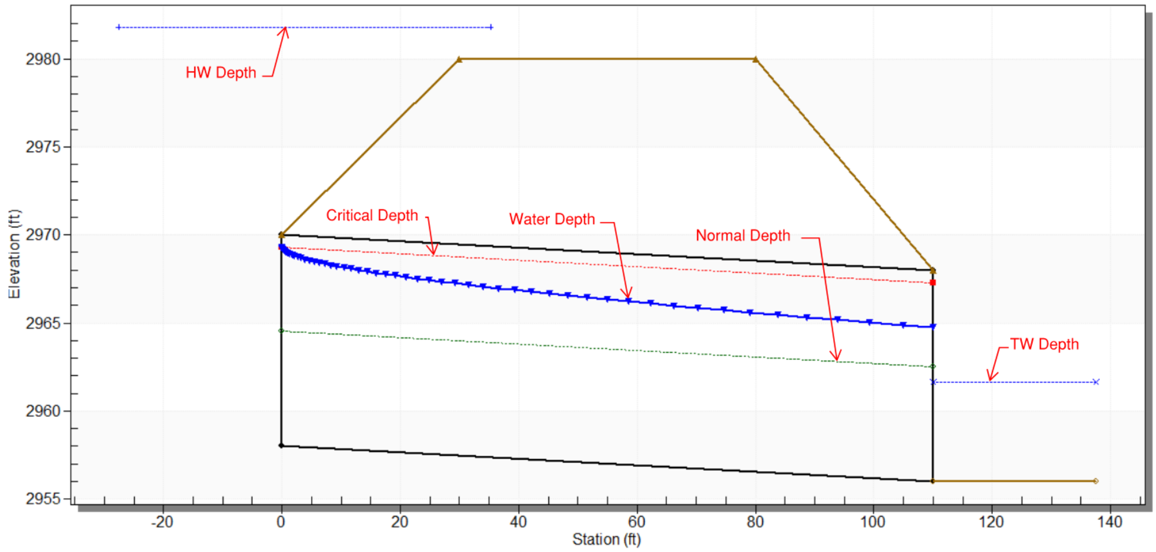

Figure 7.23: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under current design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.9: Current 12 foot x 12 foot – side by side – box culvert parameters generated by HY-8 Part 2

10 year return period					
Culvert		Units	Downriver		Units
Total Discharge	2265.00	cfs	Flow	2265.00	cfs
Culvert Discharge	2265.00	cfs	Tailwater Elevation	2959.25	ft
Overtopping Discharge	0.00	cfs	Depth	3.25	ft
Headwater Elevation	2968.89	ft	Velocity	12.97	ft/s
Inlet Control Depth	10.89	ft	Shear	3.69	psf
Flow Type	5-S2n		Froude Number	1.38	
Normal Depth	3.56	ft			
Critical Depth	6.52	ft			
Outlet Depth	4.66	ft			
Outlet Velocity	20.25	ft/s			
50 year return period					
Culvert		Units	Downriver		Units
Total Discharge	4747.70	cfs	Flow	4747.70	cfs
Culvert Discharge	4747.70	cfs	Tailwater Elevation	2960.92	ft
Overtopping Discharge	0.00	cfs	Depth	4.92	ft
Headwater Elevation	2979.44	ft	Velocity	16.41	ft/s
Inlet Control Depth	21.44	ft	Shear	5.59	psf
Flow Type	5-S2n		Froude Number	1.46	
Normal Depth	6.11	ft			
Critical Depth	10.67	ft			
Outlet Depth	8.20	ft			
Outlet Velocity	24.12	ft/s			
100 year return period					
Culvert		Units	Downriver		Units
Total Discharge	6062.80	cfs	Flow	6062.80	cfs
Culvert Discharge	5163.41	cfs	Tailwater Elevation	2961.63	ft
Overtopping Discharge	898.44	cfs	Depth	5.63	ft
Headwater Elevation	2981.78	ft	Velocity	17.69	ft/s
Inlet Control Depth	23.78	ft	Shear	6.39	psf
Flow Type	5-S2n		Froude Number	1.48	
Normal Depth	6.52	ft			
Critical Depth	11.29	ft			
Outlet Depth	8.74	ft			
Outlet Velocity	24.61	ft/s			

