THE UNIVERSITY OF ARIZONA

IN COORDINATION WITH

THE DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS
&
THE DEPARTMENT OF HYDROLOGY AND ATMOSPHERIC SCIENCES

# Helping a town with culvert design and small-scale flooding issues over a roadway critical to emergency services

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Date: 4/8/18

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## Introduction

When I first applied to the University of Arizona, I had to make a choice between two different Master's programs: a thesis or a non-thesis Master's of Science in Hydrology degree. I decided on the non-thesis option, as this was more oriented towards individuals who are looking to enter the job market and begin a career after receiving their degree. The thesis route on the other hand, was aimed at people who were interested in pursuing more research, a PhD, and teaching. Because I chose the non-thesis option, I knew that I would have more time on my hands than my thesis colleagues. With that time, I decided to pursue a graduate certificate in Hydraulics and Water Resources Engineering in addition to my Master's degree. I felt that taking five hydrology-related civil engineering courses and earning the certificate would help to hone my understanding of hydrology as it relates to engineering. I also figured that both my Master's degree and the certificate would help me stand out among other applicants when applying to jobs.

Instead of completing a thesis for my degree, I was tasked with completing a Master's Report. A Master's report was generally defined as "a professional report completed as a current work assignment or a graduate-level paper on a topic approved by the faculty advisor." I knew that the report would need to be more substantial than a regular report or paper for one of my courses. But since the Master's Report was only worth one credit hour, I determined that my report would not have to be a thesis-caliber paper. For the report, I had to decide whether I wanted a project focused on surface water hydrology or groundwater hydrology. Since I had taken several civil engineering courses that focused on surface water, I was leaning towards a surface water based report. But I figured I would keep an open mind, as I had taken two courses devoted to groundwater hydrology.

To start my search for a Master's report topic, I consulted with my faculty advisor, Ty Ferré. We met and he had suggested a project that was groundwater based with a surface water interaction element. While I thought the topic was interesting, I told Ty that I was not quite sure if it was the topic for me. Ty told me that I should find a topic that I was really interested in, because the more interest I had in the topic, the better report I would be able to complete. After analyzing the groundwater topic, I wanted to see if I could find a project that was surface water based. That way I could compare the two options and choose the project that I felt the most interested in and could get the most out of. Having taken a number of classes in the Civil Engineering department, I decided to look there to see if I could find a project that I would find interest in.

I decided to meet with Kevin Lansey, the department head of Civil Engineering and Engineering Mechanics, as well as my professor for two of my certificate courses. He gave me a few ideas, but one that stuck out to me was a project that involved a small town with some flooding issues. I contacted the town manager, and he explained the flooding problems they were having. I told him that I would think on the two possible report topics and get back to him. After a couple of days, I decided on the town with the flooding issues as the topic for my Master's report. When I told the town manager, he seemed very excited to be getting some help with the flooding problems they were having. The town was Superior, Arizona, and after a couple of weeks since making contact, I took a road trip up to Superior to check out the town for myself.

Superior, Arizona is a small town of about 3,000 people approximately one hour east of Phoenix. Several movies had film shoots in Superior, including the critically acclaimed *How the West Was Won*. It is also home to Boyce Thompson Arboretum, the largest and oldest botanical

garden in the state of Arizona (founded in 1924). Figure 1 shows the town of Superior, and its relative location to Phoenix and Tucson.

*Figure 1: State map of Arizona, with Superior featured prominently, and its relative location to Phoenix and Tucson.*

Before I arrived in Superior, I had a basic idea of the problems they were having. There was a low-lying road (Mary Drive), in which a creek (Queen Creek) would flow over during flooding events. These flooding events would generally only happen during the monsoon season in the summer. The problem was that 8-10 days of the summer, the road would get flooded out, making it impassable to even the most off-road of vehicles. An even more serious issue was that the town's fire department was situated a few hundred yards south of the crossing. When the road is flooded, the fire trucks have to take a 3-5 minute detour to respond to emergencies. 3-5 minutes can mean life or death in an emergency such as a fire. Not only fire trucks, but also ambulances and police vehicles would have to make this detour should they be on the south side of the crossing. Because of the effect of flooding at this crossing on emergency vehicle response

times, the town manager ranked it as the flooding issue with the highest priority, as Superior has other flooding issues they deal with.

After my meeting with the town manager, the sheriff, and others, we decided that there would be two main parts to this project: hydrologic analysis and culvert design. Luckily my hydrology classes prepared me for the first part of the project, and my civil engineering courses helped prepare me for the second part. Once the meeting concluded, the last step of my visit was to get a first-hand look at the crossing. The sheriff took me over to the crossing so that I could take pictures and document my findings. Figure 2 shows a picture of Queen Creek upstream of the Mary Drive crossing.

*Figure 2: Upstream view of Queen Creek, just ahead of the Mary Drive crossing.*

Figure 3 is also a picture taken of the upstream section of Queen Creek, but with Mary Drive included for context. Note the channel invert elevation is almost equal to the road invert elevation.

*Figure 3: Upstream view of Queen Creek, with Mary Drive being the road pictured.*

Figure 4 shows the downstream section of Queen Creek, immediately after it crosses over Mary Drive.

*Figure 4: Downstream view of Queen Creek, just after the Mary Drive crossing.*

Figure 5 is a picture taken of one of the upstream channel banks. Note the serious amount of erosion that has occurred.

*Figure 5: Picture of the upstream left bank of Queen Creek, noting the erosion.*

For a larger perspective of the crossing, Figure 6 is an aerial image of Queen Creek as it crosses Mary Drive. Note that the Superior Fire Department is also featured.

*Figure 6: Aerial image of the Mary Drive-Queen Creek crossing. Note the crossing in the blue circle and the fire department in the red oval.*

**Methods –** Hydrologic analysis

Before I could do any calculations, the first step was to delineate the watershed that would drain into the outlet (the Mary Drive – Queen Creek crossing). I went through a trove of files that were given to me by the town manager, and found a USGS topographic map of Superior and some surrounding areas. I needed to print out a large version of the map, but I did not have quick access to a print shop to have a large map printed. So I decided to MacGyver my way through the situation by printing out nine separate pages of the map, cutting them precisely, and taping them back together to form the whole map. Now that I had a large map and could clearly see all of the features, I decided to first delineate the watershed by highlighting all of the waterways. I did this so it would be easier to see the watershed and delineate its boundaries. I then delineated the watershed.

Afterwards, I went onto Adobe Acrobat and measured the hydraulic length of the watershed (the length between the outlet and the hydraulically most distant point). This happened to be from the outlet at Mary Dr. all the way up around Fortuna Peak, where Queen Creek begins. I measured the hydraulic length because I knew it would be an important component to the equations I would need later on. After measuring the hydraulic length, the area was then measured in Adobe Acrobat by drawing a polygon matching the watershed boundary delineation I did on the paper map. Figure 7 shows my hand drawn delineation of the watershed that drains through Queen Creek and over Mary Drive.

*Figure 7: Hand drawn delineation of the Superior watershed that drains through the outlet at the Mary Drive – Queen Creek crossing.*

Now that I had the watershed area, I had to determine how I was going to make my calculations. For starters, the main goal of the hydrologic analysis was to quantify the flows coming to the crossing. Specifically, I needed to calculate the peak flows for six statistical storm events: 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storms. I had to choose software and a method that would be able to perform these calculations. For my report, I chose HEC-HMS. I figured that HEC-HMS would be appropriate, as HEC-HMS is designed to simulate the precipitation-runoff processes of dendritic drainage basins. For HEC-HMS to run properly, several inputs had to be calculated.

The first input needed was the area of the drainage basin. Fortunately, I already calculated the area previously. The next step was to define five basin methods: canopy, surface, loss, transform, and baseflow. To make the calculations simpler, I decided to assume that the canopy, baseflow, and surface methods would have a negligible effect on the flow values for this project. For the runoff losses, I chose the SCS Curve Number method. For the hydrograph transformation, I chose the SCS Unit Hydrograph. Figure 8 shows the subbasin inputs in HEC-HMS.

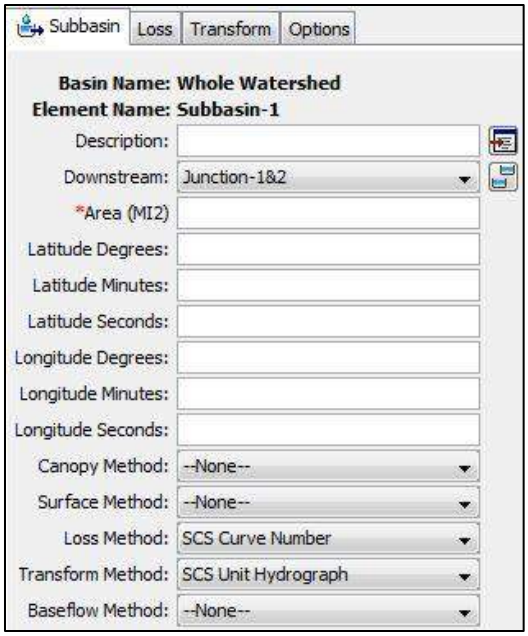


*Figure 8: Subbasin inputs required to run a simulation in HEC-HMS.*

For the runoff losses, two inputs were required: a curve number and a percentage of impervious area. To determine a curve number for the drainage area, I used the free Web Soil Survey with the NRCS (Soil survey information included in Appendix A). I drew an area of interest that represented the drainage area. The program then returned information on the soil types within the watershed I drew. I used the soil data in the survey and, looking at satellite imagery, did my best to match up information from Figure 9 and 10 to what was given in the soil survey. A lot of the curve number work was using my own judgment and assumptions, so that is a part of this project that could use more study.

| Cover description | | Curve numbers for hydrologic soil group | | | |
|---|---|---|---|---|---|
| Cover type | Hydrologic condition[b] | A[c] | B | C | D |
| Herbaceous—mixture of grass, weeds, and low-growing brush, with the minor element | Poor | | 80 | 87 | 93 |
| | Fair | | 71 | 81 | 89 |
| | Good | | 62 | 74 | 85 |
| Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush | Poor | | 66 | 74 | 79 |
| | Fair | | 48 | 57 | 63 |
| | Good | | 30 | 41 | 48 |
| Pinyon-juniper—pinyon, juniper, or both; grass understory | Poor | | 75 | 85 | 89 |
| | Fair | | 58 | 73 | 80 |
| | Good | | 41 | 61 | 71 |
| Sagebrush with grass understory | Poor | | 67 | 80 | 85 |
| | Fair | | 51 | 63 | 70 |
| | Good | | 35 | 47 | 55 |
| Desert shrub—major plants include saltbrush, greasewood, creosotebrush, blackbrush, bursage palo verde, mesquite, and cactus | Poor | 63 | 77 | 85 | 88 |
| | Fair | 55 | 72 | 81 | 86 |
| | Good | 49 | 68 | 79 | 84 |

*Note*. Source: Soil Conservation Service (1986).

[a]Average runoff condition, and $I_a = 0.2S$. For range in humid regions, use Table 3.2c.

[b]*Poor:* <30% ground cover (litter, grass, and brush overstory). *Fair:* 30 to 70% ground cover. *Good:* >70% ground cover.

*Figure 9: Runoff Curve Numbers for Arid and Semiarid Rangelands.*

| Land use description | Hydrologic soil group | | | |
|---|---|---|---|---|
| | A | B | C | D |
| Cultivated land[a] | | | | |
| Without conservation treatment | 72 | 81 | 88 | 91 |
| With conservation treatment | 62 | 71 | 78 | 81 |
| Pasture or range land | | | | |
| Poor condition | 68 | 79 | 86 | 89 |
| Good condition | 39 | 61 | 74 | 80 |
| Meadow | | | | |
| Good condition | 30 | 58 | 71 | 78 |
| Wood or forest land | | | | |
| Thin stand, poor cover, no mulch | 45 | 66 | 77 | 83 |
| Good cover[b] | 25 | 55 | 70 | 77 |
| Open Spaces, lawns, parks, golf courses, cemeteries, etc. | | | | |
| Good condition (grass cover on 75% or more of the area) | 39 | 61 | 74 | 80 |
| Fair condition (grass cover on 50 to 75% of the area) | 49 | 69 | 79 | 84 |
| Commercial and business areas (85% impervious) | 89 | 92 | 94 | 95 |
| Industrial districts (72% impervious) | 81 | 88 | 91 | 93 |
| Residential[c] | | | | |
| Average lot size — Average percentage impervious[d] | | | | |
| $\frac{1}{8}$ acre or less — 65 | 77 | 85 | 90 | 92 |
| $\frac{1}{4}$ acre — 38 | 61 | 75 | 83 | 87 |
| $\frac{1}{3}$ acre — 30 | 57 | 72 | 81 | 86 |
| $\frac{1}{2}$ acre — 25 | 54 | 70 | 80 | 85 |
| 1 acre — 20 | 51 | 68 | 79 | 84 |
| Paved parking lots, roofs, driveways, etc.[e] | 98 | 98 | 98 | 98 |
| Streets and roads | | | | |
| Paved with curbs and storm sewers[e] | 98 | 98 | 98 | 98 |
| Gravel | 76 | 85 | 89 | 91 |
| Dirt | 72 | 82 | 87 | 89 |

[a]For a more detailed descrlption of agricultural and land use curve numbers refer to "National Engineering Handbook," Sect. 4, "Hydrology" Chap. 9, 1972.

