Town of Camp Verde Community Park Site Plan

Final Design recommendations

Prepared For:

Ron Long Camp Verde Town Engineer

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1.0 Project Personnel

Elwid Mubarak

Currently an Environmental Engineering undergraduate student at Northern Arizona University. Expected date of graduation is December 2013.

The student has taken the following classes that are related to the capstone project, and has the following background information for each class:

- CENE 333: Water Resources
- CENE 410: Unit Operations in Environmental Engineering
- CENE 332: Solid and Hazardous Waste Management
- CENE 383: Geotechnical Engineering

Dejan Dudich

Working towards a B.S. in Civil Engineering, at Northern Arizona University. With an expected graduation date of December 2013.

Has taken several classes that pertain to the project and form a background from which to satisfactorily work with.

- Cene 420: traffic and signal studies
- Cene 543: Urban Transportation Planning
- Cene 333: Hydraulics/ water resources
- Cene 383: Geotechnical Engineering
- Cene 336: Water resources 2/ hydrology and flood control.

LeAnne Little

Working towards a B.S. in Civil Engineering, at Northern Arizona University.

Has experience with working in a team on engineering fundamental designs.

The following classes taken are relevant to technical engineering writing and in designing an entry road.

- CENE 186 :Introduction to Engineering Design
- CENE 286: Engineering Design: Process
- CENE 386: Engineering Design: The Methods
- CENE 180: Computer Aided Drafting
- CENE 333L: Water Resources Lab
- CENE 270: Surveying
- CENE 420: Traffic Study and Signal
- CENE 418: Highway Engineering

LeAnne Little will be designing the entry road to Camp Verde's new 118-acre park. The design of horizontal and vertical alignments, cross-sections, roadside design, and drainage system are the following perimeters LeAnne will aid in design.

Steven Tallas

Currently an Environmental Engineering undergraduate student at Northern Arizona University. Expected date of graduation is December 2013.

Has taken several classes that pertain to the project and form a background from which to satisfactorily work with.

- CENE 333: Water Resources
- CENE 410: Unit Operations in Environmental Engineering
- CENE 332: Solid and Hazardous Waste Management
- CENE 383: Geotechnical Engineering
- CENE 485: Leupp Family Farms conservation/solar planning

2.0 WORK PLAN

2.1 Introduction:

The Town of Camp Verde is planning to construct a Community recreation Park that will include baseball fields, BMX course, Soccer and football fields, trails, picnic areas and more recreation activities. The park is planned to be built on an undeveloped 118 acres of land that the city has purchased. There are currently no engineered plans except topography maps, minor surveying maps, and a conceptual plan for the future.

2.2 Understanding and Approach

The proposed Camp Verde Community Park is located on a 118 acre parcel of land on the east side of Camp Verde located between McCracken Lane and State Route HWY 260. While a Conceptual plan for the park layout is available and no Engineering plans have been developed paper at this time. The 118 acres are situated right in front of an ADOT drainage basin and the parcel has significant topographical challenges with an estimated 5% slope falling from the northeast to the southwest approximately.

The proposed project will provide preliminary engineering and environmental services to successfully guide and facilitate the construction and completion of the Community Park in Camp Verde. Key objectives for this project include:

- Designing a road for the main park entrance.
- Develop a rough grading plan.
- Layout of Park Water Resources system which includes the parks irrigation, drinking water, and wastewater.

To achieve these objectives a carefully planned approach that emphasizes these elements is summarized below and detailed in the scope of services.

2.3 Tasks:

1. Team Management

    a. Client and Technical advisor meetings

    Technical design meetings will be coordinated with the Town of Camp Verde engineer Ron Long and the groups Technical Advisor Mark Lamar. A minimum of 8 meetings will be held during the projects duration.

    Deliverable: Meeting minutes and action items

    b. Group meetings

    Meetings will serve as the primary forum for reviewing the status of the project and identifying and resolving project issues. Meetings will be held at a minimum once a week.

    Deliverable: Meeting minutes and action items

2. Review documents and existing plans

    a. Review concept plan

    b. Review existing utilities, roads, and drainages

    Deliverable: memo with subtasks

3. Existing maps

    a. Topography

    b. Other

    Deliverable: site plan

4. Site Visit

    A site visit is essential to the understanding of the scope of the project. A visit will be coordinated with the Client to conduct a field review. This review will be to identify and document physical features, potential design constraints, and environmental considerations. Field information will be recorded using field notes and digital photos.

    Deliverables: Field notes and photographs of site

5. Determine expected use

    a. Population

    b. Traffic

    c. other

    Deliverable: Park usage

6. Develop Rough Grading Plan

    Deliverable: Rough Grading Plan

7. Water Resources Systems - This task is composed of irrigation, sewer, storm drains, potable water, and well drilling. There is no development on the site so our task would be to develop the best possible locations for each of the utilities. There would be a cost analysis on each locations analyzed. The first location to be investigated would be the location given to us by the client. The client also mentioned there was a sewer plant with than half a mile away. Also mentioned was the fact that the Verde River flows nearby and is a potential source of both irrigation and potable water.

    a. Irrigation

        Deliverable: irrigation plans and data on different types of irrigation methods

    b. Drainage

        Deliverable: topography map of possible drainage sites

    c. Drinking water

        Deliverable: possible drinking water locations

    d. Wastewater

        Deliverable: Possible wastewater management areas and methods

    Deliverable: a description of different scenarios of each plan. The scenarios would be composed of going the traditional route or using the new sustainable techniques. There would be a cost analysis of each system.

8. Roadway Design Task:

    A proposal of one-fifth a mile roadway design which is off of Highway 260 will contain the following perimeters: horizontal and vertical alignment subtasks, cross sections, roadside design concepts, and drainage systems. All subtasks will comply with American Association of State Highway and Transportation Officials (AASHTO), Arizona Department of Transportation (ADOT), and Maricopa County Department of Transportation (MCDOT) regulations/requirements.

a. Horizontal Alignment Subtask:
   - Bearings
   - Distances
   - Stations
   - Calculations

   Deliverables: Horizontal alignment design and calculations

b. Vertical Alignment Subtask:
   - Vertical curve designs
   - Elevations
   - Sight distances
   - Earthworks (cut and fill)
   - Calculations

   Deliverables: Vertical alignment design and calculations

c. Cross-Sections Subtask:
   - Lanes
   - Crowns
   - Shoulders
   - Ditches
   - Control drawings
   - Calculations

Deliverables: Design control drawings and cross-sectional calculations

d. Roadside Design Subtask:
   - Clear zones
   - Roadside geometry (fore slope, back slope, and drainage)
   - Longitudinal barriers (guardrails where needed)
   - Bicycles and pedestrians road design
   - Calculations

   Deliverables: Typical cross-section design drawings and calculations

e. Drainage System Subtask:
   - Surface drainage (runoff, rainfall intensity, & area)
   - The rational method for small drainage areas
   - Watershed delineation
   - Culvert performance and design
   - Drainage calculations

   Deliverable: Drainage design and calculations

2.4 Staffing Plan:

Table 1.0 shows the staffing plan on how the tasks will be divided among the team members. Each member will work equal hours with a total of 530 hours for the overall project.

Task	Dejan (hrs)	LeAnne (hrs)	Elwid (hrs)	Steven (hrs)	Total (hrs)
Project Management	5	5	5	5	20
Review Documents & Existing Plans	1.25	1.25	1.25	1.25	5
Existing Maps	1.25	1.25	1.25	1.25	5
Site Visit	1	1	1	1	4
Determine Expected Use	4	4	4	4	16
Water Resource Systems	0	0	120	120	240
Roadway Design	120	120	0	0	240
Total	132.5	132.5	132.5	132.5	530

Table 1.0: Staffing Plan

2.5 Budget:

Table 2.0 shows how the each team member will have a minimum hourly pay rate of thirty dollars. The overall project is estimated to cost $15,900.

Task	Civil Engineer	Civil Engineer	Env. Engineer	Env. Engineer	
Project Management	5	5	5	5	
Review Documents & Existing Plans	1.25	1.25	1.25	1.25	
Existing Maps	1.25	1.25	1.25	1.25	
Site Visit	1	1	1	1	
Determine Expected Use	4	4	4	4	
Water Resource Systems	0	0	120	120	
Roadway Design	120	120	0	0	
Total Estimated Hours	132.5	132.5	132.5	132.5	
Base Hourly Rate ($)	30	30	30	30	
Total Cost ($)	3975	3975	3975	3975	15900

Table 2.0: Budgeting Plan

2. 6 Scheduling Plan:

Figure 1.0 provides the scheduling plan of when each task will be completed by either the whole team or by individual members. The scheduling plan may be modified in the future. See last page of the proposal report for the full size scheduling plan.

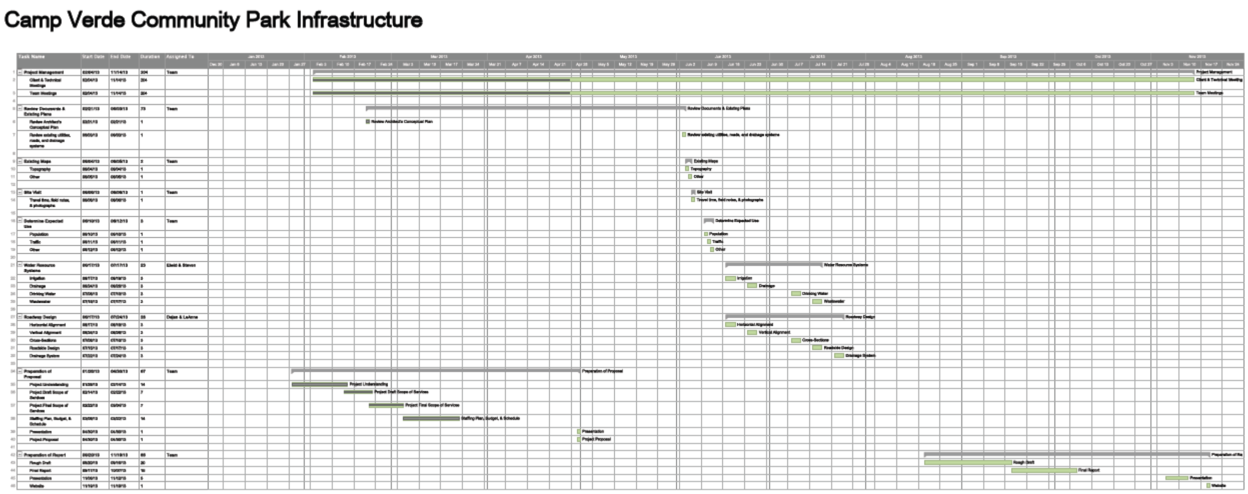


Figure 1.0: Scheduling Plan

3.0 GRADING PLAN

3.2 Introduction

The first step in developing the complete site plan for the 118 acre park was to develop a rough grading plan. The team's survey data was acquired from the Client as was a concept plan for the park.

3.3 Given Data

The client gave the team two critical pieces of data for the formation of the grading plan. The first was the survey data of the existing plot of land. The data came from an American Land Title Association (ALTA) survey. The second piece of data was the Town of Camp Verdes Conceptual master plan for the park. This last document was what the client envisioned for the park and was used as the principal guide in determining the locations of the grading pads. Figures 3.1 and 3.2 show the existing surface and conceptual plan respectfully.

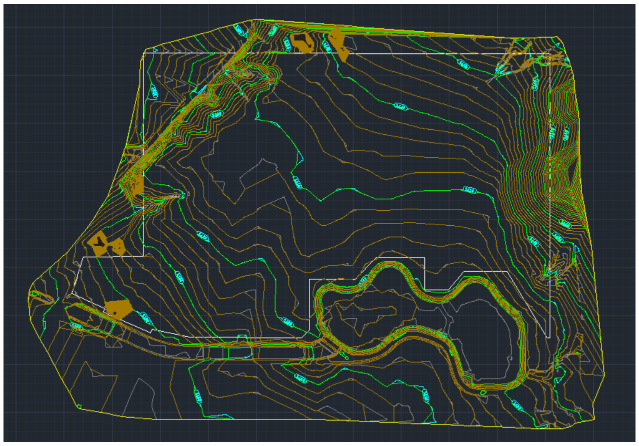


Figure 3.1: Existing surface of Park.

Figure 3.2: Concept plan

3.3 Grading Methodology

The survey data was placed into AutoCAD and a TIN Surface was created. From the TIN Surface and the resulting contour lines approximate locations for the fields and parking could be determined. The Client requested that the grading pads all be sloped at 1% to start with and that their slop cause any surface runoff to go into the ADOT drainage basin that can be seen at the south end of the property. Using AutoCAD polylines and the Grading tools five graded pads were made to accommodate three soccer fields, four baseball fields, and three separate parking lots. Originally the park was to house an area for equestrian sports; however this was scrapped in favor of two small detention ponds that could be used to irrigate the park if needed. Seen in Figure 3.3 the final graded surface has five flat graded pads that slope down toward the ADOT drainage basin at 1% and two small detention ponds.

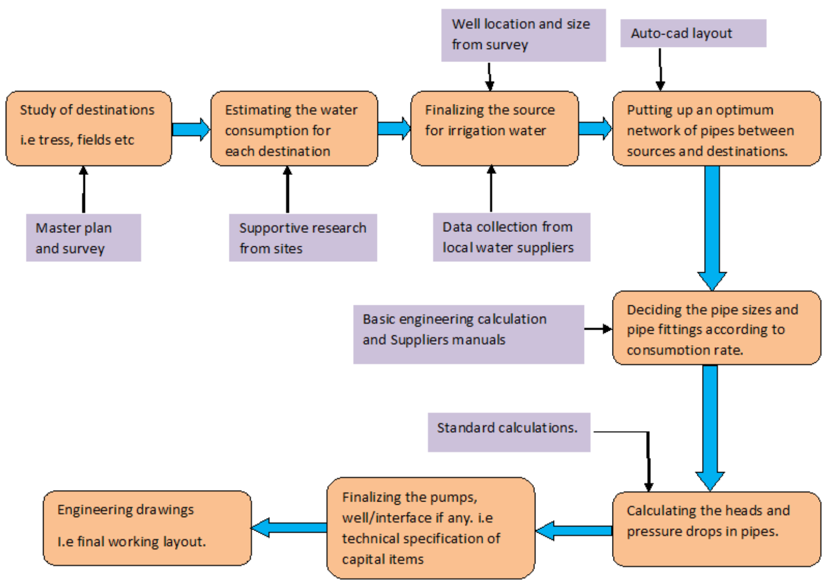
Figure 3.3: Final Graded surface

With the rough grading plan finished the team was able to move into the Water Resource System design and the park entry road design.

4.0 Water Resource System

4.1 Irrigation and Utility Water System

The strategic approach for the irrigation system can be framed according to the required tasks and the inputs. We start with the inputs and end on the final requirement.



The following are the basic steps:

a. Study of destinations:
b. Estimating the water consumption per destination:
c. Finalizing the source for irrigation and utility water:
d. Putting up an optimum network between the finalized source and the destination.
e. Deciding the pipe sizes and pipe fittings according to consumption rate.
f. Calculating the heads and pressure drops in pipes.
g. Finalizing the pumps, well/interface if any. i.e technical specification of capital items
h. Engineering drawings. I.e final working layout.

4.1.1 Irrigation System Design Notes:

The irrigation water will come from wastewater ponds north of the project site. The wastewater will be pumped from the existing ponds to a holding tank on the highest location on the property. The tank will be need to be 10 foot by 10 foot which will give a holding reservoir of approximately 23,000 Gallons. The mainline pipe will be a 4" PVC class 200 pipe, which will initially be able to supply 1,600 to 1,800 GPM at a nominal psi between 20-100. Sizing the pipe slightly larger than may be need will facilitate the irrigation of the play fields. This size of mainline will allow two sections of each system to be run simultaneously which will allow all irrigation to be done at night without interruption to daytime activities and will reduce water waste due to evaporation in the heat of the day. The irrigation mainline was designed as a loop system around the fields in order to supply an even amount of pressure to all of the irrigation sites and to allow a way to back feed from one end or the other in the event of a pipe breach. Shut off valves will be installed on either end as well as backflow preventers when necessary.

The tank will need to be disinfected by chlorinating it, and then retested once the chlorine is flushed out, to avoid bacteria contamination. This will be done by shock chlorination of the piping systems using a metering pump with high pressures.

The irrigation frequency would be daily depending on the season, as with any other irrigation. The GPD in the design note is based off of the MAG regulations.

4.1.2 Soccer Fields:

The three soccer fields consist of 259,200 square feet of lawn, which breaks down to 86,400 square feet per field. It was calculated using the MAG Regulations that the grassed area would consume 0.10 gallons of water per square foot. This time the square footage of the three soccer fields gives us a total of 25,920 gallons per day or 8,640 gal per day per field.

Each field has an underground irrigation system consisting of a 2 ½" mainline that is connected to the irrigation mainline. There are backflow preventers and shutoff valves installed at the connection points. The backflow preventers will prevent the reversal of flow and will help maintain the water pressure in this segment during use. The shut valve will be used to isolate this segment from the main trunk line during maintenance or during the off months to protect employees and the irrigation line in the winter months.

The mainline has a programmable 8-station irrigation controller that will allow only one or two of the six sections of the system to run at a time. Each lateral pipe size starts with a 2" PVC pipe and ends with a 1"pvc pipe. The reason for sizing down the pipe as we go down the line is to maintain the GPM and PSI needed but also maintain a cost efficient system. Using all 2" pipe just to maintain uniformity would increase the cost of the project significantly. Each lateral section contains a remote control valve linked to the irrigation controller, and four full circle sprinkler heads with a radius of 61 feet, that will require a minimum of 65 psi, and a minimum of 14gpm. The total lateral will require 56gpm in order to function fully. The entire system requires a minimum of 75psi downstream of the backflow preventer and a minimum of 56 GPM. At this time the calculations re not finalized and a booster pump may need to be installed in order to maintain the need pressure. In order to adequately water the field this system must run for a total of 2.5 hours per day.

4.1.3 Baseball Fields:

The four baseball fields consist of 377,133 square feet of lawn, which breaks down to 94,283 square feet per field. It was calculated using the MAG Regulations that the grassed area would consume 0.10 gallons of water per square foot. This time the square footage of the four baseball fields gives us a total of 37,713 gallons per day or 9,428 gal per day per field.

Each field has an underground irrigation system consisting of a 2 ½" mainline that is connected to the irrigation mainline. There are backflow preventers and shutoff valves installed at the connection points. The backflow preventers will prevent the reversal of flow and will help maintain the water pressure in this segment during use. The shut valve will be used to isolate this segment from the main trunk line during maintenance or during the off months to protect employees and the irrigation line in the winter months.

The mainline has a programmable 8-station irrigation controller that will allow up to two of the six section of the system to run at a time. Each lateral pipe size starts with a 2 1/2" PVC pipe and ends with a 1" PVC pipe. The reason for sizing down the pipe as we go down the line is to maintain the GPM and PSI needed but also maintain a cost efficient system. Using all 2 1/2" pipe just to maintain uniformity would increase the cost of the project significantly. Each lateral section contains a remote control valve linked to the irrigation controller, and four to seven full circle and partial circle sprinkler heads with a radius of 59 feet that will require a minimum of 50 psi, and a minimum of 15.4 GPM. The total lateral will require from 77 to 108 GPM in order to function fully. The entire system requires a minimum of 65 psi downstream of the backflow preventer and a minimum of 108 GPM. At this time the calculations re not finalized and a booster pump may need to be installed in order to maintain the need pressure. In order to adequately water the field this system must run for a total of 2.5 hours per day. These numbers are all based on my initial calculations and are subject to change based on the final irrigation design after all other utilities and systems are designed and added to the final plan.

The fields were designed to use Rainbird components. The pipe sizes and sprinkler heads used were recommended by Rainbird engineers to provide the most efficient and cost effective system.

For the turf selection, real grass will be used instead of artificial grass. Although artificial grass saves water, there is an availability of reclaimed water, which produces 250,000 gallons per day to be used. Also, due to high temperatures in Camp Verde, the artificial grass will over heat and will need to be replaced.

The plan view of the irrigation system is found in Appendix D. It shows the irrigation mainline (red lines), which was designed as a loop system around the fields. This design will allow an even supply of pressure to all of the irrigation sites.

4.2 Water System Design Notes:

The current water system will be supplied by a well. Current well data taken from 172 wells in a one mile radius around the project site indicates an average well depth of 104 feet from the surface with an average casing depth of 48 feet, an average casing diameter of six inches and wells hitting water at an average depth of 38 feet (Appendix A). The average depth water is found does not indicate that the water found at that depth is useable or in a quantity that is sustainable for the project. Once water is found during the drilling process the driller will be able to test the well to determine the quality of the water and the approximate gallons per minute the well will be able to sustain.

The well will require a nominal pressure tank and filtration system including a chlorine gas injection system. The casing diameter should be six inches and the casing depth may be set by the county department of Health or the state Department of Ecology when the well permit is applied for.

Using the MAG regulations regarding water usage and flows, the drinking water usage at peak daily rates for all buildings on the complex was estimated to have 637.5 GPD for the complex (Appendix B). This does not take into consideration any extraneous drinking fountains or any other use for fresh potable water other than the buildings listed in the master plan. As such the recommended water pipe size should be 2” sch. 40 HDPE pipe buried at a minimum of 36” to prevent accidental dig-ins and possible freezing temperatures. According to an online source (flexpvc.com), the two-inch line will supply approximately 127 gallons per minute at pressures between 20-100psi. See figure 1 below to compare pipe sizes with the amount of water it supplies. This will allow for future expansion and for the system to be integrated into the city’s water system in the future.

The Tot lot/Splash pad was not included into the water usage plans. However, the water used for the Tot lot/Splash pad should be on a recirculating system that is highly chlorinated. This system could be fed from either the potable water system or from the irrigation system on a closed loop system from an external valve adding water to the system as needed and preventing backflow with an inline backflow preventer.

Since the groundwater is pretty clean in the area, samples of water will be tested for arsenic twice a year. If needed, it will be treated with filtration.

The plan view of the water system is found in Appendix C. The well location was picked because it does not disturb nearby existing wells, and also because it is relatively close to the Verde River.

			Assume Gravity to Low Pressure. About 6f/s flow velocity, also suction side of pump		Assume Average Pressure. (20-100PSI) About 12f/s flow velocity		Assume "High Pressure" PEAK flow. About 18f/s flow velocity*	
Sch 40 Pipe Size	ID (range)	OD	GPM (with minimal pressure loss & noise)	GPH (with minimal pressure loss & noise)	GPM (with minimal pressure loss & noise)	GPH (with minimal pressure loss & noise)	GPM (with significant pressure loss & noise)	GPH (with significant pressure loss & noise)
1/2"	.50-.60"	.85"	7 gpm	420 gph	14 gpm	840 gph	21 gpm	1,260 gph
3/4"	.75-.85"	1.06"	11 gpm	660 gph	23 gpm	1,410 gph	36 gpm	2,160 gph
1"	1.00-1.03"	1.33"	16 gpm	960 gph	37 gpm	2,220 gph	58 gpm	3,510 gph
1.25"	1.25-1.36"	1.67"	25 gpm	1,500 gph	62 gpm	3,750 gph	100 gpm	5,940 gph
1.5"	1.50-1.60"	1.90"	35 gpm	2100 gph	81 gpm	4,830 gph	126 gpm	7,560 gph
2"	1.95-2.05"	2.38"	55 gpm	3300 gph	127 gpm	7,650 gph	200 gpm	12,000 gph
2.5"	2.35-2.45"	2.89"	80 gpm	4800 gph	190 gpm	11,400 gph	300 gpm	17,550 gph
3"	2.90-3.05"	3.50"	140 gpm	8400 gph	273 gpm	16,350 gph	425 gpm	25,650 gph
4"	3.85-3.95"	4.50"	240 gpm	14,400 gph	480 gpm	28,800 gph	700 gpm	42,000 gph

Figure 4.1: Water flow based on pipe size

4.3 Storm Water:

The purpose of the storm water management and use is to ensure that the Camp Verde Park does not flood. Flooding can result in property damage and/or cause harm to park visitors

Safety Requirement -

The safety of the park during the wet season relies on the parks landscape ability to manage storm water.

Below is a list of the constraints and criteria of the storm water management system for the Camp Verde Park.

Criteria:

- Ensure the safety of the park and its inhabitants

Constraints:

- Use detained storm water for irrigation water during the wet season
- The storm water management solution must be cost effective
- Minimize the amount of water on roadways, parking lots, and playing fields
- Ensure storm water generated by pervious surfaces on the park does not negatively affect the surrounding area
- Ensure onsite buildings are safe from high intensity storm events such as the 100 year storm

Storm water is found in Appendix E. The red arrows show the flow of runoff during a major storm event. And during any small storm event, the water would be absorbed on contact.

The arrows are pointing towards the elevation change and the shape of the contours. The curves in the contours show peaks and valleys as well.

4.4 Estimated Costs:

The estimated costs for the irrigation system is between $50,000-$75,000 USD. And the estimated costs for the water system is between 25,000-40,000 USD.

4.5 MAG Regulations:

MAG regulations were used for all components of this design. The regulations do not typically apply to this job because it will not be incorporated into the county or city systems at this time. But being they are the local regulations it was best to use them as guidelines and for best engineering practices.

4.6 Wastewater System Design

Figure 4.2- finished wastewater plan view

The client's objective for the wastewater produced by the park is to have the waste collected from all the restrooms and buildings flow through a gravity fed pipeline to a grinder pump located on the lowest elevation of the park at a 1% slope. The waste would be collected at a grinder pump which will then be grinded up and pumped to the local wastewater treatment plant which is over a 60 foot high hill and less than a mile away. There would be two shut off valves they would be located on the base of the grinder pump and the other shut off valve would be located on the edge of the property of the pressure pipe.

Shown in figure 4.2 is the finished wastewater plan. The circles represent the restrooms and the rectangle on the lower left corner represents the grinder pump and the lift station location. This particular location was selected due to its' relative low elevation and clearance from any other infrastructure on the park such as the drinking water well any fields and/or roads. The other blocks represent the pads of the planned fields that are to be built. Below shown in Figure 4.3 is the relation of the park boundary to the local sanitary plant.

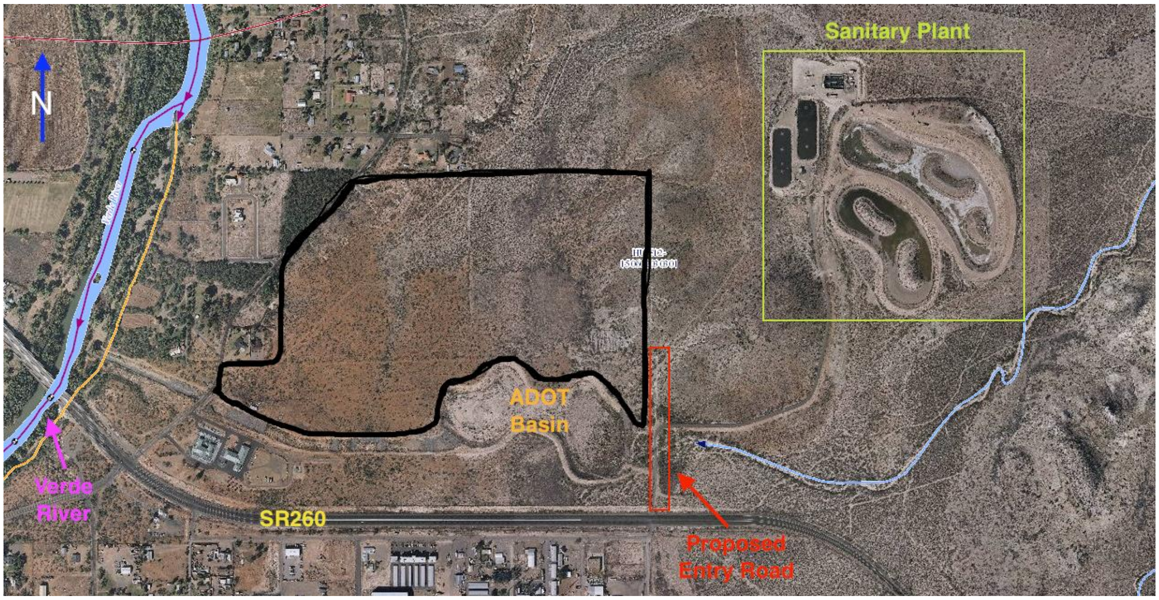


Figure 4.3- the park in relation to the wastewater plant

The client's tasks for this part of the park is to determine the size and the length of the pipes from each restroom to the grinder pump, determine the pump size to overcome the 60' head, and design a gravity flow system that would transport waste to a grinder pump and to then to the wastewater treatment plant.

4.6.1 Summary of Completed waste water system

The proposed final design computed for the wastewater system was comprised of the following specifications. The specifications were determined from following the Maricopa County MAG regulations and the design parameters. Below are the summarizations of the sewer specifications.

- Gravity pipe length= 5,679.9'
- Pressure pipe length = 3,092.77'

- Gravity flow pipe diameter= 8”
- Pressure pipe diameter= 3”
- Slope = 1 % (gravity feed system)
- Manning roughness coefficient = 0.013
- Five man holes on each intersection
- Service connection every 500 feet per length
- Pipes running along the main road (right of way)
- Peak flow 93 GPM
- Grinder pump 3 hp
- Pump 1.5 to 2 hp

4.6.3 Pipe diameter

The pipe diameter of the system was determined by using the Maricopa County MAG regulations. In the regulations from Appendices C it shows the proper diameter size for the gravity flow pipe systems as directed by the client. The gravity flow pipe diameter was listed to be 8”. The pressurized pipe was determined to be 3” due to financial constraints. The 3” diameter was a size specified specifically by the client.

4.6.4 Pipe length

The pipe was laid out in a manner that would be easily accessible and not interfere with any other park infrastructure like the fields or buildings. Therefore, most of the pipes follow the inner park roadways. The pipe would be laid out on the right of way as determined by the MAG regulations. A typical cross section of the right of way of the pipe is shown in appendix A-1. The gravity flow pipe length came out to be 5,679.9’. The pressurized system length came out to 3,092.77’ this length is from the grinder pump to the edge of the park property. The pressurized pipe that was used was a schedule 80 typical wastewater pipes.

4.6.5 Clean outs and Man holes

There are five pipe intersections in the design and as directed by the client a manhole should be in place at each pipe intersection. This is due to the heightened potential clogging of the pipes at these locations. In the MAG regulations there are also specifications of having cleanouts every 500 feet. In Appendix A-1 it shows a a typical clean out.

4.6.6 Slope

The slope was pre determined to be 1% as requested by the client. The justification for this is that the park area is fairly flat and goes from a subtle high slope to a low slope. The park site contained no high or low abnormities on the site. The client believed that a 1% slope was enough for the wastewater to flow enough so it won't clog.

