Exhibit 19

HYDRAULIC CALCULATIONS

FOR
HIGH SCHOOL AT WILDCREEK
ORR DITCH RELOCATION

DECEMBER, 2018

PREPARED BY

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Prelim For
Review

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1.0 INTRODUCTION

These calculations represent the design of an inverted siphon diverting the Orr Ditch waterway across WildCreek Golf course to allow for future construction of the High School at WildCreek. An inverted siphon is a closed conduit designed to run full and under pressure sometimes referred to as sag pipes or sag lines. Trapezoidal

The purpose of this report is to analyze the existing flow patterns within the Orr Dich to identify design parameters of the inverted siphon. Accommodation of the varying flows within the Orr Ditch along with hydraulic design and future maintenance of the system were the primary focuses within the design process. This report includes the hydraulic analyses for existing ditch and proposed ditch/siphon conditions.

The project area is contained within the existing WildCreek Golf Course property located west of Sullivan Lane (APN 027-011-08), diverting approximately 5,630 linear feet of the Orr Ditch. The project includes construction of the inlet and outlet structures, inverted siphon and various maintenance operating structures such as a trash rack and overall system drain. The site is located within Section 32 of Township 20 North, Range 20 and is a part of the City of Sparks. It is bounded by Sullivan Lane to the west, McCarran Boulevard to the south, two parcels to the north (APNs 027-011-07, APN 035-080-04), and multiple private parcels to the east as shown on the included vicinity map in Appendix A.

1.1. LIMITATIONS

The following calculations were prepared for the limited purpose of presenting a hydraulic analysis for the High School at WildCreek Orr Ditch Relocation within the project area and is based upon available record drawings and reports, field investigation, and assumptions as listed below. The results and conclusions outlined in this report should not be relied on for purposes beyond those stated within.

1.2. PREVIOUS STUDIES

Previous studies were requested from the agencies and the following reports were provided and utilized as a reference.

- *Existing Orr Ditch Flow Measurements (Office of the Water Master – Truckee River System Daily Flow Record – 2012 to 2017)*
- *Preliminary Design Report for the High School at WildCreek completed by Wood Rodgers Inc., dated September 2017*
- *Design Report for the Sun Valley Flood Control Detention Dam prepared by SEA Inc., dated August 1987*

1.3. REFERENCES

Design References utilized in the Inverted Siphon Design

- *Design of Small Canal Structures (United States Department of the Interior – Bureau of Reclamation – A Water Resources Technical Publication1978): USBR*
- *Design Note Number 15 – Submerged Weir Flow (United States Department of Agriculture – Soil Conservation Service – Engineering Division, Design Branch 1973): USDA*
- *Open Channel Hydraulics (McGraw-Hill Civil Engineering Series 1959) : CHOW*
-

1.4. REGULATIONS AND COORDINATION

The project falls under the jurisdiction of the City of Sparks, Washoe County and the Orr Ditch Company.

Design for the inverted siphon has been completed following the guidance of the Design of Small Canal Structures (United States Department of the Interior – Bureau of Reclamation; 1978). This design guidance was used for multiple components of design: Inlet Structure, Outlet Structure, Siphon Sizing and Weir calculations. These hand calculations can be found in Appendices A through J.

2.0 METHODOLOGY

2.1. INVERTED SIPHON

This inverted siphon has been designed following the guidelines and design procedures of the Design of Small Canal Structures (United States Department of the Interior – Bureau of Reclamation; 1978). Available head, economy, and allowable pipe velocities determine the size of the inverted siphon pipe. Utilizing the Manning's equation and understanding the hydraulic losses associated with long siphons is key to an efficient design. Design Calculations are included in Appendix G.

The operating flows required to design the siphon were acquired from the Office of the Water Master – Existing Orr Ditch Flows included in Appendix B. There are three primary operating flows for the Orr Ditch: Average flow of 15 cubic feet per second (cfs), Flushing Flow of 40 cfs and the Maximum Flow of 51 cfs. These flow scenarios match historical flow data provided in the Water Master Reports. The average flow is taken as the operating flow that the Orr Ditch sees throughout its season and can vary from 9 cfs to 16 cfs. The flushing flow is a scenario seen during ditch maintenance for clearing/cleaning operations. The maximum flow is based upon storm event flow that has been determined through review of the historical flow data. This flow has been chosen as the maximum flow design for this inverted siphon system. The final design flow for the inlet structure is a theoretical maximum capacity flow within the ditch immediately upstream of the inlet structure. This capacity flow is calculated at 160 cfs. This flow will be diverted through and past the inlet through an overflow structure, away from the siphon. Flow calculations for the existing Orr Ditch are included in Appendix C.