[b]Good cover is protected from grazing, litter, and brush cover soil.

[c]Curve numbers are computed assuming the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltrations could occur.

[d]The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[e]In some warmer climates of the country a curve number of 95 may be used.

*Figure 10: Runoff Curve Numbers for Selected Land Uses (Soil Conservation Service, 1986).*

The second input required for runoff losses is the percentage of impervious area. Just like determining the curve number, determining the area of impervious surfaces was based on my judgment. In the soil survey online software, I drew a polygon around the impervious surfaces in town of Superior. I included parking lots, roads, and buildings as impervious areas. Figure 10 was also used to determine the percentages of the areas that were impervious.

For the hydrograph transformation, only one input is required: lag time. The Soil Conservation Service (SCS) developed an equation for lag time based on natural watersheds. Figure 11 gives the equation that the SCS developed in 1975. *L* is the hydraulic length of the watershed in feet. The value of *L* was determined when the watershed was delineated. *S* is the potential maximum retention, which is related to the curve number by the equation in Figure 12. *Y* is the average land slope as a percentage. It was calculated by taking the difference in elevation of the hydraulically most distant point and the outlet and dividing the difference by the hydraulic length of the watershed. And $t_L$ is the lag time in hours.

$$t_L = \frac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} \qquad (50 \leq CN \leq 95)$$

*Figure 11: SCS (1975) lag time equation.*

$$S = \left(\frac{1000}{CN}\right) - 10)$$

*Figure 12: SCS (1975) potential maximum retention equation.*

After inputting the basin model parameters, the next step was to input the meteorologic model parameters. Out of the available options, the frequency storm was the best-suited precipitation method for this project. The frequency storm requires the annual-duration depth values for Superior. To acquire these values, I went online and accessed NOAA's Atlas 14, found a station in Superior, Arizona, and input those values in for all six of the storm return intervals. Figure 13 is a screen-captured image of the values given by NOAA's Atlas 14 for Superior. The last input needed to run the simulation in HEC-HMS was a control input. I set a start date, start time, end date, and end time.

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

| Duration | Average recurrence interval (years) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 5 | 10 | 25 | 50 | 100 | 200 | 500 |
| 5-min | **0.260** (0.223-0.309) | **0.339** (0.290-0.401) | **0.450** (0.382-0.529) | **0.535** (0.452-0.624) | **0.646** (0.540-0.749) | **0.732** (0.601-0.846) | **0.820** (0.663-0.945) | **0.905** (0.722-1.05) | **1.02** (0.794-1.18) |
| 10-min | **0.397** (0.339-0.471) | **0.516** (0.442-0.611) | **0.685** (0.581-0.805) | **0.814** (0.687-0.949) | **0.983** (0.821-1.14) | **1.12** (0.915-1.29) | **1.25** (1.01-1.44) | **1.38** (1.10-1.59) | **1.56** (1.21-1.80) |
| 15-min | **0.492** (0.420-0.583) | **0.640** (0.548-0.757) | **0.850** (0.721-0.998) | **1.01** (0.851-1.18) | **1.22** (1.02-1.41) | **1.38** (1.13-1.60) | **1.55** (1.25-1.78) | **1.71** (1.36-1.98) | **1.93** (1.50-2.23) |
| 30-min | **0.662** (0.566-0.786) | **0.862** (0.737-1.02) | **1.14** (0.971-1.34) | **1.36** (1.15-1.59) | **1.64** (1.37-1.90) | **1.86** (1.53-2.15) | **2.08** (1.68-2.40) | **2.30** (1.83-2.66) | **2.60** (2.02-3.01) |
| 60-min | **0.819** (0.700-0.972) | **1.07** (0.913-1.26) | **1.42** (1.20-1.66) | **1.68** (1.42-1.96) | **2.03** (1.70-2.36) | **2.30** (1.89-2.66) | **2.58** (2.08-2.97) | **2.85** (2.27-3.29) | **3.21** (2.50-3.72) |
| 2-hr | **0.961** (0.818-1.14) | **1.23** (1.05-1.45) | **1.61** (1.36-1.90) | **1.90** (1.60-2.22) | **2.28** (1.89-2.65) | **2.58** (2.11-2.99) | **2.89** (2.34-3.35) | **3.19** (2.54-3.71) | **3.60** (2.81-4.21) |
| 3-hr | **1.00** (0.857-1.18) | **1.27** (1.09-1.50) | **1.64** (1.39-1.92) | **1.93** (1.62-2.26) | **2.33** (1.93-2.70) | **2.65** (2.17-3.07) | **2.99** (2.41-3.46) | **3.33** (2.64-3.86) | **3.80** (2.94-4.42) |
| 6-hr | **1.23** (1.08-1.41) | **1.54** (1.35-1.77) | **1.92** (1.68-2.19) | **2.23** (1.94-2.53) | **2.66** (2.27-3.00) | **2.99** (2.53-3.38) | **3.33** (2.77-3.78) | **3.68** (3.02-4.17) | **4.16** (3.32-4.71) |
| 12-hr | **1.51** (1.31-1.76) | **1.88** (1.64-2.18) | **2.33** (2.02-2.69) | **2.69** (2.32-3.11) | **3.17** (2.69-3.64) | **3.54** (2.98-4.06) | **3.92** (3.27-4.52) | **4.30** (3.53-4.96) | **4.82** (3.87-5.57) |
| 24-hr | **1.86** (1.74-2.01) | **2.33** (2.18-2.51) | **2.92** (2.72-3.15) | **3.39** (3.15-3.66) | **4.06** (3.75-4.38) | **4.58** (4.20-4.95) | **5.12** (4.67-5.56) | **5.69** (5.13-6.21) | **6.46** (5.74-7.13) |

*Figure 13: Partial Duration Sequence (PDS) – based precipitation frequency estimates with 90% confidence interval (in inches) for Superior, Arizona.*

## **Methods –** Culvert Design

After HEC-HMS produced reasonable values for peak flows for storms that I was looking at, the next step was to begin some preliminary culvert designs. There were three programs that I could use to accomplish this: HEC-RAS, CulvertMaster, and the Federal Highway Administration's (FHWA) HY-8 program. HEC-RAS would be the best software to use for a full crossing design and accounting of the culvert's impacts to the area, but it was a bit too cumbersome to use for preliminary designs. CulvertMaster was not free, so that option was out. HY-8 was not only free, but it would also be the perfect software to use for basic designs of the culvert, so I chose it as the program to use for the first set of culvert designs.

It is important to note that I do not have a lot of experience designing culverts, since my Master's degree is in hydrology and not civil engineering. That being said, I have had some experience with culverts and their design in a few of the civil engineering classes that I have taken towards a graduate certificate in Hydraulics and Water Resources engineering. While I am more confident in the hydrology aspect of this report, I decided that I could try to help the town manager and his team with the culvert designs. There were two rounds of preliminary culvert designs. The first round was based on passing the 50-year return period flow completely through the culvert, with the 100-year return interval flow overtopping the road at various depths. The first round had 5 designs: 4 concrete box culvert designs and 1 circular concrete culvert design. Each design contained a side view, front view, the crossing data, and the culvert analysis. Because there are many pictures related to the culvert designs, I have included them in Appendix B rather than in the Methods section of this report. Figure 14 shows the input window for HY-8 and I will explain below the figure how I came up with all of the inputs.

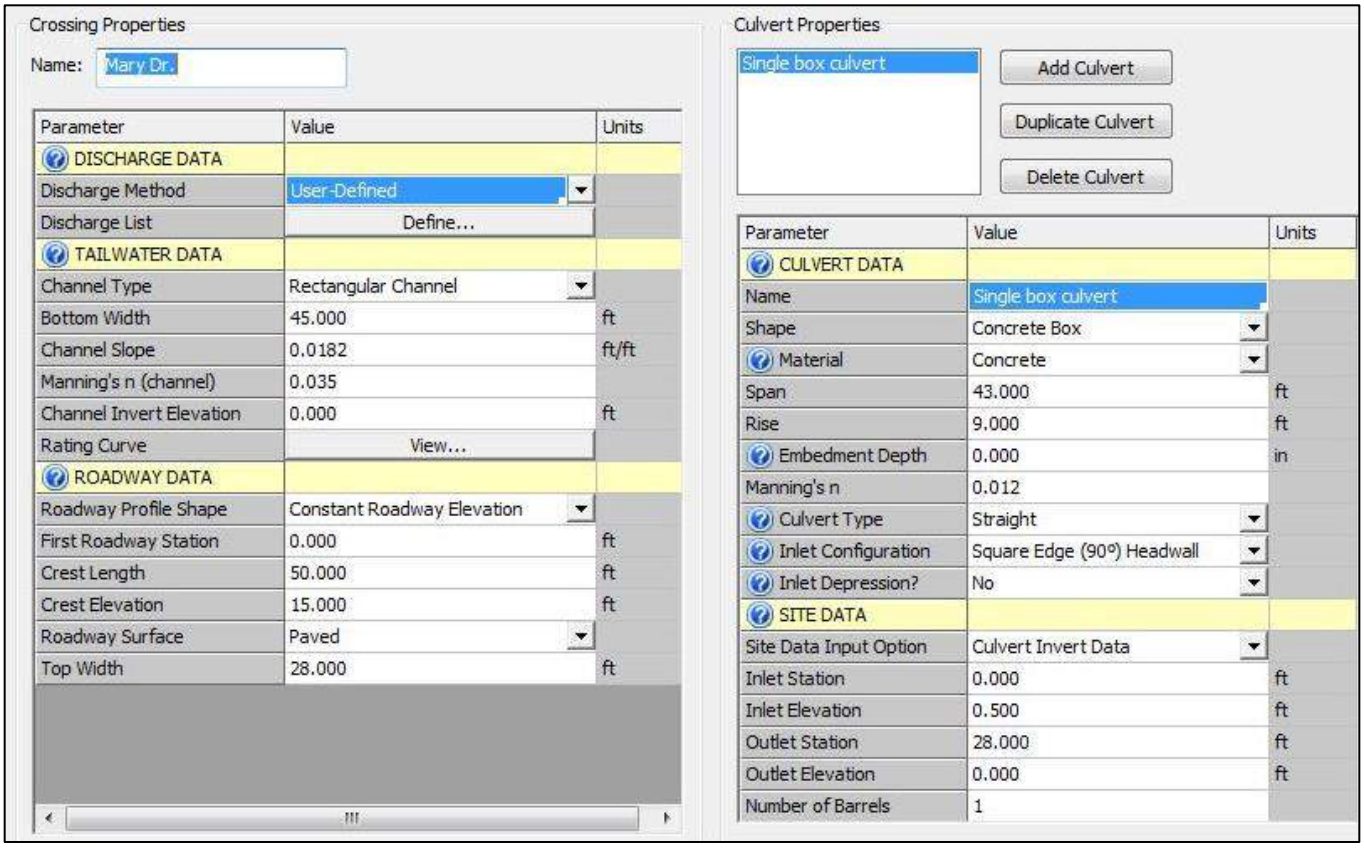


*Figure 14: Main input screen to FHWA's HY-8 culvert design program.*

Discharge method: Instead of a minimum, maximum, and design flows, I chose the user defined option and input the 50-year and 100 year flow values. Channel type: I chose a rectangular channel (as opposed to a trapezoidal, triangular or irregular channel) because I knew it would be the simplest to get preliminary culvert designs. Based on Figures 2, 3, and 4, it appears that the channel is naturally somewhat rectangular, but it will end up being an irregular channel once the channel is mapped with elevation data in ArcMap and transferred to HEC-RAS. Bottom width: Estimated with Google Maps. A more accurate value was obtained when using elevation data with ArcMap and HEC-RAS. Channel slope: Found by taking 40 feet (a topographic map with 40 feet contours) divided by the distance between the 40-foot contours (around the crossing).

Manning's n: Estimated by looking at tables, established n values and pictures of channels, and comparing theses to Queen Creek (see Figure 2). Channel Invert Elevation: Made an assumption that it was 0 feet, actual value was found with ArcMap. Roadway Profile Shape: Chose constant roadway elevation (even though the road does not have a constant elevation) since this was the simpler option for a preliminary design. Ended up with an irregular roadway profile once elevation data was used in ArcMap and HEC-RAS. First Roadway Station: Chose 0.0 feet as the start point for the roadway. Crest length: Chose 50 feet based on Google Maps.

Crest Elevation: Assigned a value of 15 feet based on the size of the culvert that needed to be built. Roadway Surface: Paved. Top width: Estimated based on Google Maps. Culvert shape: Concrete box or circular; chosen because those two designs seemed to be the most popular. Material: Concrete. Only option for concrete box culvert, did not investigate the other materials for a circular culvert. Span: Estimated based on a reasonable value for "rise" in order to allow 50-year flow to flow through culvert. Rise: Estimated based on a reasonable value for "span" in order to allow 50-year to flow through culvert.

