



National Park Service

**Glen Canyon National Recreation Area – Bullfrog
Launch Ramp**

Project Basis of Design

January 20, 2023

Prepared by:

Jacobs Government Services Company

Greenwood Village, CO

Contract No. 140P2021D0007

Task Order No. 140P2021F0391

Jacobs



GLCA Launch Ramps Design Criteria

Date: January 20, 2023

I. Introduction

In partnership with the Park, and input from other stakeholders during early listening sessions, Jacobs developed site specific design criteria to address project needs for the launch ramp concept designs.

II. Design Standards

1. Codes

- a. Local Building Codes for Arizona and Utah
 - i. 2018 International Building Code (IBC 2018)
 - ii. Building Code requirements for Structural Concrete (ACI 318-14)
- b. Launch Ramp Materials
 - i. Concrete – Building Code requirements for Structural Concrete (ACI 318-14)
 - ii. Steel – American institute of Steel Construction Steel Manual (AISC-14)
- c. Ramp and Marina
 - i. California Boat Launching Facilities – Design Handbook
 - ii. UFC 4-152-07 Design: Small Craft Berthing Facilities
- d. Roads
 - i. AASHTO Green Book (GDHS 2018)
 - ii. Park Road Standards – National Park Service 1984
- e. Water System
 - i. Kane County Standard Specifications and Drawing Details for Design and Construction (2014)
 - ii. Utah Division of Drinking Water, Series 500 Rules, Drinking Water Facility Construction Design and Operation (2014)
 - iii. American Public Works Association (APWA)
 - iv. American Water Works Association (AWWA)
 - v. 2018 International Building Code (IBC 2018)
 - vi. International Fire Code (2021)
- f. Sanitary Sewer System
 - i. Kane County Standard Specifications and Drawing Details for Design and Construction (2014)
 - ii. Uniform Plumbing Code (UPC 2021)
 - iii. 2018 International Building Code (IBC 2018)
- g. Electrical and Communications Systems
 - i. National Electrical Safety Code (2023)
 - ii. National Electrical Code (2020)

- h. ITA/EIA Standards
- 2. Loads (Dead, Live, Snow and Wind)
 - a. Seismic is not considered since for a slab on ground (without piles), the slab will move with the ground and there is not a structure period associated with it (different from pile-supported slab).
 - b. However, seismic shear may need to be considered for the grade beams; further work required to confirm.
 - c. Load combinations (per Table 5.3.1 of ACI 318-14)

III. Environmental Conditions

- 1. Water level
 - a. Current site water level is 3535 ft. (as of January 10, 2022)
 - b. Existing ramps were designed for a water level between 3540 to 3580 ft.
 - c. Design low water level for this project is 3450 ft.
- 2. Precipitation
 - a. ASCE 7-16 Ice/Snow
- 3. Wind
 - a. ASCE 7-16 Basic Wind Speed 120 mph based on a Risk Category of III or IV
 - b. Wind is applied to the side of the large houseboat (described under the Vessels section for each site), which creates an overturning moment. This overturning moment is applied to the ramp as a compression load on the leeward side.
- 4. Atmospheric Conditions
 - a. Atmospheric Conditions are derived from the following NOAA stations:
 - i. BULLFROG BASIN, UT US – ID GHCND:USC00421020
 - ii. Page, Page Municipal Airport (KPGA)

IV. Bullfrog Site Design Criteria

- 1. Existing Public Ramp Geometry
 - a. Existing Ramp– (Reference 3128 in NPS eTic Data). Main Boat ramp – 124 feet by 1,250 feet (running surface 110 feet wide), slope 9%. Low Water Boat ramp – 60 feet by 720 feet, slope 5.56%.
 - b. Low Water Ramp Extension – ramp extended and expanded to be 150 feet by feet by 1,165 feet (reference 60024 in NPS eTic Data). Slope 10.5%.
- 2. New Ramp Location Siting and Geometry
 - a. Bullfrog bay has very low water, siting will have to be closer to the main river channel within the bay or over to Stanton Creek site.
 - b. Desired ramp width 150 feet, 5- to 30-foot-wide ramp lanes.
 - c. Desired ramp slope is 10.5% in order to put in and take out larger houseboats.
 - d. Top of slab elevation at bottom of ramp will be 3445 ft. and have a

minimum of 5 ft of water.

- e. Two options have been considered:
 - i. Combined high and low water ramp to go from Elevation 3700 to 3450.
 - ii. Separate high and low water ramps. From Elevation 3700 to 3580 (existing ramp) and 3580 to 3450 (new ramp).

3. Vessels

- a. UDOT Ferry – Approximately 155 feet long, 50 feet wide and 8 feet draft.
- b. Large Houseboats – Up to 125 feet long, 22 feet wide, maximum of 5 feet draft, registered 250 Tons of displacement and 100 Tons of actual displacement as determined through engineering judgment.
- c. Small Recreational Craft

4. Vehicles

- a. Vehicle Types and Loads – Some of the large houseboats require multiple trucks or even tractor trailers, loading assumptions:
 - i. Loading from large houseboat vessels is spread over enough axles to make it road legal
 - ii. Loading assumed in structural model is a variation on HS-20 loading condition with a houseboat trailer attached. This critical loading condition consists of a 6-axle vehicle with a 16-kip front axle (Axle 1), followed by a 32-kip axle (Axle 2) set back 14 feet on center from Axle 1, followed by four 32-kip tandem axles (Axles 3 through 6) with the first setback 30 feet on center from Axle 2 and each subsequent tandem axle spaced 4 feet on center axle to axle.

5. Gangways

- a. CSA 46 Boarding Floats and Gangways (Reference 70383 in NPS eTic Data)
 - i. Loading floats 114-foot-long, Gangway 40-foot-long.
- b. Ramp utilizes UDOT Ferry Gangways/Floats – Wedges are 22 feet wide

6. Marina

- a. Requested minimum water depth – 20 feet, but it has been stated that the concessionaire could operate in 10 feet of water, or even as little as 6 feet of water as an extreme scenario.
- b. The intent of the concept design has been to replace the marina in-kind. Jacobs has proposed a relocation location for each of the two Bullfrog concept options, both allowing for a minimum of 20 feet of water. The marina will need to be reconfigured to fit in the desired new footprint, but the number of slips and docking areas remains the same as the existing marina.
- c. Phasing/Temporary storage – Jacobs has identified an intermediate marina location. This intermediate move provides a temporary location for the marina to be located entirely within the 3,495-foot water elevation contour. It is anticipated that this temporary relocation could be used down

to water elevation of 3505 feet, which allows for a minimum of 10 feet of water depth.

7. Roadways, Parking and Turnaround

- a. Pavement – Attached find the Bullfrog Traffic Data Summary from 2018 through 2022. The document was provided by Chuck Bullington, the Northlake Fee Supervisor on March 13, 2023. The file of raw data is the total number of vehicles in a given month. The dip in Sept. of 2019 was due to the traffic counting equipment being down. This information is being included to facilitate pavement design efforts.
- b. Roadway - The roadway is classified as a Class II - Connector Park Road with rolling to mountainous terrain. The roadway alignment was refined from the concept phase to avoid culturally sensitive areas within the park and to better balance earthwork. Specific roadway design criteria used are listed in Appendix A.
- c. Parking Lot - the direction for the parking lot design was to provide the same area currently at the existing Bullfrog ramp which is approximately 16 acres. Guidance provided by the Park was to design for a 2 lane circulatory ingress/egress around the perimeter of the parking lot.
- d. Truck Turnaround – the truck turnaround was designed to accommodate a semi with 125-ft trailer, based on the largest vessel at Antelope should it need to come out at Bullfrog before the Antelope low water ramp is constructed. The largest vessel at Bullfrog is 90-ft. The roadway, truck turnaround, and parking area were modeled for each of these vehicles as well as a car with boat trailer. Auto turn templates for each are provided in Appendix B.
- e. Earthwork - earthwork was balanced between the ramp and parking in order to utilize the large amount of cut within the ramp for fill within the parking area to reduce haul length. The roadway was designed to balance earthwork from the connection to SH276 to the parking area.

8. Geotechnical conditions

- a. Geotechnical investigations are being planned, and data shall be incorporated in the next phase of design.
- b. Currently assumed to be a stiff rock subgrade.
- c. Assumed cut and fill slopes of 1:2 (V:H).

9. Utilities

Four (4) utilities are designed to provide service to the relocated Bullfrog launch and will provide connection points for concessionaire services. The basis of utility design is to match the concessionaire's current level of service. The utilities include Potable Water System, Sanitary Sewer System, Electrical System and Telecommunications System. Design considerations for each utility are described in detail below:

- a. Potable Water System
 - i. Existing Water Usage

In response to a request by the Jacobs design team, the Park provided data on September 30, 2022, containing monthly potable water usage for Bullfrog Marina concessionaire services between February 4, 2015 and July 5, 2022. The design team analyzed the Park data and derived a monthly maximum usage design value of 2,500,200 gallons. This translates into an average usage of 57.85 gallons per minute (gpm). The design team also looked for physical indicators on-site as a check for the calculated flow rate. A utility survey performed by Jacobs indicates three (3) two-inch (2") potable water pipes are serving the current concessionaire facilities. Based on this piping, flow is estimated to be between 90 gpm and 150 gpm. This more conservative range of values will be the basis of new system design.

ii. Proposed Water Usage

The proposed connection point for the new system is to the existing 8-inch water main near the intersection of Nra Sr-276k and the access drive to the water storage tanks. The new water main will follow Nra Sr-276k to the road leading to Lake Powell School, and will follow the road to the point where an existing sanitary sewer force main runs "cross-country" toward Stanton Creek Road. The water main will run parallel to the force main eastward to Stanton Creek Road, where it will then follow Stanton Creek Road to the launch ramp.

The design flow rate for the new water main is based on the more conservative existing supply line capacity, estimated to be between 90 gpm and 150 gpm. This range of flow rates provide 1.5 to 5 times the peaking factor to the existing average water usage under the maximum month (57.58 gpm, above). This peaking factor takes peak hour demand into consideration. Based on design flow rate, a four-inch (4") diameter pipe is proposed, which provides a design fluid velocity of 2.5 to 4 feet per second (fps). Industry standard minimum fluid velocity is 3 to 6 fps. Flow pressure is assumed to be 20 pounds per square inch (psi). The proposed potable water system is not designed to provide fire flow. Two (2) post hydrants will be provided near the top of ramp and parking lot for localized water supply as needed. No water modeling was conducted in this phase of design.

b. Sanitary Sewer System

i. Existing Sanitary Sewage Flow

Existing sanitary sewer flow is based on estimated potable water usage, 90 gpm to 150 gpm (see Potable Water System).

ii. Proposed Sanitary Sewage Flow

The proposed connection point to the existing system is at an existing manhole along the east side of Stanton Creek Road, north of the intersection of Stanton Creek Road and the Bullfrog Sewer

Lagoon service road.

The design sanitary sewage flow is to match existing: 90 gpm to 150 gpm. The entire proposed sanitary sewer system provides conveyance via two (2) lift stations and pressure force main. Two (2) lift stations are proposed due to flow rate, pipeline length, losses and elevation gain. The lift stations are duplex systems, each with one pump operating and one on standby. Pump design flow rate is 150 gpm. Refer to Appendix C for detailed lift station design. Design force main diameter is four inches (4").

c. Electrical System

i. Existing System

The existing electrical system is a medium voltage (MV) system. Insulation rating is 15-kV, based on existing drawings received from the Park.

ii. Proposed System

The proposed system connection point is at the existing distribution line near the intersection of Stanton Creek Road and Nra Sr-276k. The MV system voltage is 12.47kV/7200 grounded Y. Insulation rating is 15-kV. MV circuit ampacity is 100-amp. Lift stations, parking lot and marina system voltages design values are 480V and 208/120V as required.

d. Communication System

The proposed system connection point is at the existing distribution line near the intersection of Stanton Creek Road and Nra Sr-276k. Cabling to be 36-strand single mode outside plant fiber optic cable, to be tied into the existing telecommunication system.