We accomplished design of a system to handle these varying flows with a double barrel weir controlled inlet structure effectively spreading flow to appropriately sizes siphons. For the average daily flow a single 24" Steel pipe is sufficient and has the capacity to handle up to 15 cfs. During flushing flow scenarios and maximum storm events, the water will back up in the inlet structure and flow over a weir to access a 36" Steel pipe that has the capacity to handle the remaining 38 cfs. Any additional flow provided to the system from a storm event and not from Orr Ditch operations will be released out an overflow spillway and spread over the golf course as it has historically. The flow calculations and hydraulic losses can be seen in the calculations provided in Appendix G. These losses were calculated using Manning's equation for friction losses, minor head losses, and exit/entrance head losses. Understanding these losses determined our pipe size and overall pipe material needed to have additional operating head throughout the system. Inlet structure and transition into the siphon are required to provide minimal hydraulic entrance losses and proper siphon operation. This is accomplished by ensuring a hydraulic seal on the pipe to ensure full flow conditions under pressure. This is further explained below. Calculations for this structure can be seen in Appendix E.

2.2. INLET STRUCTURE

The inlet structure has been designed following the guidelines and design procedures for the Design of Small Canal Structures (United States Department of the Interior – Bureau of Reclamation; 1978). Controlling design for the inlet structure is the transition to the siphon. This design sets the water surface elevation requirements through the rest of the inlet system. From this we determined the rectangular channel dimensions to ensure depths in the channel to obtain a hydraulic seal on the pipes regardless of flow scenarios. An 8'x6' reinforced concrete rectangular channel accommodates these depth requirements, calculations have been included in Appendix E. The four separate flow scenarios set the requirements for the other inlet structure components. For the 24" system, an eight (8) foot wide sharp crested weir is designed to control the water surface elevation under average flow conditions. For the 36" system, an eight (8) foot long side weir spillway has been designed to accommodate the extra flow during flushing and maximum flow conditions. The final overflow twelve (12) foot wide sharp crested weir has been designed to handle all other flow conditions that exceed the maximum Orr Ditch operation flows. This overflow weir was sized to control and direct any excess water to the historical drainage areas currently located over the WildCreek Golf Course. The weir loading calculations and water surface elevations can be found in Appendix E.

2.3. OUTLET STRUCTURE

The outlet structure has been designed following the guidelines and design procedures for the Design of Small Canal Structures (United States Department of the Interior – Bureau of Reclamation; 1978). The outlet structure is comprised of a double barrel outlet system that ensures proper operation during all three flow scenarios. The outlet structure was designed to control backward flow during average flow conditions to limit standing water in the 36" system when it is not needed. This was accomplished with a submerged weir that controls the 36" outlet structure water surface elevation. Controlling channel velocities was accomplished by the overall design of the outlet structure dimensions and a 1.17' high bottom step within the channel. All velocities within the concrete section of the outlet structure are slowed to less than 2 feet per second (fps) prior to release in the existing earthen channel to reduce downstream erosion. The weir loading calculations and water surface elevations can be found in Appendix H.

2.4. SOFTWARE APPLICATIONS

Bentley System FlowMaster V8i was utilized for the hydraulic modeling of the existing and proposed Orr Ditch Channel. Previous reports, field surveying and site investigation were utilized in modeling the flow conditions previously referred to in this report.

3.0 HISTORICAL ORR DITCH CONVEYANCE

The historical flow data was acquire from the Office of the Water Master, Truckee River System Daily Flow Record 2012 to 2017, included in Appendix B. This flow data values were used to come up with the three flow conditions that our system would encounter during normal operation of the Orr Ditch. These values have been coordinated with the the Orr Ditch Company during the design process.