Embedment depth: Assumed it was 0 inches. Culvert Type: Straight, assumption. Inlet Configuration: Square Edge (90 degree) Headwall, assumption. Inlet Depression: Assumed to be no. Site Data Input Option: Culvert Invert Data. Inlet Station: Assumption of 0.00 feet. Inlet Elevation: 0.500 feet, basically matched the channel slope with the slope that is designed for the culvert. Outlet Station: 28.00 feet, matching the width of the road. Outlet Elevation: 0.00 feet (see Inlet Elevation description). Number of barrels: Dependent on design.

For the second round of culvert designs, I turned to others for guidance. After talking with the Pinal County engineer, Christopher Wanamaker, I made some changes to my designs. I decided to narrow it down from five designs to one design per return period. We also decided

that we would focus on the 4 return intervals from the 10-year through the 100-year storms. Mr. Wanamaker also suggested that I try out a ConArch design and see how it compared to the rectangular and circular culverts. The concrete arch design performed much better than the rectangular and circular culverts. The ConArch designs were lighter as well. See Appendix B for pictures of the second round of culvert designs.

**Methods** – Integration of the hydrologic analysis and culvert design

Now that I had used HEC-HMS to get my hydrology data and HY-8 to get some preliminary culvert designs, the next step was integrating both of these parts together. HEC-RAS would be the best software for this task, as one of its main functions is channel flow analysis and floodplain determination. But before I could use my hydrology and culvert data in HEC-RAS, I needed to first work with ArcMap to compute cross sections, reach lengths, and bank stations that HEC-RAS would recognize. For that I needed a GIS extension called HEC-GeoRAS. I first began by taking the shapefile elevation data given to me, and used the 3D analyst tool to convert it into a TIN format (for GeoRAS). I was then able to use GeoRAS to create a stream centerline, bank lines, flow paths, and cross sections. After making sure all of the settings were correct, I exported the GIS data as RAS data, and imported the data into HEC-RAS. The result was that I had all my cross sections, downstream reach lengths, bank stations, and station elevation values ready to go in HEC-RAS.

After creating a model of Queen Creek in HEC-RAS, I was interested in seeing how it handled the peak flow values from the four return period storms, specifically at the Mary Drive – Queen Creek crossing. When I had finished running the simulation, I realized that the creek would need a serious channel excavation to handle the flows from these storms. Luckily there is a channel excavation tool within HEC-RAS that can take inputs such as the center cut, bottom

width, channel invert elevation, and left and right bank slope. Once an initial channel excavation had been created, the flows through the creek around the crossing were much more manageable. Results from the excavation are included in the Results section of the report.

**Results** – Hydrologic Analysis

By using Adobe Acrobat, and drawing a polygon in the shape of the watershed, I achieved an area of 13.70 square miles (see Figure 7 for delineation). Since I did not have any values to compare my result to, I decided to look within the gigabytes of data given to me. Fortunately, I found a FEMA Flood Insurance study performed in July of 1980. It had calculated a watershed area (that drains through the same point at the Mary Drive crossing) of 13.42 square miles. My calculated value was approximately 2% larger than the flood insurance value, and the Pinal County engineer deemed that acceptable. Figure 15 shows the area the flood study determined for the watershed at the Mary Drive crossing, as well as the peak flows for the 10-year, 50-year, 100-year, and 500-year return periods.

| | | Peak Discharges (cfs) | | | |
|---|---|---|---|---|---|
| Flooding Source and Location | Drainage Area (sq. mi.) | 10-Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
| Queen Creek | | | | | |
| At Mary Drive | 13.42 | 4,600 | 9,800 | 11,400 | 14,200 |

*Figure 15: Drainage area and peak discharges for storms of various return periods (10-year, 50-year, 100-year, and 500-year from left to right) at the Mary Drive crossing.*

To calculate the curve number for the watershed, I needed to determine the area-weighted average of the different curve numbers within the watershed, as the curve number for the urban region will not be the same as the curve number for the mountainous region. For the Web Soil Survey, I needed to draw an area of interest. Just like in Adobe Acrobat to determine the

watershed area, I drew a polygon to approximate the area of the watershed. The area of interest turned out to be 12.92 square miles, or approximately 3% smaller than the flood insurance area value. This value was again within the acceptable range. For reference, the soil survey information is included in Appendix A. Figure 16 shows my work for determining the curve number for the watershed. It required a combination of the Web Soil Survey from the NRCS, Figures 9 and 10, and my own judgment of satellite imagery. The curve number I determined for the watershed was 85.8.

| | | | | | | CN | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Map unit symbol | Map unit name | HSG | Acres in AOI | Percent of AOI | A | B | C | D | | |
| | 84 | Rock outcrop-La | D | 89.6 | 1.10% | 63 | 77 | 85 | 88 | | |
| | Subtotals for Soil Survey Area | | | 89.6 | 1.10% | | | | | | |
| | | | | | | CN | | | | Actual | |
| | Map unit symbol | Map unit name | HSG | Acres in AOI | Percent of AOI | A | B | C | D | CN | weighted CN |
| | 400 | Tenneco-Bodecl | (57.5% B, 42.5% A) | 52 | 0.60% | 63 | 77 | 85 | 88 | 71.05 | 0.4263 |
| | 405 | Urban land and | (50% A, 50% C) | 15.9 | 0.20% | 89 | 92 | 94 | 95 | 91.5 | 0.183 |
| | 410 | Urban land and | (50% B, 50% C) | 570.9 | 6.90% | 77 | 85 | 90 | 92 | 87.5 | 6.0375 |
| | 420 | Deloro-Andrada | (25% C, 75% D) | 6.6 | 0.10% | 63 | 77 | 85 | 88 | 87.25 | 0.08725 |
| | 450 | Andrada extrem | D | 39.3 | 0.50% | 63 | 77 | 85 | 88 | 88 | 0.44 |
| ASSUMPTION | 500 | Mined land (ass | C | 401.1 | 4.90% | 72 | 81 | 88 | 91 | 88 | 4.312 |
| | 570 | Rock outcrop-M | D | 954.2 | 11.50% | 63 | 77 | 85 | 88 | 88 | 10.12 |
| | 600 | Oxyaquic Torrifl | (82% A, 16% D) | 32.5 | 0.40% | 63 | 77 | 85 | 88 | 67.5 | 0.27 |
| | 655 | Bodecker soils a | A | 22.9 | 0.30% | 63 | 77 | 85 | 88 | 63 | 0.189 |
| | 710 | Rock outcrop-W | D | 533.6 | 6.50% | 63 | 77 | 85 | 88 | 88 | 5.72 |
| ASSUMPTION | NOTCOM | No Digital Data A | D | 5,550.20 | 67.10% | 63 | 77 | 85 | 88 | 88 | 59.048 |
| Subtotals for Soil Survey Area | | 8,179.20 | 98.90% | | | | | | | | |
| Totals for Area of Interest | | 8,268.80 | 100.00% | | | | | | | | 86.8 |
| | | | | | | | | | | | |
| | | | | | CN if 2/3 of watershed is Soil Group C | | | | | 84.8 | |
| | | | | | CN if 2/3 of watershed is Soil Group D | | | | | 86.8 | |
| | | | | | Average between the two scenarios | | | | | 85.8 | |

*Figure 16: Excel spreadsheet work to determine the curve number for the watershed.*

Similar to determining the curve number, determining the area of impervious surfaces was in part based on my judgment. However for impervious surfaces, I used the 13.70 square miles value for drainage area, because I believe it is better to slightly overestimate the area of impervious surfaces rather than underestimate it. In the soil survey online software, I drew a polygon around the town of Superior to get an idea of the number of acres that had a significant area of impervious surfaces. I determined that about 655 acres of the drainage area had some

degree of impervious surfaces. This meant that about 8110 acres, or around 92% of the watershed, basically did not have any impervious surfaces, as it was mainly in the desert and mountains. (I did take out 0.5% for roads though). Figure 17 shows my work for determining the percent area of impervious surfaces. I determined that the watershed has 5.16% impervious surfaces by area.

| | | | | | | |
|---|---|---|---|---|---|---|
| 655.8 | # of acres of watershed with some degree of impervious area | | | | | |
| 8768 | total acres of watershed | | | | | |
| | | | | | 8768 | acres |
| 8112.2 | # of acres that have essentially no impervious area | | | | 13.7 | $mi^2$ |
| 92.5205292 | % of watershed area that has essentially no impervious area | | | | | |
| -0.50% | account for some roads in this area | | | | | |
| **92.02** | **realistic % of watershed area that has no impervious area** | | | | | |
| | | | | | | |
| | | | | | | |
| About 80% of 655.8 acres is residential, 20% is commercial | | | | | | |
| | | | | | | |
| | | Impervious % | Acres impervious | | | |
| 524.64 | acres of residential 1/8 acre lots | 65 | 341.016 | | | |
| 131.16 | acres of commercial | 85 | 111.486 | | | |
| | | | | | | |
| | | | 452.502 | acres of impervious | | |
| | | | | | | |
| | **percent of total watershed that is impervious** | | **5.16** | | | |
| | | | | | | |

*Figure 17: Excel spreadsheet work to determine the percent of impervious area in the watershed.*

For the unit hydrograph transformation, all I needed to calculate was the lag time. Using the equations in Figures 11 and 12, I determined the lag time to be 2.36 hours. Figure 18 contains the work I completed in Excel.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Hydraulic length of the watershed | | L= | 8.71 | miles | | | | |
| | Area of the watershed | | A= | 13.7 | $mi^2$ | | | | |
| | | | | | | | highest elevation | 5240 | ft |
| | | | | | | | lowest elevation | 2650 | ft |
| | | | S = (1000/CN)-10 | | | | Elev diff | 2590 | ft |
| | | | CN = | 85.8 | | | length | 45988.8 | ft |
| | | | S = | 1.65501166 | | | slope | 5.63 | % |
| | | | | | | | Y | 5.63 | % |
| | | $t_L$= | 2.35988435 | hours | | | | | |

| CN | | $t_L$ (hrs) | $t_L$ (mins) |
|---|---|---|---|
| **84.8** | If 2/3 of watershed is Soil Group C | **2.45** | **147** |
| **86.8** | If 2/3 of watershed is Soil Group D | **2.28** | **136.8** |
| **85.8** | Average between the two scenarios | **2.36** | **141.6** |

*Figure 18: Excel spreadsheet work to determine the lag time for the watershed.*

For the meteorologic model parameters, the frequency storm was the best-suited precipitation method. The process is described in detail in the Methods section, but Figure 19 shows a screenshot from the input screen from HEC-HMS.

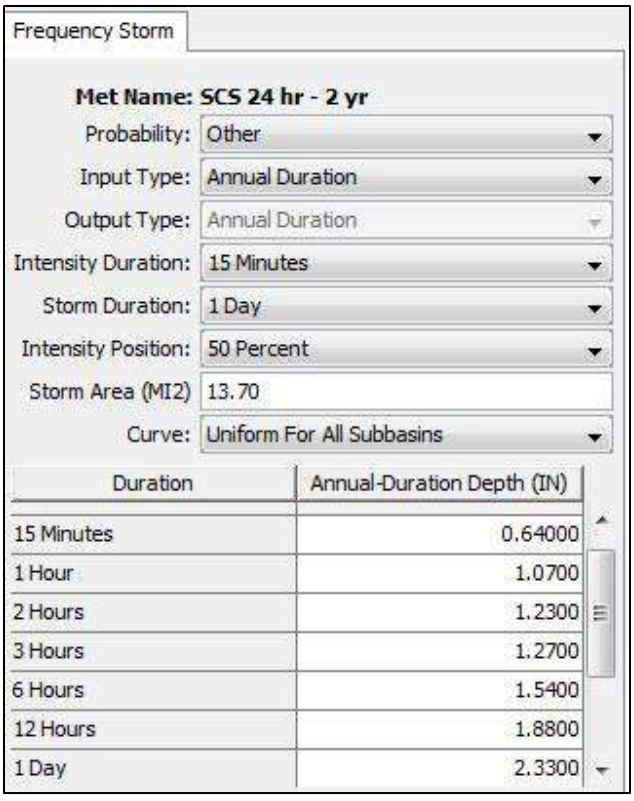


*Figure 19: Screenshot of the frequency storm inputs for HEC-HMS. The Annual-Duration Depth values were taken from Figure 13.*

After calculating all of the parameters required for HEC-HMS to run properly, a simulation was ran and results were achieved. Table 1 shows the peak flow values for the 10-year, 25-year, 50-year, and 100-year storms. The 2-year and 5-year values were not included because the Superior team determined that if they are going to design a culvert, it should be big enough to at least handle a 10-year return flow. These values will be compared to those in Figure 15 in the Discussion section.

*Table 1: Peak flow values for storms of various return periods at the Queen Creek – Mary Drive crossing.*

| Return Period | Peak Discharge (cfs) |
|---|---|
| 10-year | 3,335 |
| 25-year | 4,345 |
| 50-year | 5,250 |
| 100-year | 5,885 |

In addition to a table of values, it is often helpful to be able to analyze a figure related to peak flow values. Figure 20 shows the Rainfall depths and outflow hydrograph for a 50-year return period flow.