4.6.7 Peak flow

The peak flow was determined using bases of 400 cars per day with four people per car. That was resulting in 1600 people at the peak time. This number was used in relation to the average wastewater production per person as determined by the EPA. The result was 93 gallons per minute. The client wanted to assume that the flow was continuous for the sake of simplicity of the calculations. Any wastewater flow that is generated in the park would occur mostly in the daytime or mid afternoon. In the night and other times were people aren't less active in parks the flow would be minimal or there wouldn't be a flow all together.

4.6.8 Pump size

The pump size for the lift station was determined to be 1.5 hp. The velocity would be 4.24 ft/s with a specific head loss of 2.1 ft per 100 ft of pipe. The total dynamic head was very small and proved to be irrelevant and this is due to the flow of the pipe being a straight path to the wastewater plant. This size will be able to transport the expected 93-gpm-peak flow comfortably over the 60' head and to the wastewater treatment plant. But the recommended size of pump would be a 2hp pump. This is due to sizing the pump to accommodate an increase of the size of the park due to the growth the town. In Appendix A-3 it shows a typical lift station.

4.6.9 Grinder pump

The grinder pump was sized to accommodate the peak flow discharge. The grinder pump that was chosen was a 3hp KG-31 industrial grade pump. It was a class F motor and has single-phase options. In Appendix B it shows the specifications of the grinder pump.

4.6.10 Cost estimation

- Standard PVC pipe 8": $11.95 per foot = $ 67,864
- Standard PVC pipe 80 3": $4.28 per foot = $ 13,233.76
- Pump: $2000 to $3000
- Grinder pump: $2000 to $ 3000
- Total= $85,097.76 to $87,097.76

4.6.11 Conclusion

The wastewater system was designed to be the cheapest alternative to direct the wastewater. The client specified to only use one pump and grinder to keep cost down. The use of the gravity feed system is to reduce the chances a system will fail if the system was comprised of many pumps and or grinders.

5.0 Entry Roadway Design

5.1 Introduction

The proposed entry road to the community park will have a new paved road section approximately 1,134.07 feet long, which is off of State Route (SR) 260. This proposed road is going to follow the existing alignments of a paved and dirt road (See Figure 1.0). A new road will not only be redesigned to accommodate both the park and sanitary treatment plant's future average daily traffic volume, but also because several sections on the existing road experience overtopping floods each year. The new roadway will contain the following design parameters: existing site observations, design controls and criteria, horizontal and vertical alignment subtasks, cross sections, and drainage systems. All subtasks will comply with the American Association of State Highway and Transportation Officials (AASHTO), Arizona Department of Transportation (ADOT), United States Department of Transportation's (USDOT) Federal Highway Administration's (FHWA) Hydraulic Design of Highway Culverts (HDHDC), Maricopa Association of Governments (MAG), Highway Capacity Manual (HCM), and Maricopa County Department of Transportation (MCDOT) regulations/requirements.

5.2 Existing Site Observations

5.2.1 Existing Roads:

Two existing roads make up the proposed road entrance to the park. Approximately 600 feet comes from Camp Verde's Sanitary District paved road and the remaining 534.07 feet from an existing dirt road (See Figures 1.0 & 1.1). The paved sanitary district road has a posted speed of 15 miles per hour (mph), has no current traffic information, and contains four corrugated culvert pipes that serve as the only point of drainage along the pipe (See Figure 1.2 for the posted speed limit sign & Figure 1.3 for culvert pipes). Figure 1.3 also shows that the current placement of the culvert pipes does not convey the total recurring annual peak flows that overtop the road. As seen in the figure, a large volume of flow overtops the road at a further distance from the existing pipes. In addition, the pipes' exit exhibits signs of scour ("Erosion of streambed due to flowing water," (HDHC, 2012)) as seen in Figure 1.4. As seen in Figure 1.5, the dirt road has several minor water crossings that may pose drainage problems for the entry road. Future park traffic will need to use the SR260 road to enter the park's entry road; the SR260 road has a total of five lanes (four traveled way lanes & one center lane). SR260 has functional classification as a minor arterial (ADOT Map Book, Page #8) and the site is located in the Yavapai County (ADOT Map Book, Page #5) on the Transportation Board District #6 (ADOT Map Book, Page #10). The overall terrain of the site is considered rolling, because there are several slope changes, but a majority of the road can be viewed from beginning to end.

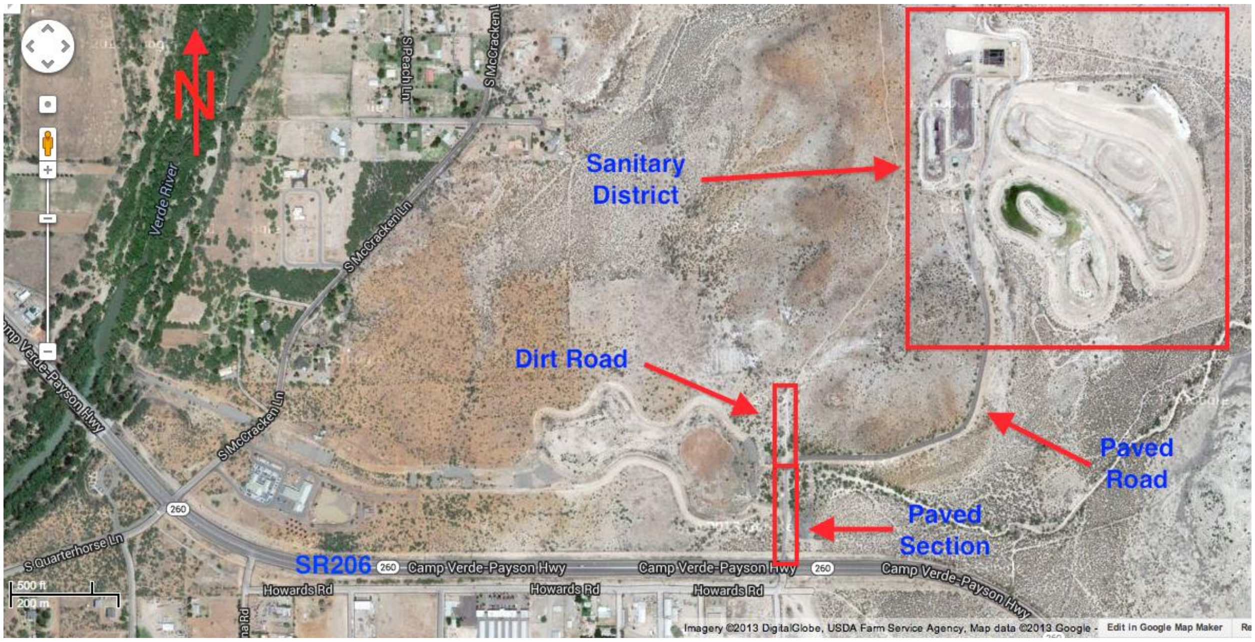


Figure 5.0: Google Maps' Top View of the Site.

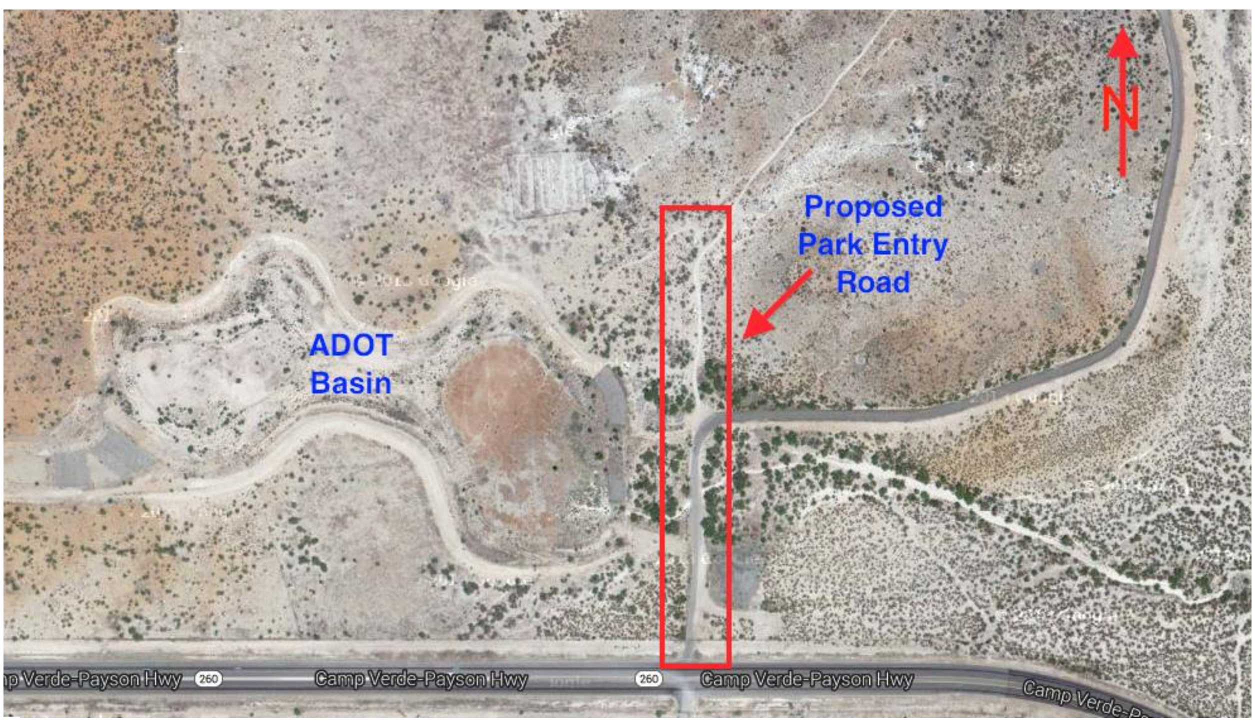


Figure 5.1: Proposed Park Entry Road & ADOT Basin Locations

Figure 5.2: Posted Signs of the Existing Paved Roadway

Figure 5.3: Existing Culvert (flow entrance) with Road Overtopping View

Figure 5.4: Pipes Exit with some scour forming

Figure 5.5: Existing Dirt Road Conditions: Drainage on side of road

5.2.2 Review of the Client's Given Materials:

The client has given the team a basic site plan showing the park and city boundaries, the proposed town access easement dimensions, ADOT's drainage basin, etc. (See Appendix #), two conceptual architectural plans where future site road and facility locations (See Appendix #), and an AutoCAD drawing of the entire existing site that shows the community park area, entrance road, site topography (contour lines), and property boundaries, etc. (See Appendix #). Keep in mind that the proposed road shown in Appendices A&B is the first park access suggestion, but due to frequent overtopping floods as seen in Figure 1.3, the latest proposal can be seen in Figures 1.1 & 1.2.

5.3 Design Controls & Criteria

The upcoming park entry road of 1,134.07 feet in length will need to accommodate the 2012 Camp Verde population of approximately 10,925 with a population change since the year 2000 of +15.6% (City-Data.com, Camp Verde, Arizona, accessed on 8/25/13). Therefore, the proposed road's traffic projections will be based on a 20-year design period (AASHTO's *A Policy on Geometric Design of Highways and Streets, 2001*, Page #424 & ADOT's *Roadway Design Guidelines, 2012,* Page #100-4), which will be designed for the year 2033. According to Institute of Transportation Engineers (ITE) Trip Generation Manual, the park's future average daily traffic (ADT) was estimated to be approximately 400 vehicles per day for the year 2033. Since the present speed is 15 mph, the design speed (V) will also be 15 mph. The following list provides the client's requirements for the proposed entry road:

- Lane width: 12 feet (ft)
- Number of lanes: 2
- Shoulder widths:
  - As entering the park (North direction):
    - Right side shoulder: 4 ft (for roadside safety stops)
    - Left side shoulder: 8 ft (where the majority of pedestrians & bicyclists will travel on)
- Shoulders are to have a thickened edge design (See Appendix D) to slow erosion
- Total Right-of-way (ROW): 100ft
- Provide a left turn lane for traffic exiting the park onto SR260
- A right turn lane will not be required to design for adding an extra lane to the existing SR260 is out of the site location. The client will take care of the SR260's right turn lane into the park entry road with ADOT
- Design entry: approximately 20 ft. from property fence line
- Provide box culvert designs where the existing road experiences overtop flooding as seen in Figure 1.3

After reviewing the client's design requirements, the road's functional classification was chosen to be a "rural minor collector," because the road will be serving a purpose of moving traffic between a arterial road (SR260) and the park's local streets to access the community park's facilities. Also the typical road cross-section is similar to that of MCDOT's *Pavement Marking Manual, 2005*'s "rural minor collector" standard drawing (See Figure 2.0). With the functional classification, the roadway capacity is to be designed for a level of service (LOS) C, which provides "acceptable operating service for facility users" on the rural minor collector road (HCM, 2000, Page 2-3 & AASHTO's *A Policy on Geometric Design of Highways and Streets, 2001*, Page #426 & ADOT's *Roadway Design Guidelines, 2012,* Page #100-6 Table 103.2A). "LOS is a quality measure describing operational conditions within a traffic stream, generally in terms of such service measures as speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience (HCM, 2000, Page #2-2)." By selecting the functional classification of the proposed entry road, the rural minor collector road section of AAHTO's *A Policy on Geometric Design of Highways & Streets, 2001* & *Roadside Design Guide, 2002* can be utilized in obtaining design requirements such crown slopes, foreslopes and backslopes, etc.

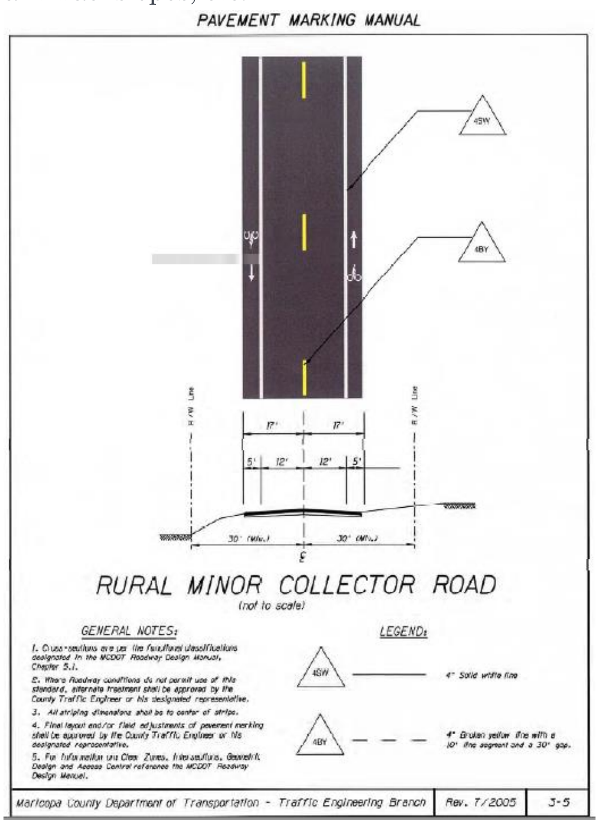


Figure 5.6: MCDOT Pavement Marking Manual's Typical Cross Section

5.4 Horizontal Alignment

5.4.1 Center Line Stations, Bearing, & Elevations:

The new road's center-line (CL) will be redesigned on top of the existing paved and dirt road sections. The AutoCAD drawing provided by the client was used in developing both the existing and proposed road CL's stations (every 50 ft. including major drainage facilities), one bearing (proposed), and elevations of each station points. Since the new roadway's beginning of project (BOP) is at the intersection of SR260 and the end of project (EOP) is where a wild animal trough is located on the dirt road (See Figure 5.7 & 5.8), there is no need for horizontal curves for the proposed CL is straight. Therefore, only one bearing is listed and can be seen in Appendix E. This S 00° 00' 35" E bearing does not need to be calculated for it is in the same direction as the boundary line to the right of the road's CL. Figure 3.2 is a closer picture of the road's plan view to better see the different colors that associate the CL (white line dotted line with a X at the center of the line to specify stations & elevations), property fenced line (white line with a square at the center of the line), boundary line (white), edge of the shoulder (blue), and traveled way (TW) in red. Elevations of every 50-foot stations (including major drainage facilities) of the CL were estimated and calculated using the slope-intercept formula y=mx+b and a printed contour map (See Table 3.0 for the existing & proposed road's stations, elevations, & bearing). Calculations of using the slope-intercept formula to find the elevations of each station can be found in Appendix #.

Figure 5.7: Wild Animal Trough at the EOP

Figure 5.8: Enlarged Wild Animal Trough

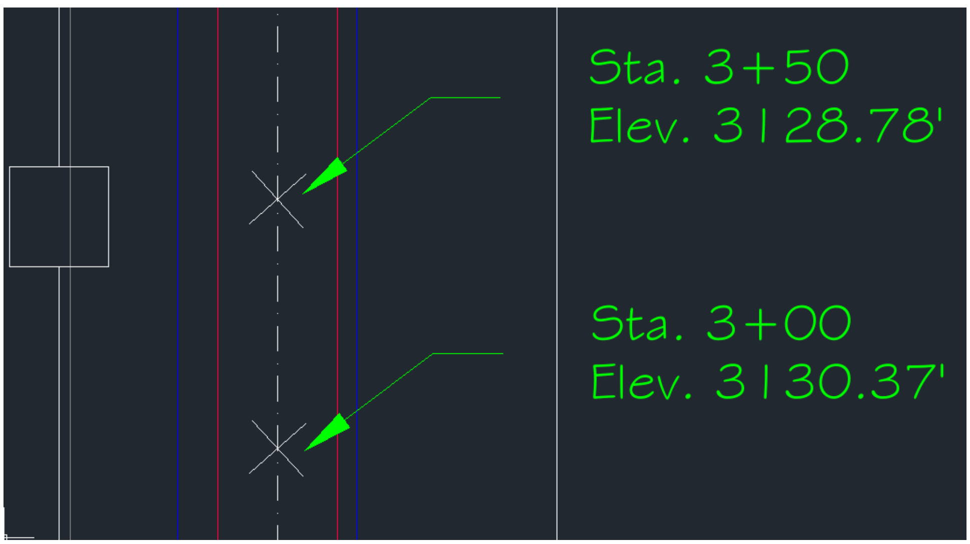


Figure 5.9: A Magnified Road Plan View Section

Stations	Proposed Elevations	Existing Elevations
00+00	3122.09	3122.09
00+25.79	3124.23	3123.24
00+50.00	3125.06	3124.48
00+65.50	3126.05	3125.03
01+00	3126.28	3125.89
01+50	3127.58	3126.58
02+00	3129.38	3128.65
02+50	3130.07	3129.49
03+00	3130.37	3130.12
03+50	3128.78	3128.65
04+00	3130.38	3129.21
04+36.50	3129.75	3128.95
04+50	3129.74	3128.77
05+00	3128	3127.3
05+50	3127.76	3127.43
05+75	3127.57	3127.57
06+00	3127.75	3127.78
06+50	3127.97	3127.86
07+00	3128.37	3127.9
07+50	3129.31	3128.82
08+00	3129.95	3129.77
08+50	3131.51	3131.51
09+00	3134.63	3134.61
09+50	3137.75	3135.95
10+00	3141.29	3138

10+50	3141.86	3141.85
11+00	3141	3141
11+34.07	3144.25	
BOP to EOP Bearing: S 00° 00' 35" E		

Table 3.0: Existing & Proposed Stations, Elevations, & Bearing

5.4.2 Sight Distances:

First, the stopping sight distance (SSD) is calculated (See Appendix G), and the decision sight distance (DSD) is also computed (See Appendix H) to assure roadway safety. SSD is the required distance that is needed to stop when the driver sees a person or object on the travelled way and the instant the brakes are applied plus the distance that the vehicle comes to a complete stop. DSD is the distance the driver needs to make when approached with complex decision maneuvering tasks. The future road will not have much difficulty when it comes to making a complete stop or maneuver decisions for the design speed is a low 15 mph on an intermediate terrain (between leveled & rolling) where the entire roadway can be seen at any point without vegetation or different road grade sight blockages.

5.4.3 Left-Turn Lane:

Since there will be more traffic using the entry road in the future, a left turn lane needs to be designed at the new road intersection with SR260. ADOT *Traffic Engineering Policies, Guidelines, and Procedures Section 400 – Pavement Markings, 2000* (PGP) provided guidelines in designing a left-turn lane. A left-turn lane consists of a taper, gap, and storage lengths as seen in Figure 3.3 with design calculations in Appendix I. Taper lengths are comprised of the design speed (15 mph) and width of the lane added (12ft.); the gap length is given in Table 3.1 to be 60 ft. (PGP, 2000, Page 430-2); and the storage length is the braking distance (20 ft. from Table 3.2) plus the queue length, where the queue length should provide space for two passenger cars at 25 ft. each when the truck percentage is less than 10% (ADOT PGP, 2000, Page 430-5). Figure 3.4 shows a closer top view of the left turn lane, which was taken from the road's plan.

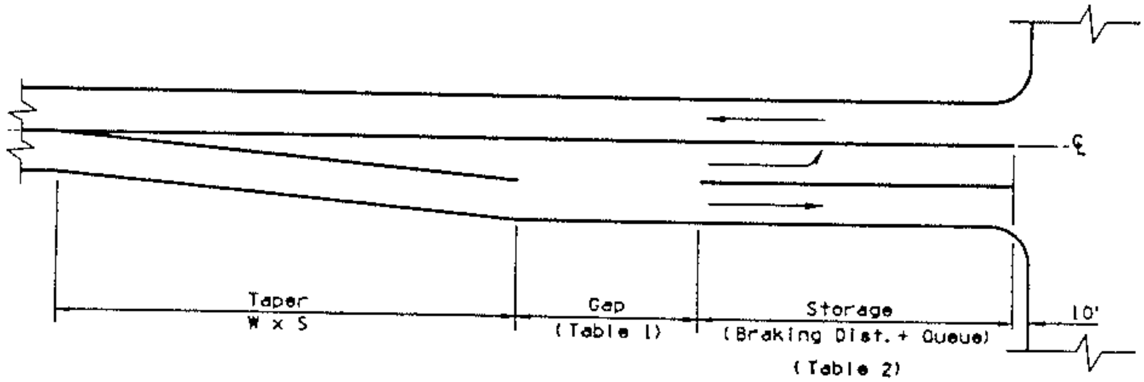


Figure 5.10: ADOT PGP, 2000, Page 430-1; Left-Turn Lane

POSTED or DESIGN SPEED (mph)	GAP (feet)
< 40	60
40 - 50	90
> 50	140

Table 3.1: ADOT PGP Gap Lengths Table

POSTED or DESIGN SPEED (mph)	DESIRABLE		MINIMUM		
	BRAKING SPEED (mph)	BRAKING DISTANCE (feet)	ENTERING SPEED (mph)	BRAKING SPEED (mph)	BRAKING DISTANCE (feet)
30	29	80	20	20	20
35	34	115	25	25	40
40	38	150	30	29	50
45	43	200	35	34	85
50	47	245	40	38	120
55	52	300	45	42	145
60	56	360	50	47	200
65	60	415	55	52	265
70	64	490	60	56	315
75	70	585	65	61	400

Table 3.2: ADOT PGP Braking Distance Table

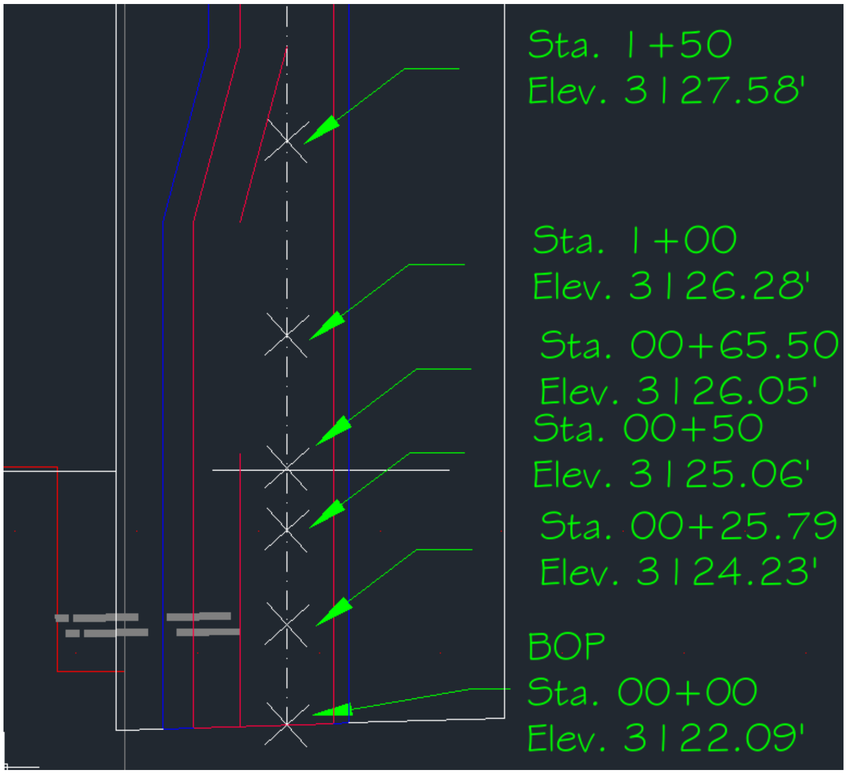


Figure 5.11: A Magnified Section of the Left Turn Lane

5.5 Vertical Alignment

5.5.1 Vertical Curves:

First, an elevation vs. stations was plotted to create a vertical profile of the road's existing (in blue) and proposed road's centerlines (in red) using the Excel software (See Appendix O for profile). From the profile, straight lines were drawn closer to the existing road's CL (in green). Having the vertical alignment closer to the existing CL leads to a reduced need for compaction, cut, and fill during construction. After the straight lines were drawn (a total of 5 lines), the Point of Vertical Intersections (PVI –where each straight line intersects) could now be used to calculate the stations and elevations of each Point of Vertical Curves (PVC –where the curve starts) and Point of Vertical Tangencies (PVT –where the curve ends) (See Appendices J, K, L, & M for vertical curve calculations). A table shown in Appendix N displays the vertical alignments' curves, grades in percentages (G), stations, elevations in feet, the absolute values of grade differences (A in percent), length of vertical curves (L – the distance between PVC & PVT), and the rate of vertical curvatures (K). By looking at the profile, there are a total of four vertical curves (VC) where Curve#1 is a Type II Crest VC (AASHTO, 2001, Exhibit 3-73, Page #269), Curve#2 is a Type II Crest VC (AASHTO, 2001, Exhibit 3-73, Page #269), Curve#3 is a Type I Crest VC (AASHTO, 2001, Exhibit 3-73, Page #269), and Curve#4 is a Type III Sag VC (AASHTO, 2001, Exhibit 3-73, Page #269). In Figure 4.0 from AASHTO, 2001, Exhibit 3-73, Page #269 shows the different types of VC's. The designer also made sure that while choosing the placements of the vertical alignment, there will be enough room for a concrete box culvert with a minimum rise of 3 feet tall to be installed with a minimum freeboard of two feet (ADOT 2012, Page 600-20).

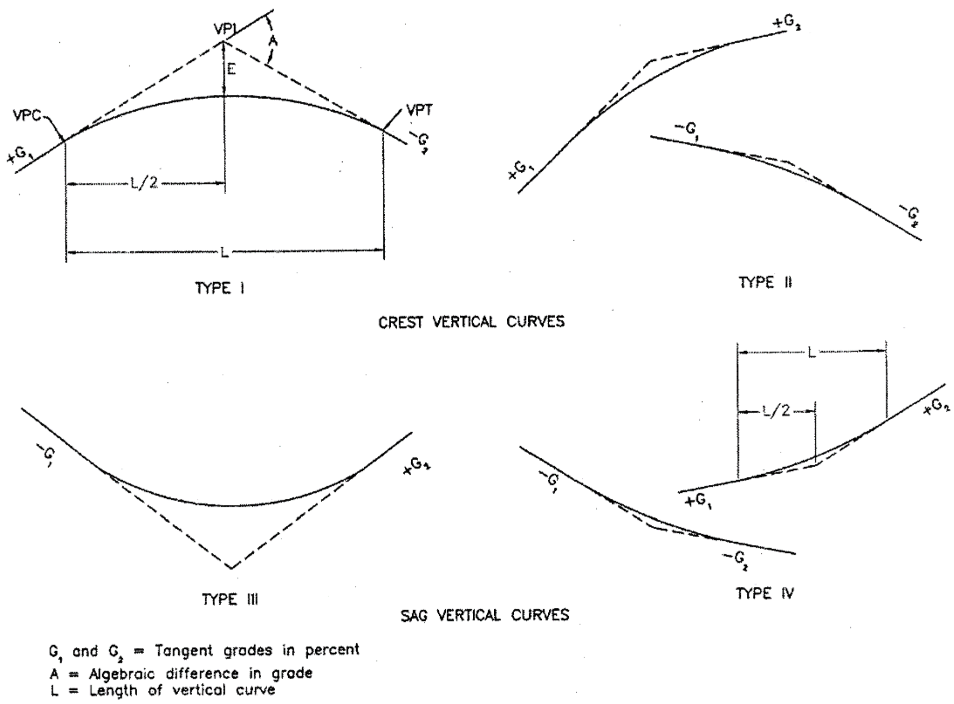


Figure 5.12: Types of Vertical Curves

5.5.2 Maximum Cut & Fill Locations:

The max fill and cut depths are determined by looking at the road profile in Appendix O. The farthest depth from the road's vertical alignment (in green) to the existing roads' (current & proposed) elevations helps specify where the max fill and cuts are located. From the road profile, Station 5+75 shows the maximum fill location with a depth of 5.274 feet (See Appendix P for Max. Fill calculations). Also from Appendix O, Station 10+00 displays the maximum cut location with a cut height of 2.59 feet (See Appendix P for Max. Cut calculations). By specifying the location and depth of the max fill, one can also speculate that 5.274 feet is enough height and width (From Stations 5+00 to 6+50 is where the majority of the roadway experiences overtop flooding) for a concrete box culvert with a rise of 3 feet to be installed during the drainage design of roadway, which is discussed in Section 5.7 of the report.