4.0 PROPOSED FACILITIES

4.1. INVERTED SIPHON

A single 24" Steel pipe is sufficient to handle 13 cfs equal to the average flow scenario. A single 36" Steel pipe is sufficient to handle remaining 27 cfs or 38 cfs equal to the flushing flow or maximum flow scenarios. Any additional flow provided to the system from a storm event and not from normal Orr Ditch operations will be released out an overflow spillway and spread over the golf course as it has historically. Inverted Siphon calculations are included in Appendix G.

4.2. INVERTED SIPHON LOW POINT DRAIN AND POND

A single 12" outlet drain pipe with Baffled outlet structure has been sized to control the outlet velocity of the inverted siphon during maintenance and shutdown procedures. The pond that this drains into has been sized to accommodate the 215 cfs flows from the Sun Valley Dam and is sufficient to spread the flow over a spillway to resemble historical flooding conditions. This flood event volume was taken from the Design Report for the Sun Valley Flood Control Detention Dam prepared by SEA Inc., dated August 1987. Inverted Siphon low point drain and baffled outlet structure calculations are included in Appendix G.

4.3. INLET STRUCTURE

The inlet structure has been designed to have the corresponding components: 8'x6' reinforced concrete rectangular channel, double barrel inverted siphon inlet type 5 transitions per the Design of Small Canal Structures, 8' sharp-crested weir with maintenance access catwalk, 8' side spillway weir with maintenance access catwalk, 12' overflow spillway with maintenance access catwalk, trash rack and reinforced concrete box section for excavator access during cleaning operations and a riprap protected transition connection to the existing earthen channel. Structure calculations are included in Appendix E. Rip Rap sizing calculations have been provided in Appendix F.

4.4. OUTLET STRUCTURE

The outlet structure has been designed to have the corresponding components: 8'x6' reinforced rectangular channel, double barrel inverted siphon outlet transitions per the Design of Small Canal Structures, 16.67' reinforced concrete rectangular channel, 12' submerged weir with maintenance access catwalk, 16.67'x1.17' channel bottom step, and a riprap protected transition connection to the existing earthen channel. Structure calculations are included in Appendix H. Rip Rap sizing calculations have been provided in Appendix J. System outlet velocity check calculation are included in Appendix I.

4.5. CONCLUSION / RECOMMENDATION

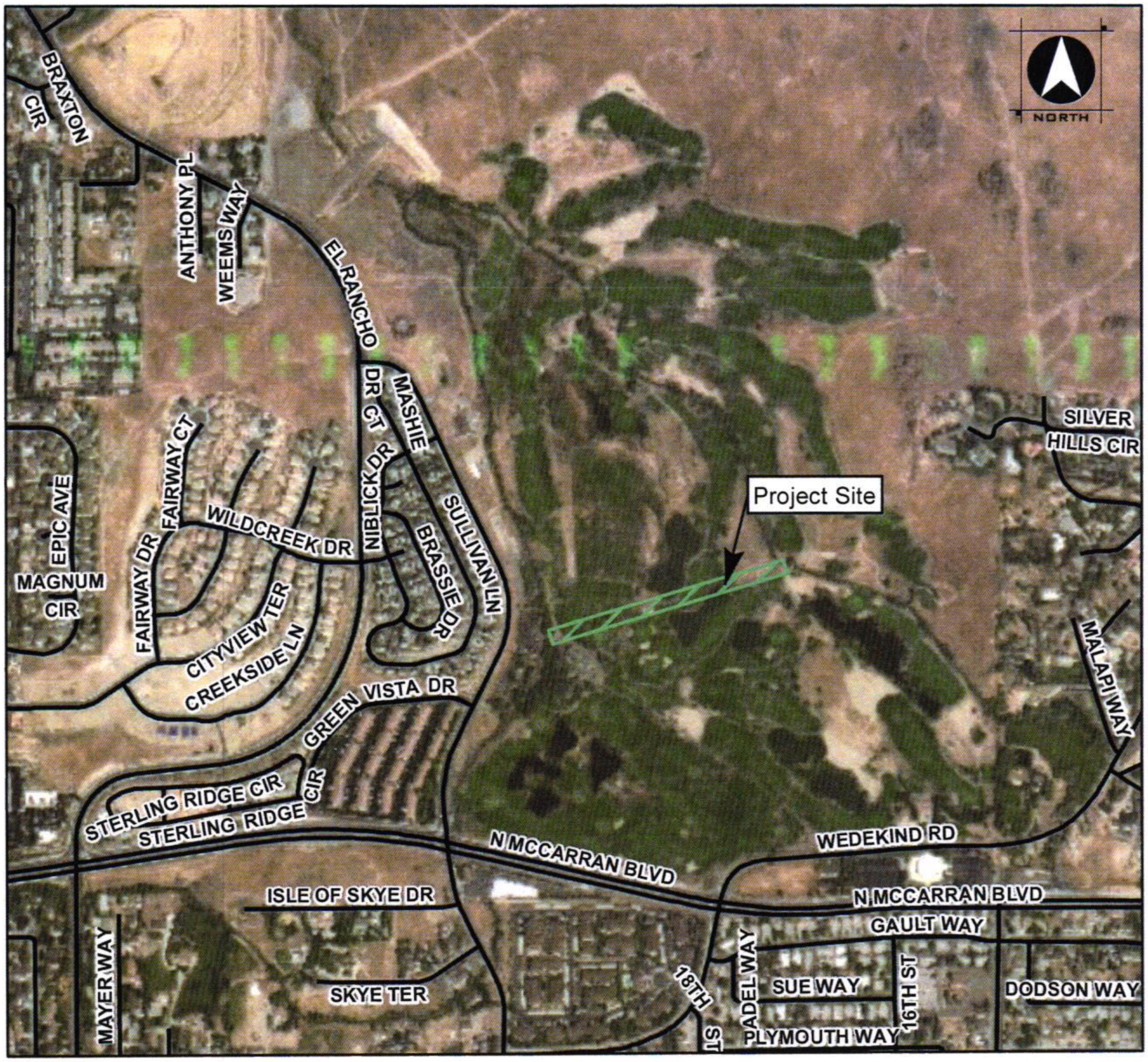
The Orr Ditch operates within an earthen channel providing irrigation water to downstream patrons. The construction of the proposed High School is reason to relocate the ditch across the golf course and this has been accomplished through the inverted siphon design. Impacts to downstream systems are negligible both for standard irrigation as well as during storm events.