V. Hydrologic and Hydraulic Analysis

1. Hydrology

The offsite hydrology was completed using the SCS unit hydrograph method applied within the HEC-HMS model version 4.10. Drainage basins were delineated to potential culvert crossing locations along the roadway, parking lot and ramp alignment. Hydrologic analysis for the ditch sizing was completed using the Rational Method. The 10-, 50-, and 100-year floods were analyzed. See Appendix D for detailed report.

2. Hydraulics

Hydraulic analysis was not included in the scope of services. Estimates were made for culvert sizing based on the hydrology results for cost estimating purposes. Preliminary hydraulic analysis was performed for roadside ditches to estimate earthwork quantities. It was determined that in most cases, a 1-foot ditch depth would convey the 100-year flow.



Appendix A – Design Criteria

GLCA Bullfrog Design Criteria

Acceptance and Agreement

Friday, January 20, 2023

NPS

Mike Will
Project Manager

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Assistant Project Manager

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Roadway Design

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Roadway Design

Design Manuals and References

Design Element	Design Criteria Used	Notes	Park Road Standards	Design Criteria Reference and Notes	AASHTO (2018)	Design Criteria Reference and Notes
General						
Roadway Classification	Class II - Connector Park Road		Class II - Connector Park Road		Rural Collector	
Terrain	Rolling to Mountainous		Rolling to Mountainous		Rolling to Mountainous	
Design Speed (mph)	40		40	Table 1. Based on rolling terrain and assuming AADT of 1000-4000 veh/day	40	Table 6-1. Based on rolling terrain and ADT of 400-2000, ADT over 2000=50mph
Posted Speed (mph)	35	Based on posted speed of existing access road		5-10mph less than design speed		
ADT	unknown	assume 1000-4000 veh/day				
Percent Trucks	unknown					
Design Vehicle	Custom	Semi with 90-ft trailer				
No. of General Purpose Lanes	2		2	Table 10. assume ADT 1000-4000		
Lane Width (ft)	12		11	Table 10	11	Table 6-5.
Pavement Type	asphalt		Asphalt	Table 10		
Econ. Analysis Period (Years)						
Design Life (Years)						
Horizontal Curvature/Alignment Criteria						
Minimum Curve Radius			509-ft	Table 5		
Superelevation (E_{max})	6%		6%	Recommended (pg 22)		
Max Relative Gradient						
Normal Cross-Slope	2%		1-3%	for paved surfaces (pg 31)	1.5-2%	for paved surfaces (3-6% unpaved)
Method of Attaining Superelevation (% on Tangent)						
Maximum Algebraic Difference at Crossover Line (%)						
Vertical Curvature/Alignment Criteria						
Maximum Grade (%)	9%		9%	Table 3. rolling terrain	8%	Table 6-2. rolling terrain.
Minimum Grade (%)	0.50%		0.5%	pg 19		
Critical Length of Grade (ft)				usually not a major concern for park roads		
K_{eq}						
	Minimum (ft)	64	60	Table 4	64	Table 3-37
	Desirable (ft)		70	Table 4		
K_{crest}						
	Minimum (ft)	44	60	Table 4	44	Table 3-35
	Desirable (ft)		80	Table 4		
Stopping Sight Distance (ft)	354	AASHTO, 9% Downgrade	325	Table 6. 275 min, 325 desirable	305	Table 3-1
Stopping Sight Distance (ft) (grade +3%/+6%/+9%)					289, 278, 269	Table 3-2
Stopping Sight Distance (ft) (grade -3%/-6%/-9%)					315, 333, 354	Table 3-2
Passing Sight Distance (ft)			730	Table 4	800	Table 3-4
Decision Sight Distance (ft)						
Cross Section Criteria						
Clear Zone on Tangent - 4:1 Foreslope						
	Minimum (ft)				12	AASHTO: 7-10ft desirable for low-speed rural collectors. RDG: 12-16ft for 4:1 foreslope, ADT 750-6000
	Desirable (ft)					
Clear Zone on Curve						
	Minimum Adjustment Factor					
	Maximum Adjustment Factor					
Side Slope Distance, "Z" (ft)						
Ditch Foreslope (max)	4		4 to 1	pg 32		
Ditch Backslope (max)	4					
Fill Slope	4		4 to 1	pg 32		
	Surfacing Taper Slope					
	Desirable Slope					
	Maximum Slope					
Shoulder Width (ft)	3		3	Table 10. assume ADT 1000-4000	4-6	Table 6-5.
Shoulder Cross Slope (%)	4					
Shoulder Surface Type	asphalt					

Drainage Design			
Design Manuals and References			
Design Element	Design Value	Reference	Remarks
Flood Frequency - Roadway			
Design	50-year	UDOT	
Review/Check	100-year	UDOT	

Structural Design

Design Manuals and References

Codes		
2018 International Building Code		
Building Code Requirements for Structural Concrete		
American Institute of Steel Construction Steel Manual AISC-14		
Reference Manuals		
California Boat Launching Facilities - Design Handbook		
UFC 4-152-07 Design: Small Craft Berthing Facilities		
Design Element	Design Value	Remarks
Environmental Conditions		
High Water Level	3700ft	
Low Water Level	3450 ft	
Basic Wind Speed	108mph	Wind is applied to the side of the houseboat which creates an overturning moment
Risk Category	III	
Ramp Design		
Ramp Width	150ft	5- 30ft wide ramps
Ramp Slope	10.50%	
Bottom of Ramp Elevation	3445ft	5ft below design low water elevation
Gangways	CSA 46 Boarding Floats and Gangways. Loading floats 114ft long Gangway 40ft long	Reference 70383 in NPS eTic Data
Ramp utilizes UDOT Ferry Gangways/Floats	Wedges are 22ft wide	
Design Vessels		
UDOT Ferry	155ft long, 50ft wide, 8 ft draft	
Large Houseboats	125 ft long, 22 ft wide, 5 ft draft	registered 250 Tons of displacement and 100 Tons of actual displacement as determined through engineering judgement
Small Recreational Craft		

Water System Design

Design Manuals and References

Kane County Standard Specifications and Drawing Details for Design and Construction (2014)	https://kane.utah.gov/gov/dept/planning/kane-county-standard-specifications-and-drawing-details-and-design-and-construction/	For installation, testing, and disinfection of new potable water systems
Utah Division of Drinking Water, Series 500 Rules, Drinking Water Facility Construction Design and Operation (2014)	https://deq.utah.gov/division-drinking-water	For installation, testing, and disinfection of new potable water systems
American Public Works Association (APWA)	https://www.apwa.net/	
American Water Works Association (AWWA)	https://www.awwa.org/	
International Building Code	https://codes.iccsafe.org/	
International Fire Code (2021)	https://codes.iccsafe.org/	For clear space around hydrants and vehicle impact protection.

Design Element	Design Value	Remarks
Existing Water Usage		
Monthly Peak Usage	2,500,200 gal	Park provided data from 9/30/22 containing monthly usage for Bullfrog Marina from 2/5/15 to 7/5/22. Design value is monthly maximum usage from 7/5/15 to 8/5/15.
Average Usage	57.85 gpm	Park provided data from 9/30/22 containing monthly usage for Bullfrog Marina from 2/5/15 to 7/5/22. Design value is average usage in the peak month (7/5/15 to 8/5/15).
Physical Indicators - Supply Piping Size & Quantity	90 - 150 gpm	Flow estimate is based on a utility survey, which identified three 2-inch potable water pipes serving the current concessionaire facilities.
Proposed Water Usage		
Minimum Fluid Velocity	3 to 6 feet per second	Industry standard practice.
Flow Rate	90 - 150 gpm	Design Value is based on more conservative supply line capacity, which provides 1.5 to 5 times the peaking factor to the current average water usage under the maximum month (57.85 gpm, above). This peaking factor takes peak hour demand into consideration.
Pipeline Size	4 inches diameter	Based on Design Flow Rate, Design Fluid Velocity is 2.5 to 4 feet per second.
Flow Pressure	20 psi	Assumed. No water modeling was conducted.
Fire Flow	--	System is not designed to provide fire flow; two (2) hydrants will be provided for top of ramp and parking lot water access as needed.

Sanitary Sewer System Design

Design Manuals and References

Kane County Standard Specifications and Drawing Details for Design and Construction (2014)	https://kane.utah.gov/gov/dept/planning/kane-county-standard-specifications-and-drawing-details-and-design-and-construction/	For installation and testing of new sanitary sewer systems
Uniform Plumbing Code	https://www.iapmo.org/publications/read-uniform-codes-online/	
International Building Code	https://codes.iccsafe.org/	
Design Element	Design Value	Remarks
Existing Sanitary Sewage Flow		
Physical Indicators - Supply Piping Size & Quantity	90 - 150 gpm	Sanitary sewage flows are designed to match potable water usage.
Proposed Sanitary Sewer System Criteria		
Flow Rate	90 - 150 gpm	Match existing
Pipeline Size	4 inches diameter	Pressure force main
Lift Stations	2	Based on flow rate, sewer length, losses and elevation gain. Duplex systems (each with one operating pump; one standby). See also Appendix A.
Pump Flow Rate	150 gpm	Maximum anticipated flow; serves about 2,000 people at an average daily flow rate of 100 gallons per day per capita (Kane County Design Specs.)

Electrical and Communications System Design

Design Manuals and References

National Electrical Safety Code 2023	Institute of Electrical and Electronics Engineers (IEEE)	For installation of underground medium-voltage utility equipment.
National Electrical Code 2020	National Fire Protection Association	For installation of electrical distribution systems downstream of service point.
TIA/EIA Standards		