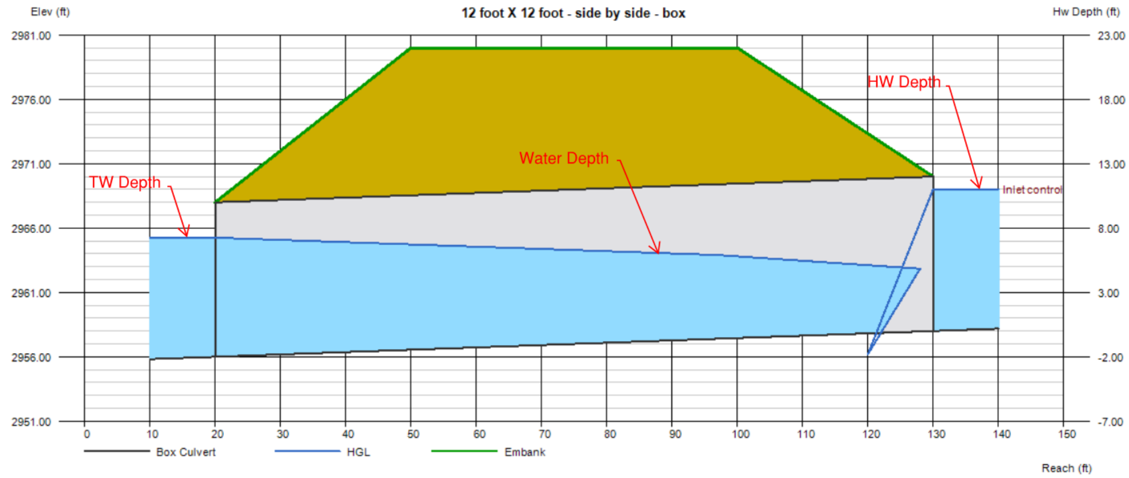

Figure 7.24: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under current design, 10 year return period 15 min intensity duration simulated flow conditions

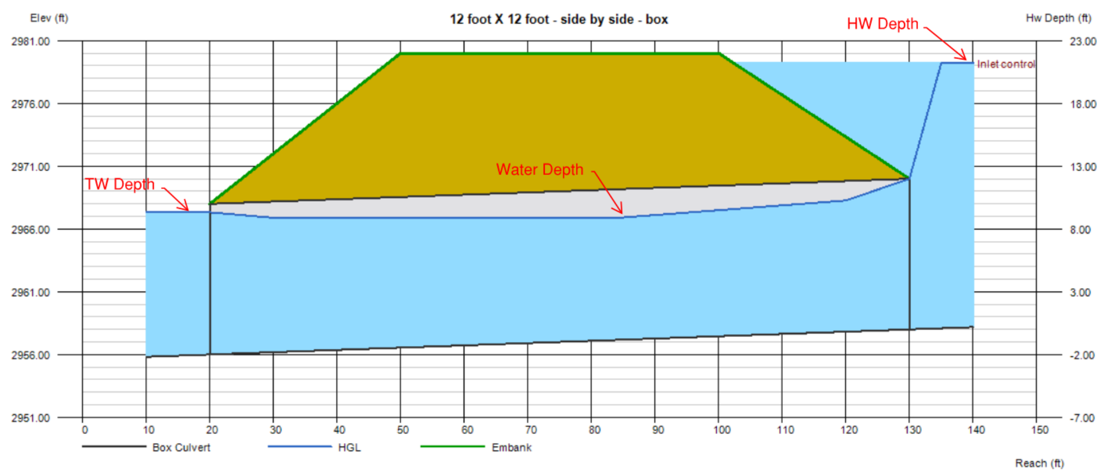

Figure 7.25: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under current design, 50 year return period 15 min intensity duration simulated flow conditions