5.6 Cross-Sections

5.6.1 Typical Cross-Sections:

A typical road cross-section was designed according to ADOT's 2012 Roadway Design Guidelines, AASHTO's *Geometric Design of Highways & Streets, 2001* and the *Roadside Design Guide, 2002*. Shown in Figure 5.0, the road lanes, in red, will have a width of 12 feet, a total of two lanes, the left shoulder, in blue, will be 8 feet wide, and the right shoulder will have a width of 4 feet as requested by the client (Also See Appendix Q for a Typical Cross-Section with scale bars). The left shoulder is wider than the right shoulder, due to pedestrian, bicyclist, and vehicle emergency stops, driver comfort and confidence. Vehicle emergency stops, driver comfort and confidence are the main purpose for the right shoulder width. Also requested by the client is to have a thickened edge design (See Appendix D for MAG's Thickened Edge Type A Detail) for the shoulders to slow down erosion. To double-check the client's requests, a 12-foot lane (ADOT 2012, Page 300-2), two lanes (AASHTO 2001, Page 428), and a minimum of 4 foot shoulders (AASHTO 2001, Page 318-319) are all desirable or accepted values according to regulations. Given in Appendix Q is also a detail drawing of the TW's pavement. This pavement detail shows 3 inches of Asphalt Concrete (AC) and 6 inches of Aggregate Base (BC), but these values may change to better suit the soil conditions of the roadway.

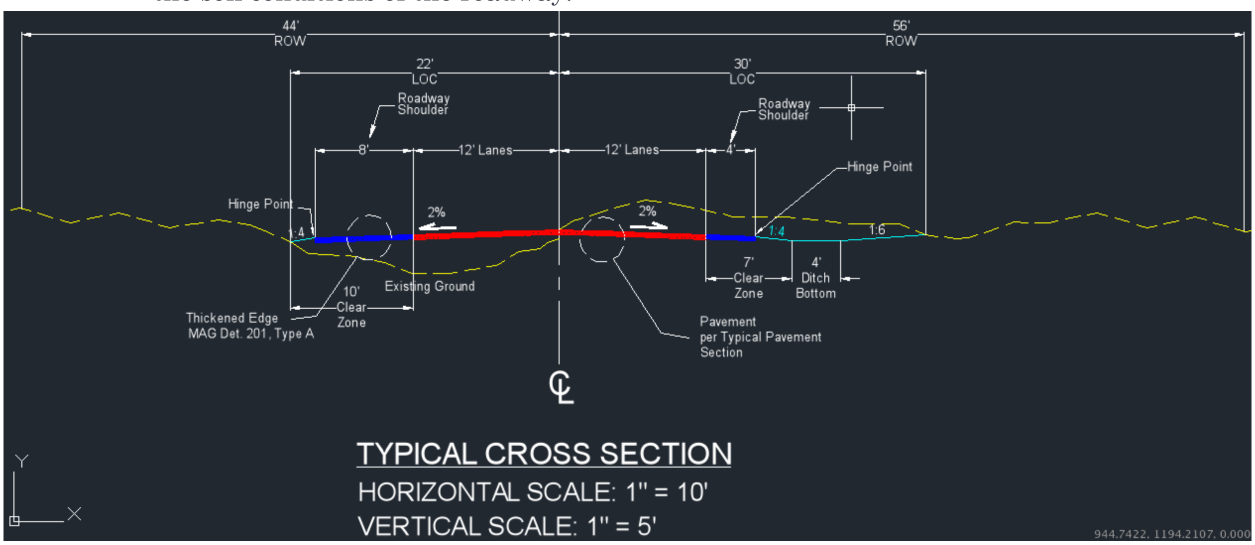


Figure 5.13: Typical Cross-Section

As shown in Figure 5.13, the crown will have a 2% slope (ADOT 2012, Page 300-1) for drainage, 1Vertcial:4Horizontal foreslopes (AASHTO 2001, Page 429), a 4 foot wide ditch bottom (AASHTO 2002, Page 3-12), 1V:6H backslopes for cut areas (AASHTO 2001, Page 331), and clear zone distances of 10 and 7 feet (AASHTO 2002, Page 3-6) for out-of-control vehicles to recover and reenter the TW safely. Figure 5.1 shows the suggested clear zone distances by using the designed ADT, forslopes, and backslopes. The values of the Limit of Construction (LOC – where construction will take place), ROW (area reserved for transportation purposed), and the location of the hinge points are shown in Figure 5.13 or in Appendix Q.

[U.S. Customary Units]

DESIGN SPEED	DESIGN ADT	FORESLOPES			BACKSLOPES		
		1V:6H or flatter	1V:5H TO 1V:4H	1V:3H	1V:3H	1V:5H TO 1V:4H	1V:6H or flatter
40 mph or less	UNDER 750	7 – 10	7 – 10	**	7 – 10	7 – 10	7 – 10
	750 – 1500	10 – 12	12 – 14	**	10 – 12	10 – 12	10 – 12
	1500 – 6000	12 – 14	14 – 16	**	12 – 14	12 – 14	12 – 14
	OVER 6000	14 – 16	16 – 18	**	14 – 16	14 – 16	14 – 16

Figure 5.14: Suggested Clear Zone Distances

5.7 Drainage Design Systems

5.7.1 Introduction:

The following hydrology and hydraulic analysis follows ADOT's *2012 Roadway Design Guidelines* and ADOT's *Drainage Structures' details* (ADOT.com, Drainage Structures), and FHWA's *2012 Hydraulic Design of Highway Culverts*. The proposed road will have a total of two newly installed culverts (Culvert #1 & Culvert #2), which can be seen in Appendix R along with the existing culverts (kept in final design; extension of culvert length required). Stations 5+66.02 and 6+30.37 are where Culverts #1 & #2 will be located. By looking at Appendix R, the existing culvert near the SR260 intersection has two pipe diameters of 18 inches and the other existing culvert near Station 4+36.50 have four pipe diameters of 20 inches. New culverts will be designed for the 100-year flood event as requested by the client. Figure 5.15, shows the locations of Culvert#1 & #2, the existing culvert #2, and the direction of flow.

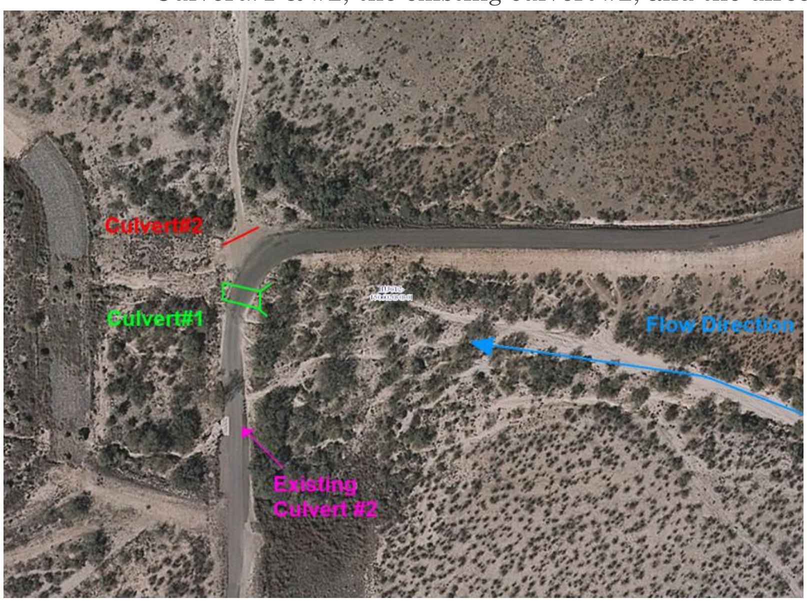


Figure 5.15: NMV's Top View of Culvert Locations (Not Drawn to Scale)

5.7.2 Culvert #1

5.7.2.1 Hydrology:

First, a watershed delineation (See Appendix U1) was completed using the U.S. Geological Survey's USGS) National Map Viewer (NMV) to obtain a drainage area (A) of 2.556 sq. miles (1,635.84 acres). Since the drainage area was found to be greater than 160 acres (ADOT 2012, Page 600-10), the Rational Method for calculating the 100-year peak flow cannot be utilized. Instead, the

National Streamflow Statistics (NSS) Program (See Appendix U2) was used. Values such as the Analysis Type (Peak), Rural location (Ungauged Site), Basin Drainage Area (A=2.556 sq. miles), Mean Basin Elevation (3,130 feet), and Crippen & Bue Region (16) were inputted into the NSS Program to obtain a 100-year Peak Flow of 2920 cubic feet per second (cfs). Next, the stream's slope (S = 3.05%) was determined from upstream and downstream elevations by using the typical LOC values.

5.7.2.2 Hydraulics Design:

By examining Figure 5.15, the main channel (wash) that drains into the ADOT basin widens before reaching the existing culvert #2, causing some (not all) of the flow to drain through the existing culvert #2 and a majority of the flood overtops approximately 150 feet of the existing road's length (From Stations 4+25 to 5+75). Therefore, the location of Culvert#1 was chosen by examining Figures 1.3, 1.4, 6.0 and also by reviewing the vertical alignment (road profile) of the road to verify that a concrete box culvert (See Figures 6.1 & 6.2 for Culvert #1 Profile drawn at skewed and Information) will be able to pass a flow of 2920 cfs without overtopping the road. A culvert with a rise of 3 feet and a span of 4 feet was selected, because the vertical alignment of the road at Culvert #1's location will allow a freeboard (top of culvert to hinge point) of about 2.25 feet (See Appendix S for a scaled drawing of Culvert #1's Profile).

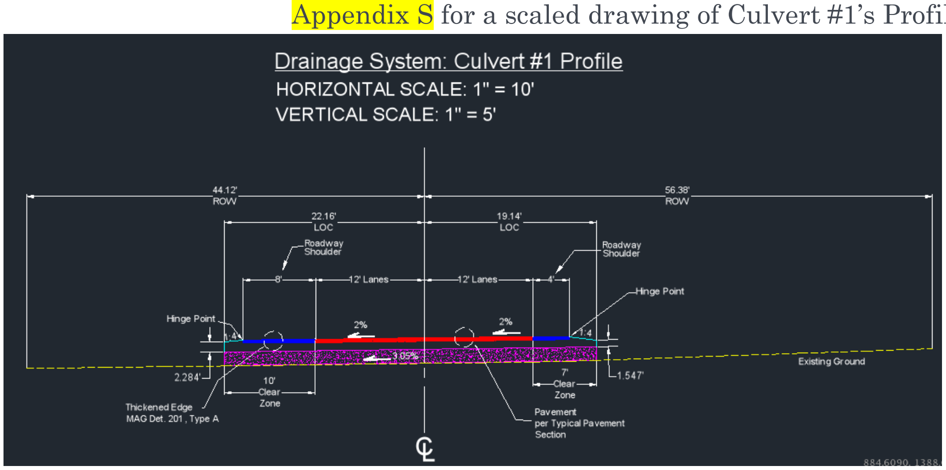


Figure 5.16: Culvert #1 Profile

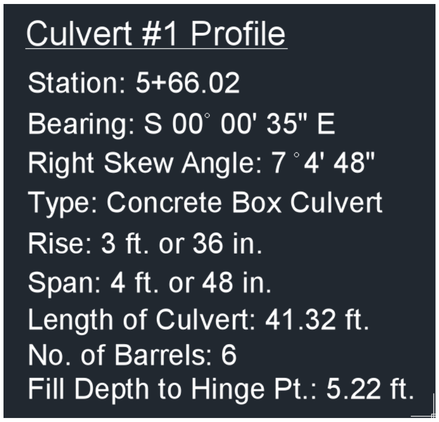


Figure 5.17: Culvert #1 Profile Information

Civil 3D's Hydraflow Express was used to determine the number of barrels needed to pass a peak flow of 2920 cfs without overtopping the roadway as requested by the client. The maximum number of barrels that could be entered into the software was four, the flow was divided by half (2920cfs/2=1460 cfs) and inputted, which allowed three barrels to pass the flow (1460 cfs) through without flooding the road. Now, since the flow was divided by half and three barrels were checked under both Inlet (See Appendix U4 for Hydraflow: Inlet Control) and Outlet Control (See Appendix U5 for Hydraflow: Outlet Control), the total number of barrels needed for a peak 100-year flow of 2920 cfs will be a total of six barrels. As shown in the Hydraflow reports in the appendices, the concrete box culverts will have a 30° to 75° flared wingwalls (See Appendix U6 for ADOT's Wingwall Detail Drawings). 30° flared wingwalls were chosen for the design of Culvert #1.

The skewed angle was calculated to be 7.08° (7°4'48" as seen in Appendix U3), which is used in selecting the correct wingwall lengths. Shown in Appendix U6 is ADOT's *Reinforced Concrete Box Culverts Inlet Wings - Skew 0° to 20°* to help obtain wingwall lengths of 7 and 8 feet from provided tables. The calculated skew angle was rounded up to 10°, a culvert height of 6 feet, and the foreslope (1V:4H) of the road are required to use the table shown in Appendix U6. A culvert height of 6 feet was used instead of the designed 3 feet, because the structure detail drawing only provides values for a culvert height from 5 to 7 feet, therefore the culvert lengths from the table were divided by half to match the height of the designed 3 foot high box culvert (See Figure 6.3 for a Plan View of Culvert#1 and the flared wingwalls).

The existing culvert shown in Figure 6.3 was extended to help Culvert #1 along with a fill boundary denoted as a dotted line (in light blue connecting Culvert #1's bottom wingwall to the existing culverts' headwall) to prevent flooding to enter the sides of the roadway, which may cause the road's embankment to erode. Headwall lengths were obtained also from ADOT's *Pipe Culvert Headwalls Inlet and Outlet 18" to 42"* structure detail drawing (See Appendix U7). 20 inches is the diameter of the four existing corrugated metal pipes; the diameter is needed to interpolate values from the table in Appendix U7 to get a length (L) of 10.2 feet. This 10.2 feet was then added to another value (E = 7.5 feet), which was interpolated from the table to get a total of 18 feet wide headwall (See Appendix U7 Plan View to define L & E values).

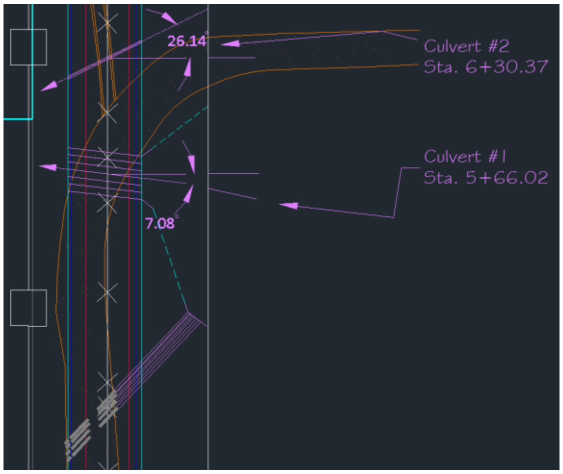


Figure 5.18: Plan View of Culverts, Wingwalls, Headwalls, & Fill lines (dotted light blue)

5.7.3 Culvert # 2

5.7.3.1 Hydrology

A watershed delineation (See Appendix V1) was completed using USGS's NMV to acquire a drainage area (A) of 30.08 acres, which is less than 160 acres. Since A≤160 acres, the Rational Method was used to get the 100-year peak discharge (Q_{100}). The equation consists of the rational method runoff coefficient (C=0.20 from Highway Engineering, PH Wright, 1996, Table 11-2), rainfall intensity (i=0.167 inches/hour or 4 inches/day), and the drainage area (A=30.08 acres). Before computing the Q_{100}, the client's given rainfall intensity was verified using the National Oceanic & Atmospheric Administration's (NOAA) National Weather Service (See Figure 6.4). NOAA's rainfall intensity came to be 0.164 inches per hour, which is very close to the value the client provided. The team used the client's rainfall intensity to evaluate the Q_{100} of 1.005 cfs (See Appendix V2 for Q_{100} calculations). Then, the stream flow's slope (S=3.02%) was determined in a similar manner as described for Culvert #1.

PF tabular | PF graphical | Supplementary information | Print Page

AMS-based 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	2.75 (2.33-3.23)	4.03 (3.42-4.74)	5.00 (4.21-5.86)	6.35 (5.29-7.40)	7.46 (6.16-8.69)	8.70 (7.09-10.2)	10.0 (8.04-11.7)	11.9 (9.36-14.0)	13.5 (10.4-15.9)
10-min	2.09 (1.78-2.45)	3.07 (2.60-3.61)	3.81 (3.21-4.46)	4.83 (4.03-5.64)	5.68 (4.69-6.61)	6.62 (5.39-7.72)	7.62 (6.12-8.90)	9.06 (7.12-10.6)	10.2 (7.93-12.1)
15-min	1.73 (1.46-2.03)	2.54 (2.15-2.98)	3.15 (2.65-3.68)	3.99 (3.33-4.66)	4.69 (3.87-5.47)	5.47 (4.46-6.38)	6.30 (5.06-7.35)	7.48 (5.88-8.79)	8.47 (6.56-9.99)
30-min	1.16 (0.988-1.37)	1.71 (1.45-2.01)	2.12 (1.79-2.48)	2.69 (2.24-3.14)	3.16 (2.61-3.68)	3.68 (3.00-4.29)	4.24 (3.40-4.95)	5.04 (3.96-5.92)	5.70 (4.41-6.72)
60-min	0.719 (0.611-0.845)	1.06 (0.897-1.24)	1.31 (1.10-1.54)	1.66 (1.39-1.94)	1.96 (1.61-2.28)	2.28 (1.86-2.66)	2.62 (2.11-3.06)	3.12 (2.45-3.66)	3.53 (2.73-4.16)
2-hr	0.432 (0.375-0.500)	0.619 (0.536-0.715)	0.758 (0.652-0.872)	0.955 (0.812-1.10)	1.12 (0.938-1.28)	1.30 (1.07-1.49)	1.49 (1.22-1.72)	1.77 (1.42-2.05)	2.00 (1.57-2.32)
3-hr	0.311 (0.275-0.356)	0.436 (0.385-0.498)	0.527 (0.463-0.601)	0.654 (0.569-0.745)	0.759 (0.653-0.862)	0.876 (0.744-0.997)	1.00 (0.838-1.15)	1.19 (0.971-1.38)	1.34 (1.07-1.56)
6-hr	0.188 (0.167-0.212)	0.254 (0.226-0.287)	0.303 (0.268-0.341)	0.371 (0.325-0.418)	0.425 (0.369-0.477)	0.485 (0.416-0.546)	0.547 (0.463-0.619)	0.638 (0.530-0.726)	0.715 (0.582-0.818)
12-hr	0.110 (0.099-0.124)	0.147 (0.131-0.164)	0.172 (0.153-0.192)	0.204 (0.181-0.228)	0.229 (0.202-0.256)	0.256 (0.224-0.286)	0.282 (0.244-0.316)	0.321 (0.274-0.363)	0.357 (0.299-0.411)
24-hr	0.068 (0.061-0.076)	0.092 (0.083-0.103)	0.109 (0.097-0.121)	0.130 (0.116-0.144)	0.147 (0.130-0.163)	0.164 (0.145-0.182)	0.182 (0.159-0.201)	0.205 (0.178-0.228)	0.223 (0.192-0.249)
2-day	0.037 (0.033-0.041)	0.050 (0.044-0.056)	0.059 (0.052-0.066)	0.070 (0.062-0.079)	0.079 (0.070-0.089)	0.089 (0.077-0.100)	0.098 (0.085-0.110)	0.111 (0.095-0.125)	0.121 (0.103-0.136)

Figure 5.19: NOAA's Rainfall Intensity (i) Value

5.7.3.2 Hydraulics Design:

Culvert #2's location was determined by examining Figure 6.0 and reviewing the vertical alignment to make sure that the culvert is properly alignment with the stream's flow direction. Once the alignment of the culvert was set, the skew angle (See Appendix V3 for Skew Angle calculations) of the culvert was computed to be 26.14° at which Culvert #2's Profile was drawn at (See Figure 6.5, 6.6 and Appendix T for Culvert drawings and information). Then, a diameter of 16 inches was chosen to carry the peak 100-year flow (Q_{100} = 1.005 cfs) through one corrugated metal pipe with a low chance of having debris clogging up the barrel. By looking at the profile, a 3.5 feet free board will be provided and the culvert was designed to not overtop the roadway as seen from the Hydraflow reports shown in Appendix V4 (under Inlet Control) and V5 (under Outlet Control).

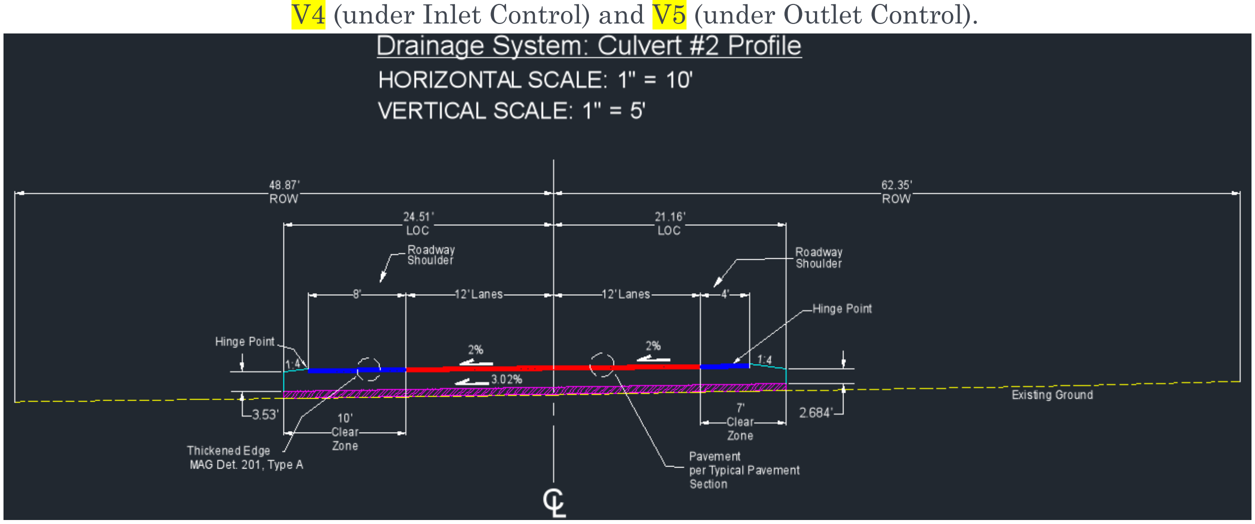


Figure 5.20: Culvert #2 Profile

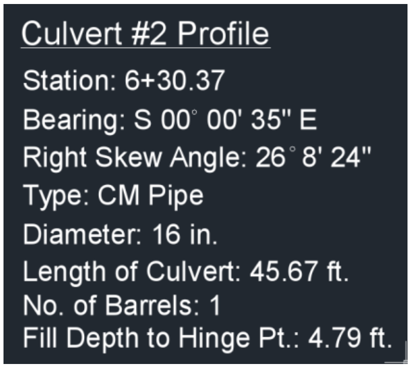


Figure 5.21: Culvert#2 Information

5.7.3.3 Culvert Maintenance:

Finally, yearly maintenance of the new and current culverts is required to attain their highest performances. Maintenance consists of culvert performance, erosion, and debris blockages to best achieve the designed culverts' performances. A concrete slab should be installed at the downward stream of existing culvert #2 to stop the erosion (scouring – See Figure 1.4) and maybe considered for all other culverts after yearly checkups.

6.0 References

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7.0 Appendixes

Appendix A:

WELL_DEPTH	CASING_DEPTH	CASING_DIAMETER	WATER_LEVEL	PUMPRATE
410.00	22.00	10.00	67.00	0.00
300.00	74.00	6.00	255.00	0.00
300.00	200.00	6.00	250.00	0.00
270.00	80.00	8.00	150.00	30.00
260.00	40.00	6.00	108.00	32.00
235.00	60.00	6.00	84.00	14.00
235.00	21.00	8.00	73.00	0.00
235.00	60.00	6.00	84.00	0.00
225.00	42.00	6.00	60.00	0.00
220.00	162.00	8.00	98.00	11.00
210.00	42.00	6.00	48.00	0.00
200.00	0.00	10.00	0.00	30.00
200.00	40.00	6.00	90.00	20.00
180.00	45.00	6.00	0.00	0.00
165.00	0.00	6.00	75.00	0.00
156.00	125.00	6.00	16.00	16.00
150.00	20.00	12.00	77.00	190.00
150.00	40.00	6.00	22.00	0.00
150.00	150.00	6.00	19.00	0.00
150.00	150.00	6.00	21.00	0.00
150.00	150.00	6.00	20.00	0.00
150.00	42.00	6.00	21.00	0.00
150.00	53.00	8.00	40.00	0.00
140.00	100.00	6.00	135.00	20.00
140.00	31.00	6.00	88.00	20.00
140.00	40.00	6.00	30.00	30.00
140.00	68.00	6.00	41.00	0.00
135.00	38.00	6.00	10.00	30.00
135.00	63.00	6.00	10.00	0.00
132.00	33.00	6.00	80.00	30.00
130.00	44.00	6.00	45.00	0.00
130.00	43.00	6.00	14.00	16.00
130.00	52.00	6.00	40.00	17.00
130.00	35.00	6.00	80.00	21.00
130.00	40.00	6.00	115.00	24.00
130.00	53.00	6.00	39.00	0.00
130.00	53.00	6.00	17.00	20.00
130.00	47.00	6.00	45.00	15.00
130.00	54.00	6.00	18.00	12.00
130.00	51.00	6.00	23.00	18.00
130.00	20.00	6.00	70.00	15.00
130.00	52.00	6.00	50.00	0.00
130.00	0.00	0.00	0.00	20.00
130.00	120.00	4.00	10.00	13.00

130.00	55.00	6.00	45.00	0.00
125.00	41.00	6.00	30.00	20.00
125.00	60.00	6.00	43.00	15.00
125.00	26.00	6.00	47.00	15.00
120.00	0.00	1.00	0.00	29.00
120.00	60.00	6.00	25.00	20.00
120.00	70.00	6.00	30.00	20.00
110.00	66.00	8.00	25.00	0.00
110.00	110.00	4.00	12.00	0.00
110.00	32.00	6.00	65.00	14.00
110.00	110.00	6.00	35.00	14.00
105.00	105.00	6.00	48.00	14.00
105.00	32.00	6.00	55.00	15.00
105.00	25.00	6.00	47.00	30.00
104.00	37.00	6.00	54.00	12.00
101.00	28.00	6.00	0.00	20.00
100.00	43.00	6.00	28.00	20.00
100.00	42.00	6.00	0.00	0.00
100.00	28.00	6.00	20.00	15.00
100.00	60.00	6.00	50.00	0.00
100.00	40.00	8.00	0.00	35.00
100.00	35.00	6.00	17.00	15.00
100.00	28.00	6.00	50.00	15.00
100.00	45.00	6.00	30.00	15.00
100.00	40.00	6.00	0.00	0.00
100.00	48.00	6.00	28.00	14.00
100.00	85.00	6.00	65.00	22.00
100.00	38.00	6.00	25.00	34.00
100.00	0.00	0.00	0.00	0.00
100.00	49.00	6.00	22.00	35.00
97.00	41.00	6.00	37.00	20.00
95.00	68.00	6.00	0.00	0.00
95.00	37.00	6.00	42.00	13.00
95.00	54.00	6.00	18.00	0.00
95.00	20.00	6.00	45.00	30.00
94.00	49.00	6.00	21.00	25.00
91.00	39.00	6.00	33.00	12.00
90.00	90.00	5.00	19.00	10.00
90.00	90.00	6.00	18.00	25.00
90.00	42.00	6.00	14.00	29.00
90.00	41.00	6.00	47.00	15.00
90.00	44.00	6.00	42.00	14.00
85.00	65.00	0.00	70.00	0.00
85.00	41.00	8.00	29.00	0.00
85.00	67.00	4.00	72.00	0.00
85.00	32.00	6.00	21.00	10.00
85.00	70.00	4.00	75.00	0.00
84.00	84.00	4.00	71.00	0.00
84.00	34.00	6.00	35.00	25.00
84.00	81.00	4.00	69.00	0.00
84.00	84.00	4.00	69.00	0.00

84.00	34.00	6.00	35.00	25.00
84.00	83.00	4.00	69.00	0.00
83.00	83.00	4.00	74.00	0.00
83.00	83.00	4.00	74.00	0.00
82.00	67.00	4.00	75.00	0.00
81.00	19.00	6.00	32.00	15.00
80.00	22.00	6.00	25.00	15.00
80.00	80.00	4.00	70.00	0.00
80.00	44.00	6.00	28.00	20.00
80.00	23.00	6.00	14.00	10.00
80.00	20.00	6.00	25.00	15.00
80.00	34.00	6.00	35.00	0.00
80.00	20.00	6.00	37.00	10.00
80.00	75.00	8.00	50.00	30.00
80.00	21.00	6.00	20.00	35.00
80.00	52.00	6.00	33.00	15.00
80.00	0.00	8.00	40.00	0.00
78.00	0.00	6.00	18.00	0.00
77.00	40.00	6.00	5.00	25.00
76.00	20.00	6.00	0.00	20.00
75.00	25.00	6.00	0.00	0.00
75.00	0.00	0.00	0.00	0.00
75.00	75.00	6.00	30.00	20.00
75.00	31.00	6.00	50.00	10.00
75.00	26.00	6.00	32.00	20.00
75.00	26.00	6.00	20.00	35.00
75.00	42.00	6.00	7.00	16.00
73.00	42.00	6.00	61.00	10.00
72.00	31.00	6.00	65.00	0.00
70.00	26.00	6.00	18.00	15.00
70.00	38.00	6.00	0.00	12.00
70.00	24.00	6.00	24.00	10.00
70.00	29.00	6.00	47.00	0.00
70.00	52.00	8.00	23.00	24.00
70.00	24.00	6.00	25.00	10.00
70.00	27.00	6.00	35.00	35.00
70.00	39.00	6.00	25.00	32.00
70.00	0.00	0.00	0.00	20.00
70.00	40.00	6.00	35.00	15.00
70.00	40.00	8.00	20.00	18.00
69.00	69.00	4.00	51.00	0.00
69.00	50.00	6.00	35.00	1.00
68.00	68.00	8.00	26.00	200.00
68.00	63.00	4.00	46.00	0.00
67.00	48.00	6.00	12.00	0.00
65.00	27.00	6.00	30.00	20.00
65.00	34.00	6.00	22.00	16.00
65.00	38.00	6.00	12.00	24.00
60.00	60.00	8.00	36.00	0.00
60.00	40.00	6.00	20.00	0.00
60.00	28.00	6.00	19.00	0.00

60.00	43.00	6.00	12.00	30.00
60.00	60.00	6.00	0.00	0.00
55.00	25.00	6.00	22.00	20.00
55.00	25.00	6.00	27.00	20.00
53.00	47.00	8.00	48.00	10.00
50.00	47.00	6.00	25.00	24.00
50.00	39.00	6.00	20.00	15.00
50.00	40.00	6.00	40.00	15.00
50.00	36.00	6.00	22.00	20.00
50.00	40.00	6.00	20.00	10.00
50.00	50.00	6.00	40.00	35.00
50.00	34.00	6.00	19.00	16.00
50.00	33.00	6.00	21.00	30.00
50.00	50.00	6.00	14.00	15.00
48.00	44.00	6.00	15.00	12.00
45.00	37.00	6.00	21.00	25.00
45.00	37.00	6.00	26.00	0.00
42.00	39.00	6.00	23.00	17.00
40.00	30.00	6.00	10.00	25.00
37.00	32.00	6.00	20.00	10.00
35.00	0.00	6.00	12.00	35.00
33.00	33.00	6.00	0.00	0.00