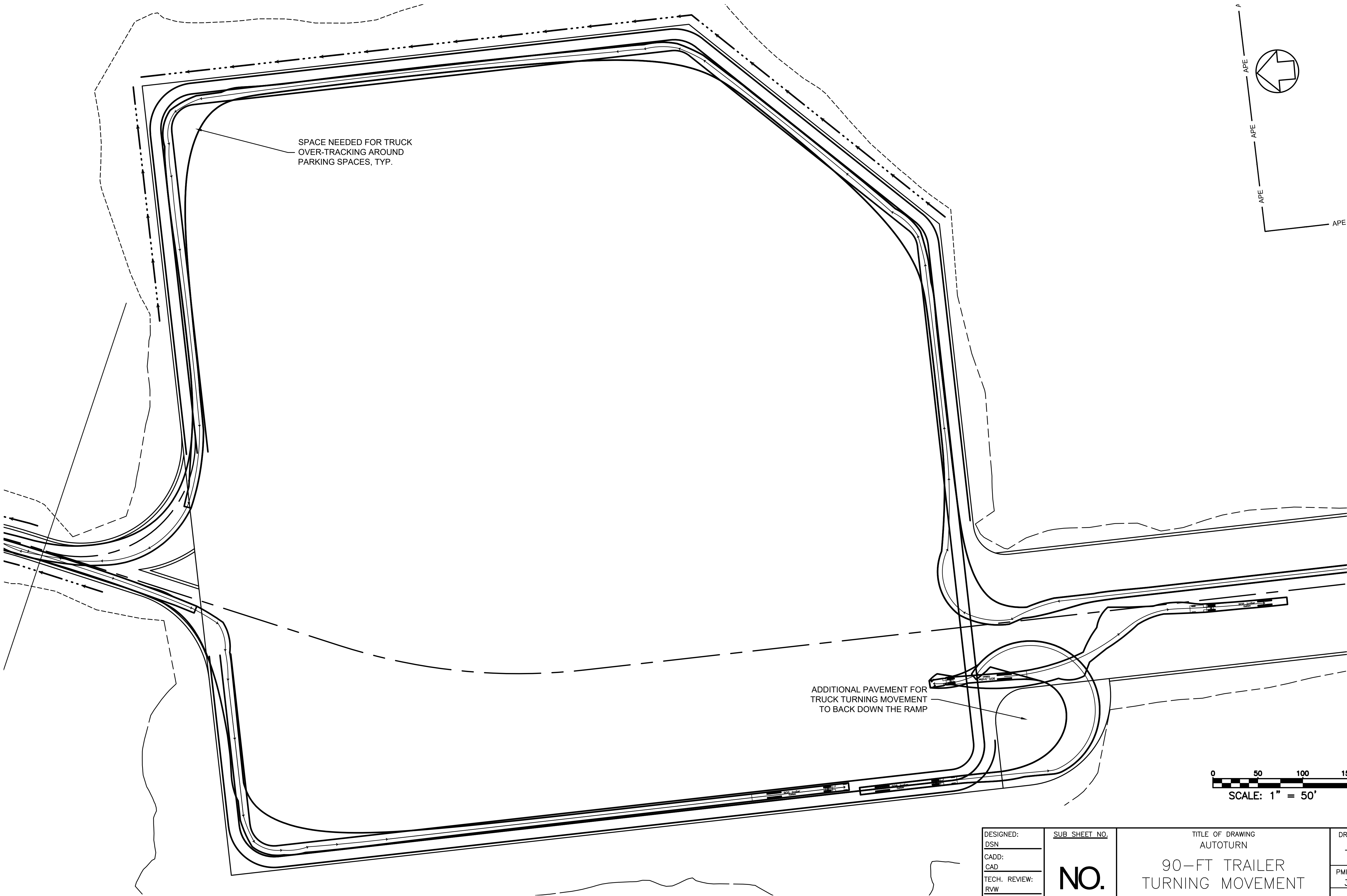
Design Element	Design Value	Remarks
Existing Electrical System Information		
Medium Voltage (MV) System Voltage		System voltage of existing system needs to be confirmed.
Insulation Rating	15-kV	Insulation rating to be confirmed with system voltage. Existing drawings received indicate 15-kV insulation rating.
Proposed Electrical System Criteria		
MV System Voltage		System voltage to be confirmed.
Insulation Rating	15-kV	Insulation rating to be confirmed.
MV circuit ampacity		Ampacity rating to be confirmed with system voltage.
Pump stations, parking lot, and marina system voltages	480V and 208/120V as required	
Proposed Communication System Criteria		
Communications system cabling	36-strand single mode outside plant fiber optic cable	

Geotechnical Design - TBD		
Design Manuals and References		
Design Element	Design Value	Remarks
Criteria		
Criteria		



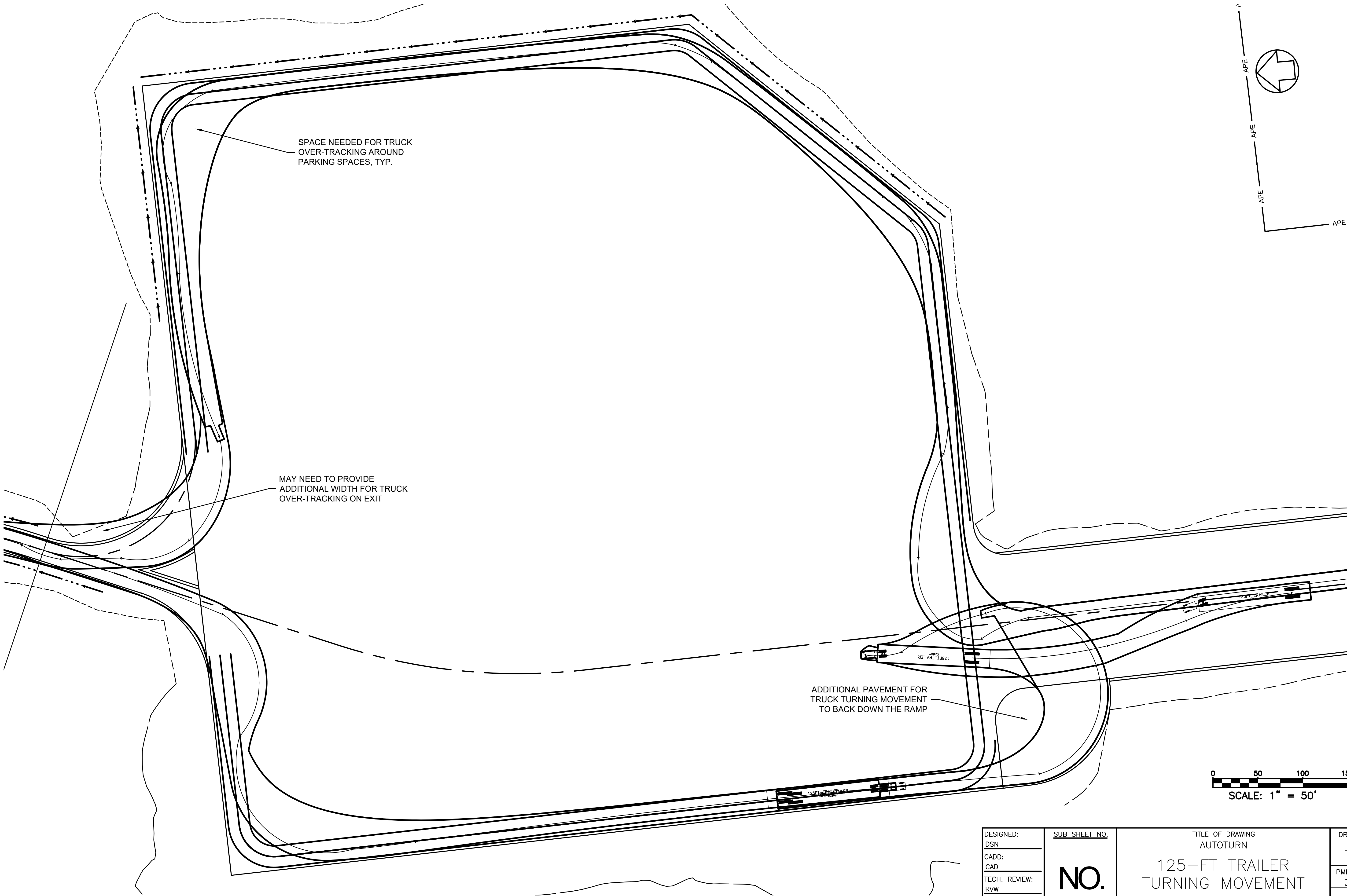
Appendix B – Roadway Auto turn Templates

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TECH. REVIEW: RVW			SHEET
DATE: 11/8/22			# OF X
GLEN CANYON NATIONAL RECREATION AREA			

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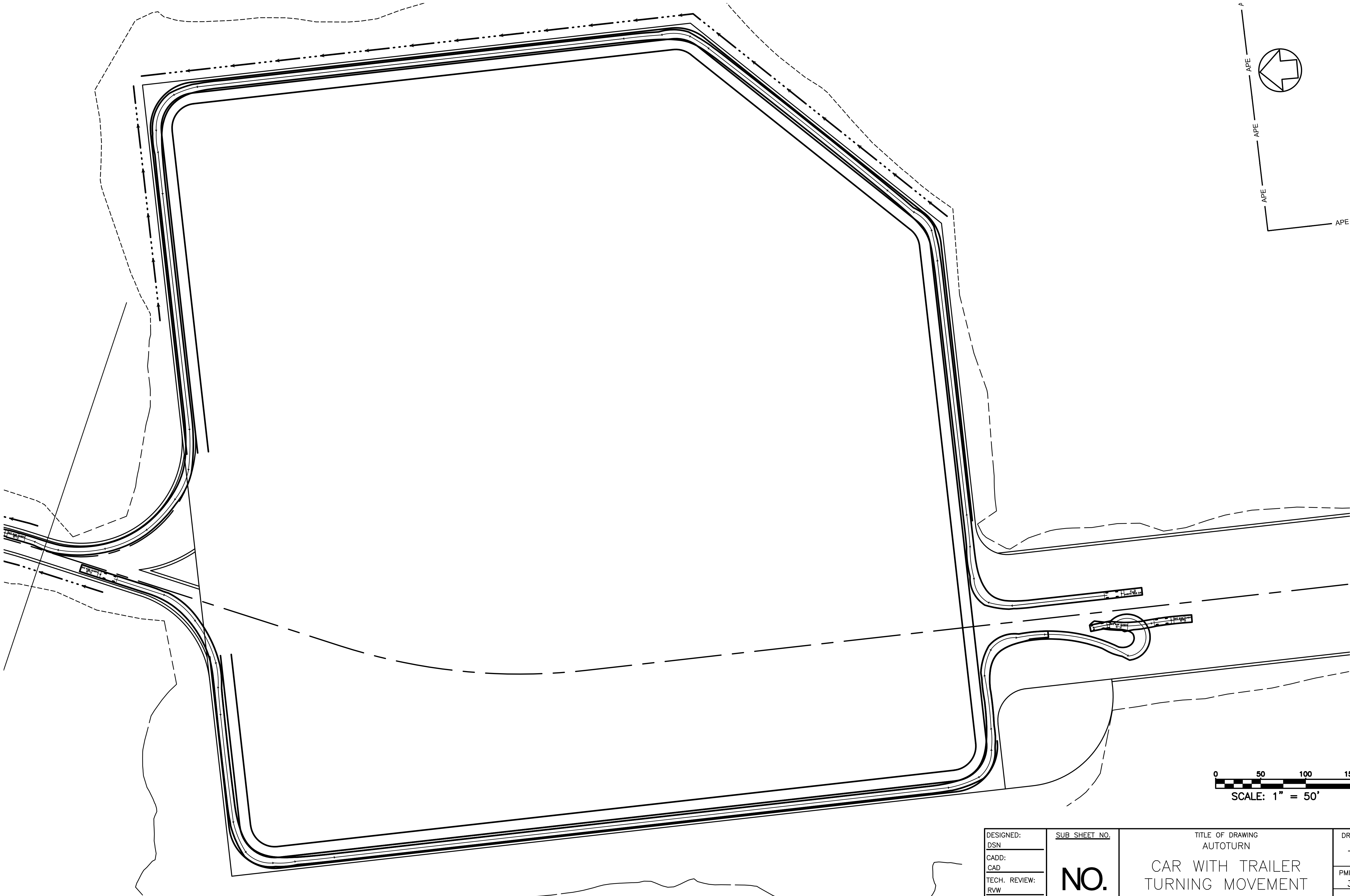
DESIGNED:
DSN
CADD:
CAD
TECH. REVIEW:
RVW
DATE:
11/8/22

SUB SHEET NO.
NO.

TITLE OF DRAWING
AUTOTURN
**125-Ft TRAILER
TURNING MOVEMENT**
GLEN CANYON NATIONAL RECREATION AREA

DRAWING NO.
#XXX
XXXXXX
PMIS/PKG NO.
318882
SHEET
OF X

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DESIGNED:
DSN
CADD:
CAD
TECH. REVIEW:
RVW
DATE:
11/8/22

SUB SHEET NO.

NO.