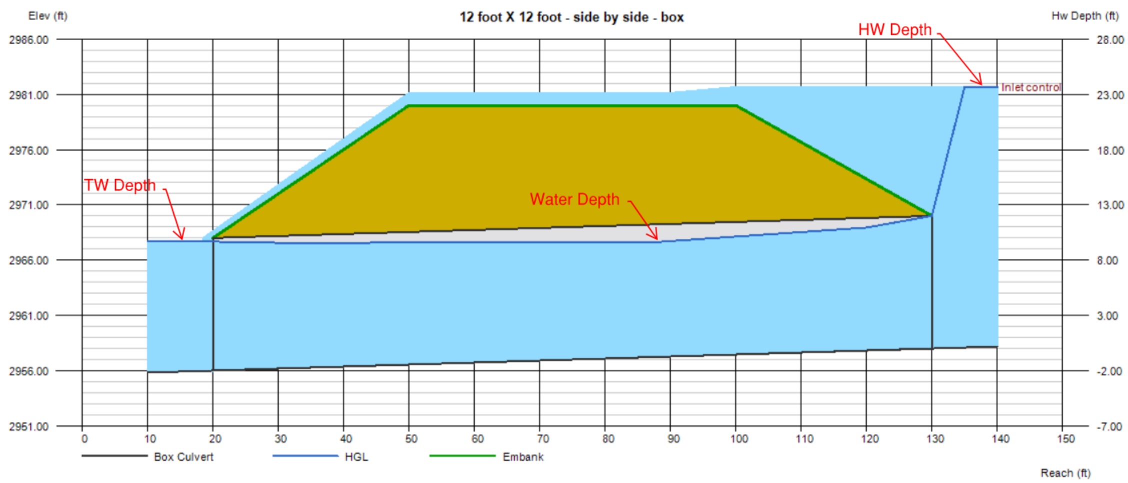

Figure 7.26: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under current design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.10: Current 12 foot x 12 foot – side by side – box culvert parameters generated by Hydraflow Express Part 2

10 year return period				50 year return period			
Category	Parameter	Value	Units	Category	Parameter	Value	Units
Flow	Total	2265.00	cfs	Flow	Total	4747.70	cfs
	Culvert	2265.00	cfs		Culvert	4747.70	cfs
	Overtopping	0.00	cfs		Overtopping	0.00	cfs
Velocity	Outlet	10.20	ft/s	Velocity	Outlet	17.47	ft/s
	Inlet	14.51	ft/s		Inlet	18.57	ft/s
Depth	Outlet	9.25	ft	Depth	Outlet	11.33	ft
	Inlet	6.51	ft		Inlet	10.65	ft
Hydraulic Grade Line	Outlet Elevation	2965.25	ft	Hydraulic Grade Line	Outlet Elevation	2967.33	ft
	Inlet Elevation	2964.51	ft		Inlet Elevation	2968.65	ft
	Headwater Elevation	2962.02	ft		Headwater Elevation	2979.23	ft
	HW/D	0.92			HW/D	1.77	

100 year return period			
Category	Parameter	Value	Units
Flow	Total	6062.80	cfs
	Culvert	5226.91	cfs
	Overtopping	835.89	cfs
Velocity	Outlet	18.65	ft/s
	Inlet	19.18	ft/s
Depth	Outlet	11.68	ft
	Inlet	10.36	ft
Hydraulic Grade Line	Outlet Elevation	2967.68	ft
	Inlet Elevation	2969.36	ft
	Headwater Elevation	2961.66	ft
	HW/D	1.97	