Average Well Depth	Average Casing Depth	Average Casing Diameter	Average Water Level
104	48	6	38

Appendix B:

Table 3.3 Water and Sewer Design Flows

Land Use	Unit	WASTEWATER Avg Daily Flow/Unit (gal)	WATER Avg Daily Flow/Unit (gal)
Single Family Residential	dwelling	240	360
Multi-family[1]	dwelling	180	240
Commercial (retail/mall)[1]	1000sq-ft	75	125
Commercial (office)[1]	1000sq-ft	90	115
Warehousing/Big Box Retail[1]	1000sq-ft	25	30
Industrial[1,3]	1000sq-ft	50	65
Schools[1]	student	20	25
Hotel (no restaurant)[1]	room	100	140
Hotel (with restaurant)[1]	room	150	200
Resort[1]	room	210	300
Hospital (all flows)[1]	bed	300	500
LANDSCAPE WATER REQUIREMENTS			
General Landscaping	acre	NA	4,374
Public Right of Way or Streetscape	acre	NA	1,339
Surface Water	acre	NA	5,335

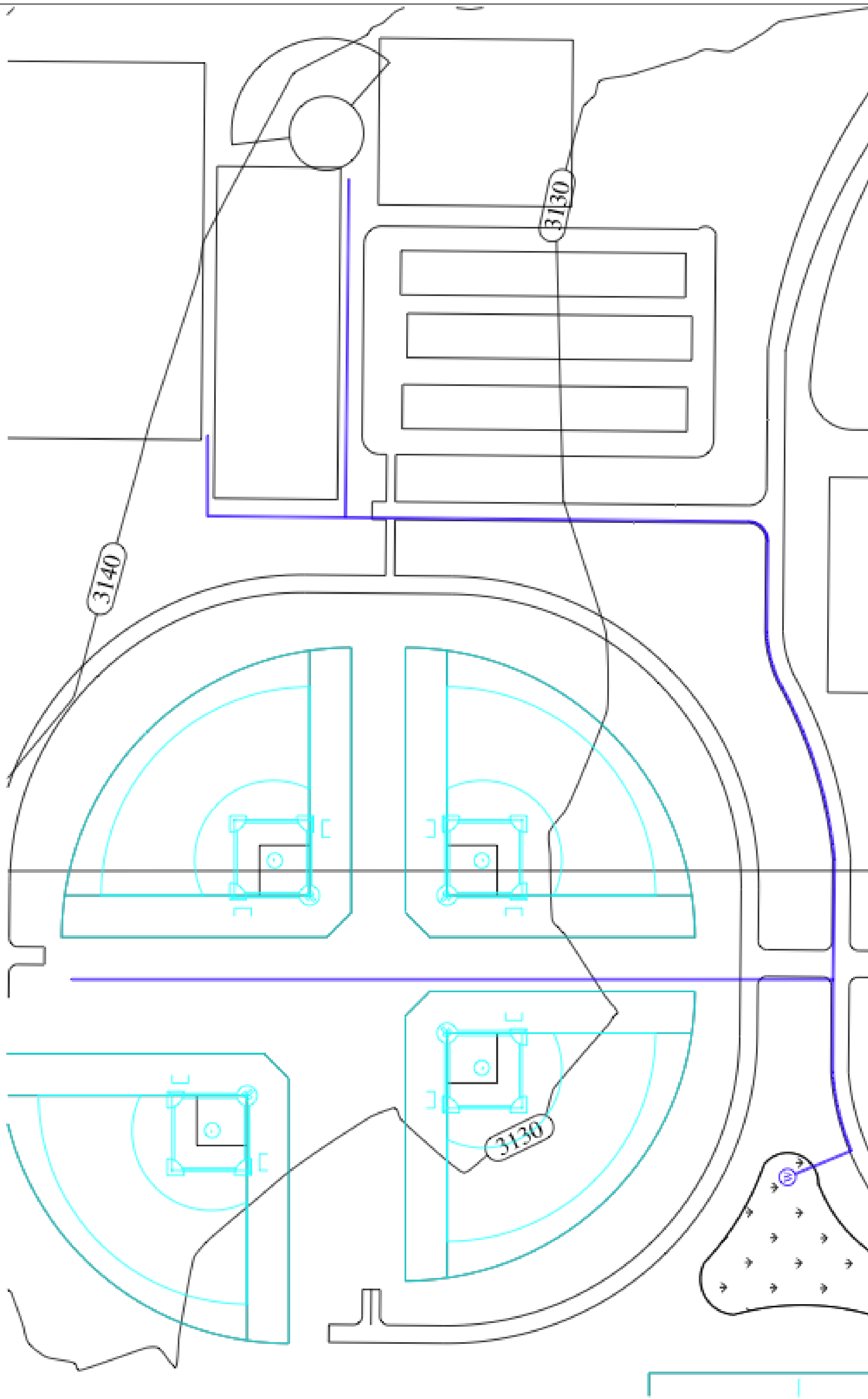
Community bldg: 125gal

Concession building: 125 gal

Comfort station: 125 gal

Equal 375 gal * 1.7 (peak Factor) = 637.5 Peak GDP

Appendix C



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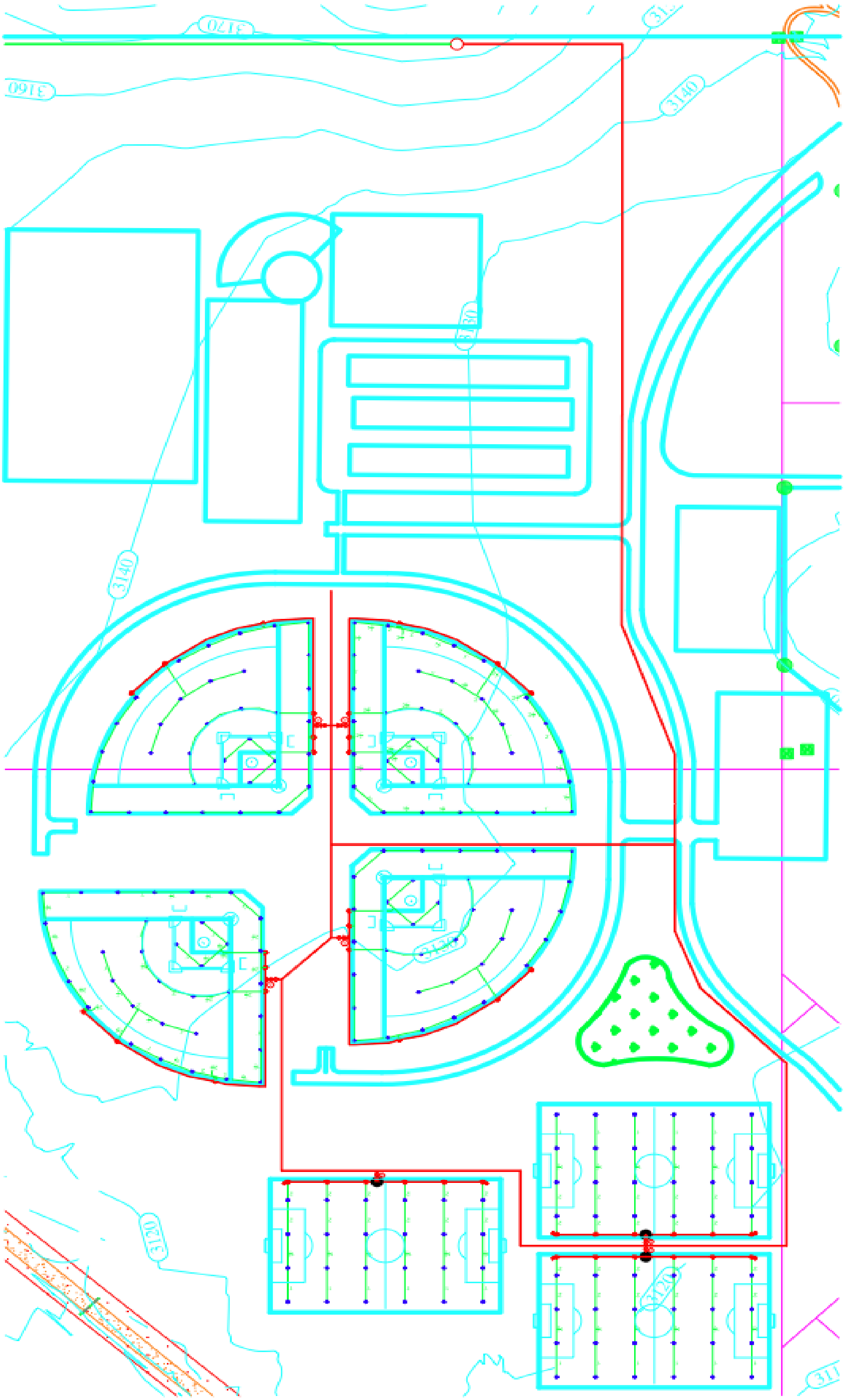

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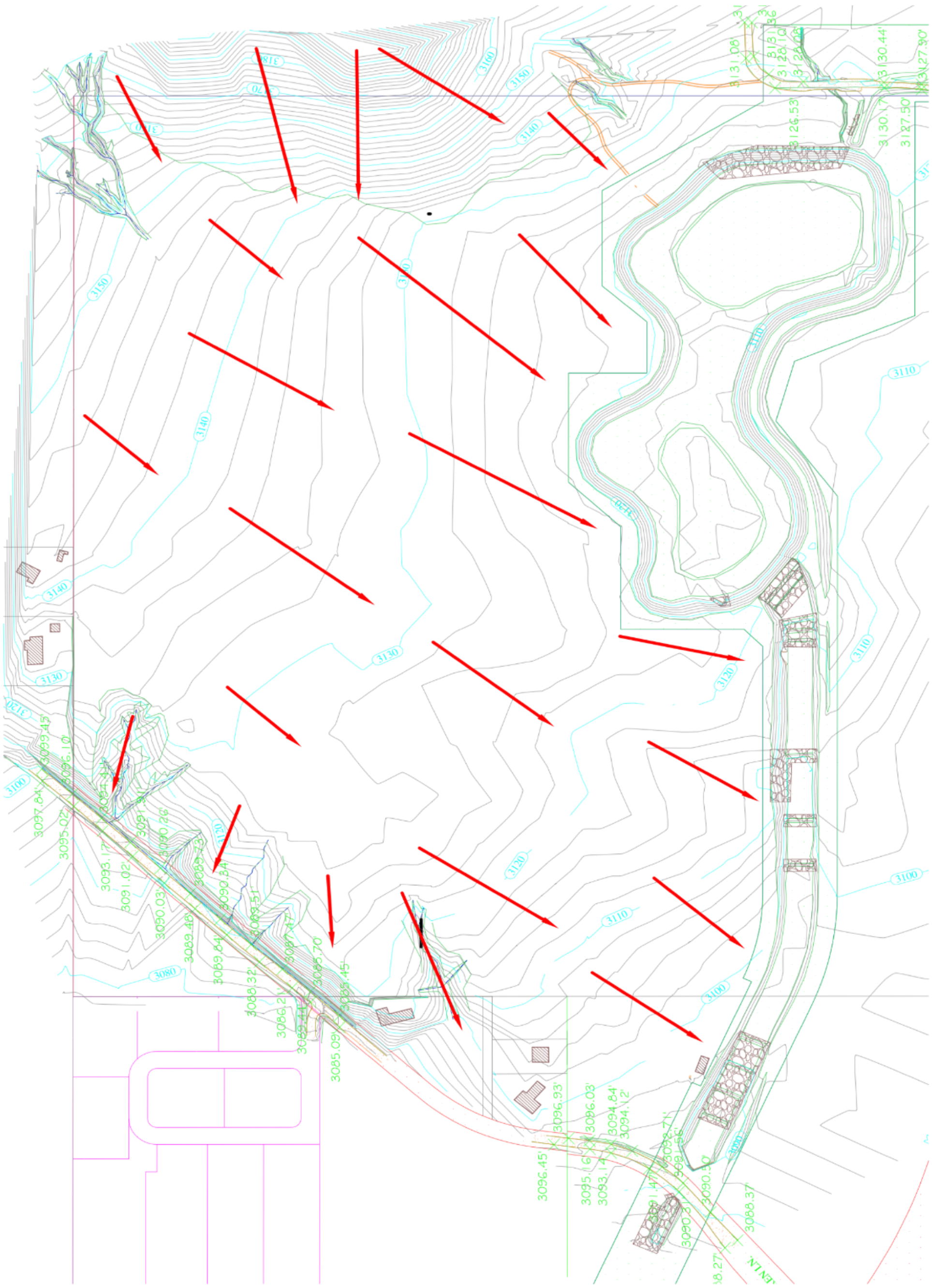

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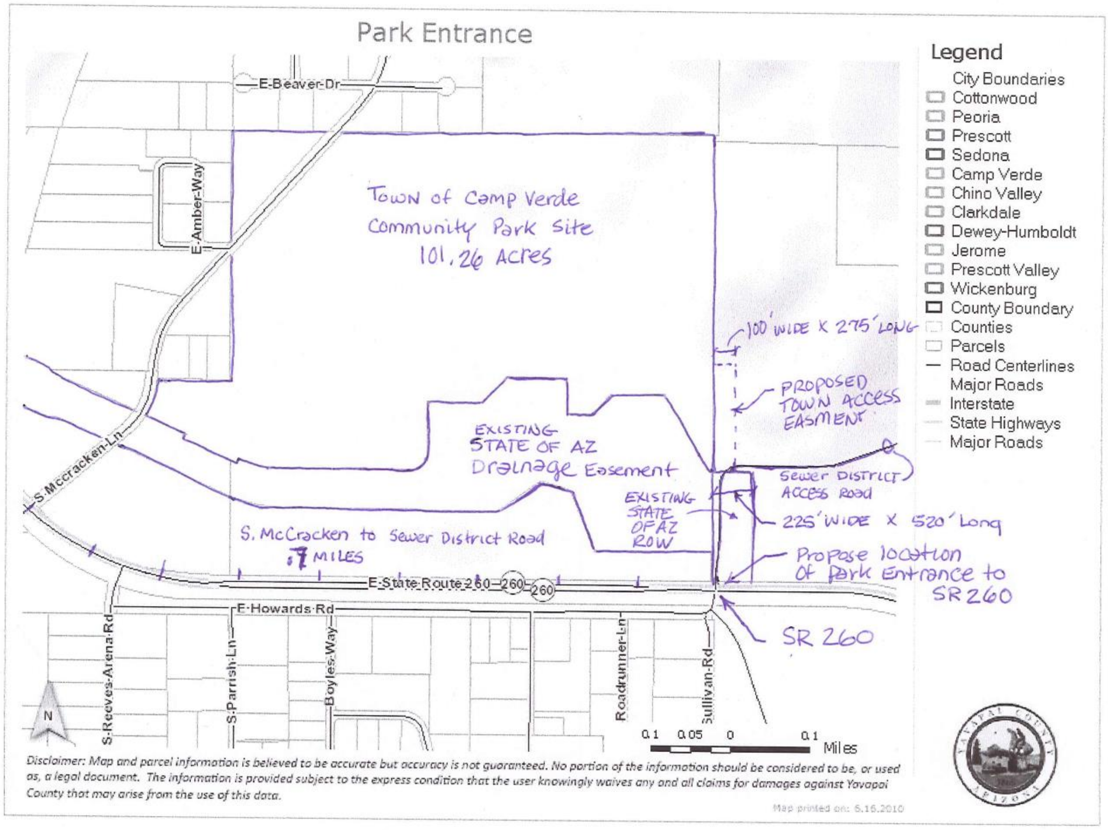
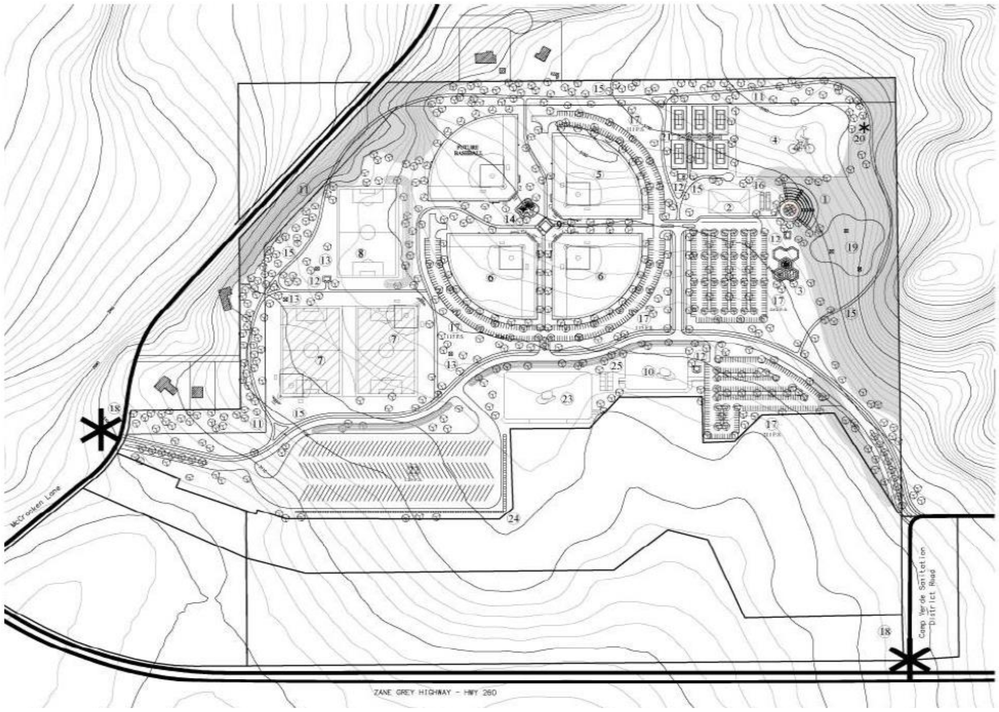

Park Entrance
Legend
City Boundaries
Cottonwood
Peoria
Prescott
Sedona
Camp Verde
Chino Valley
Clarkdale
Dewey-Humboldt
Jerome
Prescott Valley
Wickenburg
County Boundary
Counties
Parcels
Road Centerlines
Major Roads
Interstate
State Highways
Major Roads
E-Beaver-Dr
E-Amber-Way
S-Mccracken-Ln
E-State-Route-260
260
E-Howards-Rd
S-Reeves-Arena-Rd
S-Parrish-Ln
Boyles-Way
Roadrunner-Ln
Sullivan-Rd
Town of Camp Verde
Community Park Site
101.26 Acres
100' WIDE X 275' LONG
PROPOSED
TOWN ACCESS
EASMENT
EXISTING
STATE OF AZ
Drainage Easement
Sewer DISTRICT
ACCESS Road
EXISTING
STATE
OF AZ
ROW
225' WIDE X 520' Long
Propose location
Of Park Entrance to
SR 260
SR 260
S. McCracken to Sewer District Road
.7 MILES
N
0.1 0.05 0 0.1
Miles
Disclaimer: Map and parcel information is believed to be accurate but accuracy is not guaranteed. No portion of the information should be considered to be, or used as, a legal document. The information is provided subject to the express condition that the user knowingly waives any and all claims for damages against Yavapai County that may arise from the use of this data.
Map printed on: 5.16.2010

Appendix G

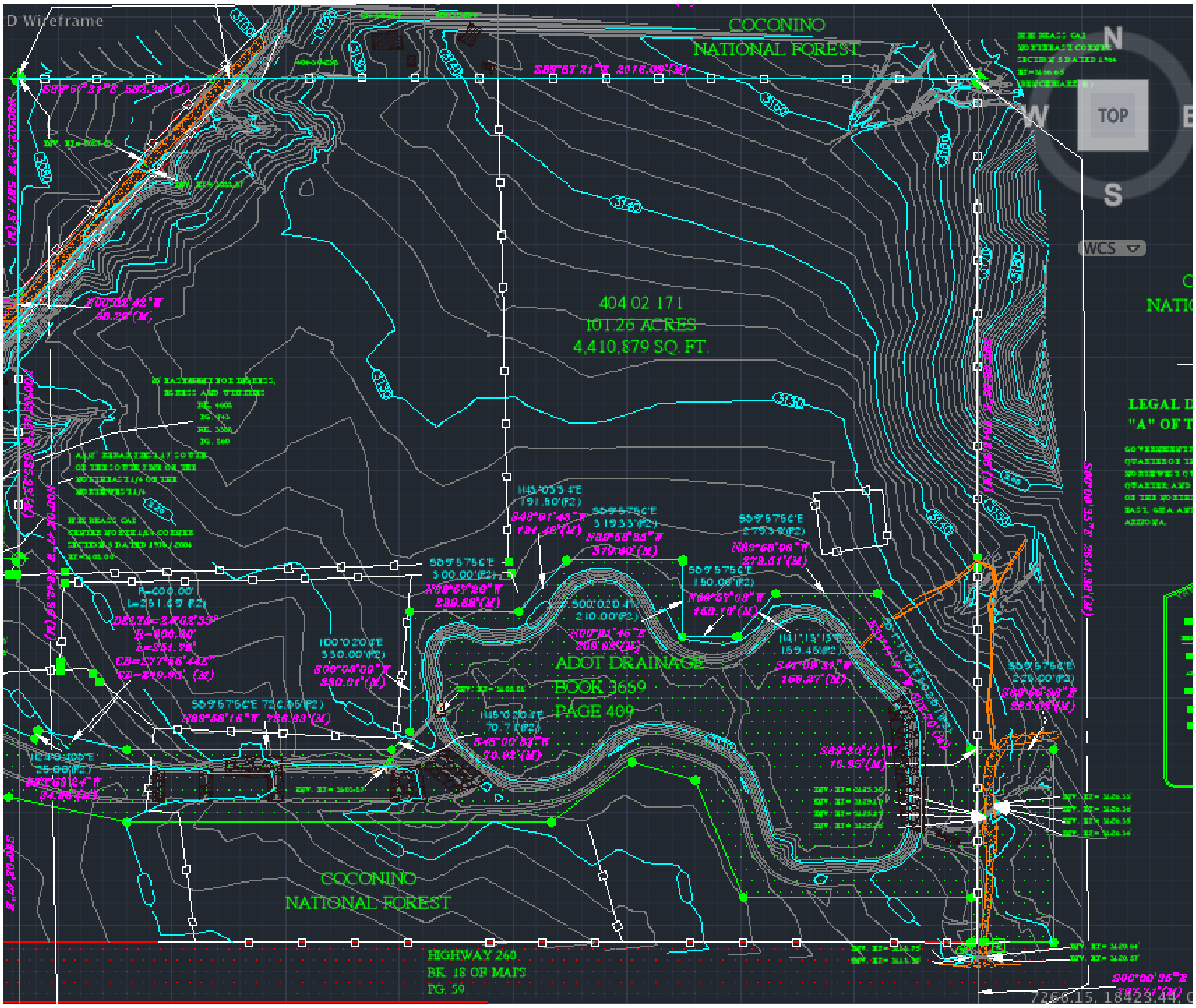
Camp Verde
Community Park
Masterplan
Legend
1 Amphitheater
2 Community Center
3 Splash Pad & Tot Lot
4 BMX Course
5 Regulation Baseball w/ mound
6 Regulation Baseball w/o mound
7 Soccer / Football
8 Soccer / Practice Baseball Field
9 Concession Building
10 Rodeo / Practice Arena
11 100' Buffer
12 Comfort Station
13 Ramada / Picnic
14 Tot Lot
15 Bike / Pedestrian Trail
16 Bocceball Court
17 Parking
18 Park Entrance
19 Picnic Area / Star Gazing
20 Trailhead
21 Tennis w/ Ramadas
22 Trailer Parking
McCracken Lane
HWY 260
RBF



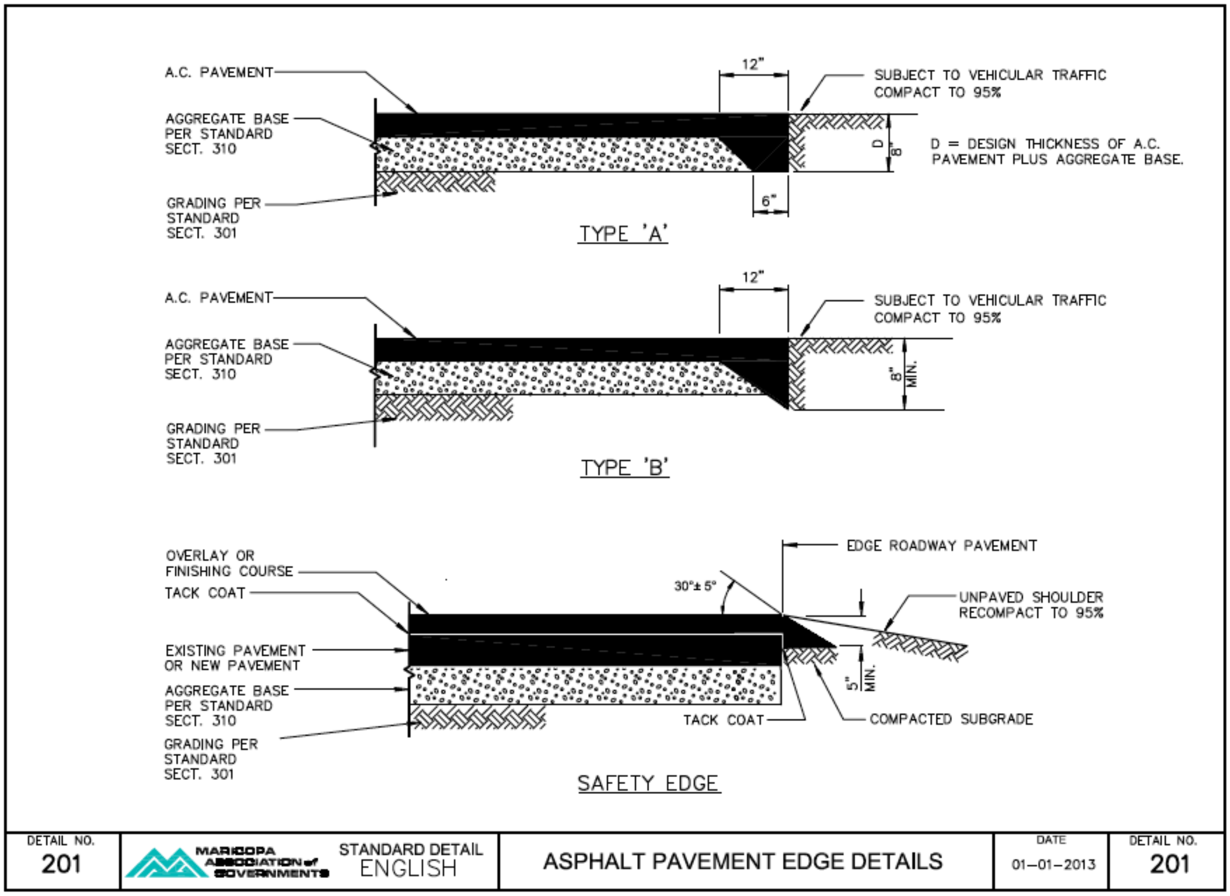
Legend

1. Amphitheater
2. Community Center
3. Splash Pad & Tot Lot
4. BMX Course
5. Regulation Baseball w/ mound
6. Regulation Baseball w/o mound
7. Soccer / Football / Baseball Practice Fields
8. Soccer w/ Berm Seating
9. Concession Building
10. Equestrian / Practice Arena
11. 100' Buffer
12. Comfort Station
13. Ramada / Picnic
14. Tot Lot
15. Bike / Pedestrian Trail
16. Bocceball Court
17. Parking
18. Park Entrance
19. Picnic Area / Star Gazing
20. Trailhead
21. Tennis w/ Ramadas
22. Trailer Parking
23. Compitition Equestrian Arena
24. 10' x 10' Animal Stalls
25. Holding Pens