TITLE OF DRAWING
AUTOTURN

CAR WITH TRAILER
TURNING MOVEMENT

GLEN CANYON NATIONAL RECREATION AREA

DRAWING NO.
#XXX
XXXXXX
PMIS/PKG NO.
318882
SHEET
OF X



Appendix C – Sanitary Lift Station Design

Lift Station Design Calculations

Project Name: **GLCA**
Lift Station - 1

Project No.:
Design Date:

Design Summary: PA 5 Duplex Lift Station

Total Number of Pumps =	2	
Pump Discharge Flow =	150	gal/min.
Pump Discharge Head =	275	feet
Force Main Diameter =		nominal diameter
Force Main Diameter =	3.786	inches
Wet Well Diameter =	12	foot diameter
Wet Well Depth =	12.4	feet
Discharge Header Static Head =	34	psi
Discharge Header Dynamic Head =	114	psi

Approximate HP required at duty point, each pump: 15.0
(Assumes 75% Efficiency)

1. Design Flows:

Detailed lift station phasing and flow calculations provided elsewhere

Peak Wet Weather Flow =	0.152	MGD	COA Requirements
=	106	gpm	
Average Daily Dry Weather Flow =	0.04	MGD	
=	27	gpm	
Minimum Daily Flow =	0.01	MGD	
	5	gpm	

2. Wet Well Design Calculations:

Pump Capacity (q) =	150	gal/min.	
Number of Pumps =	2		Pump Capacity OK
Firm Capacity =	150	gal/min.	
Minimum req. time between pump starts (θ) =	6	minutes	TCEQ Table C.4 - Minimum Pump Cycle Times. HP < 50 = 6 minutes. HP 50 - 100 = 10 minutes. HP > 100 = 15 minutes. Based on PWWF.
Minimum wet well working volume =	225	gallons	= q x θ / 4 (Minimum volume between lead pump on and all pumps off)
Using Peak Wet Weather Flow =	106	gals/min (MDF)	
Net outflow in gallons per minute =	44	(1 pump running)	= q - PWF
Using Peak Wet Weather Flow =	106	gals/min (MDF)	
Net outflow in gallons per minute =	44	(2 pumps running)	= q - PWF
Using Average Daily Flow (ADF) =	27	gals/min (ADF)	
Net outflow in gallons per minute =	124	(1 pump running)	= q - ADF
Using Minimum Daily Flow (ADF) =	5	gals/min (ADF)	
Net outflow in gallons per minute =	145	(1 pump running)	= q - ADF

3. Wet Well Detention Times and Pump Cycle Times

Wet Well Working Volume Required: 225 gallons
 Working Volume Depth: 4.00 ft
 Wet Well Diameter: 12 ft
 Calculated Working Volume: 3,384 gallons

Volume OK
 (when including calculated volume between
 lead & lag pumps)

Average Flow Detention Time: (1 pump running)

$T_f = 128$ min.
 $T_e = 27$ min.
 $T_d = 155$ min.
 Daily Cycles = 10

Minimum Flow Detention Time: (1 pump running)

$T_f = 677$ min.
 $T_e = 23.3$ min.
 $T_d = 700$ min.

Odor control will be required since detention time > 180 minutes

Minimum Time Between Pump Starts (results when inflow = 0.5*Pump Capacity): (1 pump running)

Inflow = 75 gpm
 $T_f = 45.12$ min. per pump
 $T_e = 45.12$ min. per pump
 Cycle Time = 180 min. from pump start to pump start
 Daily Cycles = 8

Minimum Cycle Time = 180.5 > = 6 OK

4. Total Dynamic Head Calculations:

Discharge Pipe: **35** linear ft
Roughness, c = 120
Inner Diameter, d = **3.91** in (8" Class 53)

Material: **Class 53 Ductile Iron (when Flanged & Exposed)**
Class 350 Ductile Iron (when Restrained & Buried)
Friction Losses, H_f (ft) = $0.2083 * (100 / c)^{1.85} * (Q^{1.85} / d^{4.8655}) * (\text{Length}/100)$

PEAK FLOWS, EACH PUMP (1 pumps running)
Flow, q = 106 gpm
f = 1.091 feet /100 ft of pipe
Total Friction Loss = **0.38** feet, both pipes

MAXIMUM FLOW EACH PUMP (1 pump running)
Flow, q = 150 gpm
f = 2.074 feet /100 ft of each pipe
Total Friction Loss = **0.73** feet

Minor Losses in Discharge Pipe:

Minor Losses, $H_{f,m}$ (ft) = $K * V^2 / 2g$

Number of 90 deg. bends = **3**
Number of 45 deg. bends = **6**
Number of 22.5 deg. bends = **0**
Number of thru flow tees = **2**
Number of branch flow tees = **1**
Number of Swing CVs = **1**
Number of Plug Valves = **1**
Total K each discharge pipe = **7.1**
Total Minor Losses = **1.78** feet (considering peak inflows, two pumps running)

K = 0.54 for 90 deg. bends
K = 0.29 for 45 deg. bends
K = 0.16 for 22.5 deg. bends
K = 0.36 for thru flow tee
K = 1.08 for branch flow tee
K = 1.71 for Swing Check Valve
K = 0.31 for Plug Valve, open full

Minor Loss K coefficients calculated based on Equivalent Lengths & Swamee-Jain Friction Factors, i.e. pipe material, diameter, fitting types, material roughness and Reynold's No. Detailed calculations shown on other calculation sheet.

Force Main: **7,500** linear ft
Roughness, c = 120
Inner Diameter, d = **3.786** in (4" PVC sch 80)

Material: **DR-25, CL 165, C905 PVC**
Friction Losses, H_f (ft) = $0.2083 * (100 / c)^{1.85} * (Q^{1.85} / d^{4.8655}) * (\text{Length}/100)$

MIN FLOW FOR FORCE MAIN (1 pump running)
Flow, q = 106.00 gpm
f = 1.28 feet/100 ft of pipe

MAX FLOW FOR FORCE MAIN (1 pump running)
Flow, q = 150 gpm
f = 2.43 feet/100 ft of pipe

Velocity = 3.0 fps
Okay.
(3 to 6 fps is acceptable to TCEQ)

Velocity = 4.3 fps
Okay.
(3 to 6 fps is acceptable to TCEQ)

Total Friction Loss = **95.7** feet

Total Friction Loss = **181.9** feet

Minor Losses in Force Main:

Minor Losses, $H_{f,m}$ (ft) = $K * V^2 / 2g$

Number of 90 deg. bends = **1**
Number of 45 deg. bends = **6**
Number of 22.5 deg. bends = **4**
Number of thru flow tees = **3**
Number of branch flow tees = **2**
Number of Swing CVs = **1**
Number of Plug Valves = **5**
Number of Gate Valves = **3**
Number of Pipe Exits = **1**
Total K force main = **7.0**
Total Minor Losses = **1.97** feet (considering two pumps running)

K = 0.33 for 90 deg. bends
K = 0.18 for 45 deg. bends
K = 0.10 for 22.5 deg. bends
K = 0.22 for thru flow tee
K = 0.66 for branch flow tee
K = 1.05 for Swing Check Valve
K = 0.18 for Plug Valve, open full
K = 0.08 for Gate Valve, open full
K = 1.00 for Pipe Exit

Minor Loss K coefficients calculated based on Equivalent Lengths & Swamee-Jain Friction Factors, i.e. pipe material, diameter, fitting types, material roughness and Reynold's No. Detailed calculations shown on other calculation sheet.

Elevation Head:

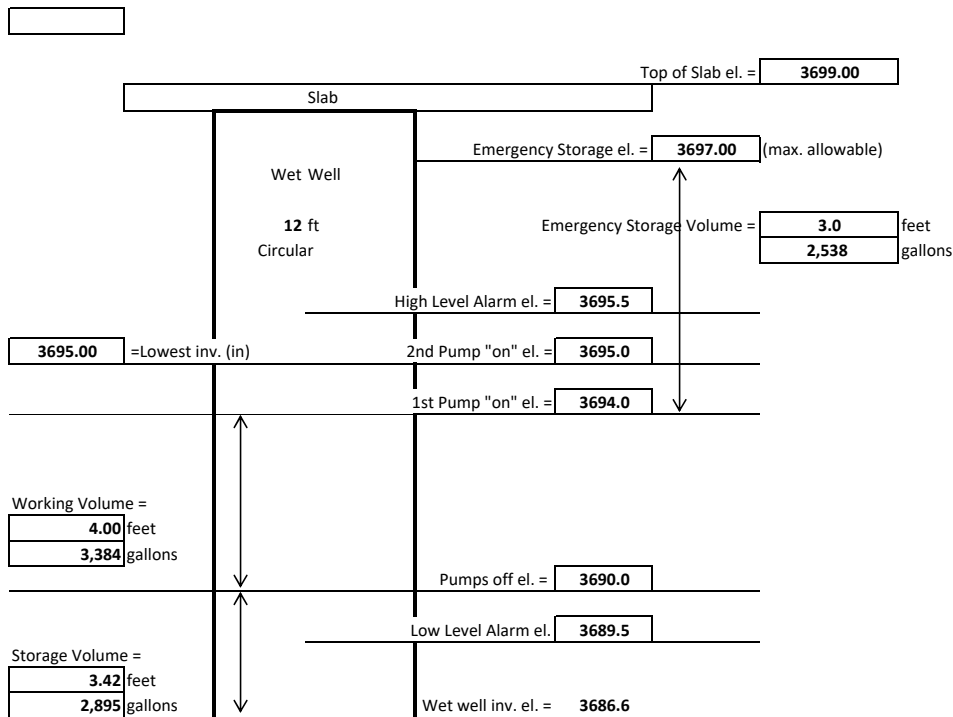
Discharge Header EL =	3700.0	MSL	79.0	Static Head on Header (ft)
Slab EL =	3699.0	MSL	34.2	psi
	3697.0	MSL	=	Max. Emergency Storage EL (Elevation lower than lowest upstream MH rim elevation)
	3695.5	MSL	=	High Level Alarm EL
Lowest Invert EL =	3695.00	MSL		influent flowline elevation
	3696.0			
	3695.00	MSL	=	2nd Pump "On" EL
	3694.00	MSL	=	1st Pump "On" EL
			Working Volume =	4.00 feet
Pumps off EL =	3690.0	MSL	or	3690.0 = Lowest elevation of FM (if < WW inv.)
Low Level Alarm EL =	3689.5			
Min Water Level EL =	3689.25		2.67	Min Water Level Allowed from Invert (ft)
Wet Well Invert EL =	3686.6	MSL	=	12.4 Wet well Depth (feet)
			5.0	feet from 1st "on" stage to slab
FM Outfall EL =	3779.00	MSL	=	89 feet of elevation head (Maximum)
			84	feet of elevation head (Minimum)
	262.90			Dynamic Head on Header (ft)
	113.8			psi

Summary of Head Losses (Two Pumps Running):

Force Main Friction Loss =	181.9	
Force Main Minor Losses =	1.97	
Discharge Pipe Friction Loss =	0.38	
Discharge Pipe Minor Losses =	1.78	
Elevation Head =	89.0	feet
Total Dynamic Head, TDH =	275	feet

5. Emergency Storage Provisions:

Wet Well Size =	12	feet	Circular
Max. Allowable Storage Elev. =	3697.0	MSL	
Depth used for Storage =	3.0	feet (1st Pump "on" to the maximum allowable elev.)	
Wet Well Storage Volume =	2,538	gallons	
Manhole Diameter =	4	feet	
Accumulated MH depth used for Storage =	159	Vertical feet	
Manhole Storage Volume =	14,945	gallons	
Total Storage Volume =	17,483	gallons	
Using an ADF inflow of	27	gpm,	Storage Duration = 660 minutes
Using a PWF inflow of	106	gpm,	Storage Duration = 165 minutes



8. Force Main Flush Time:

$$\text{Number of Pumping Cycles necessary to completely change water in Force Main} = \frac{\text{Volume of water in Force Main (gal)}}{\text{Gallons of Outflow per Pump Run Time (gal)}} = 1.3$$

$$\text{Flush Time} = \frac{(\text{Number of Pumping Cycles necessary to completely change water in Force Main}) \times (\text{Minimum Daily Flow Cycle Time})}{1}$$

$$T_{\text{flush}} = \frac{\text{Volume of water in Force Main (gal)} \times \text{Minimum Daily Flow Cycle Time (min)}}{\text{Working Volume of Wet Well (gal)}}$$

Pump Outflow =	150	gpm (1 pump running maximum flow)
Pump Run Time =	23.3	minutes for minimum daily inflows (time to empty)
Outflow per Pump Run Time =	3,501	gallons
Vol. of water in FM =	4,386	gallons = length x pi x d ² / 4 x 7.48 gal/c.f.
Minimum Daily Flow Cycle Time =	700.1	min
Working Volume of Wet Well =	3,384	gallons

$$T_{\text{flush}} = 706 \text{ min}$$

Odor control will be required.