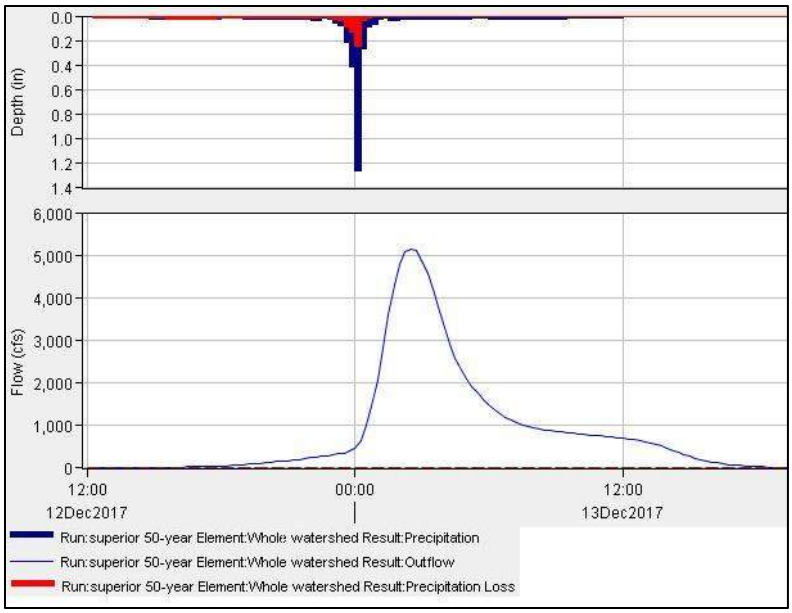


*Figure 20: A bar graph of rainfall depths and a line graph of the outflow hydrograph for a 50-year return period flow.*

**Results** – Culvert Design

Because there are two rounds of culvert designs, and each design has front view, side view, data analysis, and input figures, I have decided to include all of the figures from the culvert designs in Appendix B. The results of the first round of culvert designs were good. I only used the peak flow for the 50-year return period, but I was able to create several designs that were able to pass the 50-year flow completely through the culvert, while having the 100-year flow overtop the culvert and the road at various depths. One of the issues with my first round of designs was that the culverts were quite heavy, and thus quite expensive. So I asked the county engineer for advice, and he suggested trying a concrete arch design (ConArch), as they can sometimes perform better than a box or circular culvert.

I tried out the ConArch design, and it turned out pretty well. For example, the most efficient 10-year, box culvert design was 4 - 8'x8' barrels. Each weighed 3600 lbs/ft (total of 14,400 lbs/ft). A ConArch single barrel (24' span x11' rise) weighed in at only 6650 lbs/ft. Both designs were able to pass the 10-year return period flow. The weights of the circular concrete pipe designs were even higher than that of the box culverts. Additionally, I determined that for a 100 year flow, a 42'x11.5' ConArch would be able to pass that flow entirely through it. It weighed in at 14,968 lbs/ft, not even 600 lbs/ft more than the 10-year best box culvert design.

**Results** – Integration of the hydrologic analysis and culvert design

Before getting results in HEC-RAS, I first had to get results in ArcMap. After importing the elevation data into ArcMap, I was able to use the HEC-GeoRAS extension to successfully create the cross sections, reach lengths, and bank stations that HEC-RAS would recognize.

Figures 21 and 22 show aerial images in ArcMap of the cross sections across the entire Queen Creek in Superior and a zoomed-in image of the Queen Creek – Mary Drive respectively.

*Figure 21: Aerial image in ArcMap of the cross sections created for all of Queen Creek within Superior.*

*Figure 22: Aerial image in ArcMap of the cross sections, specifically at the Queen Creek – Mary Drive crossing. Note the cross section highlighted in blue is immediately downstream of the crossing.*

Once the GIS data was successfully exported from ArcMap, I was able to import it into HEC-RAS. Figure 23 shows the cross sections of the crossing as displayed in the geometry section of HEC-RAS.

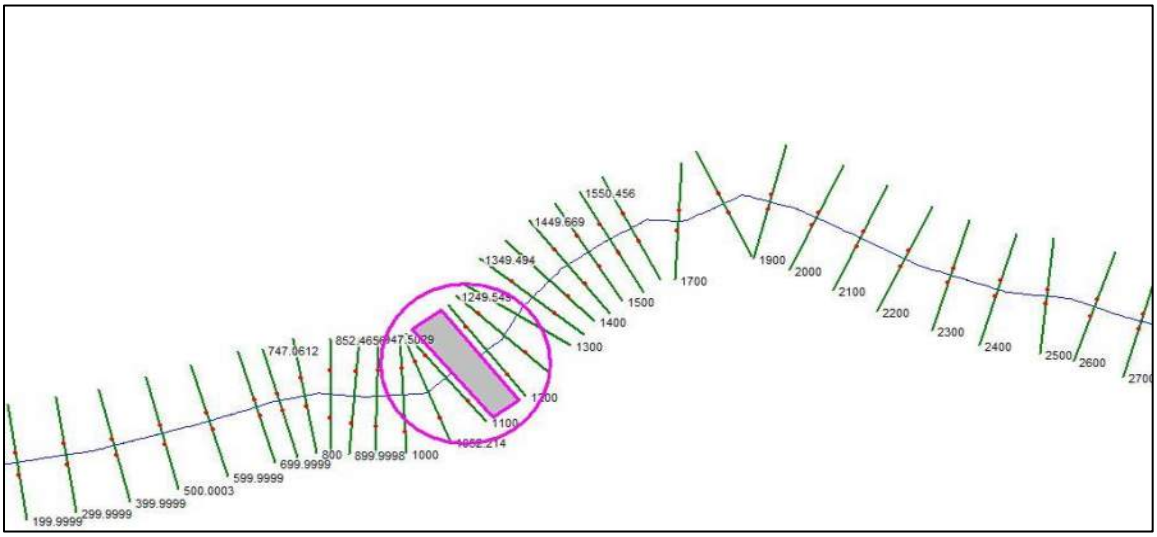


*Figure 23: Close up of the cross sections of the Queen Creek – Mary Drive crossing in HEC-RAS.*

Now that I had the geometry set up, I needed to run a simulation of the peak flow, to see how the cross sections handled it. Figures 24 and 25 show the results from the 10-year peak flow for the upstream and downstream cross sections of the crossing respectively.

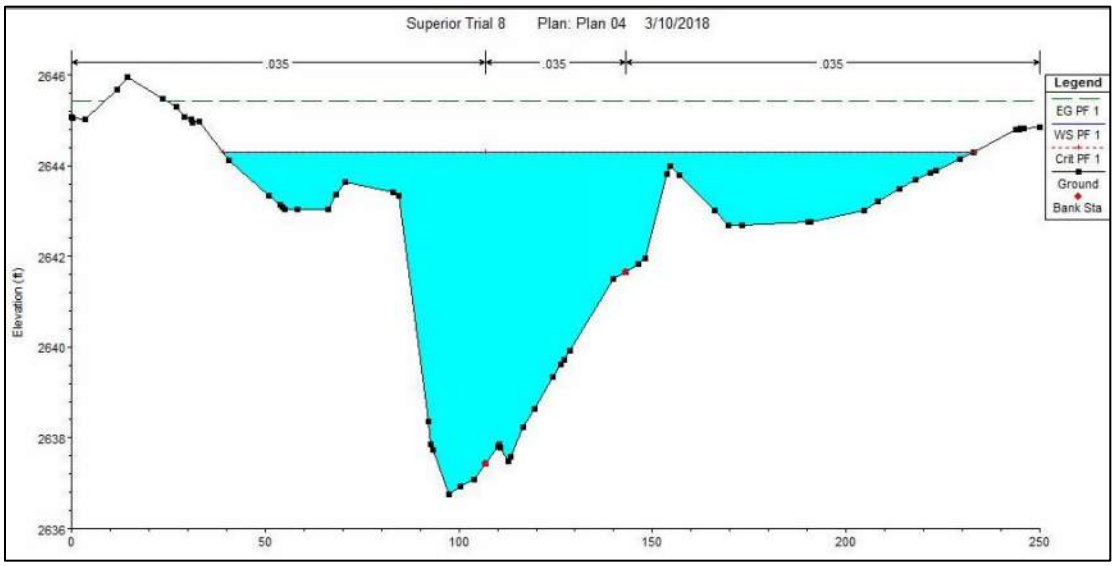


*Figure 24: 10-year peak flow through the cross section immediately upstream of the crossing.*

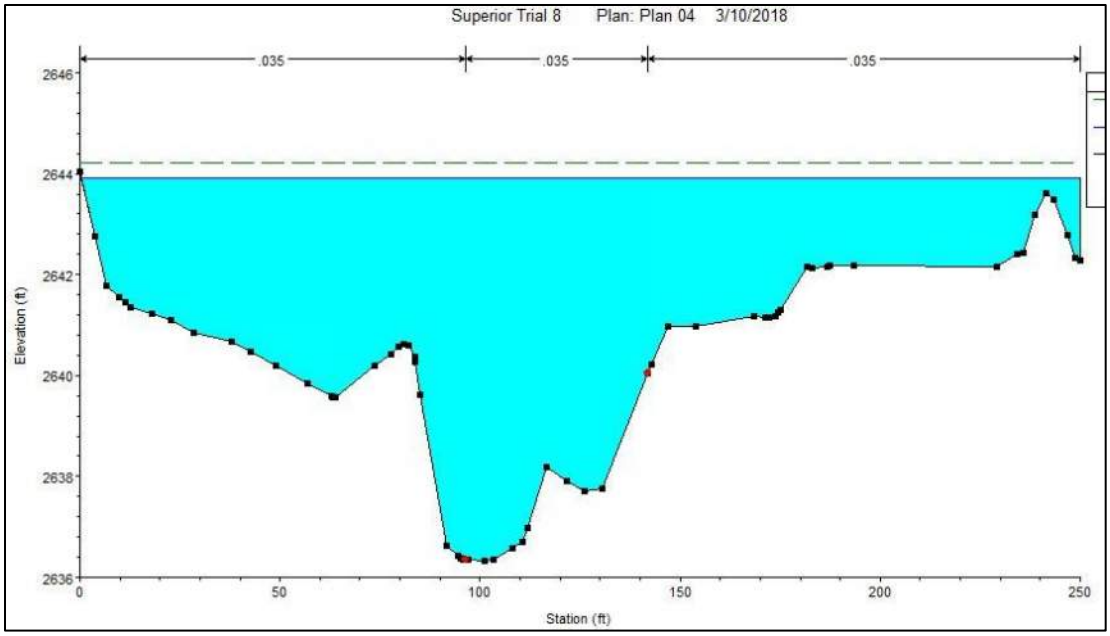


*Figure 25: 10-year peak flow through the cross section immediately downstream of the crossing.*

Now that I had run an initial simulation, I decided to perform an initial channel excavation, as this would be needed to put in a culvert. For the crossing, I had made some preliminary culvert designs. They had the road being about 15 feet above the invert of the channel (as the crossing currently sits, the invert of the channel is basically even with the road). So I knew that the channel would need to be excavated and the slope increased so that the invert of the channel was approximately 15 feet below the elevation of the road at the crossing. So I decided to take the 400 feet before the 1200' cross section (the one immediately upstream of the road) and excavate a 45 foot bottom width trapezoidal channel, 2:1 side slopes, and increase its slope from the natural value of 0.0182 ft/ft to 0.0557 ft/ft. Figure 26 shows a side view of the channel excavation.

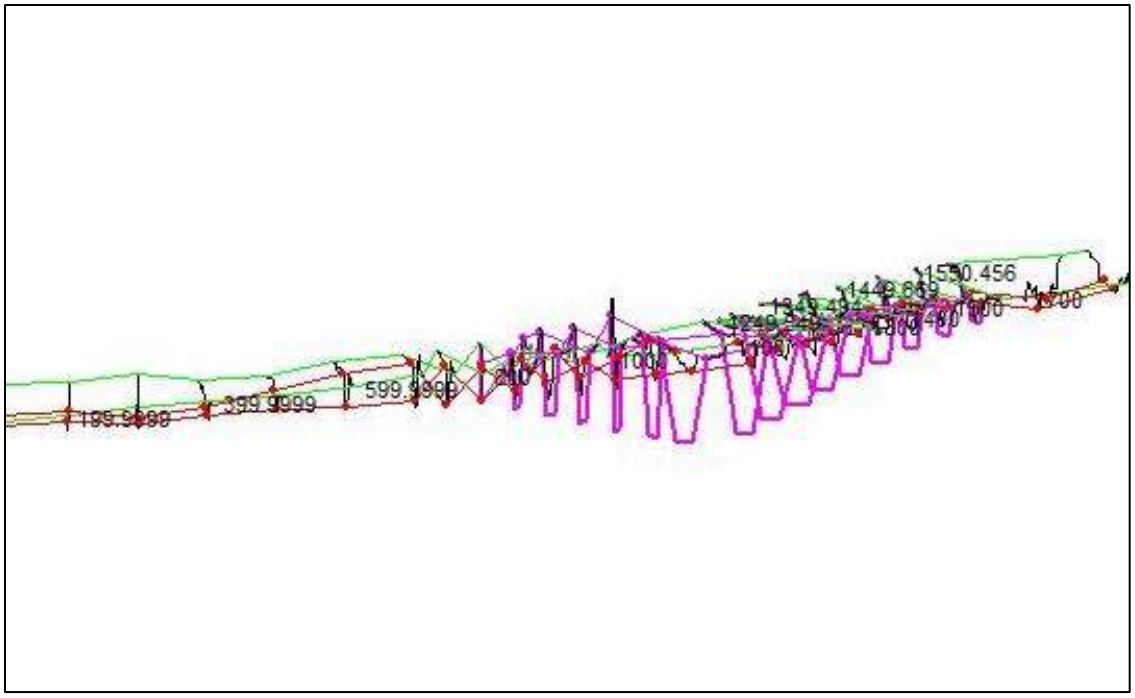


*Figure 26: Side view of the channel excavation at the crossing.*

This allowed the channel invert elevation to be about 15 ft below the road elevation at the crossing. And then I need to excavate on the downstream side of the crossing. But this time, I chose a negative slope (0.0318 ft/ft over the course of 300 ft, from 1100'-800') because both sides of the crossing have to be excavated and they both can't have a positive slope (see pictures). After creating an initial channel excavation, I decided to run a second flow simulation. I used the 10-year flow value (3,335 cfs) just as I had used it in the initial simulation run before any channel modification. Figures 27 and 28 show the results of the HEC-RAS simulation using the 10-year peak flow.