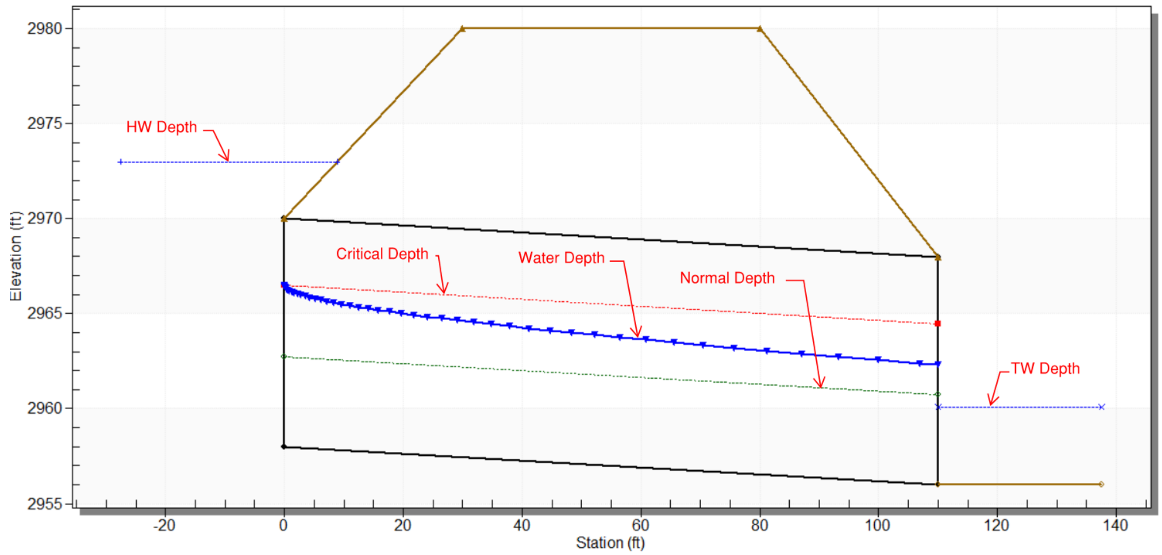

Figure 7.27: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under future design, 10 year return period 15 min intensity duration simulated flow conditions

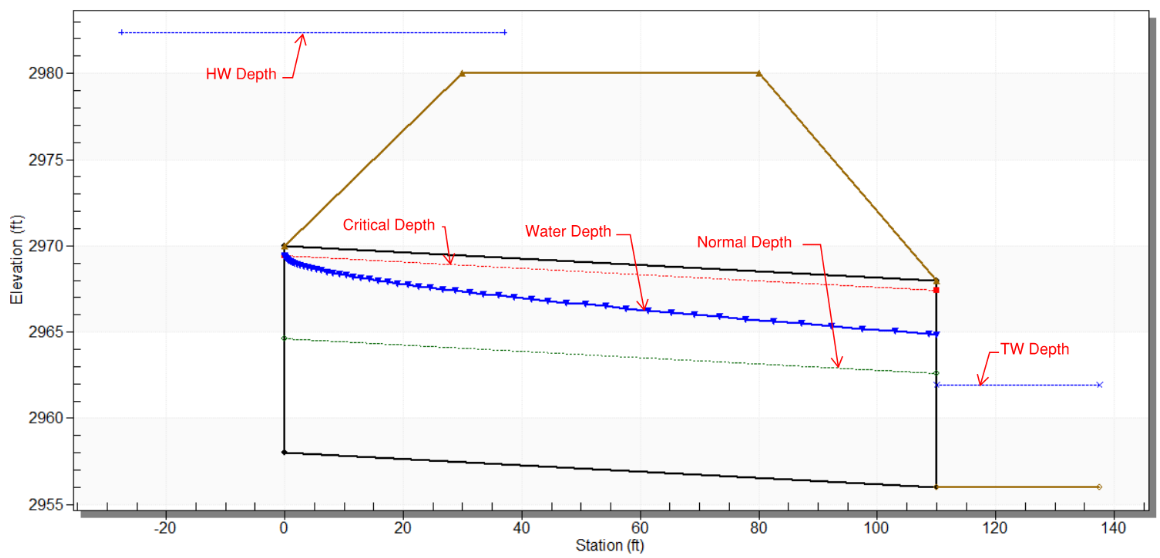

Figure 7.28: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under future design, 50 year return period 15 min intensity duration simulated flow conditions