9. Water Hammer Calculations:

Discharge Header Pipe:

$$\text{Water hammer pressure, } p = \frac{(a) \times (v)}{2.31 \times (g)} + \text{operating pressure of pipe (psi)}$$

a = pressure wave velocity (fps)

w = specific weight of water (62.4 lbs/ft³)

g = acceleration of gravity (32.2 ft/sec²)

k = bulk modulus of water (311,000 psi)

d = inside diameter of pipe (inches)

E = Young's modulus of pipe (psi)

t = pipe wall thickness (inches)

v = flow velocity of pipe (fps)

$$a = \frac{12}{[(w/g) \times ((1/k) + (d/Et))]^{0.5}}$$

operating pressure = TDH in psi = 119.2 psi

The following was calculated using the properties of the 10-inch DI CL 350 pipe:

where: d = 3.91 inches

E = 24,000,000 psi

t = 0.26 inches

v = 4.01 fps

a = 4,398 fps

p = 356.2 psi

[Ductile Iron Society, Ductile Iron Data for Design Engineers \(2013\)](#)

Class 350 Ductil. [US Pipe, Ductile Iron Pipe Design for Fire Protection, Water & Wastewater \(2013\), Table 2.](#)

Highest velocities for old system

Okay

For Ductile Iron Pipe, rated working stress + 100 psi surge pressure = 450 psi

[US Pipe, Ductile Iron Pipe Design for Fire Protection, Water & Wastewater \(2013\), Table 2.](#)

Water Hammer Calculations - Force Main:

$$\text{Water hammer pressure, } p = \frac{(a) \times (v)}{2.31 \times (g)}$$

a = pressure wave velocity (fps)

g = acceleration of gravity (32.2 ft/sec²)

k = bulk modulus of water (311,000 psi)

d = inside diameter of pipe (inches)

E = Young's modulus of pipe (psi)

t = pipe wall thickness (inches)

v = flow velocity of pipe (fps)

$$a = \frac{12}{[(w/g) \times ((1/k) + (d/Et))]^{0.5}}$$

operating pressure = TDH in psi = 119.2 psi

where: d = 3.79 inches

E = 420,000 psi

t = 0.978 inches

v = 4.28 fps

a = 2,445 fps

p = 259.7 psi

[Georg Fischer Harvel & Professional Plastics](#)

DR-25, CL 165, C905 PVC [Georg Fischer Harvel & Professional Plastics](#)

Highest velocities allowable in force main (highest flows allowable)

Surge control measures sha [Calculation example here](#)

For C900 DR25 CL165 pipe, rated working pressure = 165 psi

10. NSPH Calculations

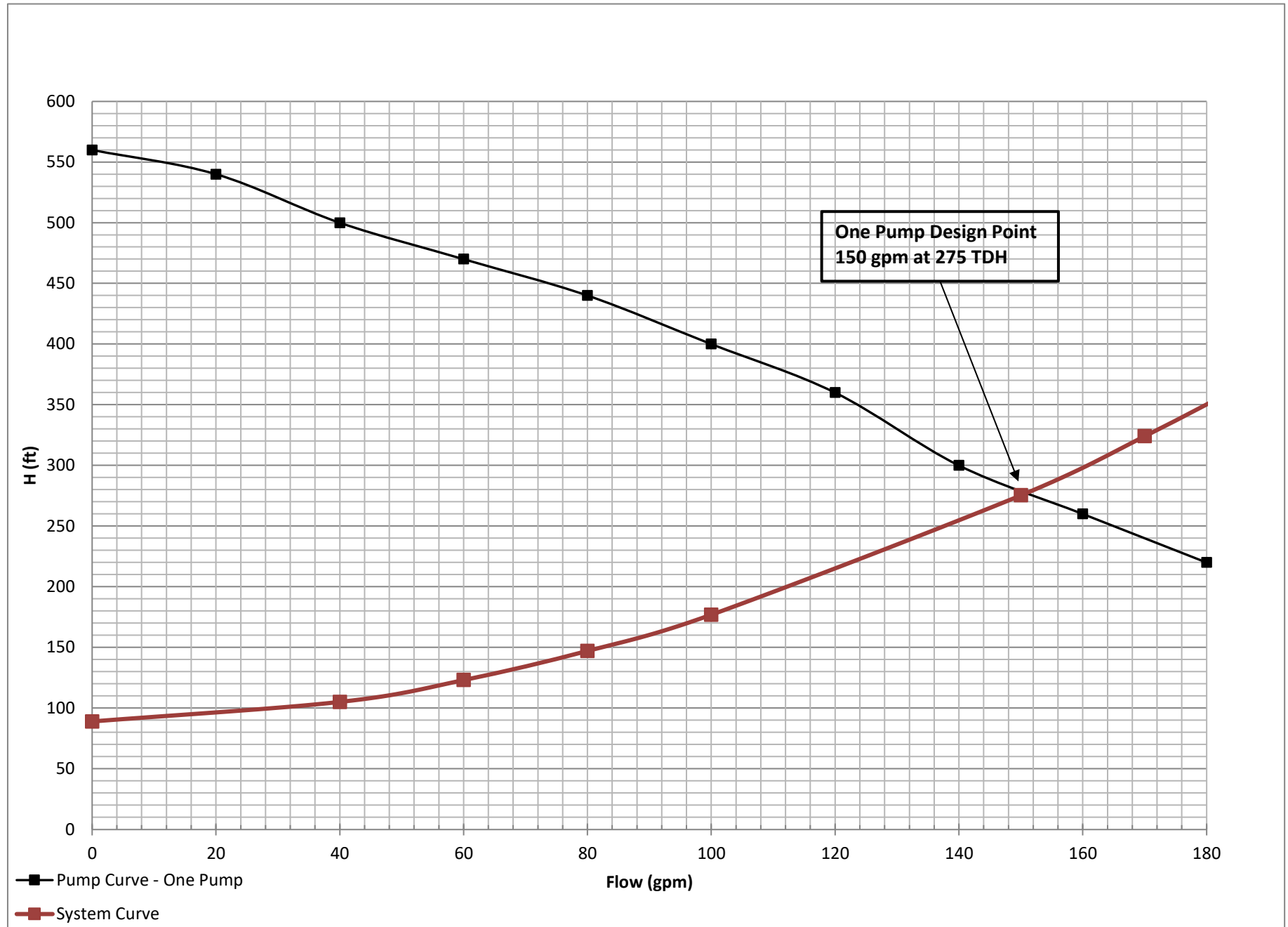
NPSHA = Atmospheric Pressure (ft) + Static Head + Surface Pressure Head - Vapor Pressure of Product - Losses in Suction Piping, Valves & Fittings

14.7	Atmospheric Pressure (psi)	Same for Kings #1, PH I
34.0	Atmospheric Pressure (ft)	Same for Kings #1, PH I
3.4	Static Suction Head (ft) MINIMUM	Same for Kings #1, PH I
8.4	Static Suction Head (ft) MAXIMUM	Same for Kings #1, PH I
0.0	Gage Pressure (the tank is open to atmospheric pressure)	Same for Kings #1, PH I
0.27	Vapor Pressure of Water at 68 def F (psia)	Same for Kings #1, PH I
0.62	Vapor Pressure of Water at 68 def F (ft)	Same for Kings #1, PH I
1.0	WW Specific Gravity	Same for Kings #1, PH I
0.0	Suction Piping Losses	Same for Kings #1, PH I
36.8	NPSHA using MINIMUM static suction head	Same for Kings #1, PH I
41.8	NPSHA using MAXIMUM static suction head	Same for Kings #1, PH I

12.0	NSPHR from pump curve at flows of 1,140 gpm each (ft)
13.02	NSPHR from pump curve at maximum flows of 1,710 gpm, one pump running (ft)
15	NSPHR from pump curve at new system maximum flows of 1,975 gpm, one pump running (ft)
32	NSPHR from pump curve at maximum possible flows of 3,000 gpm, two pumps running, new system (ft)

NSHPA > NSHPR

Lift Station-1 Duplex System Curve and Pump Curve at 15 HP



Lift Station Design Calculations

Project Name: **GLCA**
Lift Station - 2

Project No.:
Design Date:

Design Summary: PA 5 Duplex Lift Station

Total Number of Pumps =	2	
Pump Discharge Flow =	150	gal/min.
Pump Discharge Head =	441	feet
Force Main Diameter =		nominal diameter
Force Main Diameter =	3.786	inches
Wet Well Diameter =	12	foot diameter
Wet Well Depth =	12.5	feet
Discharge Header Static Head =	84	psi
Discharge Header Dynamic Head =	185	psi

Approximate HP required at duty point, each pump: 25.0
(Assumes 75% Efficiency)

1. Design Flows:

Detailed lift station phasing and flow calculations provided elsewhere

Peak Wet Weather Flow =	0.155	MGD	COA Requirements
=	108	gpm	
Average Daily Dry Weather Flow =	0.04	MGD	
=	27	gpm	
Minimum Daily Flow =	0.01	MGD	
	5	gpm	

2. Wet Well Design Calculations:

Pump Capacity (q) =	150	gal/min.	
Number of Pumps =	2		Pump Capacity OK
Firm Capacity =	150	gal/min.	
Minimum req. time between pump starts (θ) =	6	minutes	TCEQ Table C.4 - Minimum Pump Cycle Times. HP < 50 = 6 minutes. HP 50 - 100 = 10 minutes. HP > 100 = 15 minutes. Based on PWWF.
Minimum wet well working volume =	225	gallons	= q x θ / 4 (Minimum volume between lead pump on and all pumps off)
Using Peak Wet Weather Flow =	108	gals/min (MDF)	
Net outflow in gallons per minute =	42	(1 pump running)	= q - PWF
Using Peak Wet Weather Flow =	108	gals/min (MDF)	
Net outflow in gallons per minute =	42	(2 pumps running)	= q - PWF
Using Average Daily Flow (ADF) =	27	gals/min (ADF)	
Net outflow in gallons per minute =	123	(1 pump running)	= q - ADF
Using Minimum Daily Flow (ADF) =	5	gals/min (ADF)	
Net outflow in gallons per minute =	145	(1 pump running)	= q - ADF

3. Wet Well Detention Times and Pump Cycle Times

Wet Well Working Volume Required: 225 gallons
 Working Volume Depth: 4.00 ft
 Wet Well Diameter: 12 ft
 Calculated Working Volume: 3,384 gallons

Volume OK
 (when including calculated volume between
 lead & lag pumps)

Average Flow Detention Time: (1 pump running)

$T_f = 125$ min.
 $T_e = 28$ min.
 $T_d = 153$ min.
 Daily Cycles = 10

Minimum Flow Detention Time: (1 pump running)

$T_f = 677$ min.
 $T_e = 23.3$ min.
 $T_d = 700$ min.