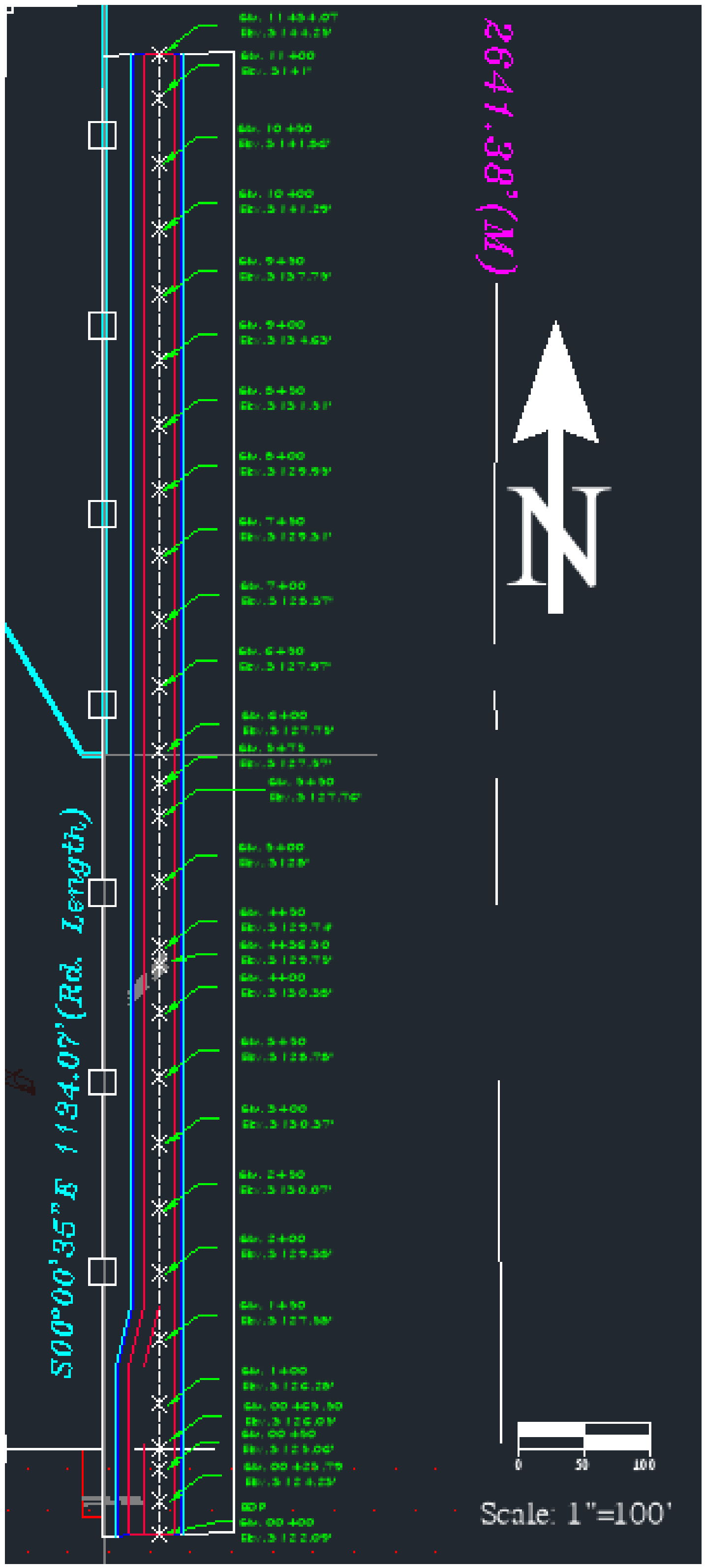
Appendix H

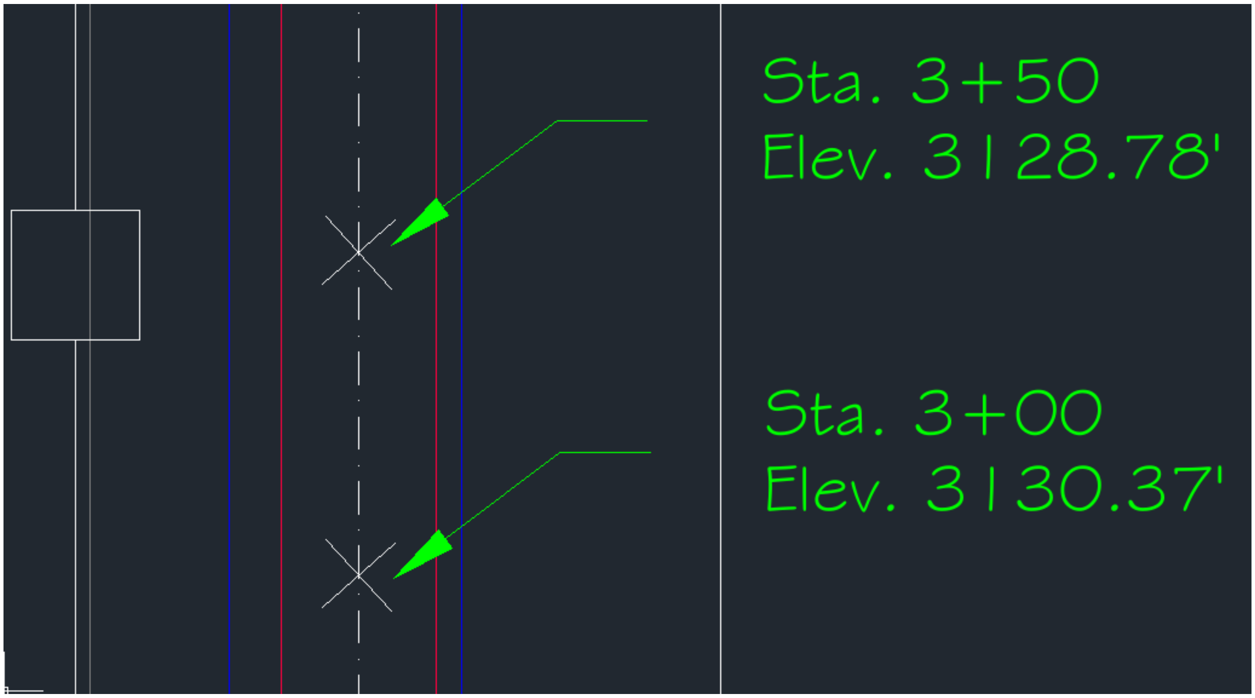

COCONINO
NATIONAL FOREST
404 02 171
101.26 ACRES
4,410,879 SQ. FT.
ADOT DRAINAGE
BOOK 3669
PAGE 409
COCONINO
NATIONAL FOREST
HIGHWAY 260
BK. 18 OF MAPS
PG. 59
N
W
S
E
TOP
WCS

Appendix I


A.C. PAVEMENT
AGGREGATE BASE PER STANDARD SECT. 310
GRADING PER STANDARD SECT. 301
12"
6"
SUBJECT TO VEHICULAR TRAFFIC COMPACT TO 95%
D
8"
D = DESIGN THICKNESS OF A.C. PAVEMENT PLUS AGGREGATE BASE.
TYPE 'A'
A.C. PAVEMENT
AGGREGATE BASE PER STANDARD SECT. 310
GRADING PER STANDARD SECT. 301
12"
SUBJECT TO VEHICULAR TRAFFIC COMPACT TO 95%
8" MIN.
TYPE 'B'
OVERLAY OR FINISHING COURSE
TACK COAT
EXISTING PAVEMENT OR NEW PAVEMENT
AGGREGATE BASE PER STANDARD SECT. 310
GRADING PER STANDARD SECT. 301
30°± 5°
EDGE ROADWAY PAVEMENT
UNPAVED SHOULDER RECOMPACT TO 95%
5" MIN.
TACK COAT
COMPACTED SUBGRADE
SAFETY EDGE
DETAIL NO.
201
MARICOPA ASSOCIATION of GOVERNMENTS
STANDARD DETAIL
ENGLISH
ASPHALT PAVEMENT EDGE DETAILS
DATE
01-01-2013
DETAIL NO.
201

Appendix J




Sta. 3+50
Elev. 3128.78'
Sta. 3+00
Elev. 3130.37'

Appendix K

Horizontal Alignment

℄ Stations	Proposed ℄ Elevations	Existing ℄ Elevations
BOP 00+00	3122.09'	3122.09'
00+25.79	3124.23'	(2/31.5)19.5+3122 = 3123.24'
00+50	(2/17)9+3124' = 3125.06'	(2/52)12.5+3124 = 3124.48'
00+65.50	(2/84)2+3126' = 3126.05'	(2/62)32+3124 = 3125.03'
1+00	(2/61)8.5+3126' = 3126.28'	(2/79)74.5+3124 = 3125.89'
1+50	(2/38)30+3126 = 3127.58'	(2/36)10.5+3126 = 3126.58'
2+00	(2/29)20+3128 = 3129.38'	(2/27.5)9+3128 = 3128.65'
2+50	(2/60)2+3130 = 3130.07'	(2/21.5)16+3128 = 3129.49'
3+00	(2/60)11+3130 = 3130.37'	(2/59.5)3.5+3130 = 3130.12'
3+50	(2/66.5)26+3128 = 3128.78'	(2/64.5)21+3128 = 3128.65'
4+00	(2/24)4.5+3130 = 3130.38'	(2/14)8.5+3128 = 3129.21'
4+36.50	(2/20)17.5+3128 = 3129.75'	(2/20)9.5+3128 = 3128.95'
4+50	(2/19.5)17+3128 = 3129.74'	(2/19.5)7.5+3128 = 3128.77'
5+00	= 3128.00'	(2/46)30+3126 = 3127.30'
5+50	(2/67)59+3126 = 3127.76'	(2/68)48.5+3126 = 3127.43'
5+75	(2/83)65+3126 = 3127.57'	3127.57'
6+00	(2/80.5)70.5+3126.00 = 3127.75'	(2/81.5)72.5+3126 = 3127.78'
6+50	(2/71.6)69.9+3126 = 3127.97'	(2/71)66+3126 = 3127.86'
7+00	(2/41)7.5+3128 = 3128.37'	(2/62.5)59.5+3126 = 3127.90'
7+50	(2/49.5)32.5+3128 = 3129.31'	(2/53.5)22+3128 = 3128.82'
8+00	(2/66.5)65+3128 = 3129.95'	(2/68.5)60.5+3128 = 3129.77'
8+50	(2/38.5)29+3130 = 3131.51'	3131.51'
9+00	(2/24)7.5+3134 = 3134.63'	(2/21)7+3134 = 3134.61'
9+50	(2/4)3.5+3136 = 3137.75'	(2/60.5)59+3134 = 3135.95'
10+00	(2/7)4.5+3140 = 3141.29'	3138'
10+50	(2/37)34.5+3140 = 3141.86'	(2/20.5)19+3140 = 3141.85'
11+00	3141.0'	3140.0'
EOP 11+34.07	(2/8)1.5+3144 = 3144.25'	0

★ Grade from BOP to EOP

- Grade $= \left(\frac{\text{Rise}}{\text{Run}}\right) * 100$

$$= \left(\frac{3146.38' - 3123.25'}{1134.07' - 0}\right) \times 100$$

$$= 2.03956\%$$

★ Bearing from BOP to EOP:

S 00°00'35" E

Appendix L

★ Sight Distance :

① SSD

- Design Speed = 15 mph
- Brake reaction distance = 55.1 ft
- Braking distance on level = 21.6 ft
- SSD for calculated = 76.7 ft
- SSD for Design = 80 ft

AASHTO 2001 pg # 112 Ex 3-1

or

$$d = 1.47\,Vt + 1.075\left(\frac{V^2}{a}\right)$$

$$= 1.47(15mph)(2.5\,sec) + 1.075\left(\frac{15mph^2}{11.2\,ft/s}\right)$$

$$= 76.721\ ft$$

AASHTO 2001 ch3 pg 113 (3-2)

d - SSD in (ft)
V - design speed (mph)
t - brake reaction time (2.5 seconds) ← AASHT 2001 Ch 3 pg. 111
a - deceleration rate, (11.2 ft/s^2) ← AASHTO 2001 ch3 pg 111

Appendix M

★ DSD :

- The decision sight distances for avoidance maneuvers C, D, & E are:

AASHTO 2001 Ch3 pg 117

$$d = 1.47Vt \quad \} \ (3\text{-}5)$$

$$= 1.47(15mph)(11.2 \ sec)$$

$$= 246.96 \ ft$$

AASHTO 2001 Ch3 pg 116 Ex. 3-3

d - DSD for LOS C
V - design speed (mph)
t - total pre-maneuver & manuver time, s } LOS C (10.2s – 11.2s)

★ Braking Distance

$$d = 1.075\left(\frac{V^2}{a}\right) \quad \} \ (3\text{-}1) \quad \text{AASHTO 2001 Ch3 pg 111}$$

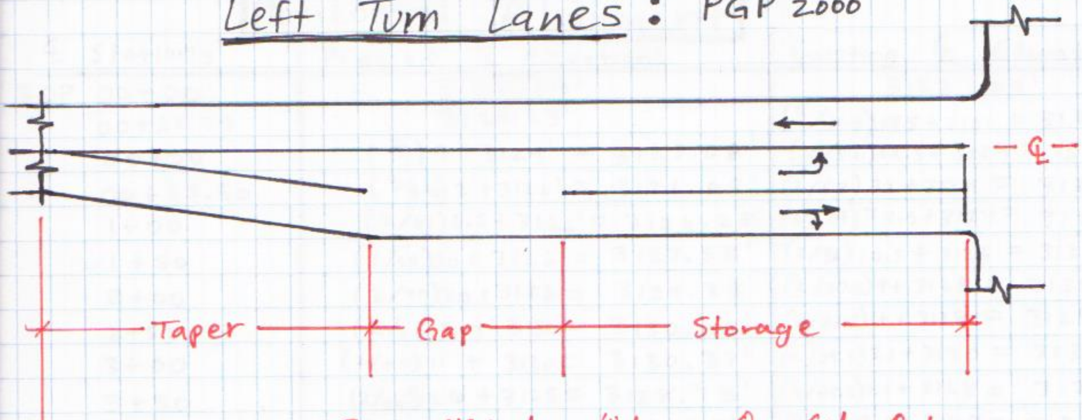
$$= 1.075\left(\frac{15mph^2}{11.2 \ ft/s^2}\right)$$

$$= 21.596 \ ft$$

Appendix N

9/29/13

Left Turn Lanes : ADOT PGP 2000



PGP: Figure 430-A – Widening One Side Only

T = length of taper (ft)
W = width of the added lane (ft)
S = posted speed for existing roadways, or design speed for new or reconstructed roadways (mph)

★ $T = \frac{S^2 \times W}{60}$ for speeds under 45 mph

$= \frac{(15\,mph)^2 \times (12\,ft)}{60} = \underline{45\ ft}$ but Client wants 200 ft

★ Gap Length:

- Table 430-1 – Gap Lengths
  ↳ Gap length = 60 ft for < 40 mph design speed

★ Storage Length:

- Storage length = braking distance + queue length
  ↳ Braking distance = 20 feet ← Table 430-2, PGP
  ↳ Queue: should provide space for two passenger cars @ 25 ft each < 10% truck %.

PGP pg 430-5

$\approx 2 \times (25') = \underline{50\,ft}$

- Storage length = 20 ft + 50 ft = 70 ft

Appendix O

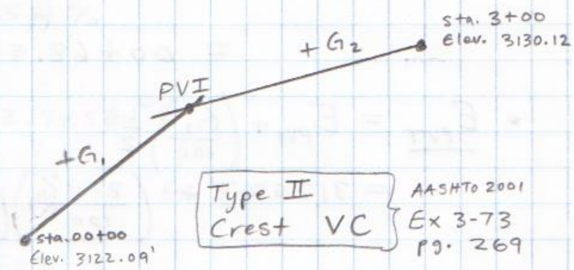
Vertical Alignment

- Roadway Functional Classification: Rural Collector

- Maximum Grade
  - Max. Grade for Rural Collectors
    - → Level : 15 mph ≈ 9%
    - → Rolling : 15 mph ≈ 12%

    AASHTO pg 427 Ex 6-4

- Vertical Curves:



  - PVI #1
    - → Sta. 00+65.5
    - → Elevation: 3125.03'

    Type II Crest VC — AASHTO 2001 Ex 3-73 pg. 269

  - $+G_1 = \left(\dfrac{3125.03' - 3122.09'}{65.5' - 0.0}\right) \times 100\%$
    $= 4.49\%$

  - $+G_2 = \left(\dfrac{3130.12' - 3125.03'}{300' - 65.5'}\right) \times 100\% = 2.17\%$

  - $A\left(\text{Algebraic diff. in grade}\right) = G_2 - G_1 = 2.17\% - 4.49\% = -2.32\%$

  - $S\left(\text{sight stopping dist.}\right) = 80$ ft for 15 mph ← AASHTO 2001 pg 274 Ex 3-76

  - $K\left(\text{rate of vertical curvature}\right) = 3$ ← AASHTO 2001 Ex 3-76 pg 274

  - $L = KA = (3)(2.32) = \boxed{6.96\text{ft} = L}$

  - S < L:
    $L_{min} = \dfrac{AS^2}{2158}$ — AASHTO 2001 pg 272 (3-45) & ADOT 2012 pg 200-28
    $= \left(\dfrac{2.32\% (80)^2}{2158}\right)$
    $= 6.88$ ft

- PVC, Station $= \text{PVI Sta.} - \frac{L}{2}$
$= 65.5' - \left(\frac{6.96'}{2}\right)$
$= 00+62.02$

- $E_{PVC} = E_{PVI} - \left(\frac{G_1}{100}\right)\frac{L}{2} = 3125.03' - \left(\frac{4.49\%}{100}\right)\frac{6.96'}{2}$
$= 3124.87'$

- PVT, Station $= \text{PVI Sta.} + \frac{L}{2}$
$= 65.5' + \left(\frac{6.96'}{2}\right)$
$= 00+68.98$

- $E_{PVT} = E_{PVI} + \left(\frac{G_2}{100}\right)\frac{L}{2}$
$= 3125.03' + \left(\frac{2.17\%}{100}\right)\left(\frac{6.96'}{2}\right)$
$= 3125.11'$

- Elevation of curve @ PVI = 3125.01
- $X_m = \left|\frac{G_1 * L}{G_2 - G_1}\right| = 1.06466 \text{ ft}$
- high pt. sta. = PVC Sta. + X_m
$= 62.02' + 1.06466'$
$= 63.085$
$= 00+63.09$

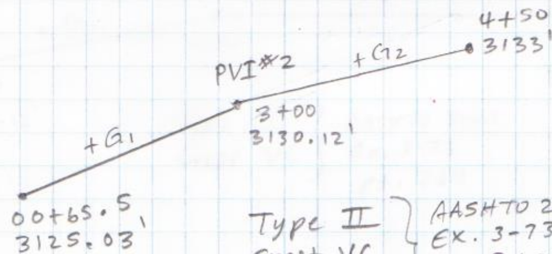
- high pt. elev. $\Rightarrow E_x = E_{pvc} + \left(\frac{G_1}{100}\right)x_m$
$= 3124.87' + \left(\frac{4.49\%}{100}\right)1.06466'$
$= 3124.92'$

Appendix P

★ Vertical Alignment:

• Vertical Curve # 2

↳ PVI # 2
→ Station: 3+00
→ Elevation: 3130.12'



Type II Crest VC } AASHTO 2001 Ex. 3-73 pg. 269

• $+G_1 = \dfrac{(3130.12' - 3125.03')}{(300' - 65.5')} \times 100\%$

$= +2.171\%$

• $+G_2 = \dfrac{(3133' - 3130.12')}{(450 - 300)} \times 100\% = +1.92\%$

• $A = G_2 - G_1 = |1.92\% - 2.17\%| = 0.251\%$

• $S = 80$ ft for 15 mph ← AASHTO 2001 pg. 274 Ex. 3-76

• $K = 3$ ← AASHTO 2001 Ex 3-76 pg 274

• $L = KA = (3)(0.251\%) = \boxed{0.752 \text{ ft}}$

• PVC Sta. = PVI sta. $- \frac{L}{2} = 300' - \frac{0.752'}{2} = 2+99.624$

• $E_{PVC} = E_{PVI} - \left(\frac{G_1}{100}\right)\frac{L}{2} = 3130.12' - \left(\frac{2.171\%}{100}\right)\frac{0.752'}{2} = 3130.11'$

• PVT Sta. = PVI sta. $+ \frac{L}{2} = 300' + \frac{0.752'}{2} = 3+00.376$

• $E_{PVT} = E_{PVI} + \left(\frac{G_2}{100}\right)\frac{L}{2} = 3130.12' + \left(\frac{1.92\%}{100}\right)\frac{0.752'}{2} = 3130.13'$

• $X_m = \left|\dfrac{G_1 * L}{G_2 - G_1}\right| = \left|\dfrac{2.171\% * 0.752'}{1.92\% - 2.171\%}\right| = 6.504'$

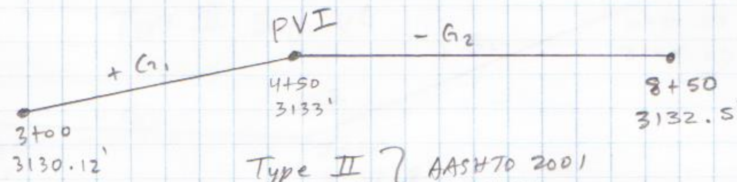
• Sta. of highest pt. on VC = PVC Sta. + X_m = 2+99.624 + 6.504'
= 3+06.128

• Elev. of highest pt. on VC $= E_{PVC} + \left(\frac{G_1}{100}\right)X_m + \dfrac{(G_2 - G_1)X_m^2}{200L}$
$= 3130.11' + \left(\frac{2.17\%}{100}\right)6.504' + \dfrac{(1.92\% - 2.17\%)\,6.504'^2}{200 \cdot 0.752'}$
$= 3130.18'$

Appendix Q

★ Vertical Curve #3

PVI #3
→ Station: 4+50
→ Elevation: 3133'



Type II Crest VC } AASHTO 2001 Ex. 3-73 pg. 269

- $+G_1 = \dfrac{(3133' - 3130.12')}{(450' - 300')} \times 100\% = 1.92\%$

- $-G_2 = \dfrac{(3132.5' - 3133')}{(850' - 450')} \times 100\% = -0.125\%$

- $A = |G_2 - G_1| = 2.045\%$

- $S = SSD = 80 ft$ ← AASHTO 2001, Ex 3-76, pg 274

- $K = 3$

- $L = KA = 3(2.045\%) = \boxed{6.135'}$

- PVC Sta. $=$ PVI Sta. $- \frac{L}{2} = 450 - \frac{6.135'}{2} = 4+46.933$

- $E_{PVC} = E_{PVI} - \left(\frac{G_1}{100}\right)\frac{L}{2} = 3133' - \left(\frac{1.92\%}{100}\right)\frac{6.135'}{2} = 3132.94'$

- PVT Sta. $=$ PVI sta. $+ \frac{L}{2} = 450 + \frac{6.135'}{2} = 4+53.068$

- $E_{PVT} = E_{PVI} + \left(\frac{G_2}{100}\right)\frac{L}{2} = 3133' + \left(\frac{-0.125\%}{100}\right)\frac{6.135'}{2} = 3133'$

- $X_m = \left|\dfrac{G_1 * L}{G_2 - G_1}\right| = \left|\dfrac{1.92\% \cdot 6.135'}{-0.125\% - 1.92\%}\right| = 5.76'$

- Sta. of high pt. on VC $=$ PVC Sta. $+ X_m = 4+46.933 + 5.76'$
  $= 4+52.693$

- Elev. of high pt. on VC $= E_x = E_{PVC} + \left(\frac{G_1}{100}\right)X_m + \dfrac{(G_2 - G_1)X_m^2}{200L}$

  $= 3132.94' + \left(\frac{1.92\%}{100}\right)5.76' + \dfrac{(-0.125\% - 1.92\%)5.76'^2}{200(6.135')}$

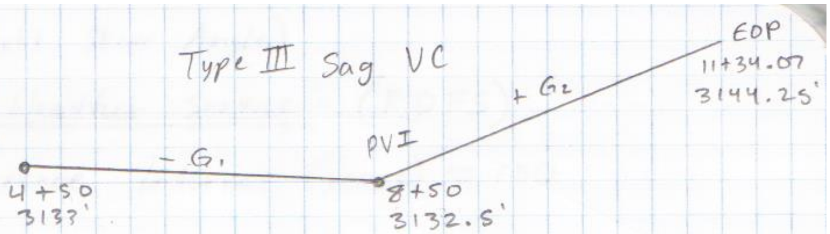
  $= 3133'$

Appendix R

* Vertical Curve # 4

Type III Sag VC

PVI:
→ Station: 8+50
→ Elevation: 3132.5'



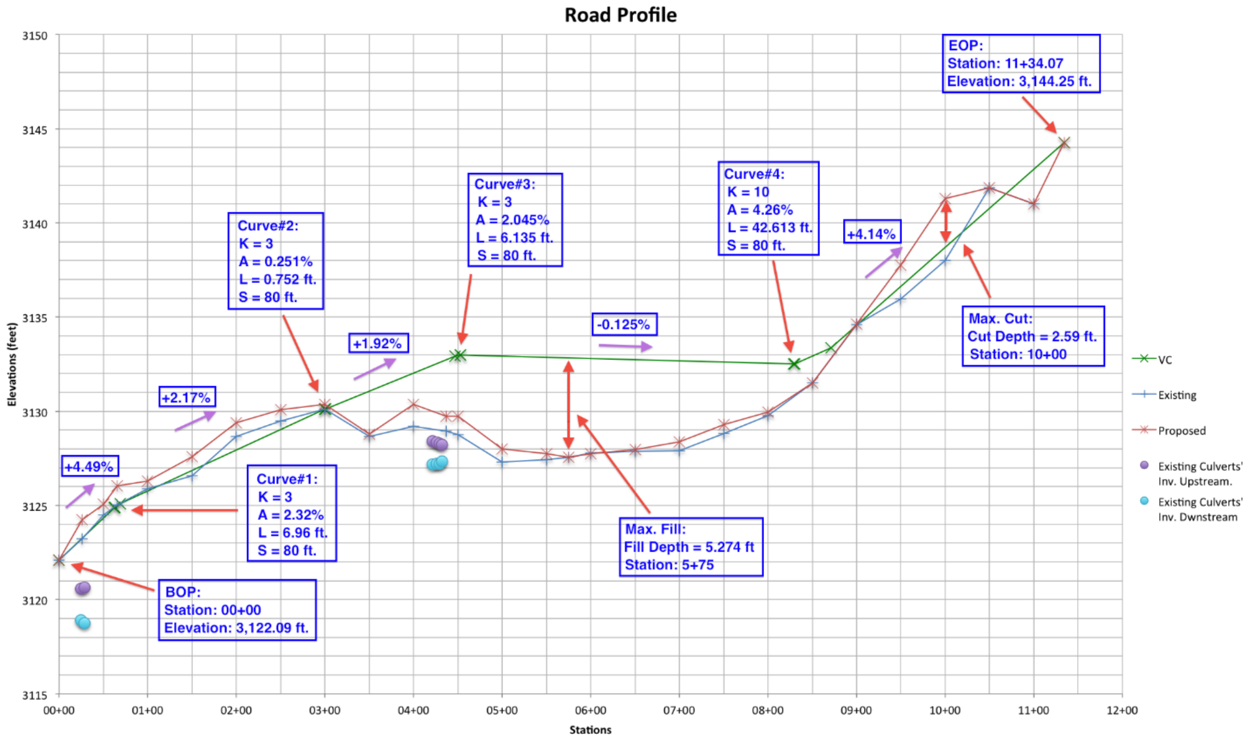
- $-G_1 = \left(\frac{3132.5' - 3133'}{850 - 450}\right) \times 100 = -0.125\%$
- $+G_2 = \left(\frac{3144.25' - 3132.5'}{1134.07' - 850'}\right) \times 100 = 4.1363\%$
- $A = |G_2 - G_1| = |4.1363\% - -0.125\%| = 4.2613\%$
- $K = 10$ ← AASHTO 2001, pg 280, Ex 3-79
- $L = KA = (10)(4.2613\%) = \boxed{42.613'}$
- PVC Sta. = PVI Sta. $- \frac{L}{2} = 850 - \frac{42.613'}{2} = 8+28.694$
- $E_{PVC} = E_{PVI} - \left(\frac{G_1}{100}\right)\frac{L}{2} = 3132.5' - \left(\frac{-0.125\%}{100}\right)\left(\frac{42.613'}{2}\right) = 3132.53'$
- PVT Sta. = PVI Sta. $+ \frac{L}{2} = 850 + \left(\frac{42.613'}{2}\right) = 8+71.307$
- $E_{PVT} = E_{PVI} + \left(\frac{G_2}{100}\right)\frac{L}{2} = 3132.5' + \left(\frac{4.1363\%}{100}\right)\left(\frac{42.613'}{2}\right) = 3133.38'$
- $X_m = \left|\frac{G_1 L}{G_2 - G_1}\right| = \left|\frac{-0.125\%(42.613')}{4.2613\% - (-0.125\%)}\right| = 1.21438'$
- Low pt. Sta. on VC = PVC Sta. $+ X_m = 828.694 + 1.21438'$
  $= 829.91$
  $= 8+29.91$
- Low pt. Elev. on VC ⇒ $E_x = E_{PVC} - \left(\frac{G_1}{100}\right)X_m$
  $= 3132.53' - \left(\frac{-0.125\%}{100}\right)1.21438'$
  $= 3132.53'$

Appendix T

Vertical Curves		Stations	Elevations (ft)	A (%)	K	L (ft.)=K*A	Critical Station	Critical Elevation (ft.
Curve #1 Crest	PVI1 PVC1 PVT1	00+65.50 00+62.02 00+68.98	3125.03 3124.87 3125.11	2.32	3	6.69	00+63.09	3124.92
Curve #2 Crest	PVI2 PVC2 PVT2	03+00.00 02+99.62 03+00.38	3130.12 3130.11 3130.13	0.251	3	0.752	03+00.00	3130.12
Curve #3 Crest	PVI3 PVC3 PVT3	04+50.00 04+46.93 04+53.07	3133 3132.94 3133	2.045	3	6.135	04+52.69	3133
Curve #4 Sag	PVI4 PVC4 PVT4	08+50.00 08+28.69 08+71.31	3132.5 3132.53 3133.38	4.2613	10	42.613	08+29.91	3132.53

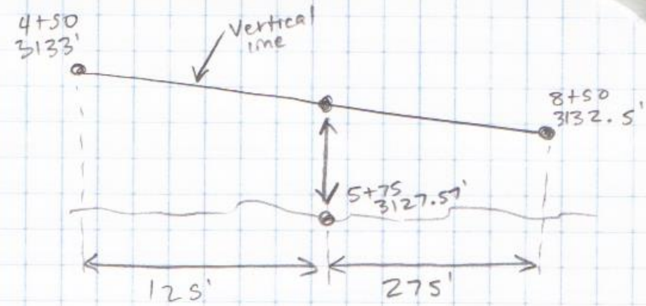
Grades (%)
G1 = 4.49
G2 = 2.17058
G3 = 1.92
G4 = -0.125
G5 = 4.1363

Appendix U


Road Profile
Elevations (feet)
Stations
3150
3145
3140
3135
3130
3125
3120
3115
00+00
01+00
02+00
03+00
04+00
05+00
06+00
07+00
08+00
09+00
10+00
11+00
12+00
EOP:
Station: 11+34.07
Elevation: 3,144.25 ft.
Curve#3:
K = 3
A = 2.045%
L = 6.135 ft.
S = 80 ft.
Curve#4:
K = 10
A = 4.26%
L = 42.613 ft.
S = 80 ft.
+4.14%
Curve#2:
K = 3
A = 0.251%
L = 0.752 ft.
S = 80 ft.
-0.125%
+1.92%
Max. Cut:
Cut Depth = 2.59 ft.
Station: 10+00
+2.17%
+4.49%
Curve#1:
K = 3
A = 2.32%
L = 6.96 ft.
S = 80 ft.
Max. Fill:
Fill Depth = 5.274 ft
Station: 5+75
BOP:
Station: 00+00
Elevation: 3,122.09 ft.
VC
Existing
Proposed
Existing Culverts' Inv. Upstream.
Existing Culverts' Inv. Dwnstream

Appendix V

* $\boxed{\text{Max. Fill}}$:



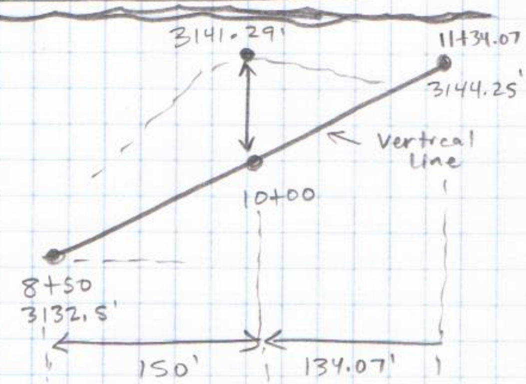
- [Existing Ground] ⇒ Station: 5+75.00
  Elevation: 3127.57'
- [Vertical line] ⇒ Station: 5+75
  Elevation: 3132.84'
- $y = mx + b$

$$= \left| \left(\frac{3132.5' - 3133'}{850' - 450'}\right)(275') - (3132.5') \right| = 3132.84'$$