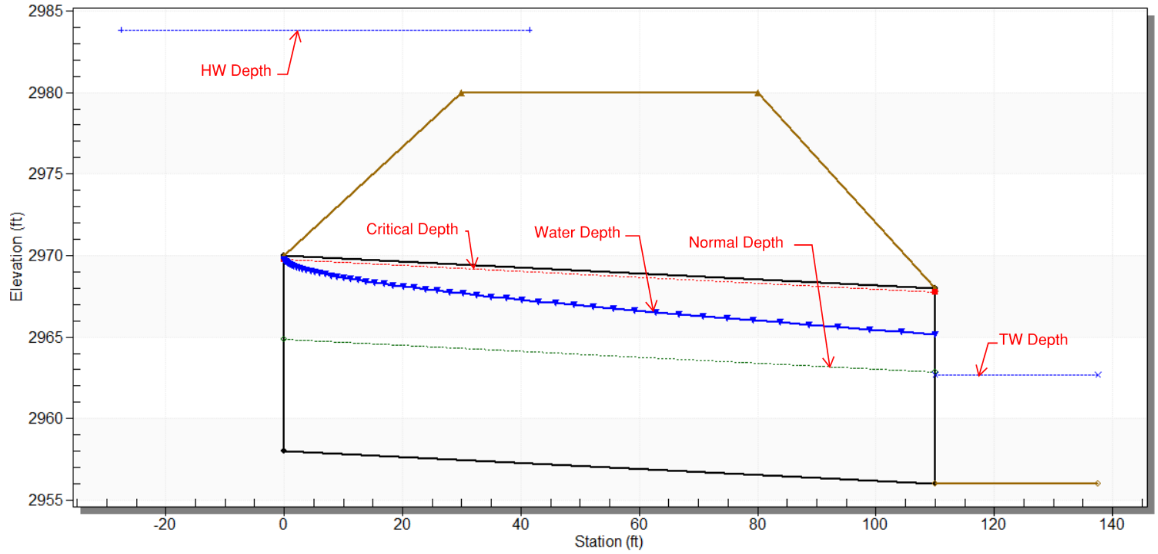

Figure 7.29: HY-8 analyzed 12 foot x 12 foot – side by side – box culvert under future design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.11: Future 12 foot x 12 foot – side by side – box culvert parameters generated by HY-8 Part 2

10 year return period					
Culvert		Units	Downriver		Units
Total Discharge	3359.90	cfs	Flow	3359.90	cfs
Culvert Discharge	3359.90	cfs	Tailwater Elevation	2960.06	ft
Overtopping Discharge	0.00	cfs	Depth	4.06	ft
Headwater Elevation	2972.97	ft	Velocity	14.73	ft/s
Inlet Control Depth	14.97	ft	Shear	4.61	psf
Flow Type	5-S2n		Froude Number	1.42	
Normal Depth	4.73	ft			
Critical Depth	8.48	ft			
Outlet Depth	6.30	ft			
Outlet Velocity	22.21	ft/s			
50 year return period					
Culvert		Units	Downriver		Units
Total Discharge	6635.50	cfs	Flow	6635.50	cfs
Culvert Discharge	5261.53	cfs	Tailwater Elevation	2961.91	ft
Overtopping Discharge	1373.92	cfs	Depth	5.91	ft
Headwater Elevation	2982.36	ft	Velocity	18.17	ft/s
Inlet Control Depth	24.36	ft	Shear	6.72	psf
Flow Type	5-S2n		Froude Number	1.49	
Normal Depth	6.61	ft			
Critical Depth	11.43	ft			
Outlet Depth	8.87	ft			
Outlet Velocity	24.72	ft/s			
100 year return period					
Culvert		Units	Downriver		Units
Total Discharge	8331.50	cfs	Flow	8331.50	cfs
Culvert Discharge	5499.98	cfs	Tailwater Elevation	2962.69	ft
Overtopping Discharge	2831.22	cfs	Depth	6.69	ft
Headwater Elevation	2983.82	ft	Velocity	19.45	ft/s
Inlet Control Depth	25.82	ft	Shear	7.59	psf
Flow Type	5-S2n		Froude Number	1.52	
Normal Depth	6.84	ft			
Critical Depth	11.77	ft			
Outlet Depth	9.17	ft			
Outlet Velocity	24.99	ft/s			