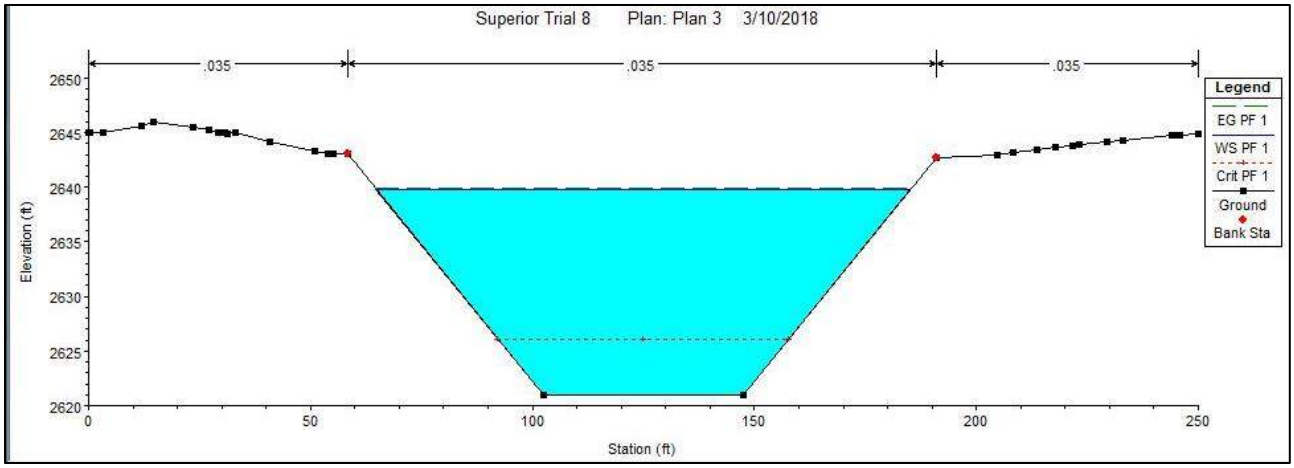


*Figure 27: 10-year flow through the cross-section immediately upstream of the crossing after channel modification*

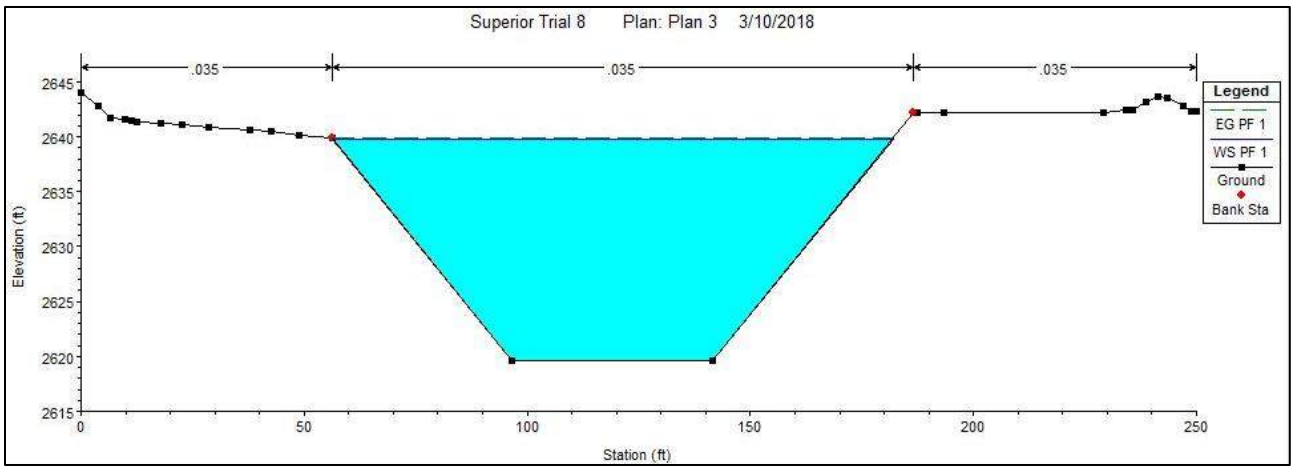


*Figure 28: 10-year flow through the cross-section immediately downstream of the crossing after channel modification*

## Discussion

When I first began work on this project, I knew that the first step was to determine the watershed area. What I did not know was that the watershed area had already been determined by a FEMA flood insurance study done in 1980. While it could have saved me some time by not delineating the watershed, it turned out to not only be beneficial for my own experience, but necessary as well. I needed to know the general area of the watershed for the soil survey, so even

if I had found the FEMA watershed area value beforehand, I would have still needed to delineate the watershed for use with the soil survey.

For this project, I actually went through and calculated the curve number for the watershed twice. The first time I calculated a curve number of about 70 for the entire watershed. This resulted in peak flow values that the county engineer deemed to be too low for a watershed of its size. My mistake originated from the large part of the watershed that the soil survey did not have data on. I assigned that area, approximately two-thirds of the entire watershed, a curve number of 63. This was based on selecting desert shrub – poor hydrologic condition – hydrologic soil group 'A' (see Figure 9). As the county engineer had exponentially more experience with the area of Superior, he said that due to the mountainous terrain and general lack of 'A' soils throughout the mountainous areas of Pinal County, he would expect the soil group for Superior's watershed to be a C or D. After taking the average of the C and D curve numbers, I achieved peak flow values that the county engineer deemed reasonable.

Going back over this project, if there was any part of it that I would do more work on, it would be determining the curve number and impervious surfaces. In Figures 16 and 17, I show my work for determining the curve number and percent of the area covered with impervious surfaces. Both of these determinations were done with educated guesses, and even assumptions in some case. What I would do, if I had more time, would be to give someone the same aerial images and soil survey, and have them determine the curve number and impervious surfaces. That way the two values could be averaged and eliminate a possible high or low unconscious bias in the calculations. For the purposes of this report, I believe that my calculations were acceptable given my methods and the fact that the county engineer approved the final numbers.

Table 1 shows the peak flow values I calculated at the Queen Creek – Mary Drive crossing for storms of various return periods. These values can be compared to the ones in Figure 15 from the FEMA flood insurance study. It is very apparent that the peak flow values FEMA calculated are much higher than the ones I calculated. The county engineer said that my values were still reasonable, and that the FEMA study was done in 1980, almost forty years ago. Since they did not have the same technology and capabilities as we do today, the fact that FEMA's numbers were much higher was to be expected. Figure 20 shows the rainfall depths and the outflow hydrograph for the 50-year return period flow. The county engineer said that based on the precipitation data available from NOAA and the curve number I calculated, both of the graphs seemed to reflect the outcome of a real-world event.

Just like calculating the curve numbers, I had two rounds of preliminary culvert designs. The first round of designs can be seen in the first section of Appendix B, while the second round of designs can be seen in the second section of the same Appendix. After the first round of designs, the county engineer gave me ample amounts of advice, which I used to create my second round of designs. This advice ranged from looking at standard size structures, to weighing the pros and cost of cast in place or pre-cast structures, to trying a whole new design (ConArch). For the second round of culvert designs, I decided to go with the ConArch design over the rectangular or circular culverts. But every design has its pros and cons, and a ConArch design is no different.

For instance, concrete boxes and pipes can be precast, while the ConArch is cast in place. What I personally do not know is the price differences between all the designs. I also do not know how much more time and effort would go into precast culverts versus cast in place culverts. This means that I cannot fully recommend a design because I do not know all that I

need to about it. But I do know that the ConArch designs would be significantly lighter than a comparable box or pipe culvert. And so I thought, as a general idea, that the lighter the design was, the less expensive it would be. The county engineer will be able to take my findings, fill in the blanks, and hopefully come up with a culvert design that will fit the needs and the budget of Superior.

After working through some issues with the elevation data in ArcMap, I was able to successfully get ArcMap and HEC-GeoRAS to run properly and produce the cross sections I needed (Figures 21 and 22). After importing the cross sections, bank stations, and reach lengths into HEC-RAS (Figure 23), I ran an initial simulation with the 10-year return period flow. The results were quite astonishing (Figures 24 and 25). The flooding immediately upstream and downstream of the Queen Creek – Mary Drive crossing was very significant. I made an initial channel excavation (Figure 26) and ran the same simulation again. The results from the second simulation reduced flooding at the crossing significantly (Figures 27 and 28). But it is important to point out that both simulations were run without a culvert in it. So HEC-RAS ran the simulation and thought that there was nothing in between the two cross sections. Putting a culvert there would increase the water surface level of the flood. So it was clear that an even more expansive channel excavation operation would be needed to reduce the water surface level even further.

My initial channel excavation was fairly simple (as described in the Results section). But I still had many questions. I was not sure whether to have a really long, very gradually sloping excavation; a short, steep sloping excavation; or something in the middle. I was also not sure whether to adjust the bottom width, invert elevation, Manning's n of the trapezoidal cut, whether the plan would be to line the bottom and/or sides of the new channel, etc. At this point, I decided

that my work on this project had come to a close. I was able to do quite a lot for the team in Superior, ranging from hydrologic analysis, to preliminary culvert design, to creating a model of Queen Creek in ArcMap, and finally integrating all three of these design elements into HEC-RAS. All that remains on this project is to determine how much channel excavation the town is looking to perform and put a final culvert design into HEC-RAS. I have confidence that the town manager, the county engineer, and all of the members of the Superior team will be able to use what I have created and come up with a culvert design that will improve the lives of the residents of Superior and the response times of emergency services.

## Conclusion

There were four main conclusions to this report. First, after delineating the watershed and using HEC-HMS to determine peak flow values, the county engineer confirmed that the values were reasonable. There are other methods to determine peak streamflow values. The USGS has a program called StreamStats, which uses Regional Regression equations to determined discharge values. But this method is very simplistic and has large margins of error. So the county engineer and myself agreed that my method was more accurate. The second was that by using HY-8, several different culvert designs were successfully created to pass the various peak flows. This was an important step as it allowed the Superior team to get a general idea of the size and cost of the culvert.

The third conclusion was that I was able to successfully create geometry data in ArcMap (with the help of HEC-GeoRAS) and import the data into HEC-RAS. This step was crucial as HEC-RAS was the software needed to look at how the flooding of Queen Creek at the crossing could be mitigated by a culvert design. And the fourth conclusion was that after initial simulations were run in HEC-RAS, a significant channel excavation would be needed, in

addition to the implementation of a culvert to the channel. Flood simulations were run with and without channel excavation, and even the initial channel excavation would not have been sufficient since a culvert design had not been built to the model yet. The only steps remaining in the project are a final channel excavation design and a final culvert design. Once they determine how much of the channel they will need to excavate, the Superior team will then have a better idea of the size the culvert will need to be.

When I first began this Master's Report, I knew it would not be nearly as in-depth as a thesis, as a thesis is at least four credit hours, while the report was one credit hour. But I still wanted to work on a project that would be meaningful. Dr. Lansey introduced me to the Superior team and they had a few flooding issues that they were trying to fix. They told me about an issue they were having that was causing problems with the response times of emergency services. That issue stuck out to me as the most important water issue they were facing that I could have a hand in helping to fix. While the town of Superior only has a population of around 3,000 people, I knew I immediately had a connection and wanted to help them with their problems. Although I did not grow up in a small town, I lived with fraternity brothers who did, and they gave me some perspective on the benefits of the small town life.

Overall, I am glad I chose this project as my Master's report. I feel that I have learned a lot, applied my knowledge to solve problems, and have helped the Superior team with the issues they are facing. While my patience was tried numerous times working with some of the computer programs, I kept thinking positively and pushed through. I look forward to using the knowledge and skills I have acquired with this project in my future career.