- Max. Fill Depth = 3132.84' − 3127.57'

$$= \boxed{5.274 \text{ ft @ Station } 5+75}$$

* $\boxed{\text{Max. Cut}}$



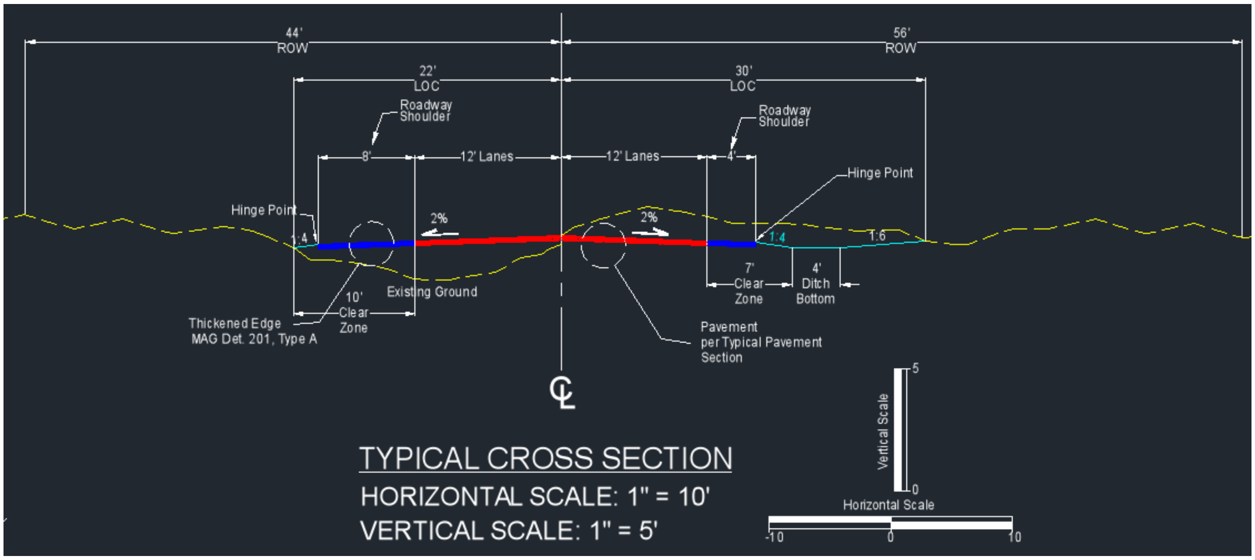
- [Proposed Ground] ⇒ Station: 10+00
  Elevation: 3141.29'
- [Vertical line] ⇒ Station: 10+00
  Elevation: 3138.7'
- $y = mx + b$

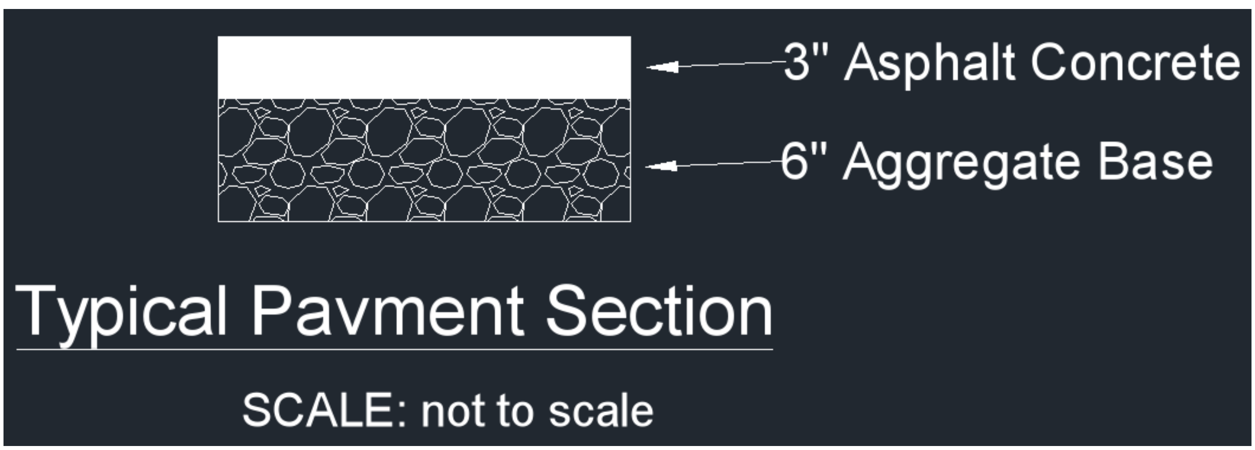
$$= \left| \left(\frac{3144.25' - 3132.5'}{1134.07' - 850'}\right)(150') + 3132.5' \right| = 3138.7'$$

- Max. Cut Depth = 3141.29' − 3138.7'

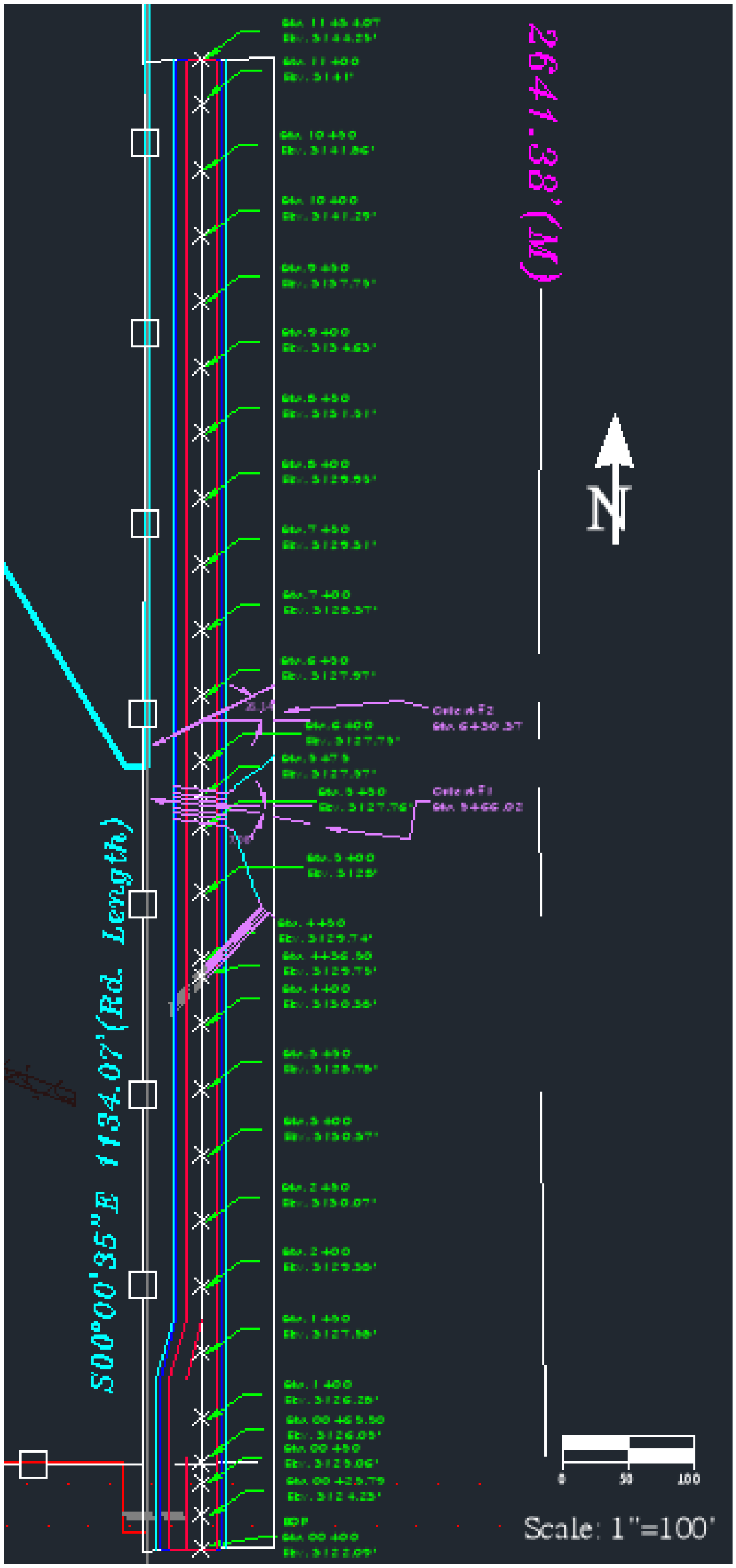
$$= \boxed{2.59 \text{ feet @ Station } 10+00}$$

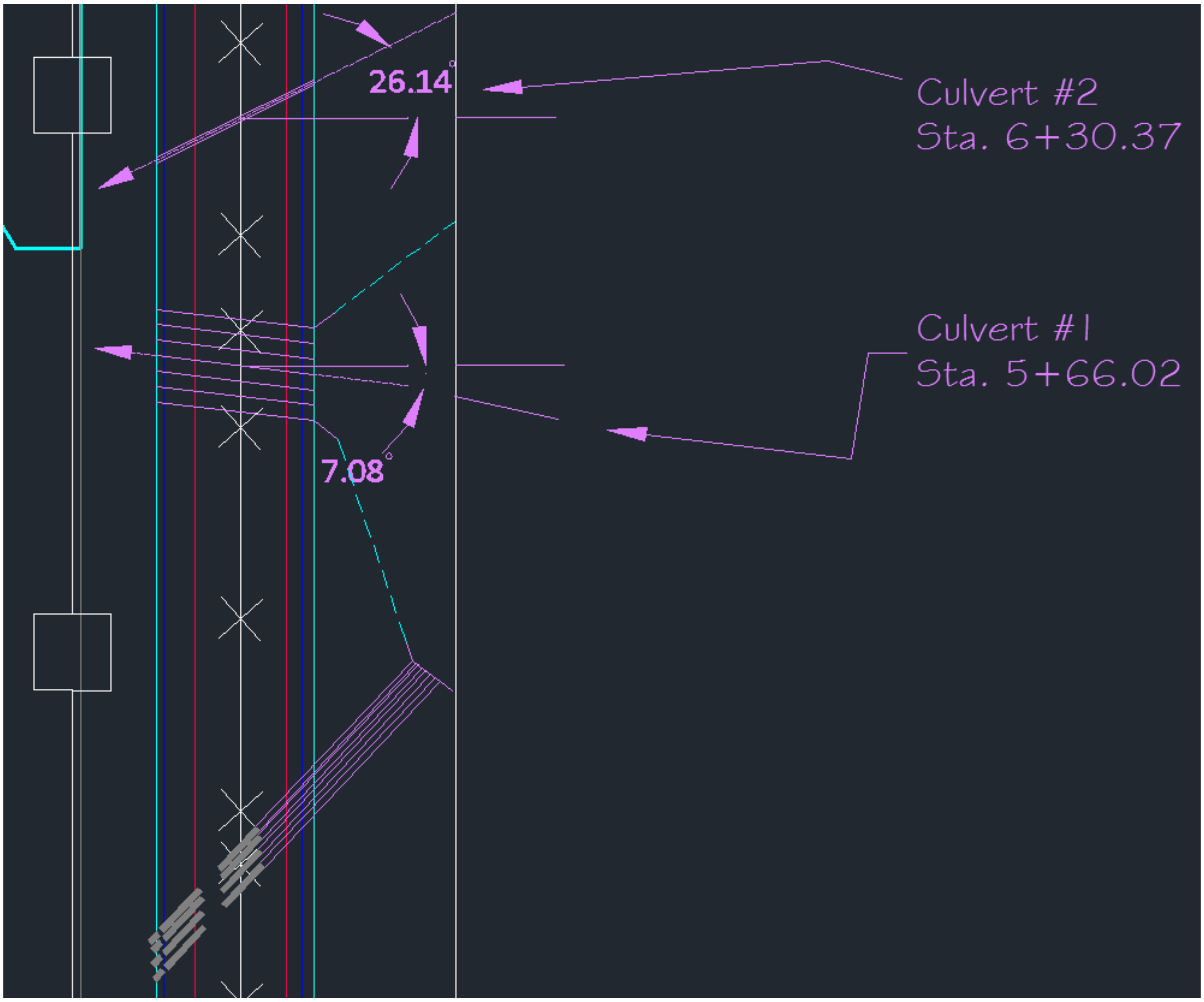
Appendix W


44'
ROW
56'
ROW
22'
LOC
30'
LOC
Roadway
Shoulder
Roadway
Shoulder
8'
12' Lanes
12' Lanes
4'
Hinge Point
Hinge Point
2%
2%
1:4
1:4
1:6
7'
Clear
Zone
4'
Ditch
Bottom
10'
Clear
Zone
Existing Ground
Thickened Edge
MAG Det. 201, Type A
Pavement
per Typical Pavement
Section
CL
TYPICAL CROSS SECTION
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 5'
Vertical Scale
5
0
Horizontal Scale
-10
0
10

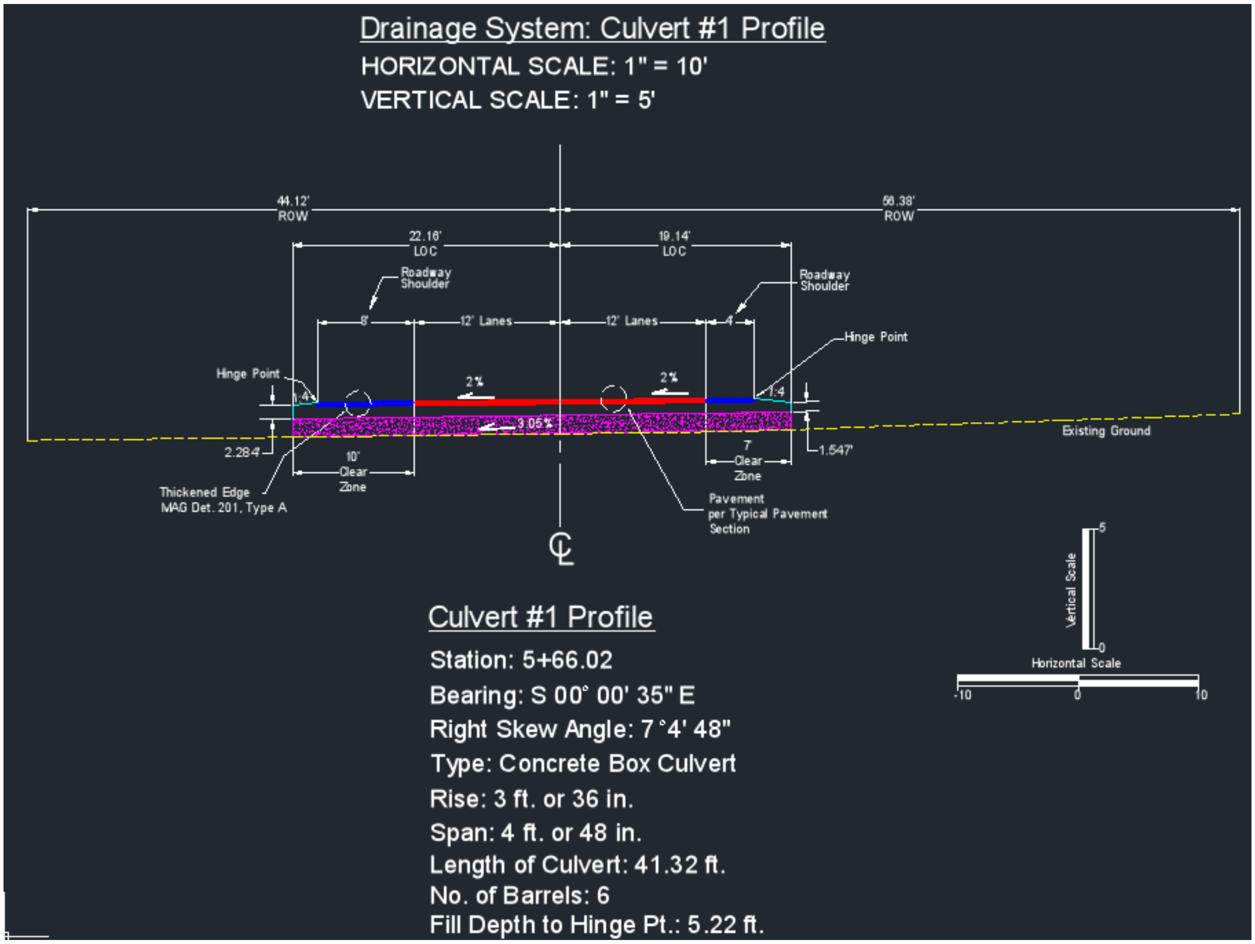

3" Asphalt Concrete
6" Aggregate Base
Typical Pavment Section
SCALE: not to scale

Appendix X

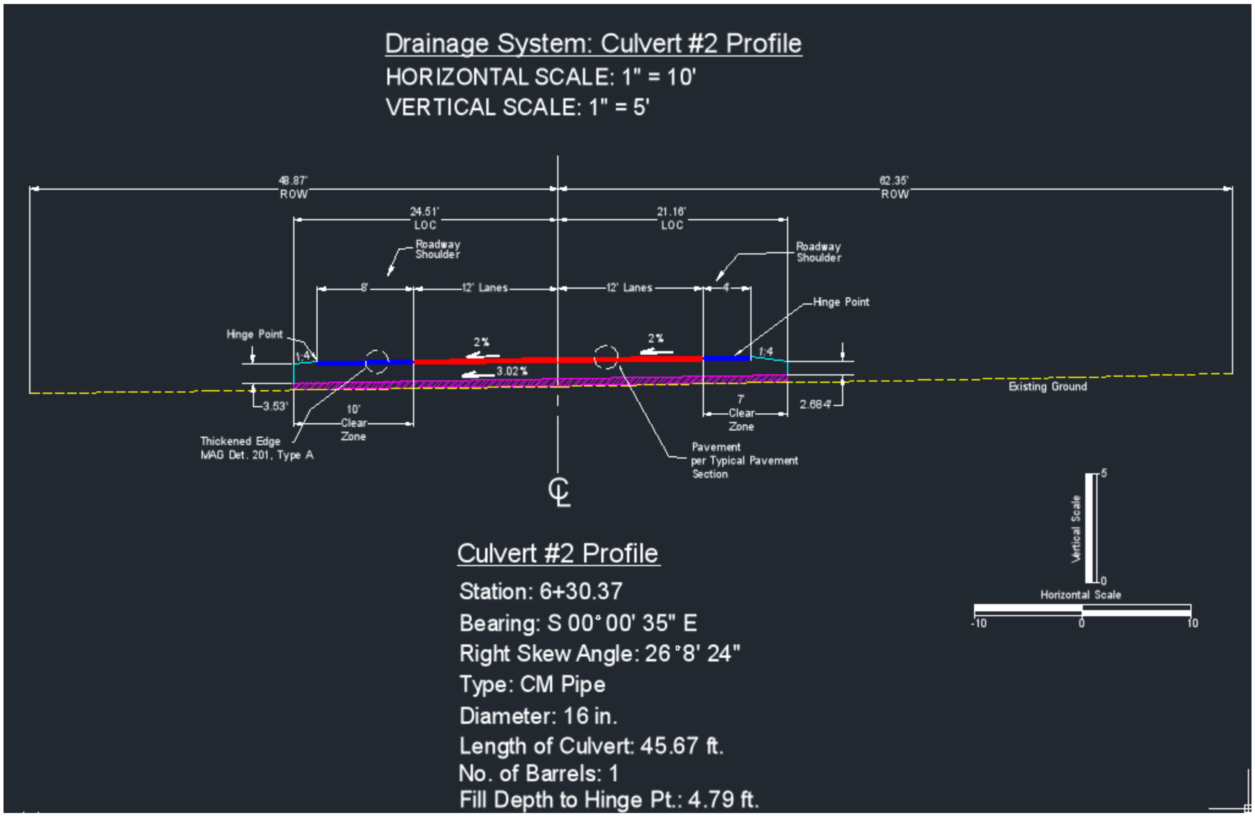



26.14°
Culvert #2
Sta. 6+30.37
Culvert #1
Sta. 5+66.02
7.08°

Appendix Y

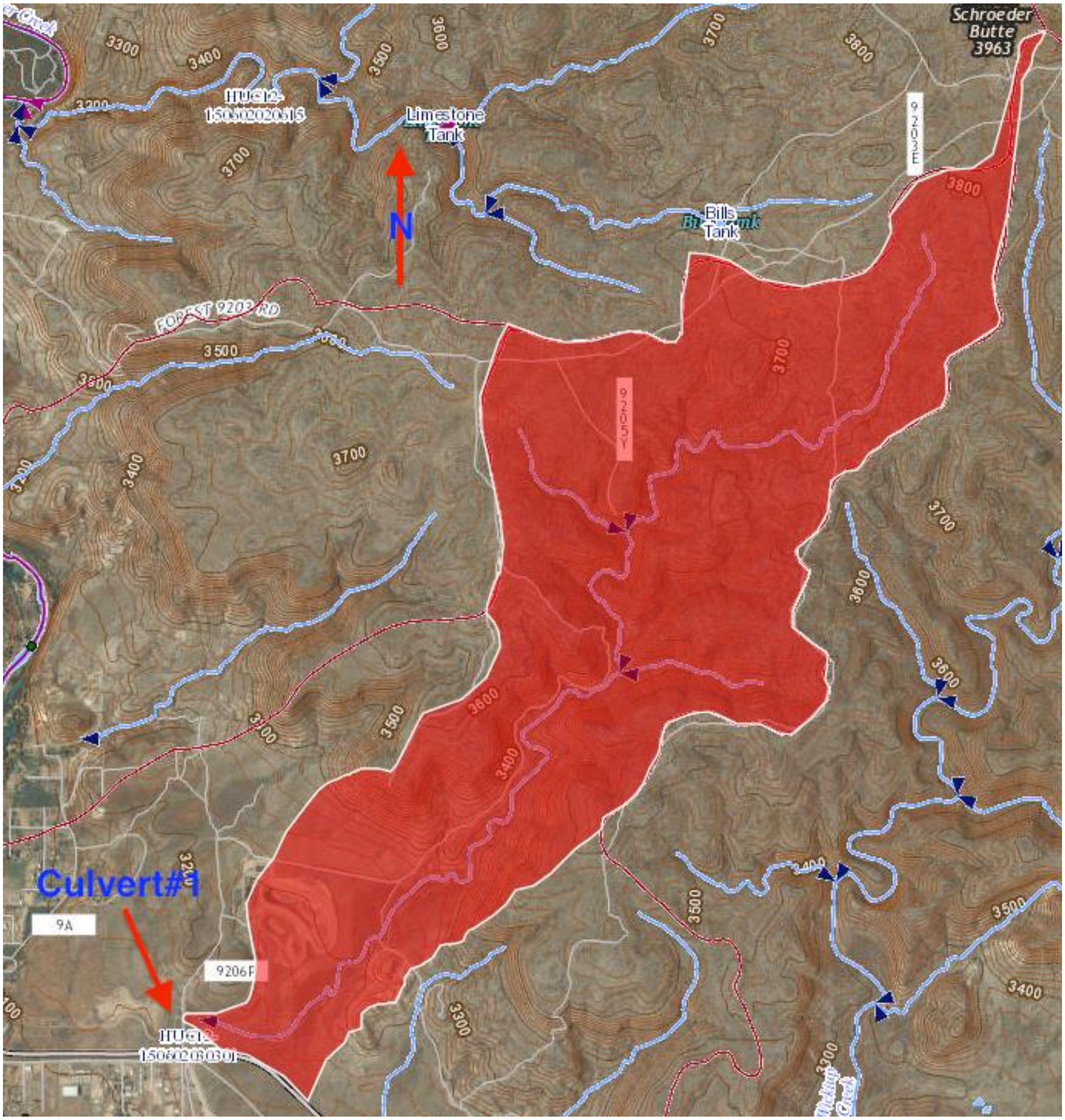

Drainage System: Culvert #1 Profile
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 5'
44.12' ROW
56.38' ROW
22.16' LOC
19.14' LOC
Roadway Shoulder
Roadway Shoulder
8'
12' Lanes
12' Lanes
4'
Hinge Point
Hinge Point
2%
2%
1:4
1:4
3.05%
2.284'
1.547'
10' Clear Zone
7' Clear Zone
Existing Ground
Thickened Edge MAG Det. 201, Type A
Pavement per Typical Pavement Section
Vertical Scale
5
0
Horizontal Scale
-10
0
10
Culvert #1 Profile
Station: 5+66.02
Bearing: S 00° 00' 35" E
Right Skew Angle: 7°4' 48"
Type: Concrete Box Culvert
Rise: 3 ft. or 36 in.
Span: 4 ft. or 48 in.
Length of Culvert: 41.32 ft.
No. of Barrels: 6
Fill Depth to Hinge Pt.: 5.22 ft.

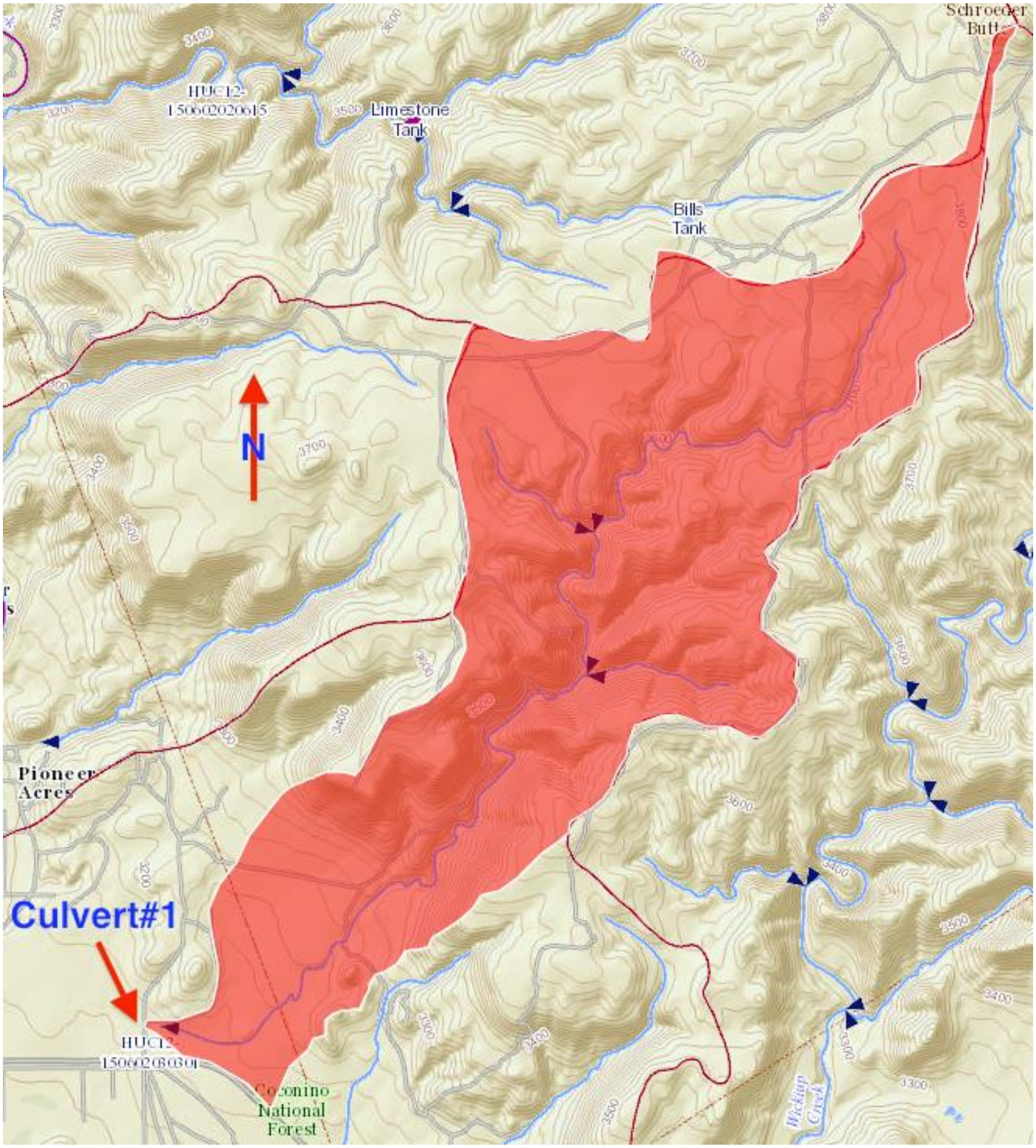
Appendix Z



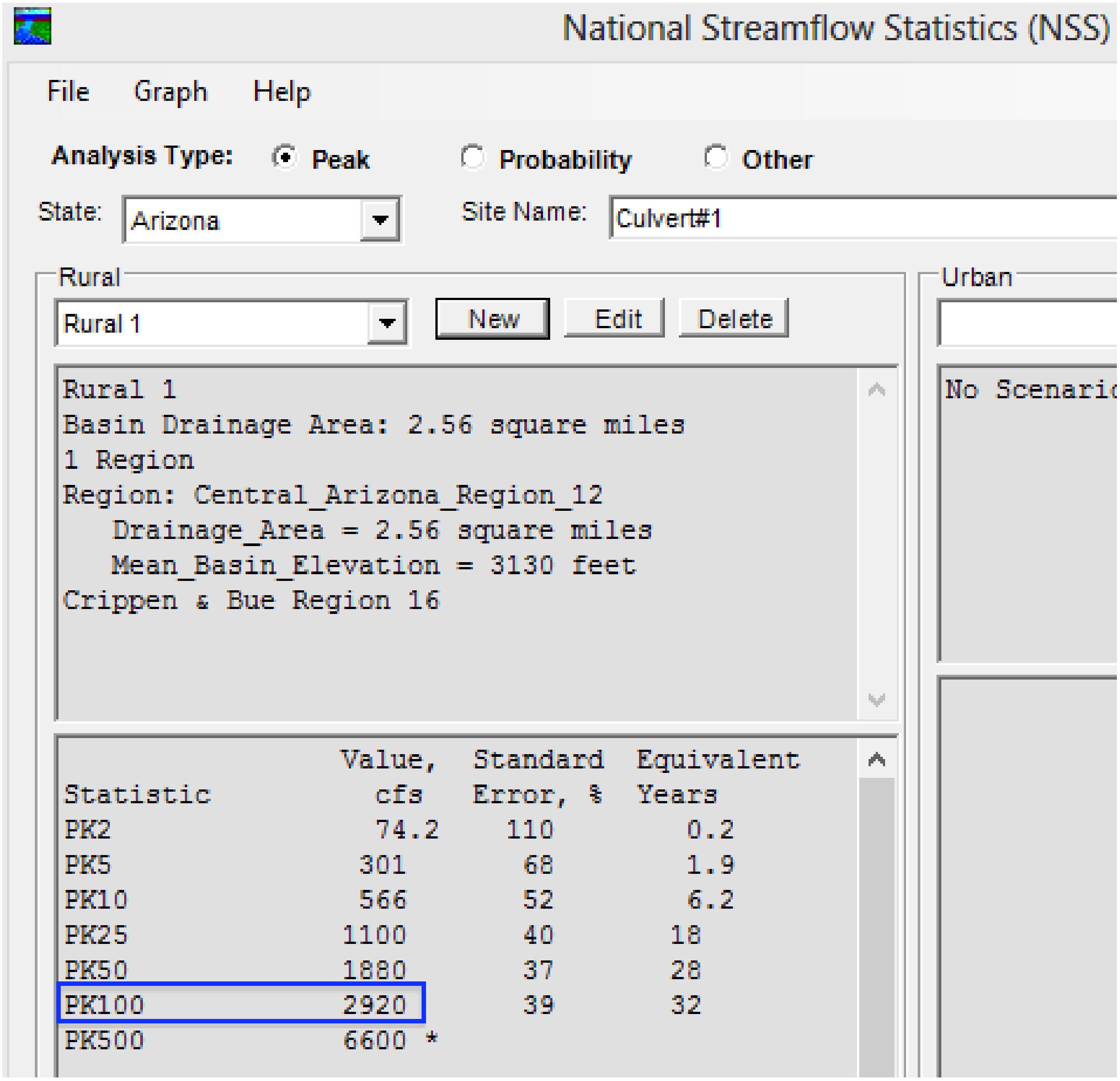
Drainage System: Culvert #2 Profile
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 5'
48.87' ROW
62.35' ROW
24.51' LOC
21.16' LOC
Roadway Shoulder
Roadway Shoulder
8'
12' Lanes
12' Lanes
4'
Hinge Point
Hinge Point
1:4
1:4
2%
2%
3.02%
3.53'
10' Clear Zone
7' Clear Zone
2.684'
Existing Ground
Thickened Edge MAG Det. 201, Type A
Pavement per Typical Pavement Section
Vertical Scale
Horizontal Scale
Culvert #2 Profile
Station: 6+30.37
Bearing: S 00° 00' 35" E
Right Skew Angle: 26°8' 24"
Type: CM Pipe
Diameter: 16 in.
Length of Culvert: 45.67 ft.
No. of Barrels: 1
Fill Depth to Hinge Pt.: 4.79 ft.