Odor control will be required since detention time > 180 minutes

Minimum Time Between Pump Starts (results when inflow = 0.5*Pump Capacity): (1 pump running)

Inflow = 75 gpm
 $T_f = 45.12$ min. per pump
 $T_e = 45.12$ min. per pump
 Cycle Time = 180 min. from pump start to pump start
 Daily Cycles = 8

Minimum Cycle Time = 180.5 > = 6 OK

4. Total Dynamic Head Calculations:

Discharge Pipe: **35** linear ft
Roughness, c = 120
Inner Diameter, d = **3.91** in (4" Class 53)

Material: **Class 53 Ductile Iron (when Flanged & Exposed)**
Class 350 Ductile Iron (when Restrained & Buried)
Friction Losses, H_f (ft) = $0.2083 * (100 / c)^{1.85} * (Q^{1.85} / d^{4.8655}) * (\text{Length}/100)$

PEAK FLOWS, EACH PUMP (1 pumps running)
Flow, q = 108 gpm
f = 1.129 feet /100 ft of pipe
Total Friction Loss = **0.40** feet, both pipes

MAXIMUM FLOW EACH PUMP (1 pump running)
Flow, q = 150 gpm
f = 2.074 feet /100 ft of each pipe
Total Friction Loss = **0.73** feet

Minor Losses in Discharge Pipe:

Minor Losses, $H_{f,m}$ (ft) = $K * V^2 / 2g$

Number of 90 deg. bends = **3**
Number of 45 deg. bends = **6**
Number of 22.5 deg. bends = **0**
Number of thru flow tees = **2**
Number of branch flow tees = **1**
Number of Swing CVs = **1**
Number of Plug Valves = **1**
Total K each discharge pipe = **7.1**
Total Minor Losses = **1.85** feet (considering peak inflows, two pumps running)

K = 0.54 for 90 deg. bends
K = 0.29 for 45 deg. bends
K = 0.16 for 22.5 deg. bends
K = 0.36 for thru flow tee
K = 1.08 for branch flow tee
K = 1.71 for Swing Check Valve
K = 0.31 for Plug Valve, open full

Minor Loss K coefficients calculated based on Equivalent Lengths & Swamee-Jain Friction Factors, i.e. pipe material, diameter, fitting types, material roughness and Reynold's No. Detailed calculations shown on other calculation sheet.

Force Main: **9,579** linear ft
Roughness, c = 120
Inner Diameter, d = **3.786** in (4" PVC sch 80)

Material: **DR-25, CL 165, C905 PVC**
Friction Losses, H_f (ft) = $0.2083 * (100 / c)^{1.85} * (Q^{1.85} / d^{4.8655}) * (\text{Length}/100)$

MIN FLOW FOR FORCE MAIN (1 pump running)
Flow, q = 108.00 gpm
f = 1.32 feet/100 ft of pipe

MAX FLOW FOR FORCE MAIN (1 pump running)
Flow, q = 150 gpm
f = 2.43 feet/100 ft of pipe

Velocity = 3.1 fps
Okay.
(3 to 6 fps is acceptable to TCEQ)

Velocity = 4.3 fps
Okay.
(3 to 6 fps is acceptable to TCEQ)

Total Friction Loss = **126.5** feet

Total Friction Loss = **232.4** feet

Minor Losses in Force Main:

Minor Losses, $H_{f,m}$ (ft) = $K * V^2 / 2g$

Number of 90 deg. bends = **1**
Number of 45 deg. bends = **6**
Number of 22.5 deg. bends = **4**
Number of thru flow tees = **3**
Number of branch flow tees = **2**
Number of Swing CVs = **1**
Number of Plug Valves = **5**
Number of Gate Valves = **3**
Number of Pipe Exits = **1**
Total K force main = **7.0**
Total Minor Losses = **1.97** feet (considering two pumps running)

K = 0.33 for 90 deg. bends
K = 0.18 for 45 deg. bends
K = 0.10 for 22.5 deg. bends
K = 0.22 for thru flow tee
K = 0.66 for branch flow tee
K = 1.05 for Swing Check Valve
K = 0.18 for Plug Valve, open full
K = 0.08 for Gate Valve, open full
K = 1.00 for Pipe Exit

Minor Loss K coefficients calculated based on Equivalent Lengths & Swamee-Jain Friction Factors, i.e. pipe material, diameter, fitting types, material roughness and Reynold's No. Detailed calculations shown on other calculation sheet.

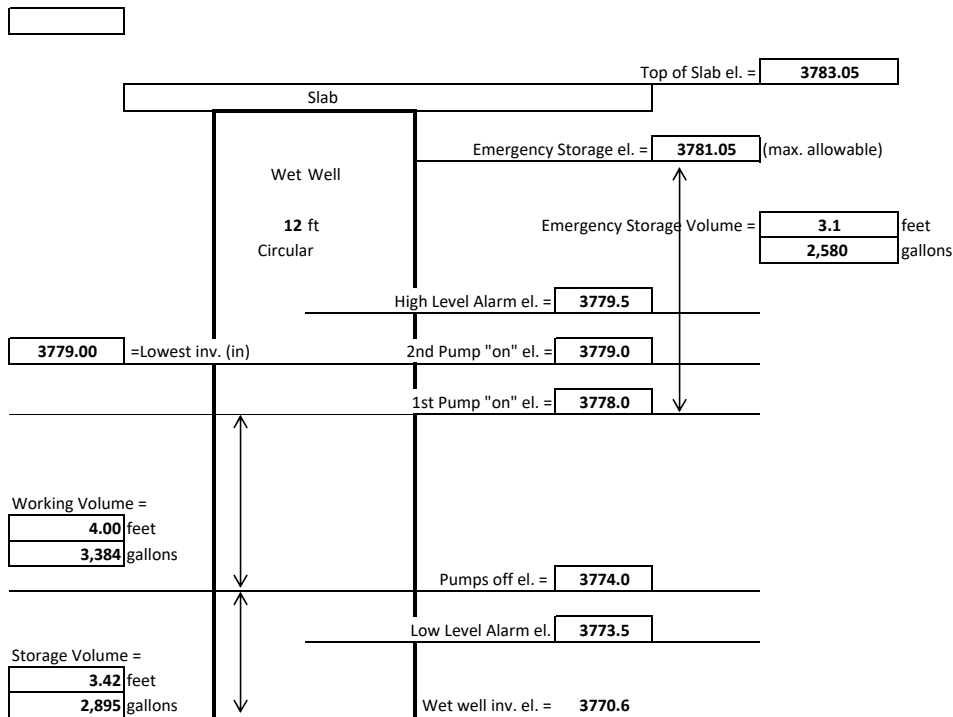
Elevation Head:

Discharge Header EL =	3784.1	MSL	194.0	Static Head on Header (ft)
Slab EL =	3783.1	MSL	84.0	psi
	3781.1	MSL	=	Max. Emergency Storage EL (Elevation lower than lowest upstream MH rim elevation)
	3779.5	MSL	=	High Level Alarm EL
Lowest Invert EL =	3779.00	MSL		influent flowline elevation
	3780.0			
	3779.00	MSL	=	2nd Pump "On" EL
	3778.00	MSL	=	1st Pump "On" EL
			Working Volume =	4.00 feet
Pumps off EL =	3774.0	MSL or	3774.0	= Lowest elevation of FM (if < WW inv.)
Low Level Alarm EL =	3773.5			
Min Water Level EL =	3773.25		2.67	Min Water Level Allowed from Invert (ft)
Wet Well Invert EL =	3770.6	MSL	=	12.5 Wet well Depth (feet)
			5.1	feet from 1st "on" stage to slab
FM Outfall EL =	3978.00	MSL =	204	feet of elevation head (Maximum)
			199	feet of elevation head (Minimum)
	428.28	Dynamic Head on Header (ft)		
	185.4	psi		

Summary of Head Losses (Two Pumps Running):			
Force Main Friction Loss =	232.4		
Force Main Minor Losses =	1.97		
Discharge Pipe Friction Loss =	0.40		
Discharge Pipe Minor Losses =	1.85		
Elevation Head =	204.0	feet	
Total Dynamic Head, TDH =	441	feet	

5. Emergency Storage Provisions:

Wet Well Size =	12	feet	Circular
Max. Allowable Storage Elev. =	3781.1	MSL	
Depth used for Storage =	3.1	feet (1st Pump "on" to the maximum allowable elev.)	
Wet Well Storage Volume =	2,580	gallons	
Manhole Diameter =	4	feet	
Accumulated MH depth used for Storage =	159	Vertical feet	
Manhole Storage Volume =	14,945	gallons	
Total Storage Volume =	17,526	gallons	
Using an ADF inflow of	27	gpm,	Storage Duration = 649 minutes
Using a PWF inflow of	108	gpm,	Storage Duration = 162 minutes



8. Force Main Flush Time:

$$\text{Number of Pumping Cycles necessary to completely change water in Force Main} = \frac{\text{Volume of water in Force Main (gal)}}{\text{Gallons of Outflow per Pump Run Time (gal)}} = 1.6$$

$$\text{Flush Time} = \frac{(\text{Number of Pumping Cycles necessary to completely change water in Force Main}) \times (\text{Minimum Daily Flow Cycle Time})}{1}$$

$$T_{\text{flush}} = \frac{\text{Volume of water in Force Main (gal)} \times \text{Minimum Daily Flow Cycle Time (min)}}{\text{Working Volume of Wet Well (gal)}}$$

Pump Outflow =	150	gpm (1 pump running maximum flow)
Pump Run Time =	23.3	minutes for minimum daily inflows (time to empty)
Outflow per Pump Run Time =	3,501	gallons
Vol. of water in FM =	5,602	gallons = length x pi x d ² / 4 x 7.48 gal/c.f.
Minimum Daily Flow Cycle Time =	700.1	min
Working Volume of Wet Well =	3,384	gallons

$$T_{\text{flush}} = 714 \text{ min}$$

Odor control will be required.

9. Water Hammer Calculations:

Discharge Header Pipe:

$$\text{Water hammer pressure, } p = \frac{(a) \times (v)}{2.31 \times (g)} + \text{operating pressure of pipe (psi)}$$

a = pressure wave velocity (fps)

w = specific weight of water (62.4 lbs/ft³)

g = acceleration of gravity (32.2 ft/sec²)

k = bulk modulus of water (311,000 psi)

d = inside diameter of pipe (inches)

E = Young's modulus of pipe (psi)

t = pipe wall thickness (inches)

v = flow velocity of pipe (fps)

$$a = \frac{12}{[(w/g) \times ((1/k) + (d/Et))]^{0.5}}$$

operating pressure = TDH in psi = 190.9 psi

The following was calculated using the properties of the 10-inch DI CL 350 pipe:

where: d = 3.91 inches

E = 24,000,000 psi

t = 0.26 inches

v = 4.01 fps

a = 4,398 fps

p = 427.9 psi

Ductile Iron Society, *Ductile Iron Data for Design Engineers* (2013)
Class 350 Ductil. US Pipe, *Ductile Iron Pipe Design for Fire Protection, Water & Wastewater* (2013), Table 2.

Highest velocities for old system

Okay

For Ductile Iron Pipe, rated working stress + 100 psi surge pressure = 450 psi

US Pipe, *Ductile Iron Pipe Design for Fire Protection, Water & Wastewater* (2013), Table 2.