**Appendix A**

Contents page of Soil Survey Report (40 pg. survey available upon request)

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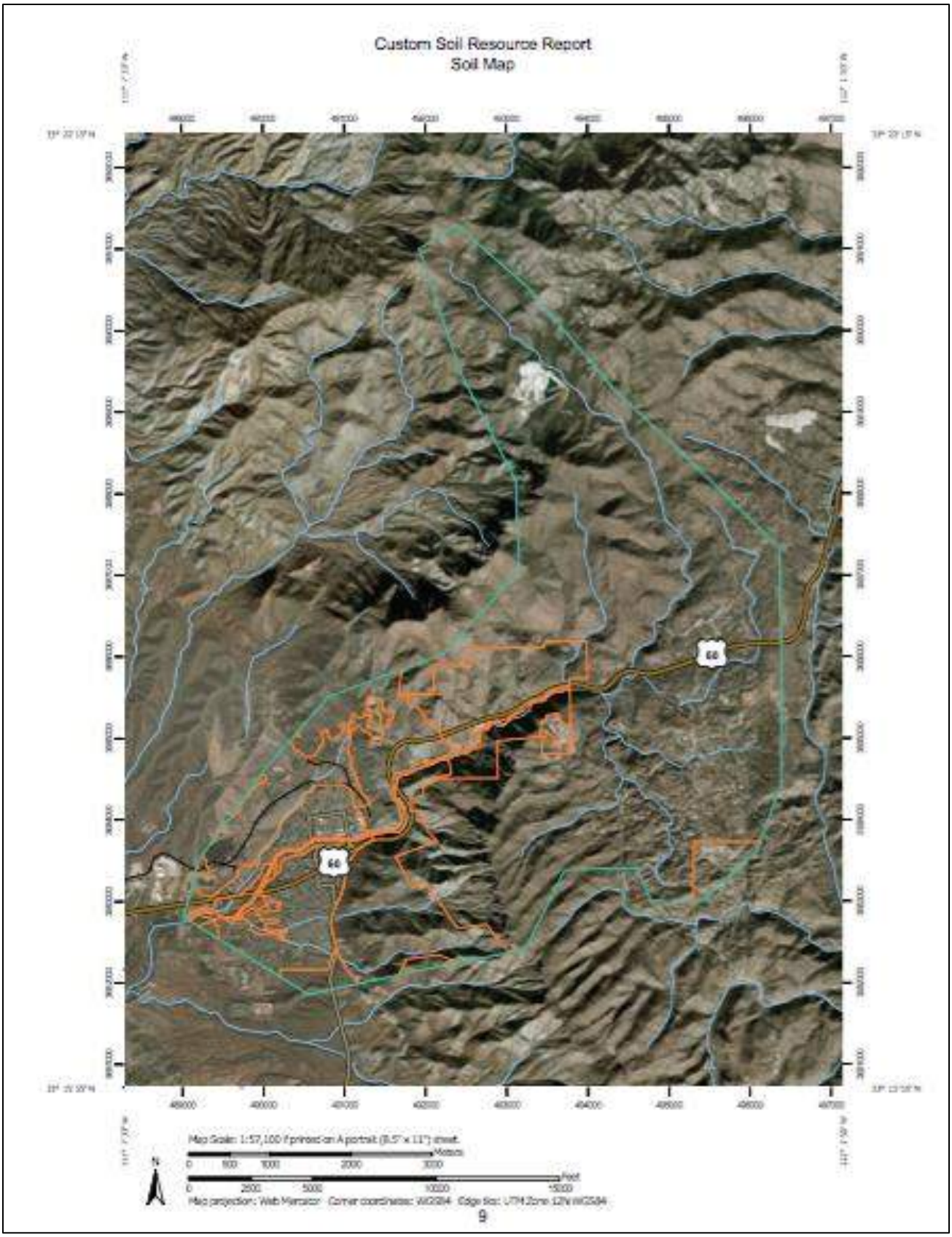
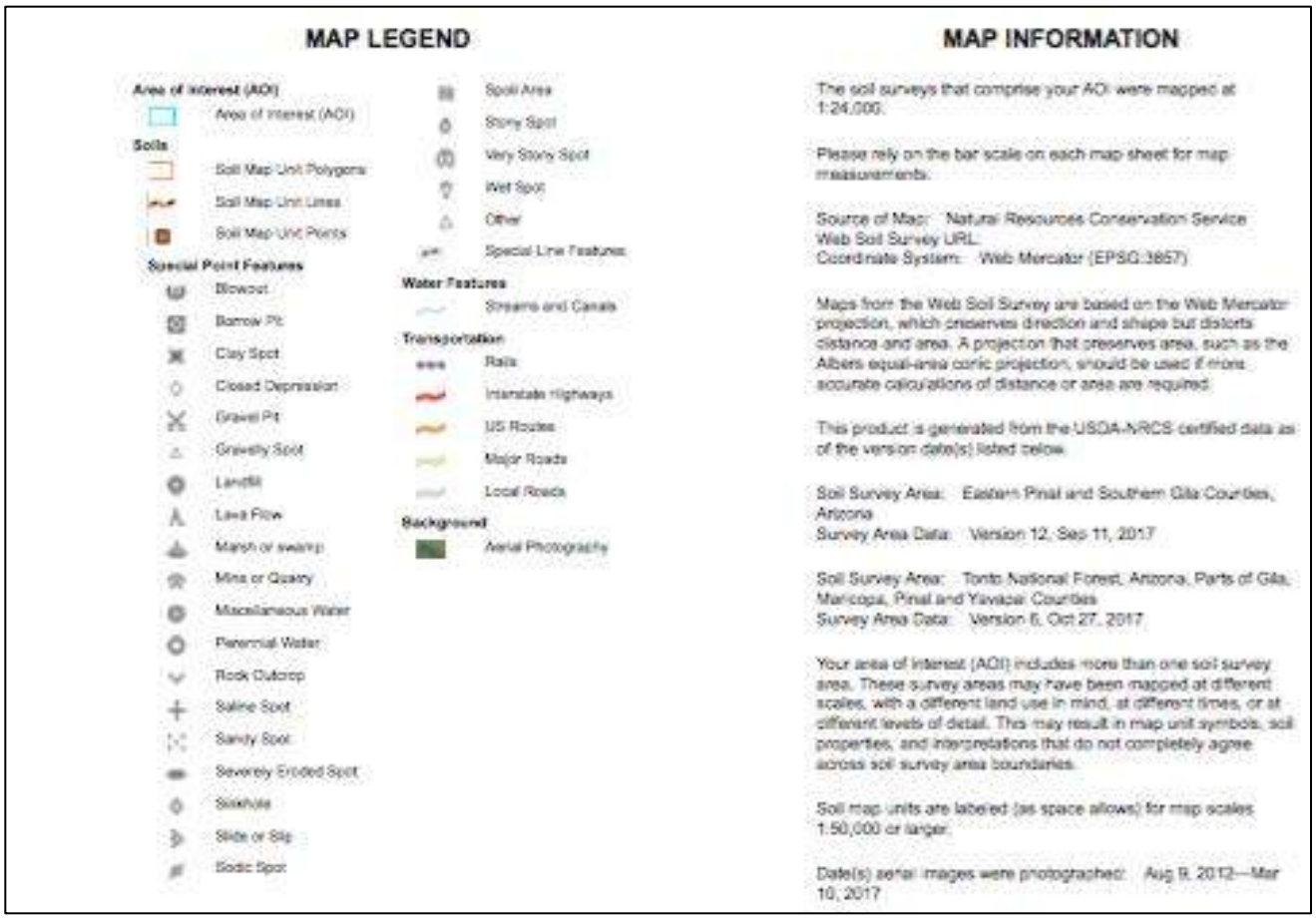
Soil Survey map



Soil map legend



Soil map breakdown

| Map Unit Symbol | Map Unit Name | Acres in AOI | Percent of AOI |
|---|---|---|---|
| 84 | Rock outcrop-Lampshire complex, chaparral, 5 to 50 percent slopes | 89.6 | 1.1% |
| **Subtotals for Soil Survey Area** | | **89.6** | **1.1%** |
| **Totals for Area of Interest** | | **8,268.8** | **100.0%** |

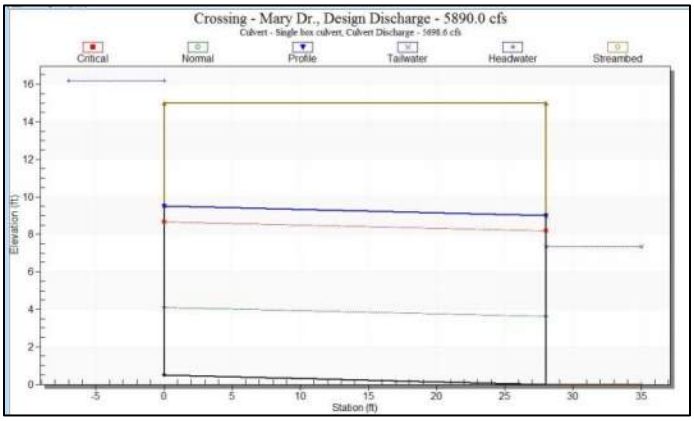
| Map Unit Symbol | Map Unit Name | Acres in AOI | Percent of AOI |
|---|---|---|---|
| 400 | Tenneco-Bodecker complex, 0 to 3 percent slopes | 52.0 | 0.6% |
| 405 | Urban land and Haplic Torrianents soils, 0 to 5 percent slopes | 15.9 | 0.2% |
| 410 | Urban land and Haplic Torrianents soils, 2 to 15 percent slopes | 570.9 | 6.9% |
| 420 | Deloro-Andrada-Sasabe, deep complex, 2 to 15 percent slopes | 6.6 | 0.1% |
| 450 | Andrada extremely gravelly sandy loam, 10 to 40 percent slopes | 39.3 | 0.5% |
| 500 | Mined land | 401.1 | 4.9% |
| 570 | Rock outcrop-Mabray-Pantak complex, 20 to 70 percent slopes | 954.2 | 11.5% |
| 600 | Oxyaquic Torrifluvents and Typic Fluvaquents soils and Riverwash, 0 to 5 percent slopes | 32.5 | 0.4% |
| 655 | Bodecker soils and Riverwash, 0 to 5 percent slopes | 22.9 | 0.3% |
| 710 | Rock outcrop-Woodcutter complex, tuff, 15 to 50 percent slopes | 533.6 | 6.5% |
| NOTCOM | No Digital Data Available | 5,550.2 | 67.1% |
| **Subtotals for Soil Survey Area** | | **8,179.2** | **98.9%** |
| **Totals for Area of Interest** | | **8,268.8** | **100.0%** |

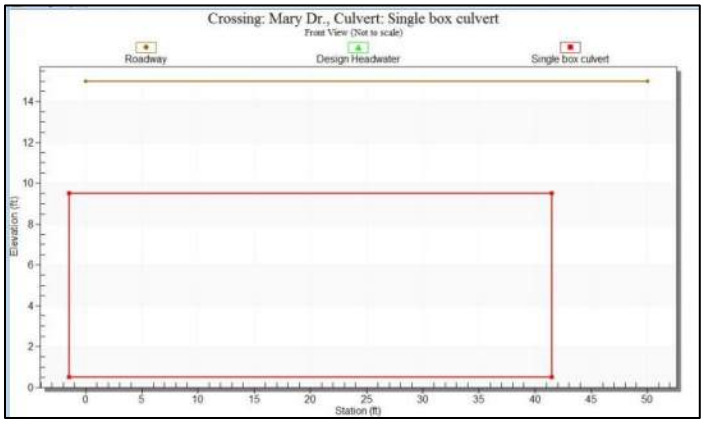
## Appendix B

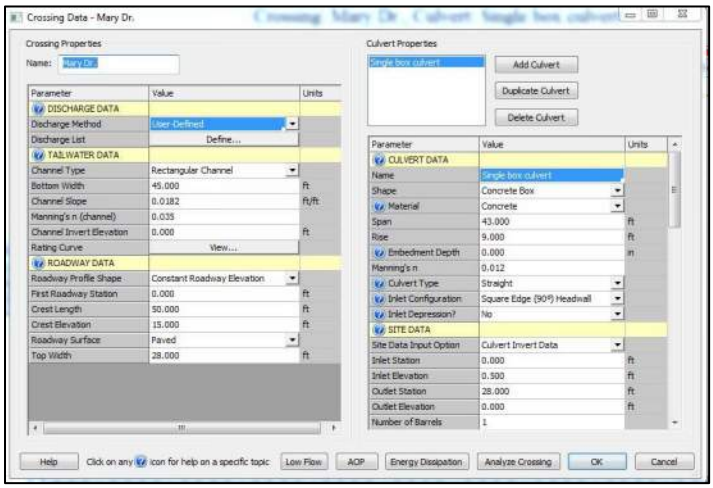
First round of culvert designs

(Clockwise from top-left: side view, front view, data analysis, inputs)