Appendix AA1




Schroeder Butte
HUC12-
150602020615
Limestone Tank
Bills Tank
N
Culvert#1
Pioneer Acres
HUC12-
150602030301
Coconino National Forest
Wickiup Creek

Appendix AA2

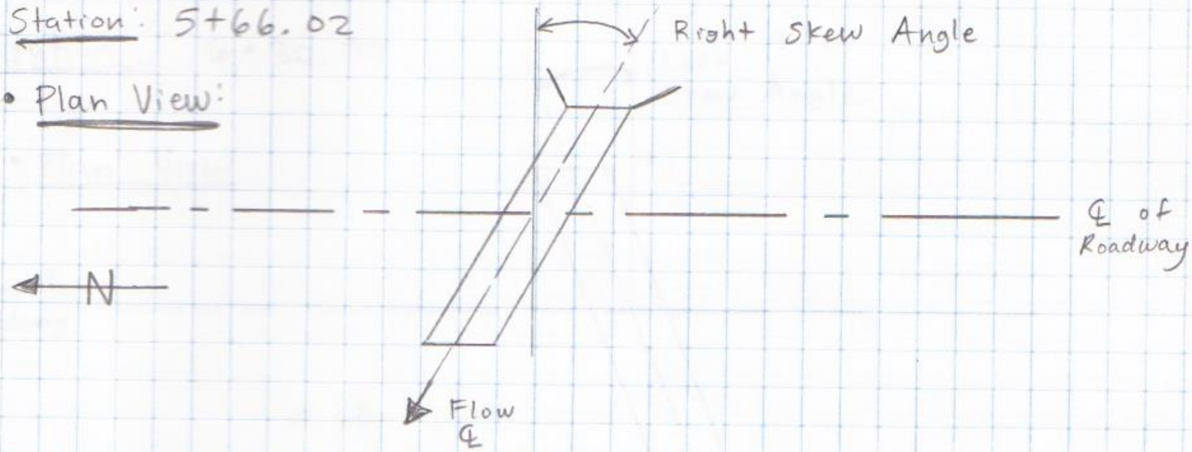


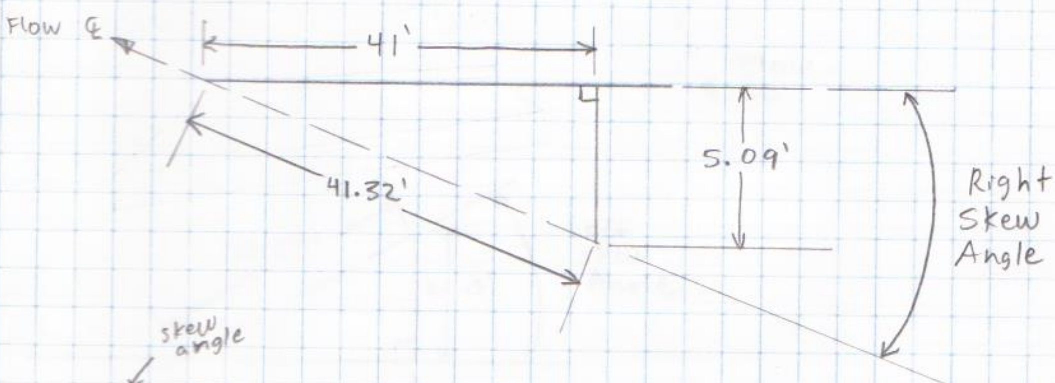
Appendix AA3

★ **Culvert #1:** Skew Angle Calculations

Station: 5+66.02

- Plan View:





skew angle

- $\sin\theta$ = opposite / hypotenuse

$$\sin\theta = \left(\frac{5.09'}{41.32'}\right)$$

$$\theta = \sin^{-1}\left(\frac{5.09'}{41.32'}\right)$$

$$= \boxed{7.08^\circ = \text{Right Skew Angle}}$$

or

$$\boxed{7^\circ\, 4'\, 48''}$$

Appendix AA4

Culvert Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 27 2013

Culvert #1

Invert Elev Dn (ft)	= 3127.05
Pipe Length (ft)	= 41.32
Slope (%)	= 3.05
Invert Elev Up (ft)	= 3128.31
Rise (in)	= 36.0
Shape	= Box
Span (in)	= 48.0
No. Barrels	= 3
n-Value	= 0.012
Culvert Type	= Flared Wingwalls
Culvert Entrance	= 30D to 75D wingwall flares
Coeff. K,M,c,Y,k	= 0.026, 1, 0.0347, 0.81, 0.4

Embankment

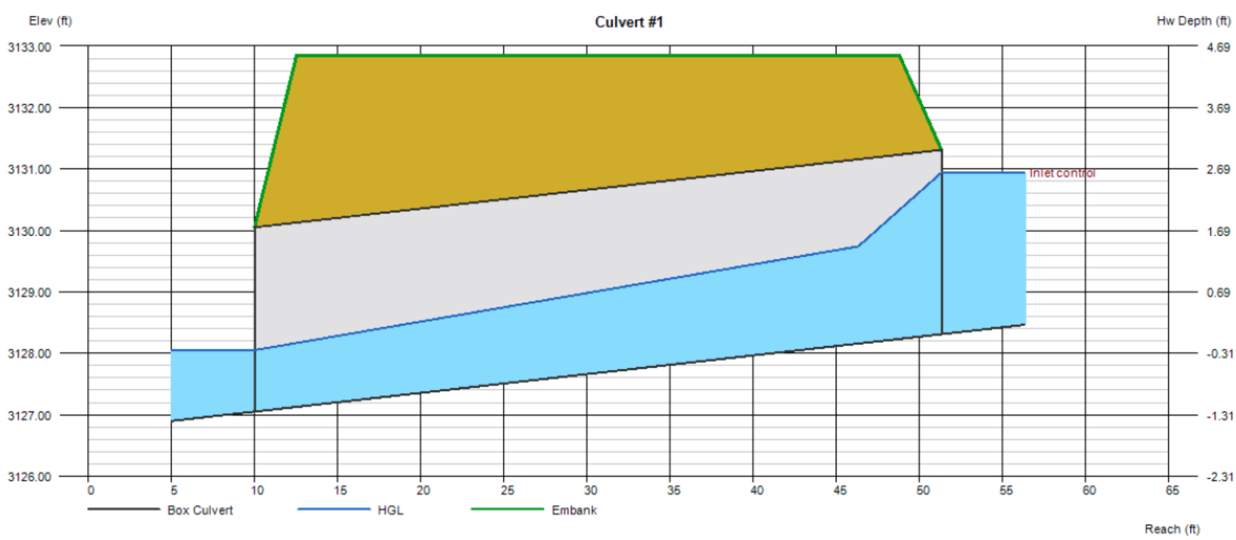
Top Elevation (ft)	= 3132.85
Top Width (ft)	= 36.26
Crest Width (ft)	= 41.32

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 1460.00
Tailwater Elev (ft)	= 0.00

Highlighted

Qtotal (cfs)	= 146.00
Qpipe (cfs)	= 146.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 12.17
Veloc Up (ft/s)	= 7.32
HGL Dn (ft)	= 3128.05
HGL Up (ft)	= 3129.97
Hw Elev (ft)	= 3130.94
Hw/D (ft)	= 0.88
Flow Regime	= Inlet Control



Appendix AA5

Culvert Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 27 2013

Culvert #1

Invert Elev Dn (ft)	=	3127.05
Pipe Length (ft)	=	41.32
Slope (%)	=	3.05
Invert Elev Up (ft)	=	3128.31
Rise (in)	=	36.0
Shape	=	Box
Span (in)	=	48.0
No. Barrels	=	3
n-Value	=	0.012
Culvert Type	=	Flared Wingwalls
Culvert Entrance	=	30D to 75D wingwall flares
Coeff. K,M,c,Y,k	=	0.026, 1, 0.0347, 0.81, 0.4

Embankment

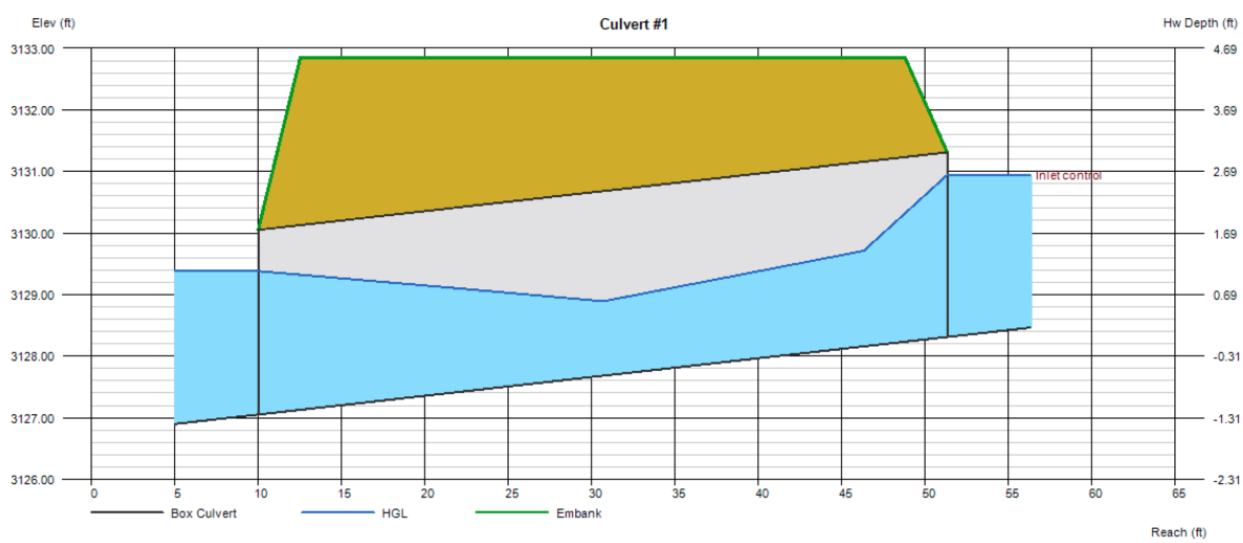
Top Elevation (ft)	=	3132.85
Top Width (ft)	=	36.26
Crest Width (ft)	=	41.32

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	1460.00
Tailwater Elev (ft)	=	(dc+D)/2

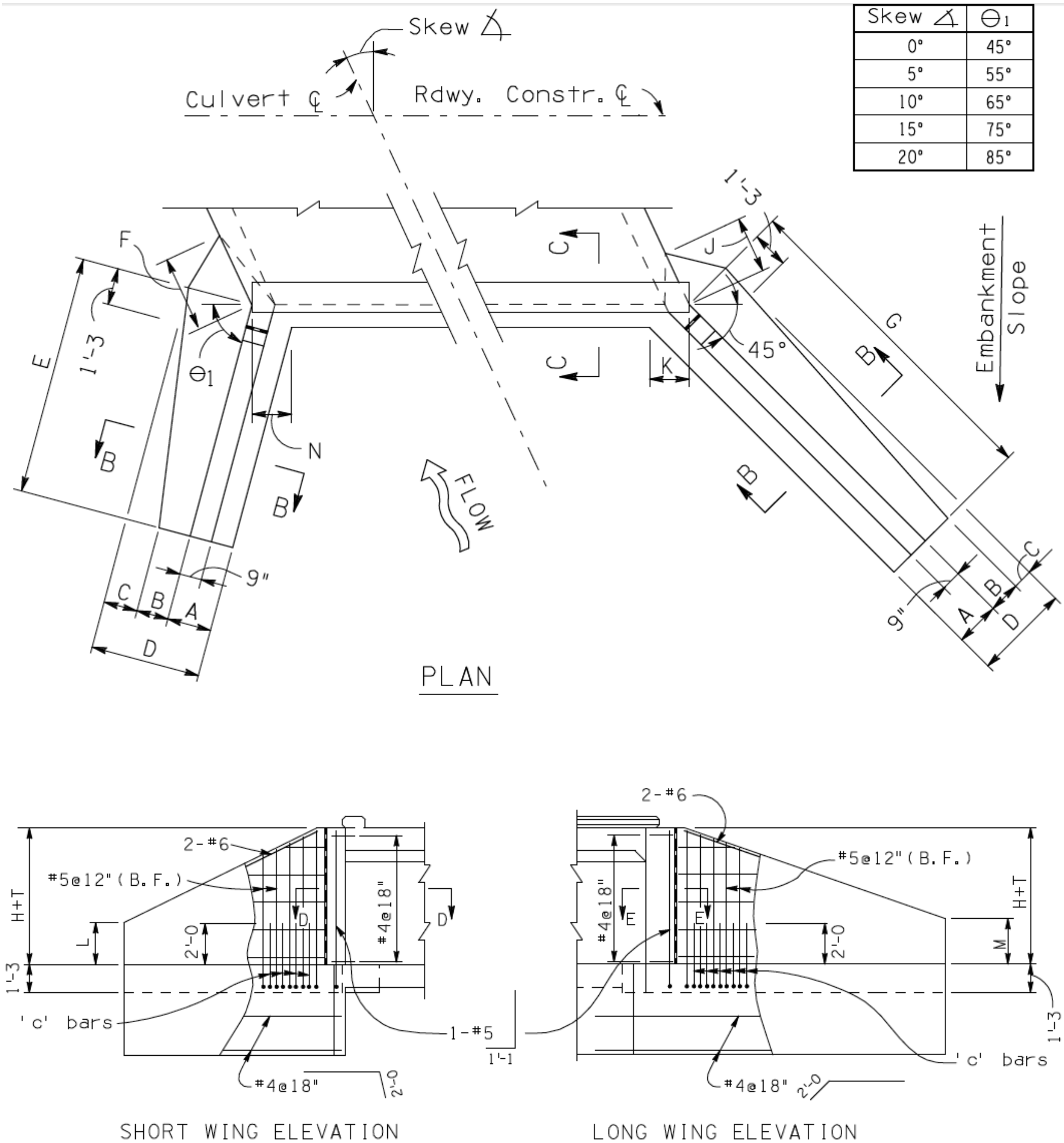
Highlighted

Qtotal (cfs)	=	146.00
Qpipe (cfs)	=	146.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.22
Veloc Up (ft/s)	=	7.32
HGL Dn (ft)	=	3129.38
HGL Up (ft)	=	3129.97
Hw Elev (ft)	=	3130.94
Hw/D (ft)	=	0.88
Flow Regime	=	Inlet Control



Appendix AA6

Skew ∡	Θ1
0°	45°
5°	55°
10°	65°
15°	75°
20°	85°



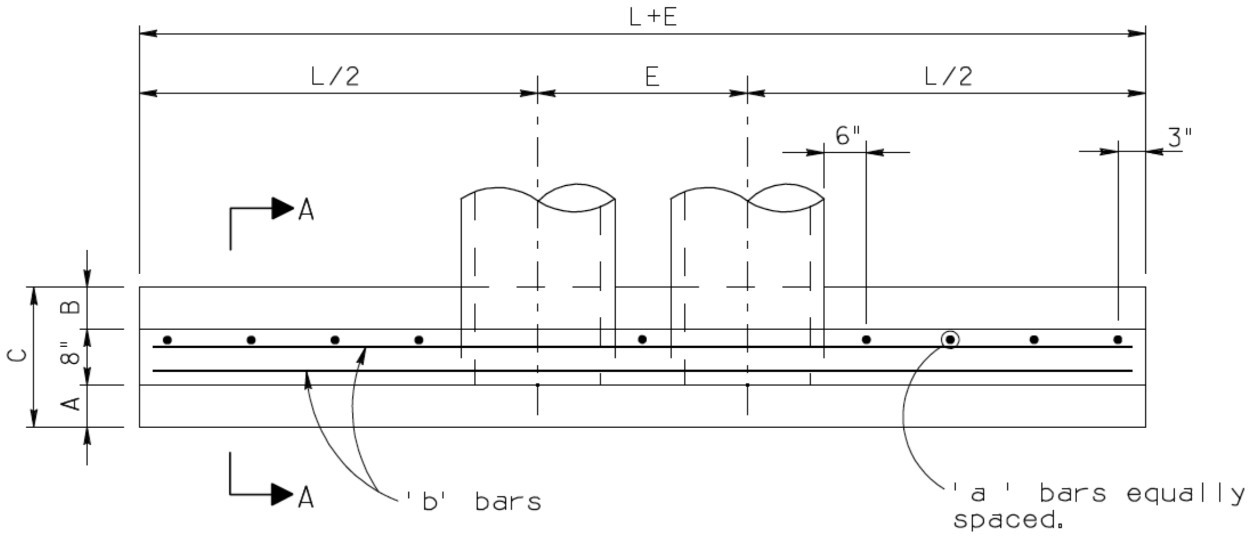
PLAN

SHORT WING ELEVATION

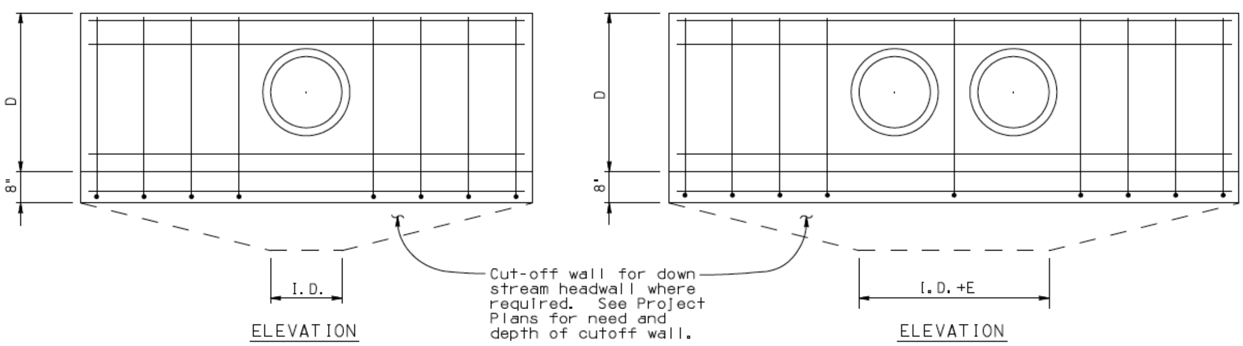
LONG WING ELEVATION

4 : 1 Slope	Skew ∡	0°			5°			10°			15°			20°		
	Height H	5'	6'	7'	5'	6'	7'	5'	6'	7'	5'	6'	7'	5'	6'	7'
Dimensions	A	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6	1'-6
	B	1'-1	1'-4	1'-8	1'-1	1'-3	1'-7	1'-1	1'-2	1'-6	1'-1	1'-2	1'-5	1'-1	1'-2	1'-3
	C	1'-0	1'-3	1'-5	1'-0	1'-4	1'-6	1'-0	1'-5	1'-7	1'-0	1'-5	1'-8	1'-0	1'-5	1'-10
	D	3'-7	4'-1	4'-7	3'-7	4'-1	4'-7	3'-7	4'-1	4'-7	3'-7	4'-1	4'-7	3'-7	4'-1	4'-7
	E	11'-6	13'-6	15'-6	11'-6	13'-6	15'-6	12'-0	14'-0	16'-0	12'-6	14'-6	16'-6	13'-6	15'-6	17'-6
	F	2'-8	3'-4	4'-1	2'-7	3'-4	4'-0	2'-7	3'-3	4'-0	2'-6	3'-3	3'-11	2'-5	3'-1	3'-9
	G	11'-6	13'-6	15'-6	12'-6	14'-6	16'-6	13'-0	15'-6	17'-6	14'-6	16'-6	19'-0	15'-6	18'-6	21'-0
	J	2'-8	3'-4	4'-1	2'-7	3'-4	4'-0	2'-7	3'-3	4'-0	2'-6	3'-3	3'-11	2'-5	3'-1	3'-9
	K	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½	1'-4½
	L	4'-0	4'-8	5'-4	3'-8	4'-4	4'-11	3'-4	3'-11	4'-5	3'-1	3'-7	4'-1	2'-8	3'-2	3'-8
	M	4'-0	4'-8	5'-4	3'-10	4'-6	5'-2	3'-9	4'-4	5'-0	3'-6	4'-2	4'-8	3'-4	3'-10	4'-4
	N	1'-4½	1'-4½	1'-4½	1'-3⅝	1'-3⅝	1'-3⅝	1'-3⅝	1'-3⅝	1'-3⅝	1'-4¼	1'-4¼	1'-4¼	1'-5¼	1'-5¼	1'-5¼

Appendix AA7



PLAN

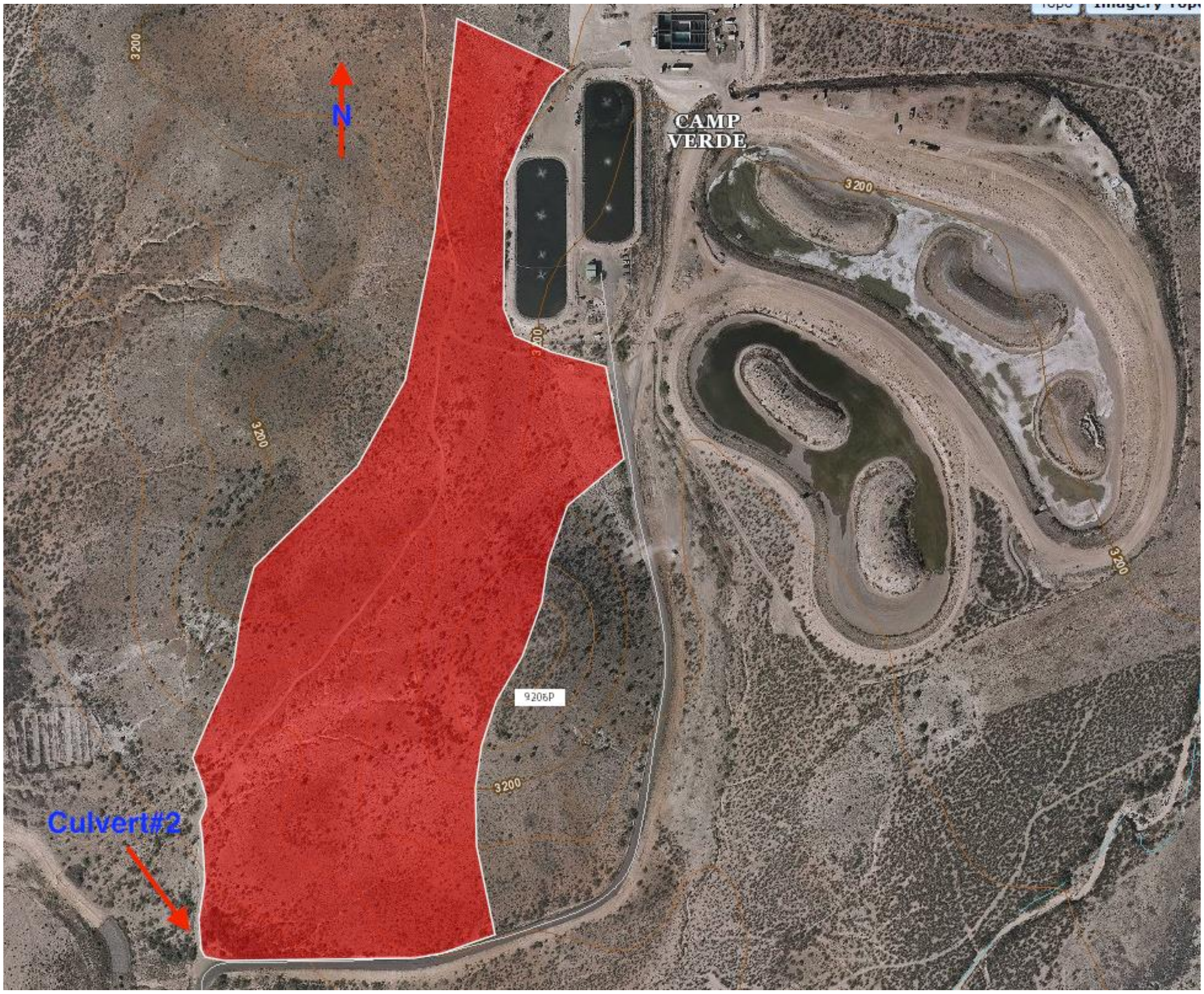


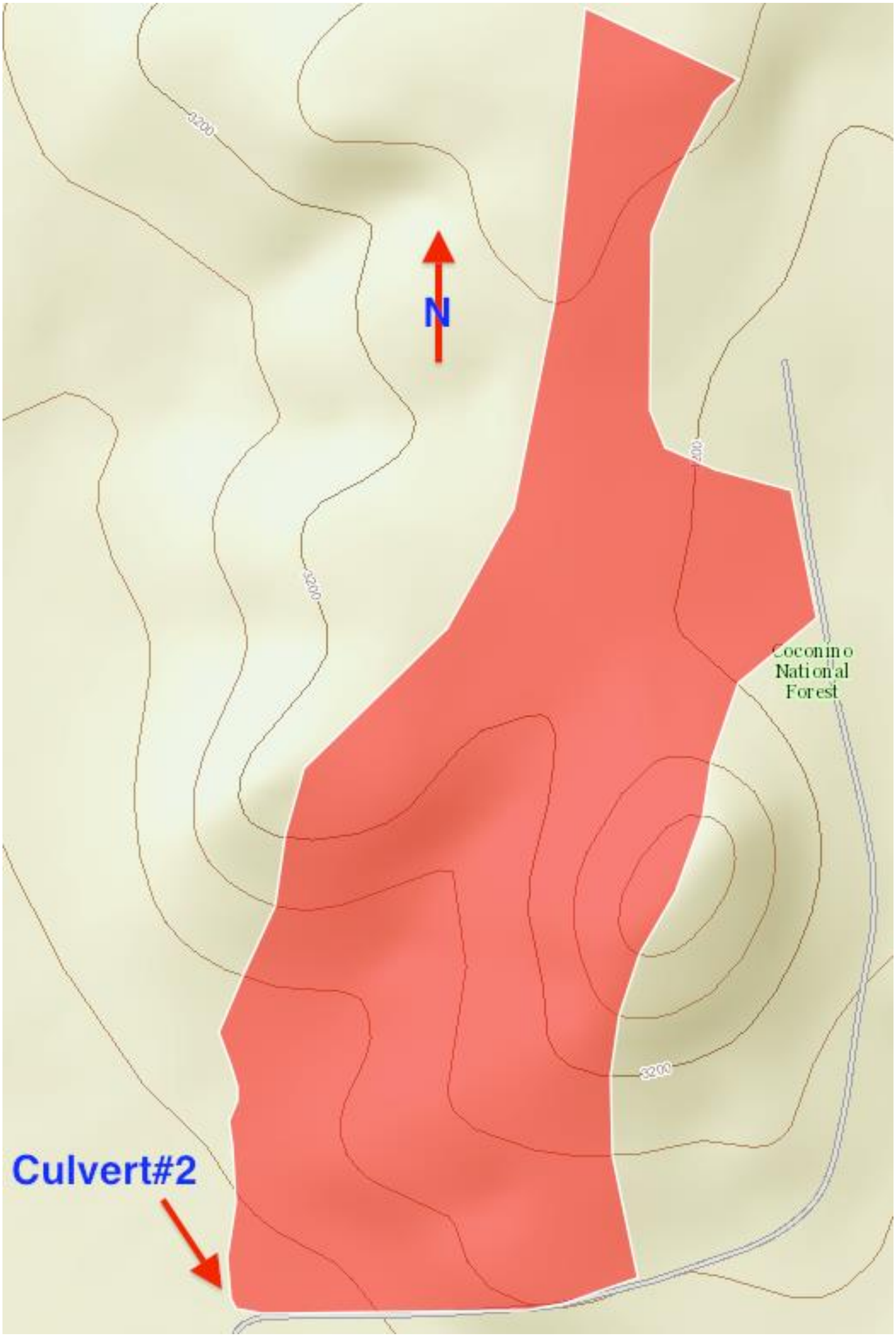
ELEVATION
SINGLE PIPE HEADWALL

ELEVATION
DOUBLE PIPE HEADWALL

Dimensions									
I.D.	A	B	C	D	E	F	G	L	L+E
18"	6"	10"	2'-0	3'-2	2'-6	1'-7	4'-6	9'-6	12'-0
24"	8"	1'-0	2'-4	3'-8	3'-0	2'-1	5'-6	11'-6	14'-6
30"	10"	1'-2	2'-8	4'-2	3'-9	2'-7	6'-6	13'-6	17'-3
36"	1'-0	1'-4	3'-0	4'-8	4'-6	3'-1	7'-6	15'-6	20'-0
42"	1'-2	1'-6	3'-4	5'-2	5'-3	3'-7	8'-6	17'-6	22'-9