Water Hammer Calculations - Force Main:

$$\text{Water hammer pressure, } p = \frac{(a) \times (v)}{2.31 \times (g)}$$

a = pressure wave velocity (fps)

g = acceleration of gravity (32.2 ft/sec²)

k = bulk modulus of water (311,000 psi)

d = inside diameter of pipe (inches)

E = Young's modulus of pipe (psi)

t = pipe wall thickness (inches)

v = flow velocity of pipe (fps)

$$a = \frac{12}{[(w/g) \times ((1/k) + (d/Et))]^{0.5}}$$

operating pressure = TDH in psi = 190.9 psi

where: d = 3.79 inches

E = 420,000 psi

t = 0.978 inches

v = 4.28 fps

a = 2,445 fps

p = 331.4 psi

Georg Fischer Harvel & Professional Plastics

DR-25, CL 165, C905 PVC Georg Fischer Harvel & Professional Plastics

Highest velocities allowable in force main (highest flows allowable)

Surge control measures sha [Calculation example here](#)

For C900 DR25 CL165 pipe, rated working pressure = 165 psi

10. NSPH Calculations

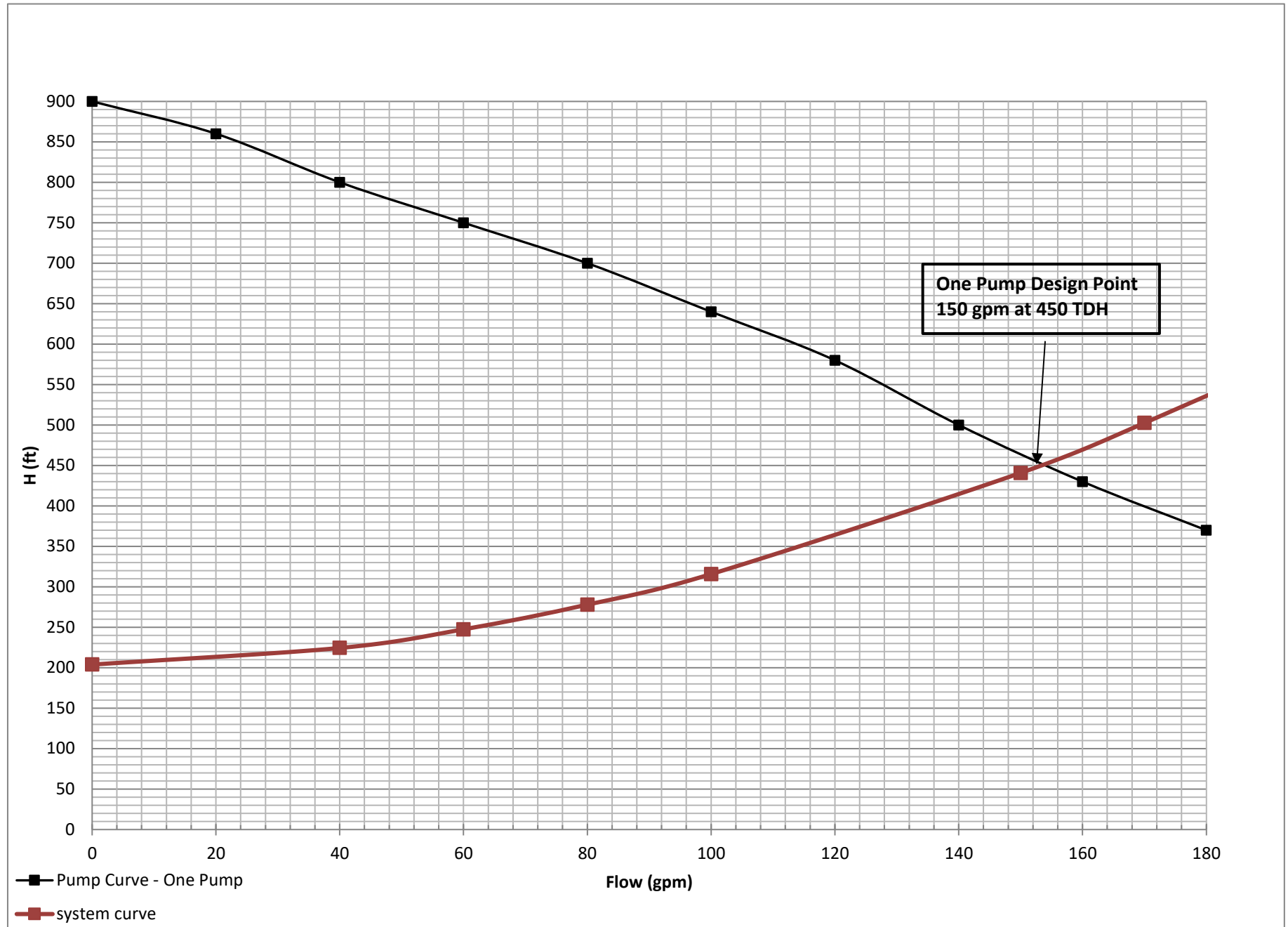
NPSHA = Atmospheric Pressure (ft) + Static Head + Surface Pressure Head - Vapor Pressure of Product - Losses in Suction Piping, Valves & Fittings

14.7	Atmospheric Pressure (psi)	Same for Kings #1, PH I
34.0	Atmospheric Pressure (ft)	Same for Kings #1, PH I
3.4	Static Suction Head (ft) MINIMUM	Same for Kings #1, PH I
8.4	Static Suction Head (ft) MAXIMUM	Same for Kings #1, PH I
0.0	Gage Pressure (the tank is open to atmospheric pressure)	Same for Kings #1, PH I
0.27	Vapor Pressure of Water at 68 def F (psia)	Same for Kings #1, PH I
0.62	Vapor Pressure of Water at 68 def F (ft)	Same for Kings #1, PH I
1.0	WW Specific Gravity	Same for Kings #1, PH I
0.0	Suction Piping Losses	Same for Kings #1, PH I
36.8	NPSHA using MINIMUM static suction head	Same for Kings #1, PH I
41.8	NPSHA using MAXIMUM static suction head	Same for Kings #1, PH I

12.0	NSPHR from pump curve at flows of 1,140 gpm each (ft)
13.02	NSPHR from pump curve at maximum flows of 1,710 gpm, one pump running (ft)
15	NSPHR from pump curve at new system maximum flows of 1,975 gpm, one pump running (ft)
32	NSPHR from pump curve at maximum possible flows of 3,000 gpm, two pumps running, new system (ft)

NSHPA > NSHPR

Lift Station-2 Duplex System Curve and Pump Curve at 25 HP





Appendix D – Drainage Reports

GLCA Hydrologic and Hydraulic Memo

Date: 12/20/2022
Project name: Glen Canyon Low Water Ramps
Task Order no: 140P2022F0267
Contract no: 140P2021D0007
Attention: Mike Will, PE, Denver Service Center
Company: National Park Service
Prepared by: Ritu Gupta, PE, CFM
Reviewed by: Thom Rutledge, PE, CFM
Copies to: Aaron Swafford, PE, Douglas P. Stewart, PE

OBJECTIVE

This memo has been prepared to summarize the hydrologic analysis, parameters, and assumptions that were used to determine the peak flow rates to prospective culvert crossings and ditch sizing along the roadway within the project area. Major basins were delineated to potential cross-culvert locations placed at natural low points along the roadway alignment. Sub basins were delineated to calculate the amount of flow in roadside ditches tributary to the prospective culvert crossings.

HYDROLOGIC ANALYSIS

The offsite hydrology was completed using the SCS unit hydrograph method applied within the HEC-HMS model version 4.10. Hydrologic analysis for the ditch sizing was completed using the Rational Method.

Subbasin Delineation and Topography

Major basins were delineated to estimate the concentrated flow at a potential culvert location. Smaller subbasins were delineated to estimate the flow that would drain directly towards the proposed roadway alignment and will need to be conveyed via ditches to one of the potential culvert locations. Major basins and ditch subbasin boundaries are shown on **Figure 1** included as **Attachment 1** with this memo.

The topography data used for the subbasin delineation included local LiDAR data flown for the project and the 10-meter USGS DEM data. ArcGIS Arc Hydro toolbox was used for an initial estimate of the overall watershed. The subbasin boundaries were then manually adjusted to account for the proposed project alignment.

Rainfall Data (Frequency, Depth, Distribution)

The hydrologic calculations are completed for the 10-year, 50-year and 100-year storm event. The rainfall data is referenced from National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 1, Version 5 Point Precipitation Frequency Estimate for Lake Powell, Utah, located at latitude 37.5195 and longitude -110.6937. The rainfall values corresponding to each storm event are provided in Table 1 below:

Table 1: Rainfall Data

Return Period (Yr)	Rainfall Intensity* (inch/hr)	Rainfall Depth** (inch)
10	2.68	1.35
50	4.39	1.90
100	5.35	2.16

*Rainfall Intensity corresponding to 5-minute time of concentration (tc) is listed and the rainfall intensity corresponding to calculated tc was interpolated based on available rainfall intensity values

**Rainfall depth values are corresponding to 24-hour storm duration

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Soils

The soil data was downloaded from ArcGIS online content. The referenced dataset was developed using the STATSGO2, 1:250,000-scale US soils database with soil descriptions from the NRCS website (<http://ortho.ftw.nrcs.usda.gov/cgi-bin/osd/osdname.cgi>). The main soil characteristics described by this dataset are soil order, texture, climate regime and caliche (special diagnostic horizon). This dataset is also uploaded to Data Basin and is available with additional information at: <http://app.databasin.org/app/pages/datasetPage.jsp?id=de1a45d142f34bbca8010903eef966d9>.

Based on the data, the project watershed lies in the Hydrologic **Soil Group Type D**. A small portion of the watershed lies within the area identified as water. For calculations, Soil Group Type D was designated in that region as well.

Land Use Data

The land use data was downloaded from the United States Geological Survey (USGS) National Land Cover Database (NLCD) 2019 Land Cover Conterminous United States. The land use categories that cover the project watershed are listed in **Table 2**. The table includes the curve number for each landuse category for soil group type D. The curve numbers are referenced from the *Urban Hydrology for Small Watersheds TR-55* by United States Department of Agriculture (USDA). The corresponding name for each land use category used in TR-55 is also provided in **Table 2**.

Table 2: Landuse Categories

NLCD Color Index	NLCD Land Cover Class	TR 55 Landuse Category	CN (Soil Type D)
11	Open water	-	100
21	Developed, open space	Open Space (Fair Condition)	84
22	Developed, Low Intensity	Residential District (2 acres)	82
23	Developed, Medium Intensity	Residential District (1/3 acres)	86
24	Developed, High Intensity	Residential District (1/8 acres)	92
31	Barren Land	Fallow - Bare Soil	94
42	Evergreen Forest	Pinyon-Juniper (Fair Condition)	80
52	Shrub/Scrub	Desert Shrub (Fair Condition)	86
71	Herbaceous	Herbaceous (Fair Condition)	89
90	Woody Wetlands	Wood (Fair Condition)	79
95	Emergent Herbaceous Wetlands	Wood-Grass (Fair Condition)	82

Curve Numbers

Rainfall losses due to soil infiltration, depression storage, and other factors were calculated using the NRCS curve number method, which is a function of soil type, land cover, and antecedent moisture condition.

GIS zonal statistical tool was used to calculate a weighted curve number for each subbasin using the subbasin delineations, soil map unit delineations, the land use layer and curve number matrix. The initial abstraction was assumed to be zero.

Lag Times

Based on United States Department of Agriculture, National Engineering Handbook - Part 630 Hydrology, Lag time (T_l) is defined using following equation:

$$T_l = 0.6T_c$$

where, T_c =time of concentration in minutes

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T_c is calculated using the definition provided in TR55 which divides T_c into two components. Initial sheet flow is calculated using Manning's kinematic solution (Overtop and Meadows 1976) and the shallow concentrated flow is calculated using velocity method that uses empirical relationship between the velocity and the slope.

Initial flow:

$$t_c = \frac{\alpha}{P_2^{0.5}} \left(\frac{nL}{\sqrt{S}} \right)^{0.8}$$

where,

α	=	unit conversion constant equal to 0.42
n	=	roughness coefficient (corresponding to fallow land is 0.05)
L	=	flow length (feet)
P_2	=	2-year, 24-hour rainfall depth (0.872 in)
S	=	slope of the surface (ft/ft)

Shallow concentrated flow:

$$T_t = \frac{L}{60V}$$

where,

L	=	flow length (feet)
V	=	velocity, ft/s = $\alpha K S^{0.5}$
α	=	unit conversion constant equal to 33
K	=	dimensionless function of land cover (Nearly bare and untilled overland flow & alluvial fans in western mountain regions = 0.305)
S	=	slope of the surface (ft/ft)

HYDROLOGY ANALYSIS AND PRELIMINARY RESULTS

After developing the input parameters described above, the HEC-HMS model was compiled, and the simulation was successfully run for 10-year, 50-year and 100-year storm events. The offsite subbasins, corresponding peak flows and preliminary culvert locations are shown in Figure 1 included with this memo.