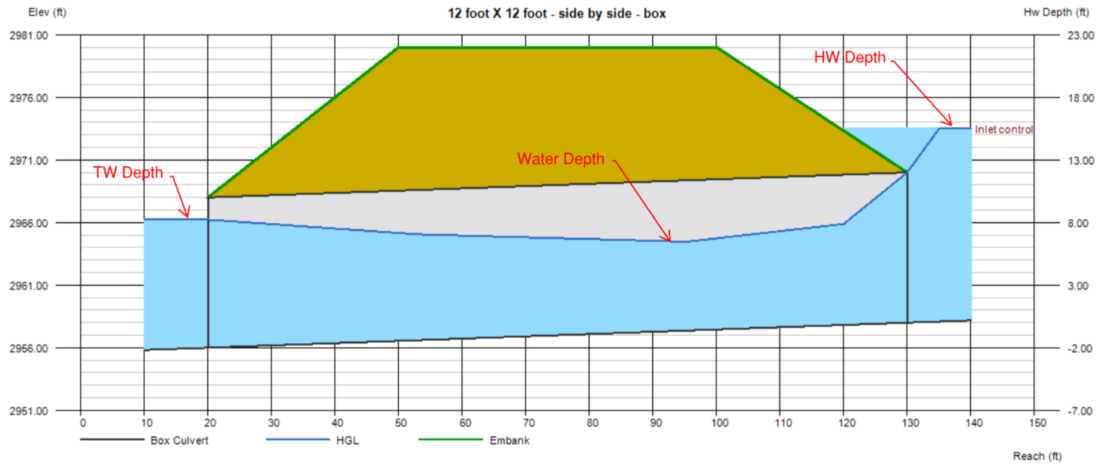

Figure 7.30: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under future design, 10 year return period 15 min intensity duration simulated flow conditions

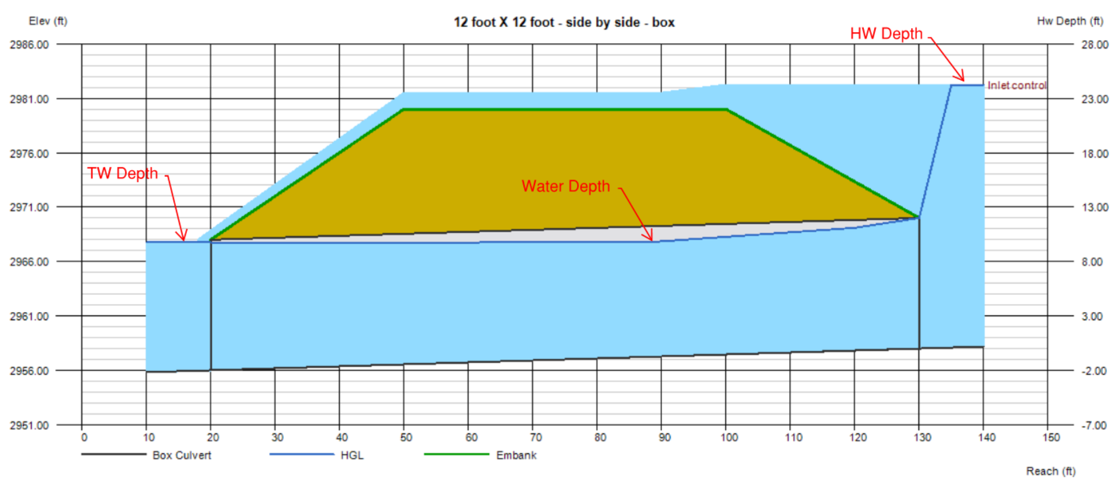

Figure 7.31: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under future design, 50 year return period 15 min intensity duration simulated flow conditions