Single large concrete box culvert, 50 year

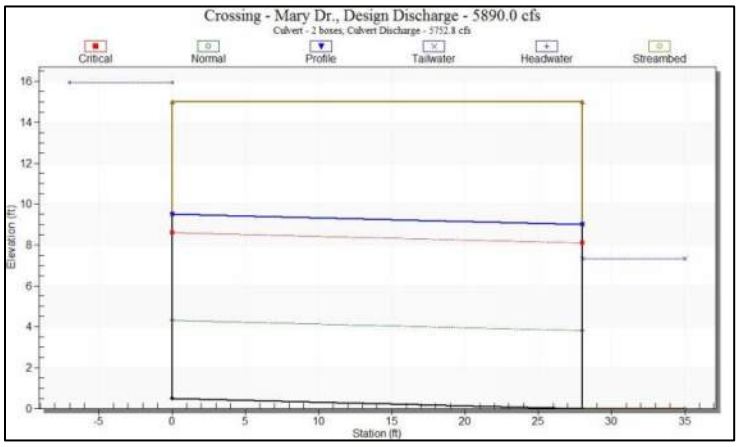


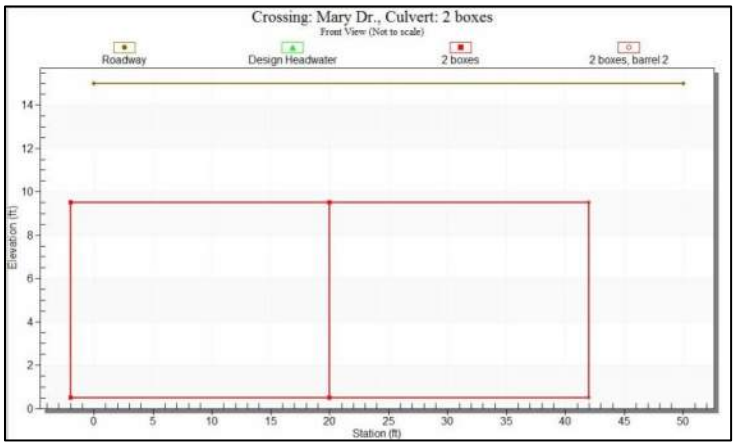



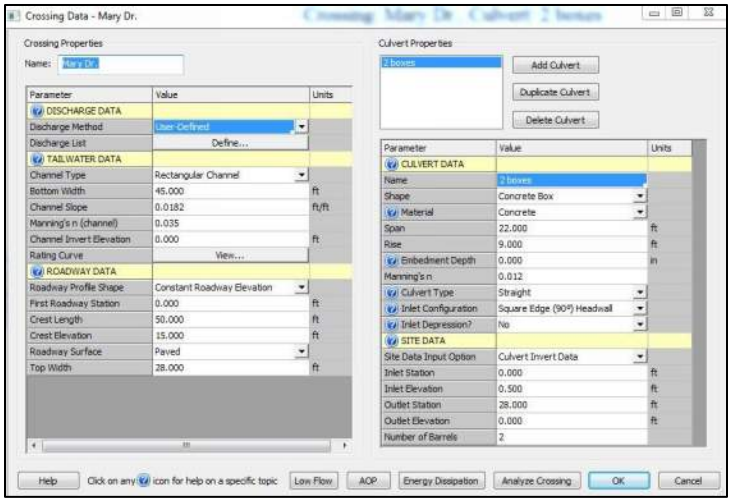




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | Single box culvert Discharge | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.85 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 16.17 | 100 yr | 5890.00 | 5698.61 | 191.00 | 4 |
| 15.00 | Overtopping | 5304.22 | 0.00 | 1.00 | |

Concrete box culvert – 2 barrels, 50 year

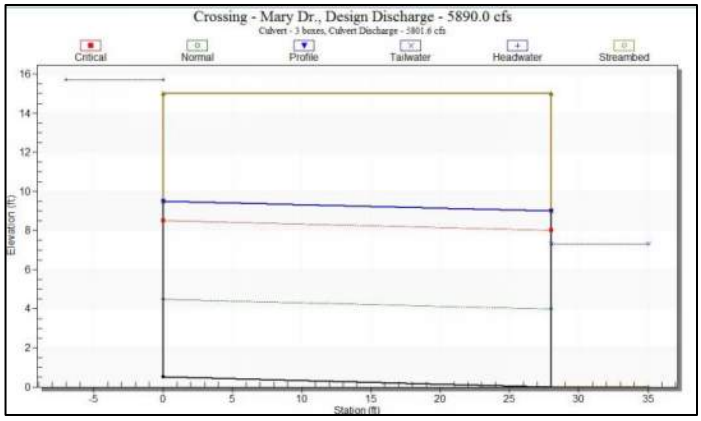


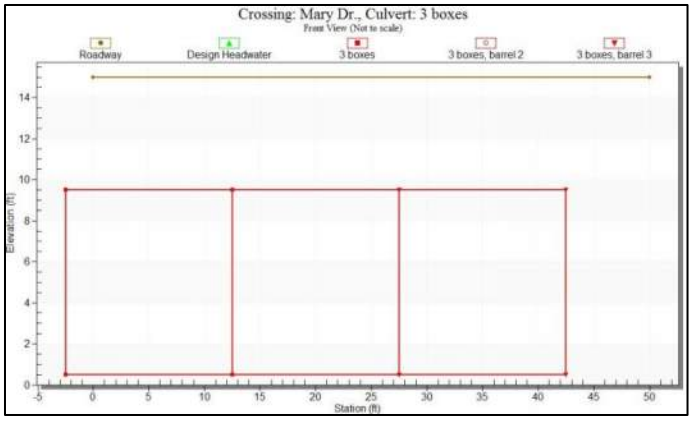



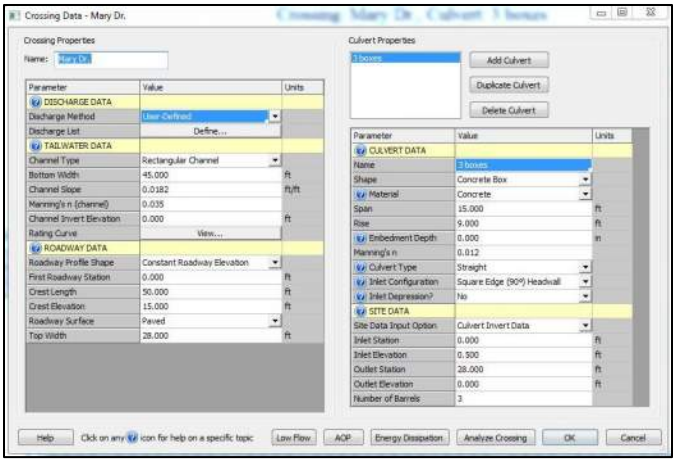




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | 2 boxes Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.51 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 15.93 | 100 yr | 5890.00 | 5752.80 | 136.69 | 4 |
| 15.00 | Overtopping | 5427.44 | 0.00 | 1.00 | |

Concrete box culvert – 3 barrels, 50 year

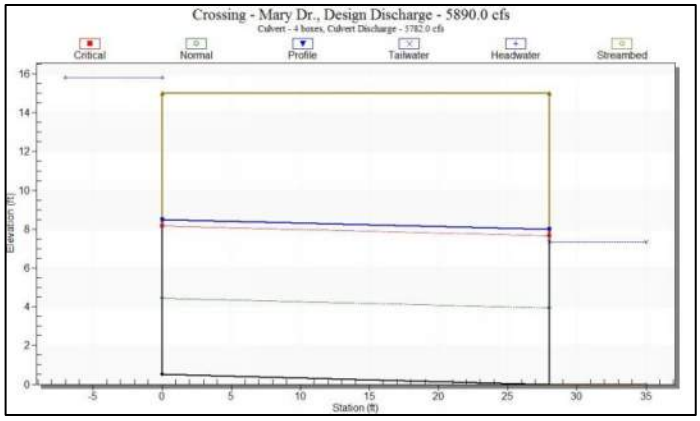


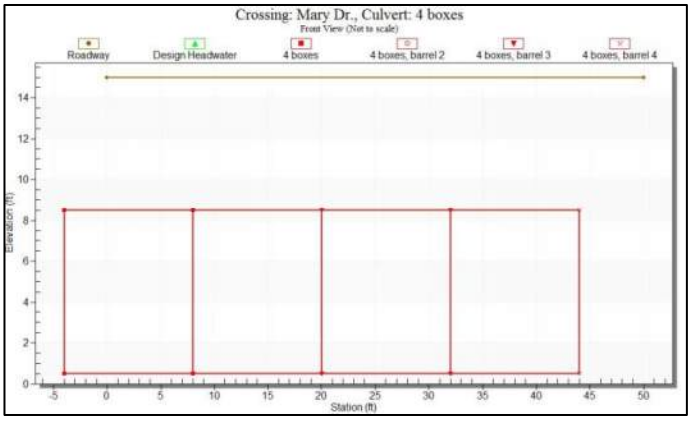



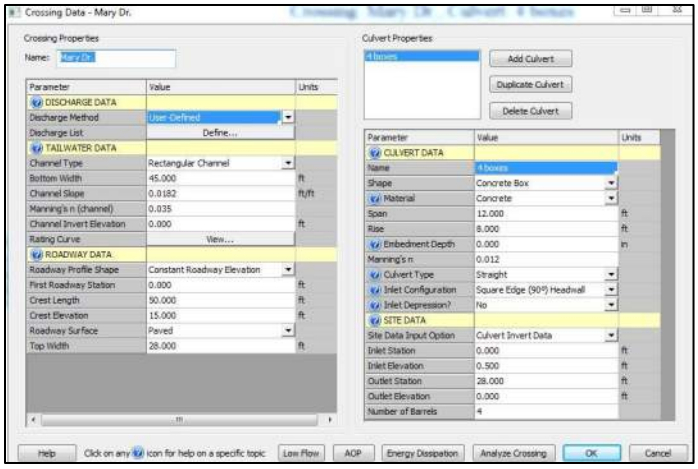




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | 3 boxes Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.20 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 15.70 | 100 yr | 5890.00 | 5801.55 | 88.42 | 5 |
| 15.00 | Overtopping | 5550.83 | 0.00 | 1.00 | |

Concrete box culvert – 4 barrels, 50 year

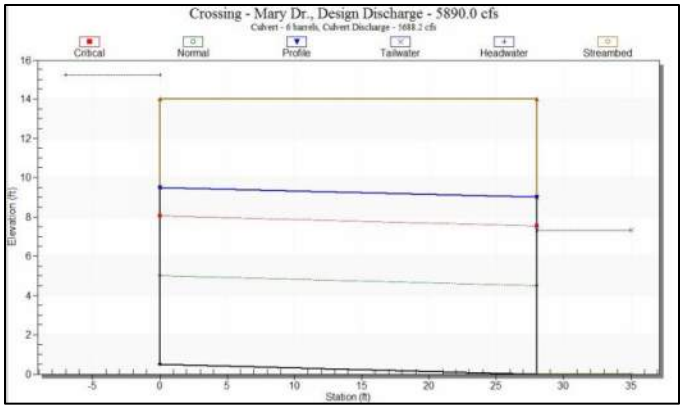


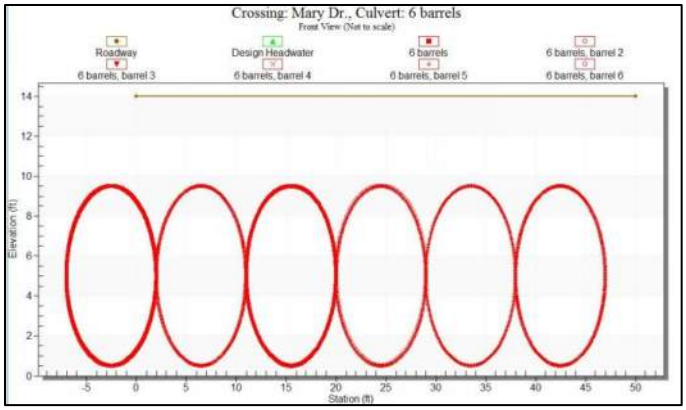



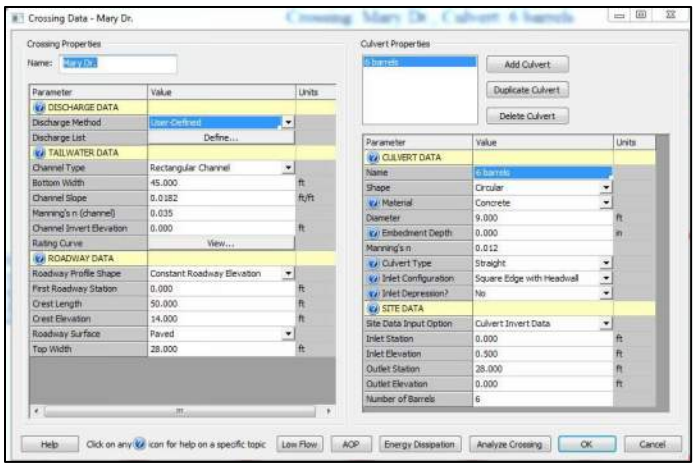




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | 4 boxes Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.19 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 15.80 | 100 yr | 5890.00 | 5782.00 | 107.47 | 4 |
| 15.00 | Overtopping | 5524.53 | 0.00 | 1.00 | |