Appendix BB1




N
3200
3200
3200
Coconino
National
Forest
Culvert#2

Appendix BB2

★ Culvert #2:

• NOAA: PDS – based Precipitation Frequency Esitmates with 90% confidence intervals (in inches/hour)

↳ Average Recurrence interval (years) = 100

↳ Duration = 24 hours

↳ i (rainfall intensity) = 3.94 inches / 24 hours = 0.164 in/hr } NOAA'S Data

↳ i (rainfall intensity) = 4 inches / 24 hours = $\boxed{0.167 \text{ in/hr}}$ } Given by client

• USGS (National Map Viewer): Watershed Delineation

Drainage Area (A) = 0.047 mi^2 = $\boxed{30.08 \text{ acres}}$

• Rational Method: $A \leq 160$ acres

$Q = CiA$

Q – Peak discharge (cfs)
C – Rational Method runoff coefficient
i – rainfall intensity (in./hr.)
A – drainage area (acre)

C – 0.20 ← (Table 11-2, Highway Engineering, PH Wright, 1996)

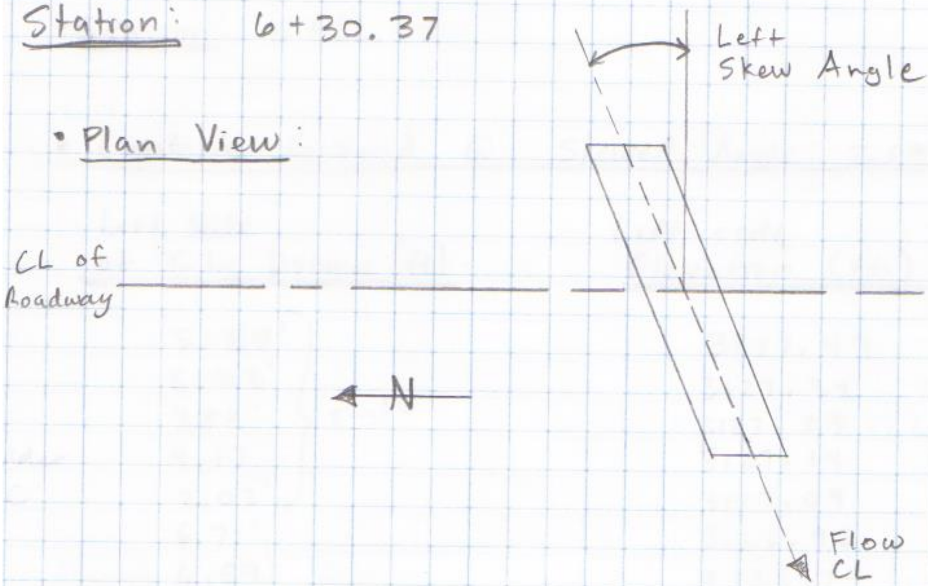
$Q_{100} = (0.20)(0.167 \text{ in/hr})(30.08 \text{ acres})$
$= \boxed{1.005 \text{ cfs}}$

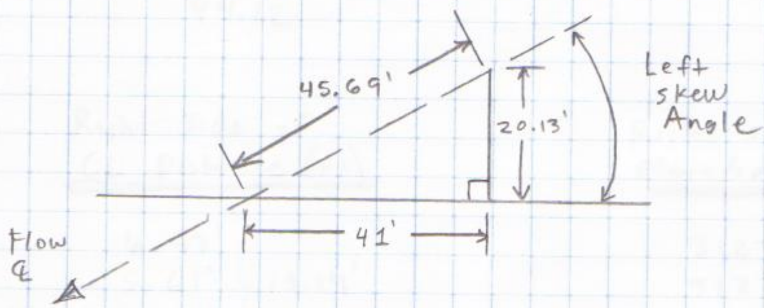
Appendix BB3

* Culvert # 2 : Skew Angle Calculations

Station: 6+30.37

• Plan View:





skew angle

• $\sin\theta$ = (opposite / hypotenuse)

$$\theta = \sin^{-1}\left(\frac{20.13'}{45.69'}\right)$$

$$= \boxed{26.14^\circ = \text{Left Skew Angle}}$$

or

$$\boxed{26^\circ\ 8'\ 48''}$$

Appendix BB4

Culvert Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 27 2013

Culvert #2

Invert Elev Dn (ft)	= 3127.35
Pipe Length (ft)	= 45.67
Slope (%)	= 3.02
Invert Elev Up (ft)	= 3128.73
Rise (in)	= 16.0
Shape	= Circular
Span (in)	= 16.0
No. Barrels	= 1
n-Value	= 0.024
Culvert Type	= Circular Corrugate Metal Pipe
Culvert Entrance	= Mitered to slope (C)
Coeff. K,M,c,Y,k	= 0.021, 1.33, 0.0463, 0.75, 0.7

Embankment

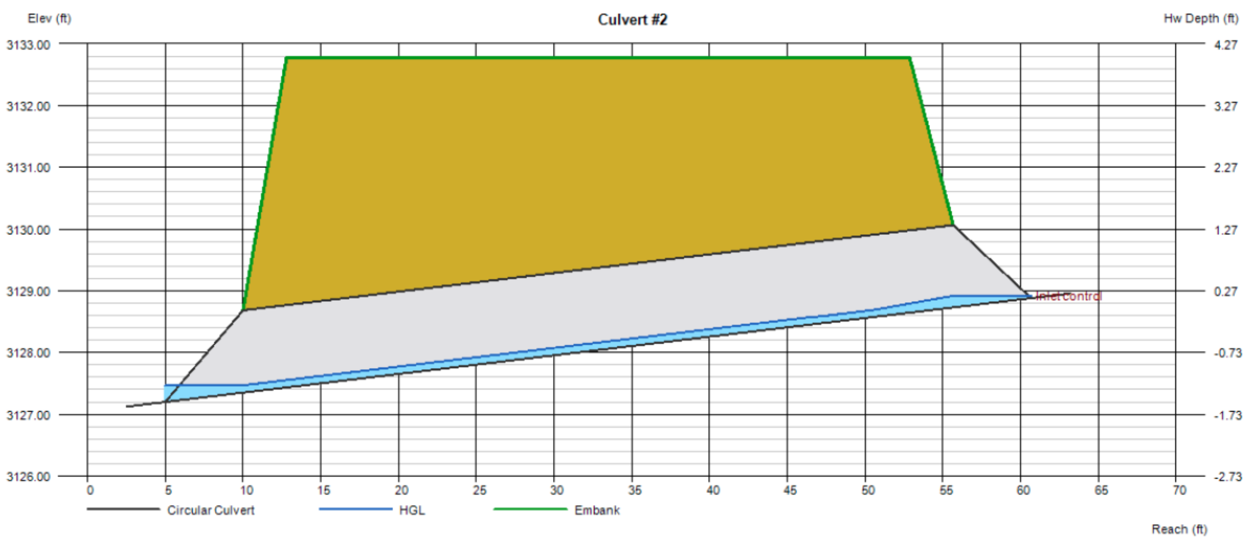
Top Elevation (ft)	= 3132.77
Top Width (ft)	= 40.08
Crest Width (ft)	= 45.67

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 1.01
Tailwater Elev (ft)	= 0.00

Highlighted

Qtotal (cfs)	= 0.10
Qpipe (cfs)	= 0.10
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 1.62
Veloc Up (ft/s)	= 1.62
HGL Dn (ft)	= 3127.47
HGL Up (ft)	= 3128.85
Hw Elev (ft)	= 3128.92
Hw/D (ft)	= 0.14
Flow Regime	= Inlet Control



Culvert Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 27 2013

Culvert #2

Invert Elev Dn (ft)	=	3127.35
Pipe Length (ft)	=	45.67
Slope (%)	=	3.02
Invert Elev Up (ft)	=	3128.73
Rise (in)	=	16.0
Shape	=	Circular
Span (in)	=	16.0
No. Barrels	=	1
n-Value	=	0.024
Culvert Type	=	Circular Corrugate Metal Pipe
Culvert Entrance	=	Mitered to slope (C)
Coeff. K,M,c,Y,k	=	0.021, 1.33, 0.0463, 0.75, 0.7

Embankment

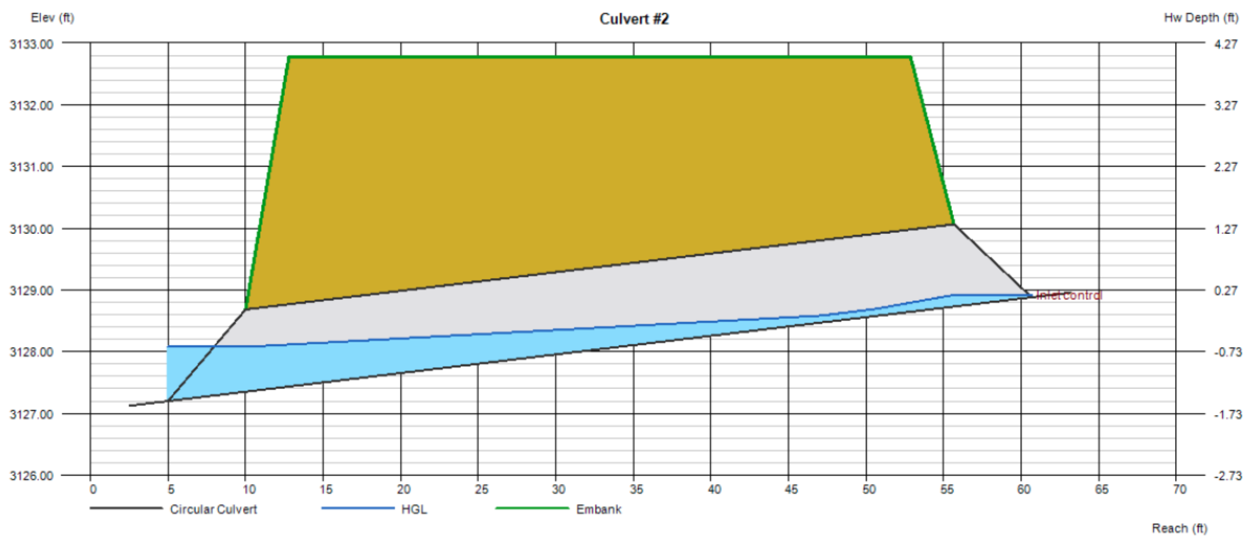
Top Elevation (ft)	=	3132.77
Top Width (ft)	=	40.08
Crest Width (ft)	=	45.67

Calculations

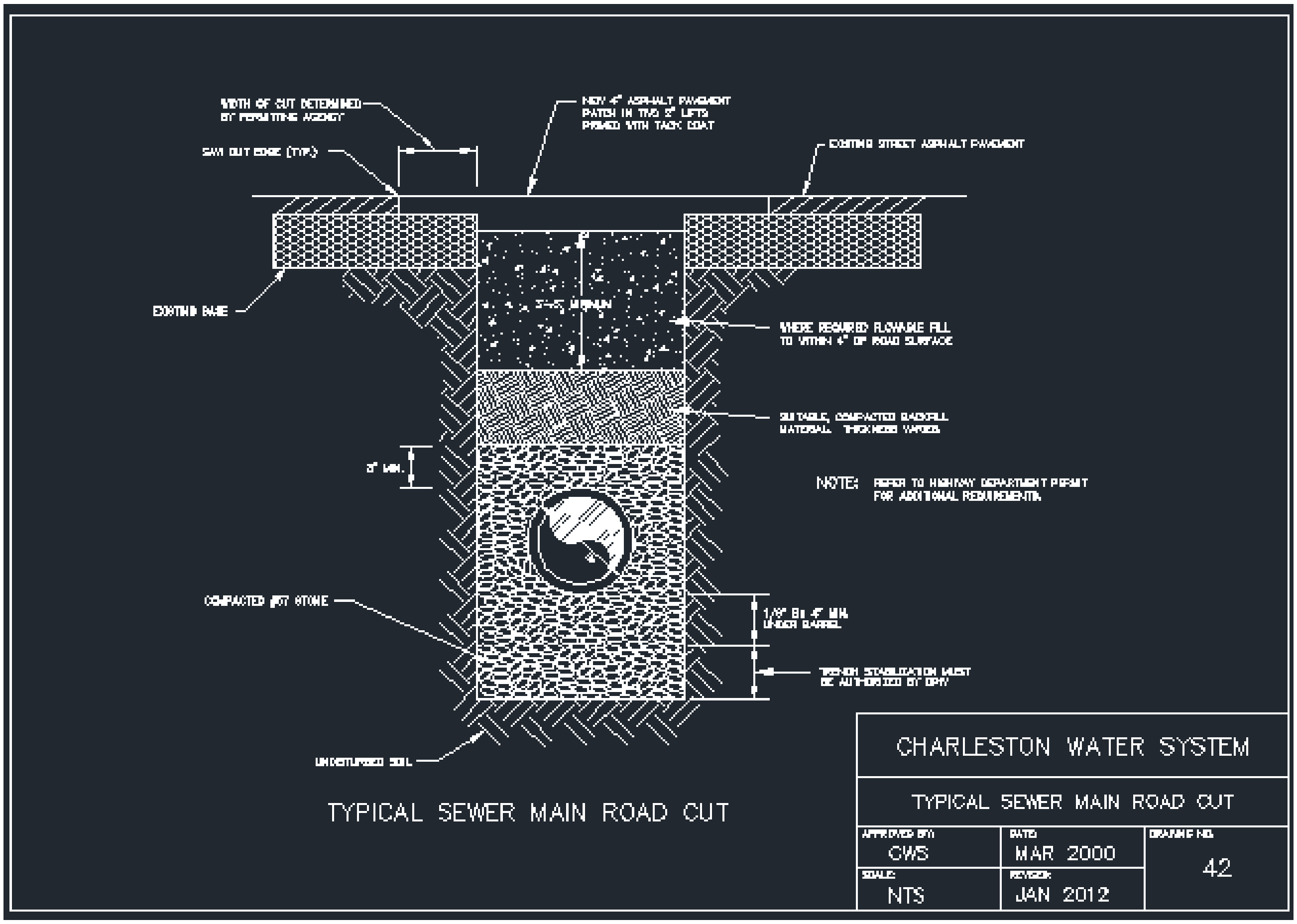
Qmin (cfs)	=	0.00
Qmax (cfs)	=	1.01
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	0.10
Qpipe (cfs)	=	0.10
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	0.13
Veloc Up (ft/s)	=	1.62
HGL Dn (ft)	=	3128.08
HGL Up (ft)	=	3128.85
Hw Elev (ft)	=	3128.92
Hw/D (ft)	=	0.14
Flow Regime	=	Inlet Control

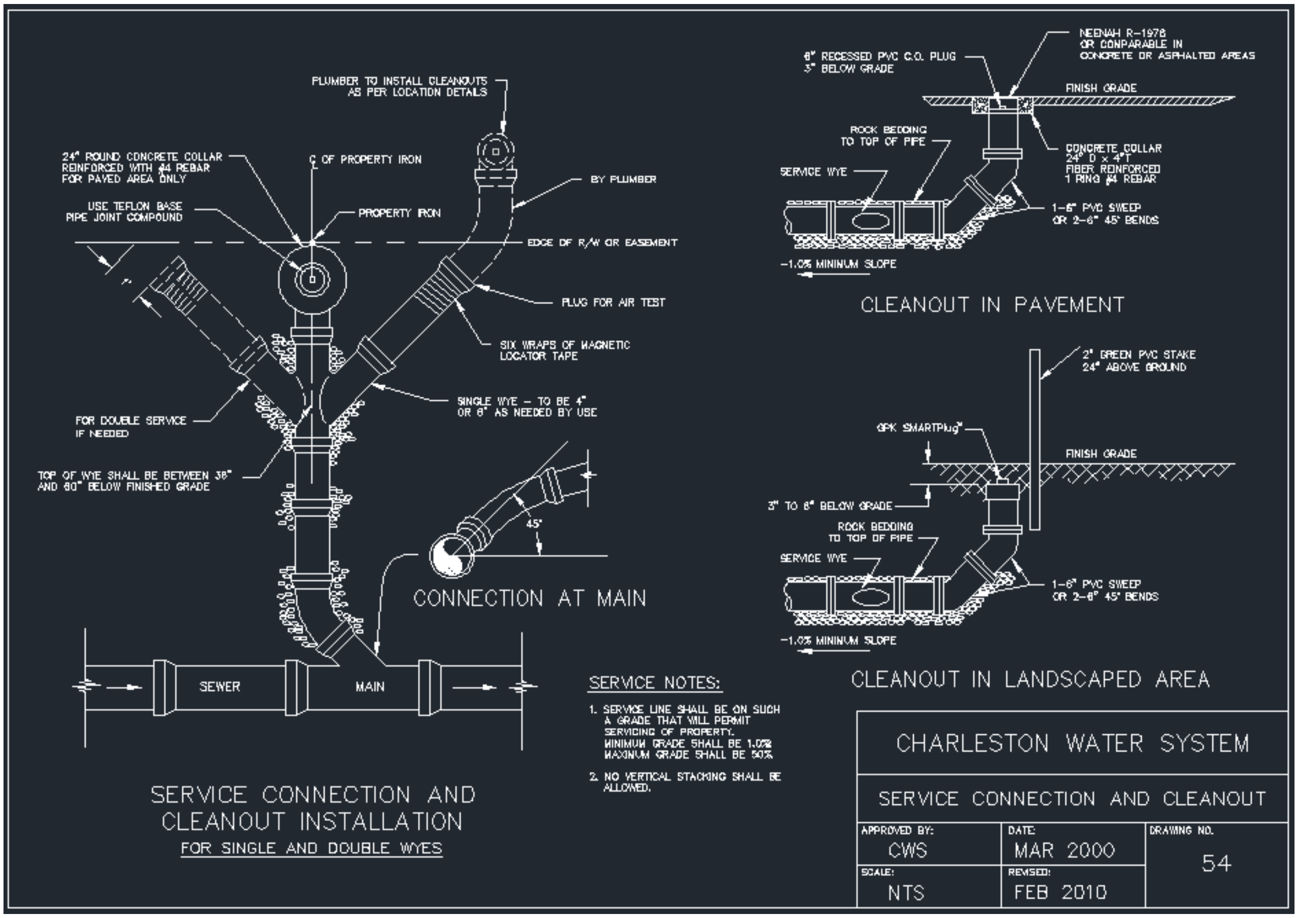


Appendix CC-1:



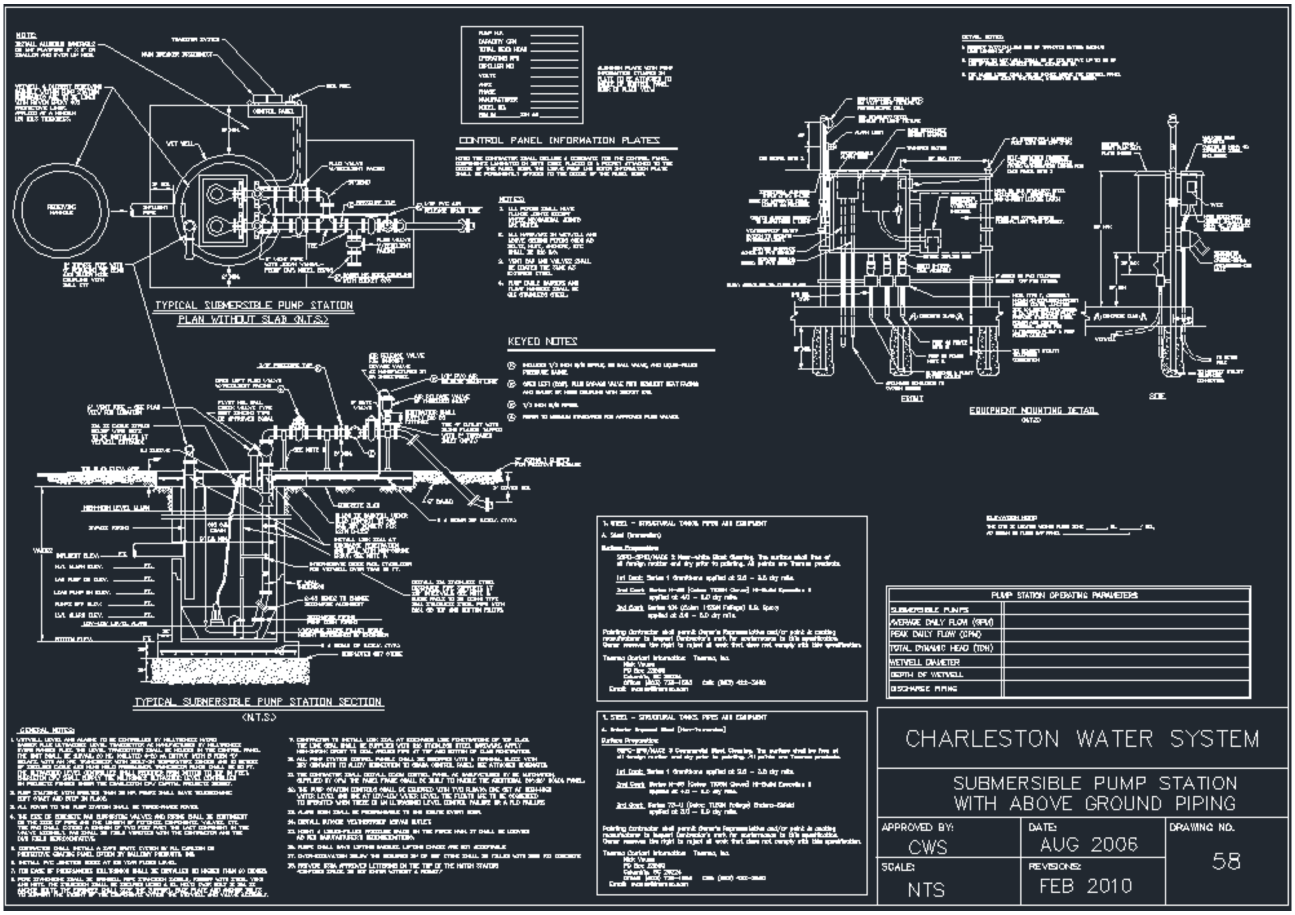
Typical main road cut

Appendix CC-2:



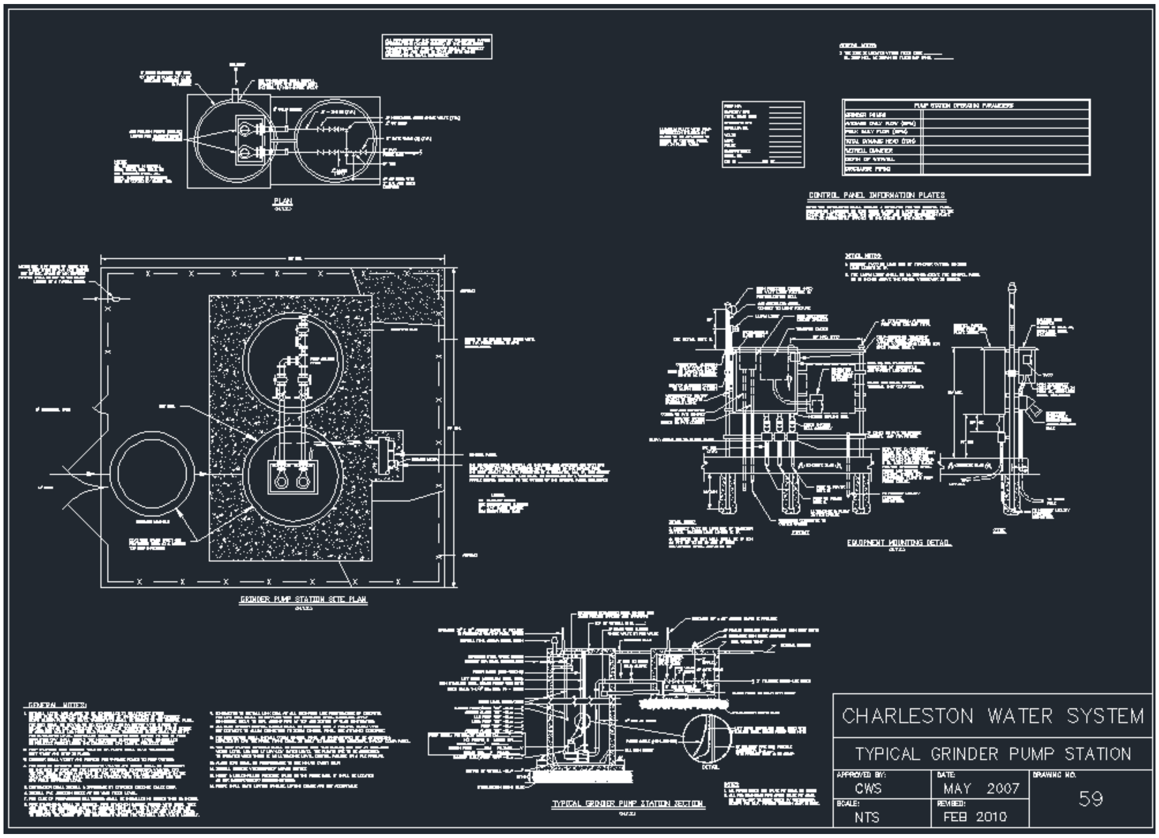
Typical service connection and cleanout

Appendix CC-3:



Typical Pump station

Appendix DD-1:



Typical Grinder Pump Station

Appendix DD-2:

Products: Grinder Pumps

Grinder 3hp KG-31

The KEEN GRINDER PUMP KG-31 series centrifugal grinder pumps easily handle residential, commercial or industrial sanitary waste, reducing it to fine slurry.

The KG-31 pump is designed for use in pressure sewer applications or any piping network.

The recessed vortex impeller design of the KG-31 grinder pump provides trouble free, non-overloading operation over the entire performance curve.

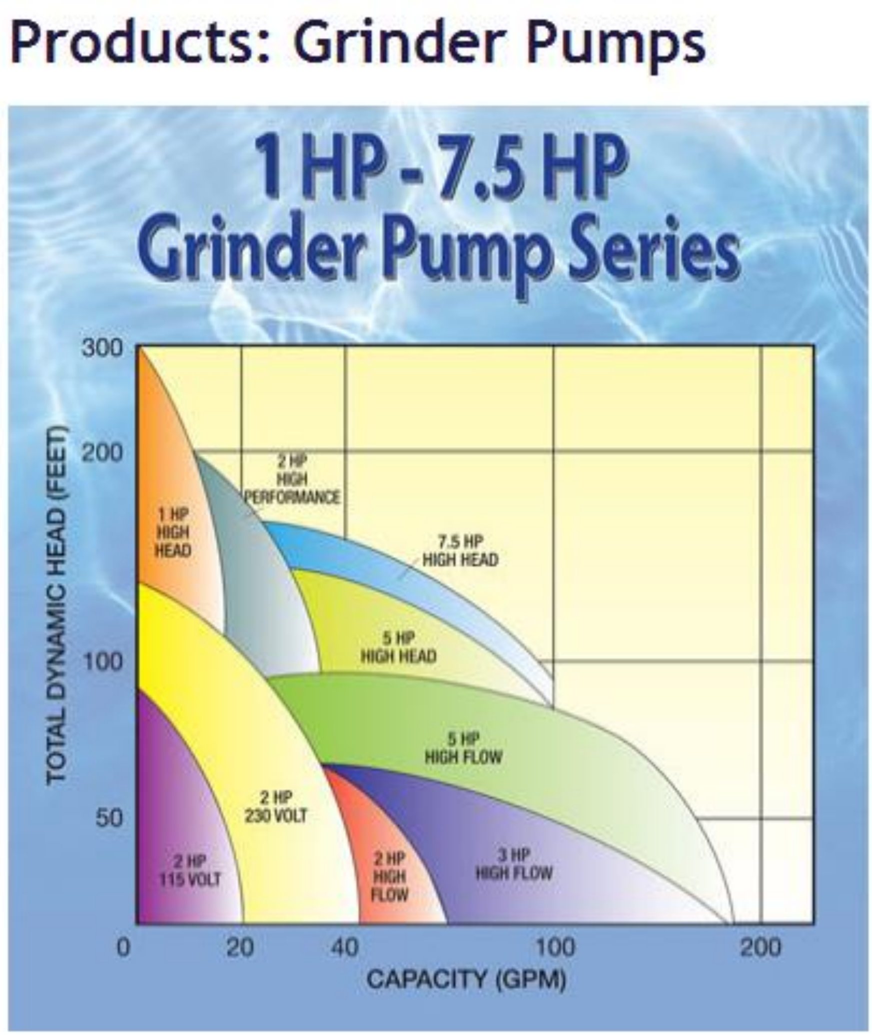
The modular design provides quick and easy serviceability. The hardened stainless steel grinder assembly provides many years of dependable operation.

The KG-31 series grinder pump features:

- 3 support bearings (upper / lower ball, sleeve)
- Dual mechanical seals (silicon carbide)
- Internal moisture detection
- Class F motor, single phase options
    - 208 / 230 volt, 1-phase

Grinder Pump specifications

Appendix DD-3:



Pump curve for Grinder pump

Appendix EE:

Table 5.1 Design Velocity

PIPE SIZING (inches)	* MINIMUM DESIGN VELOCITY (ft/s)	MINIMUN DESIGN SLOPES (%)	MAXIMUM DESIGN SLOPES (%)
8	2.1	0.380	6.980
10	2.2	0.306	5.121
12	2.3	0.256	3.919
15	2.4	0.205	2.880
18	2.4	0.140	2.390
21	2.5	0.146	1.890
24	2.6	0.127	1.520
27	2.6	0.115	1.378
30	2.7	0.102	1.113
36	2.7	0.085	0.945
42	2.8	0.073	0.754
48	2.9	0.064	0.616
54	3.0	0.058	0.522
60	3.0	0.051	0.430
66	3.1	0.047	0.396
72	3.1	0.043	0.362
78	3.2	0.040	0.316
84	3.2	0.037	0.293
96	3.3	0.032	0.238
108	3.3	0.028	0.208
120	3.4	0.026	0.182

* The velocities are based on the minimum required design shear stress recommendations provided in the American Society of Civil Engineers Manual of Practice No. 69 (MOP 69). These velocities will provide the design shear stress required to transport fine sand and grit particles less than 0.2 mm in diameter. Any slope outside the provided range will require a technical appeal to the Planning and Development Department (P&D) City Managers Representative. The Technical Appeals Procedure (P-107) can be found at the WSD's website shown below:

Shows the pipe size and minimum design velocities [1]