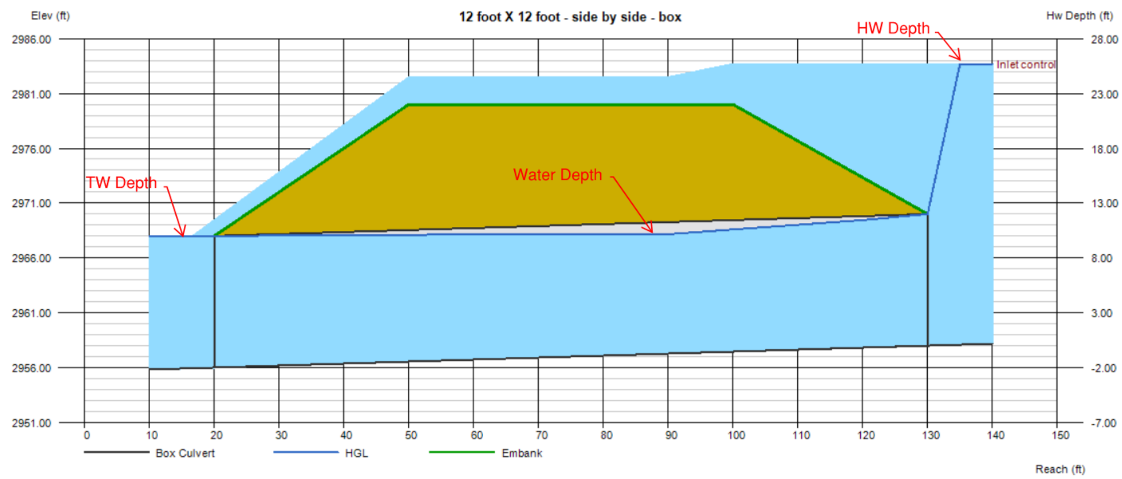

Figure 7.32: Hydraflow Express analyzed 12 foot x 12 foot – side by side – box culvert under future design, 100 year return period 15 min intensity duration simulated flow conditions

Table 7.12: Future 12 foot x 12 foot – side by side – box culvert parameters generated by Hydraflow Express Part 2

10 year return period				50 year return period			
Category	Parameter	Value	Units	Category	Parameter	Value	Units
Flow	Total	3359.60	cfs	Flow	Total	6635.50	cfs
	Culvert	3359.60	cfs		Culvert	5334.24	cfs
	Overtopping	0.00	cfs		Overtopping	1301.26	cfs
Velocity	Outlet	13.68	ft/s	Velocity	Outlet	18.91	ft/s
	Inlet	16.55	ft/s		Inlet	19.31	ft/s
Depth	Outlet	10.23	ft	Depth	Outlet	11.76	ft
	Inlet	8.46	ft		Inlet	11.51	ft
Hydraulic Grade Line	Outlet Elevation	2966.23	ft	Hydraulic Grade Line	Outlet Elevation	2967.76	ft
	Inlet Elevation	2966.46	ft		Inlet Elevation	2969.51	ft
	Headwater Elevation	2973.49	ft		Headwater Elevation	2982.24	ft
	HW/D	1.29			HW/D	2.02	

100 year return period			
Category	Parameter	Value	Units
Flow	Total	8331.50	cfs
	Culvert	5594.27	cfs
	Overtopping	2737.23	cfs
Velocity	Outlet	19.52	ft/s
	Inlet	19.62	ft/s
Depth	Outlet	11.94	ft
	Inlet	11.88	ft
Hydraulic Grade Line	Outlet Elevation	2967.94	ft
	Inlet Elevation	2969.88	ft
	Headwater Elevation	2983.69	ft
	HW/D	2.14	

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