Concrete circular culvert – 6 barrels, 50 year



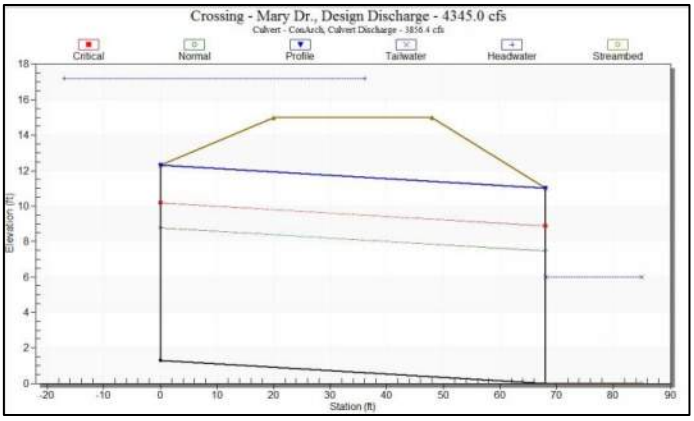



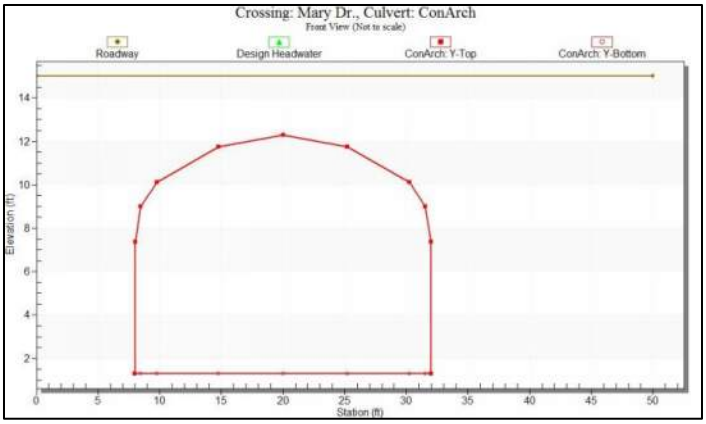

| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | 6 barrels Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 13.93 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 15.21 | 100 yr | 5890.00 | 5688.17 | 201.50 | 4 |
| 14.00 | Overtopping | 5275.37 | 0.00 | 1.00 | |

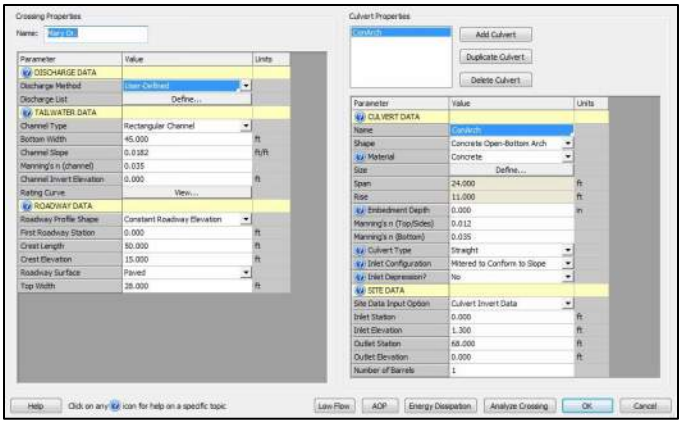
Second round of culvert designs

(Clockwise from top-left: side view, front view, data analysis, inputs)

ConArch, 10 year

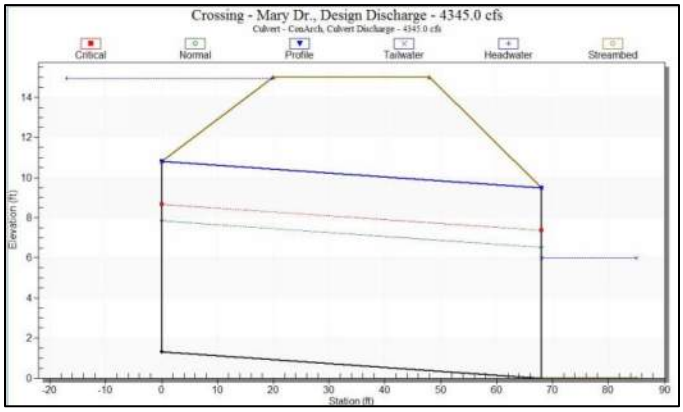


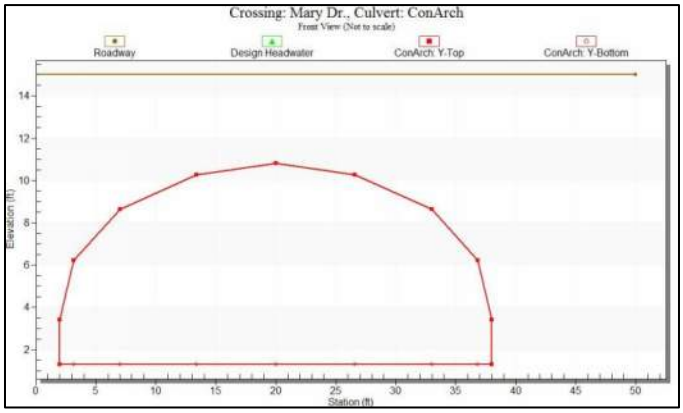



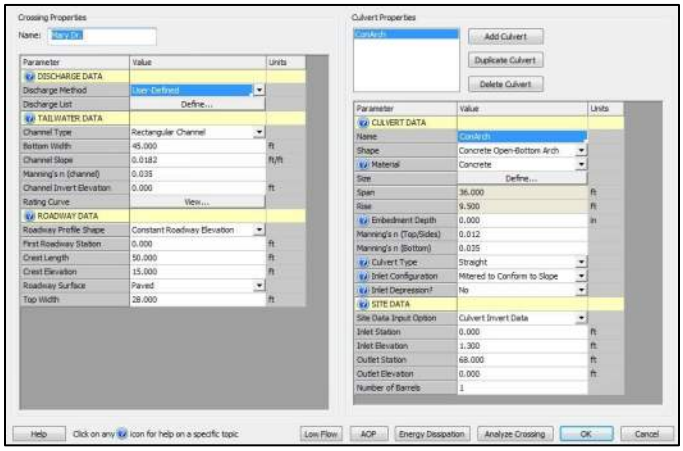




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | ConArch Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.39 | 10 yr | 3335.00 | 3335.00 | 0.00 | 1 |
| 17.18 | 25 yr | 4345.00 | 3856.43 | 488.50 | 5 |
| 15.00 | Overtopping | 3348.65 | 0.00 | 1.00 | |

ConArch, 25 year

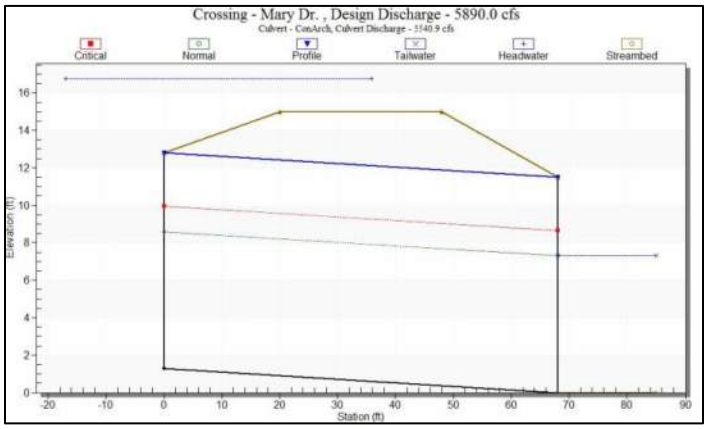


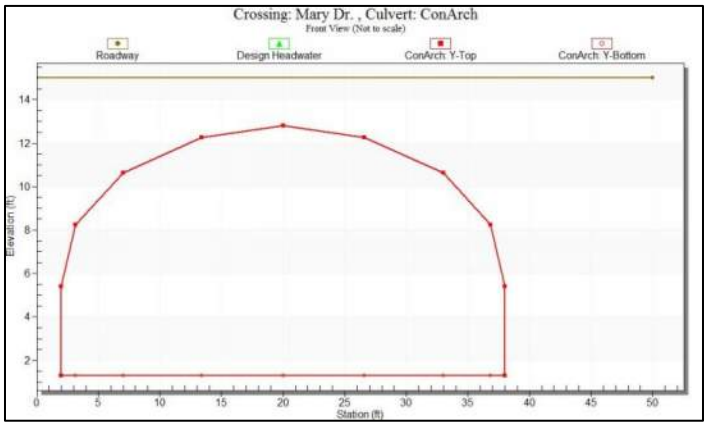



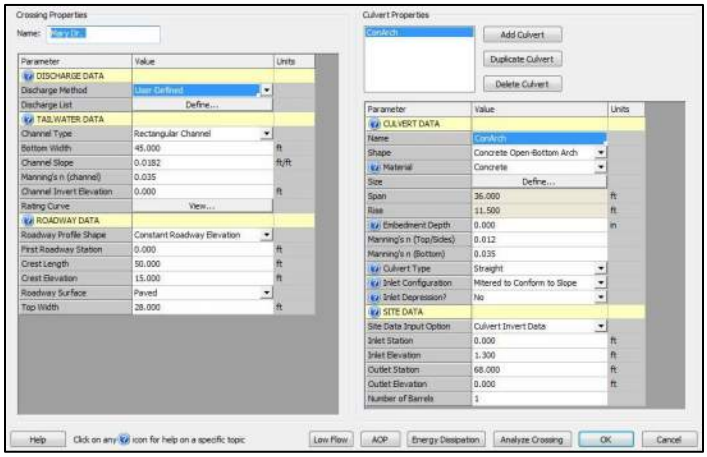




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | ConArch Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 10.26 | 10 yr | 3335.00 | 3335.00 | 0.00 | 1 |
| 14.95 | 25 yr | 4345.00 | 4345.00 | 0.00 | 1 |
| 15.00 | Overtopping | 4360.69 | 0.00 | 1.00 | |

ConArch, 50 year

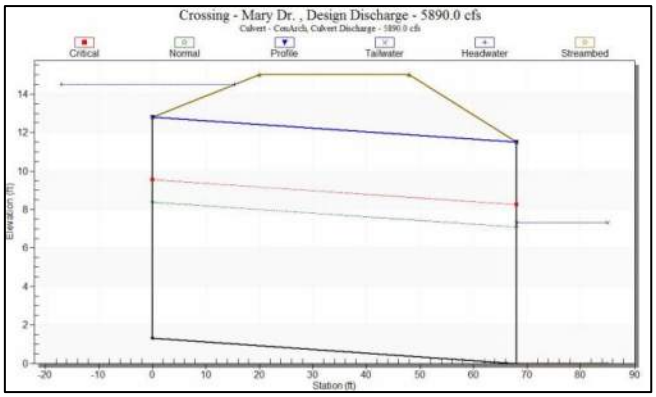


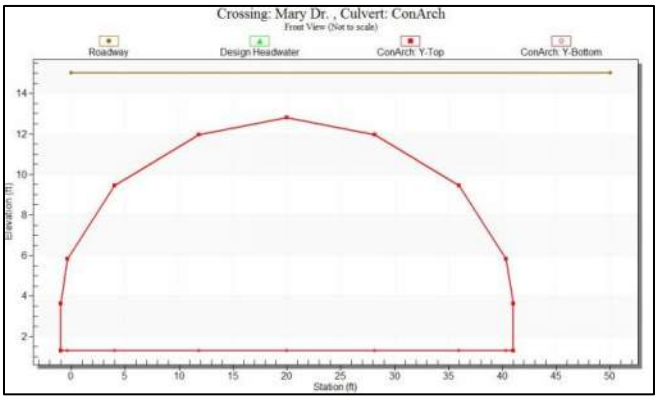



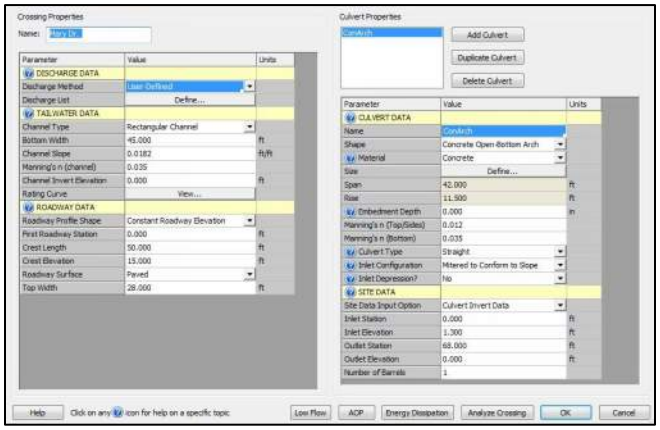




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | ConArch Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 14.36 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 16.74 | 100 yr | 5890.00 | 5540.94 | 348.51 | 6 |
| 15.00 | Overtopping | 5444.49 | 0.00 | 1.00 | |

ConArch, 100 year




| Headwater Elevation (ft) | Discharge Names | Total Discharge (cfs) | ConArch Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 12.64 | 50 yr | 5250.00 | 5250.00 | 0.00 | 1 |
| 14.49 | 100 yr | 5890.00 | 5890.00 | 0.00 | 1 |
| 15.00 | Overtopping | 6067.85 | 0.00 | 1.00 | |