HYDROLOGIC AND HYDRAULIC ANALYSIS FOR DITCHES

The flows for ditch subbasins were estimated using Rational Method which states that,

$$Q = CiA$$

where,

Q	=	flow (cfs)
C	=	runoff coefficient (10yr = 0.1; 100yr = 0.3)
i	=	rainfall intensity (inch/hr)
A	=	area (acres)

The runoff coefficient, C is referenced from Federal Highway Administration, Hydraulic Engineering Circular No. 22 (FHWA HEC-22). The values were taken corresponding to the landuse category 'undeveloped area'. The ditches were sized for the 100-year flow using Manning's equation. The ditch location, flow directions, design flows and calculated ditch sizes are provided in a summary table included as **Attachment 2** with this memo.

SUMMARY

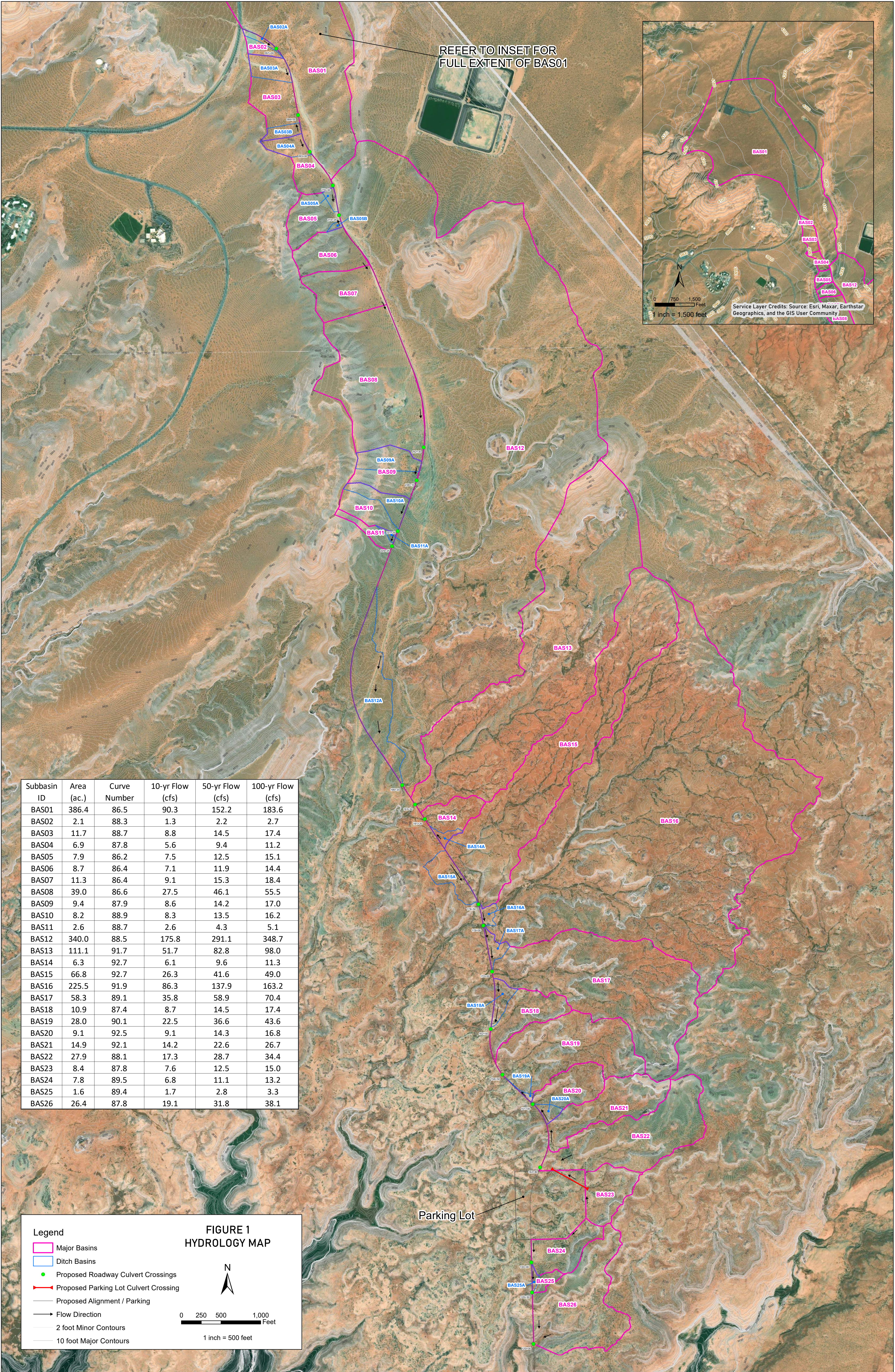
The intent of this memo is to facilitate a review by NPS of the hydrologic and hydraulic analysis, methodology and assumptions. Potential cross-culvert locations and preliminary sizes of the roadside ditches along the proposed

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project alignment have been provided. Following a review of this memo, Jacobs will update the hydrologic and hydraulic analysis as the project design progresses.

Attachments:

- Figure 1 – Hydrology Map
- Ditch Calculations



ATTACHMENT 2 - Ditch Calculations and Summary Table

Basin ID	Direction of Flow	Station from	Station to	Area (acres)	Slope (ft/ft)	Tc (min)	Intensity 10yr (in/hr)	Intensity 100yr (in/hr)	Q 10 (cfs)	Q 100 (cfs)	Ditch Geom	Manning's n	Bottom Width (ft)	Side slope	Depth of flow 10-yr (ft)	Depth of flow 100-yr (ft)	
BAS02A	South	302+00	295+50	0.69	0.0200	7.6	2.34	4.68	0.2	1.0	V Ditch	0.035	0	4	0.18	0.36	
Culvert Crossing @ 297+00																	
BAS03A	South	295+50	292+25	3.29	0.0391	5.95	2.56	5.12	0.8	5.1	V Ditch	0.035	0	4	0.30	0.59	
BAS03B	North	285+75	287+00	1.58	0.0391	5	2.68	5.18	0.4	2.5	V Ditch	0.035	0	4	0.23	0.45	
Culvert Crossing @ 288+00																	
BAS04A	South	285+75	282+63	2.59	0.0391	5.65	2.6	5.35	0.7	4.2	V Ditch	0.035	0	4	0.28	0.55	
Culvert Crossing @ 283+00																	
Culvert Crossing @ 278+00																	
BAS05A	South	277+75	274+25	0.73	0.0391	5	2.68	5.35	0.2	1.2	V Ditch	0.035	0	4	0.17	0.34	
BAS05B	North	272+50	274+25	0.29	0.0391	5	2.68	5.35	0.1	0.5	V Ditch	0.035	0	4	0.12	0.24	
Culvert Crossing @ 274+25																	
BAS06+BAS07+BAS08	South	266+75	242+30	54.55	0.0391	Flow is calculated using HMS model				43.7	88.3	Trap	0.035	4	4	0.93	1.31
Culvert Crossing @ 242+30																	
BAS09A	South	240+50	239+25	4.66	0.0121	5	2.68	5.35	1.2	7.5	V Ditch	0.035	0	4	0.43	0.85	
Culvert Crossing @ 238+10																	
BAS10A	South	235+50	231+50	3.54	0.0121	6.31	2.51	5.01	0.9	5.3	V Ditch	0.035	0	4	0.38	0.75	
Culvert Crossing @ 231+25																	
BAS11A	South	230+50	229+50	0.36	0.0121	5	2.68	5.35	0.1	0.6	V Ditch	0.035	0	4	0.17	0.32	
Culvert Crossing @ 229+25																	
BAS12A	South	217+00	212+38	15.22	0.0574	38.3	1.09	2.18	1.7	10.0	V Ditch	0.035	0	4	0.36	0.71	
BAS12A	South	212+38	212+38	15.22	0.0199	38.3	1.09	2.18	1.7	10.0	V Ditch	0.035	0	4	0.44	0.86	
Culvert crossing @196+20																	
Culvert crossing @193+50																	
Culvert crossing @191+20																	
BAS14A	North	191+20	187+00	0.84	0.0057	16.6	1.61	3.21	0.1	0.8	V Ditch	0.035	0	4	0.22	0.43	
BAS15A	South	187+00	178+50	4.27	0.0070	14.3	1.75	3.50	0.7	4.5	V Ditch	0.035	0	4	0.40	0.78	
Culvert crossing @178+50																	
BAS16A	South	178+50	176+00	0.99	0.0070	11.7	1.89	3.79	0.2	1.1	V Ditch	0.035	0	4	0.24	0.46	
Culvert crossing @176+00																	
BAS17A	South	176+00	170+00	1.56	0.0070	10.0	2.03	4.07	0.3	1.9	V Ditch	0.035	0	4	0.29	0.56	
Culvert crossing @170+00																	
BAS18A	South	170+00	162+50	2.14	0.0070	18.9	1.53	3.07	0.3	2.0	V Ditch	0.035	0	4	0.29	0.57	
Culvert crossing @162+50																	
Culvert crossing @156+50																	
BAS19A	North	156+50	151+00	0.28	0.0050	5.2	2.68	5.35	0.1	0.4	V Ditch	0.035	0	4	0.18	0.35	
Culvert crossing @151+00																	
BAS21+BAS20A	North	151+00	146+00	1.10	0.0050	8.1	2.29	4.58	14.5	28.2	Trap	0.035	4	4	0.90	1.24	
Culvert crossing @142+80																	
Culvert crossing under Parking Lot - BAS23																	
BAS24	South	133+00	130+00	-	0.0050	HMS Basins South of Parking Area				6.8	13.2	V Ditch	0.035	0	4	0.97	1.24
Culvert crossing @130+00																	
BAS25A	South	130+00	126+50	0.41	0.0050	6.3	2.55	5.09	0.1	0.6	V Ditch	0.035	0	4	0.20	0.39	
Culvert crossing @126+50																	
Culvert crossing @120+00																	



Appendix E – Bullfrog Traffic Data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	% Change
2018	3,831	2,701	6,210	9,301	15,099	20,994	11,069	18,245	17,509	3,401	3,767	1,825	113,952	10.90%
2019	1,804	1,980	5,020	9,005	14,323	20,420	13,198	11,523	1,000	6,627	4,000	2,000	90,900	-20.23%
2020	2,248	2,581	5,025	5,033	9,854	12,392	13,176	9,662	11,787	7,451	1,982	1,035	82,226	-9.54%
2021	1,313	1,352	3,098	5,639	5,925	9,432	9,432	6,727	6,723	4,142	1,946	500	56,229	-31.62%
2022	1,000	1,000	1,747	3,041	4,853	6,721	7,459	4,991	5,568	3,142	1,287	851	41,660	-25.91%

	2018	2019	2020	2021	2022
Jan	3,831	1,804	2,248	1,313	1,000
Feb	2,701	1,980	2,581	1,352	1,000
Mar	6,210	5,020	5,025	3,098	1,747
Apr	9,301	9,005	5,033	5,639	3,041
May	15,099	14,323	9,854	5,925	4,853
Jun	20,994	20,420	12,392	9,432	6,721
Jul	11,069	13,198	13,176	9,432	7,459
Aug	18,245	11,523	9,662	6,727	4,991
Sep	17,509	1,000	11,787	6,723	5,568
Oct	3,401	6,627	7,451	4,142	3,142
Nov	3,767	4,000	1,982	1,946	1,287
Dec	1,825	2,000	1,035	500	851
Year	113,952	90,900	82,226	56,229	41,660