5.0 APPENDIX A - Vicinity Map

The project area is contained within the existing WildCreek Golf Course property located west of Sullivan Lane. The site is located within Section 32 of Township 20 North, Range 20 and is a part of the City of Sparks. It is bounded by Sullivan Lane to the west and McCarran Boulevard to the south.

ORR DITCH RELOCATION
VICINITY MAP

SPARKS, NV
DECEMBER, 2018



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6.0 APPENDIX B - ORR DITCH HISTORICAL FLOW DATA

The Historical Flow data in this appendix shows the flows monitored monthly throughout the year. Agreed upon by the Orr Ditch Company. The 3 flow conditions are as follows:

- Average Flow Condition: 15 cfs
- Flushing Flow Condition: 40 cfs
- Maximum Flow Condition: 51 cfs

Another flow condition is theoretically possible during a storm event where the Existing Orr Ditch Channel is flowing full due to storm water contributing to the ordinary flow. This is the Capacity flow of the Orr Ditch and outside normal operational flows.

- Capacity Flow: 160 cfs

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2017

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01						0	13.3	12.6	12.6	0		
02						0	13.5	12.6	12.4	0		
03						0	13.2	12.6	12.5	0		
04						0	13.2	12.4	12.7	0		
05						0	13.2	12.5	12.5	0		
06						0	13.3	12.4	12.5	0		
07						13.2	13.3	12.6	12.8	0		
08						13.2	13.3	12.8	12.8	0		
09					0	13.3	13.5	12.8	12.8	0		
10					0	13.2	13.6	12.8	12.8	0		
11					0	13.2	13.7	12.5	12.5	0		
12					0	13.3	13.5	12.6	12.4	0		
13					0	12.4	12.8	12.8	12.4	0		
14					0	12	12.6	12.7	12.4	0		
15					0	11.9	12.8	12.6	12.7	0		
16					0	12	12.8	12.7	12.8	0		
17					0	13.2	12.8	12.8	12.8			
18					0	13.8	12.8	12.8	12.5			
19					0	13.9	12.4	12.7	12.6			
20					0	14.1	12.2	12.6	12.7			
21					0	13.9	12.1	12.8	12.6			
22					0	13.4	12.2	12.7	12.5			
23					0	13.3	12.4	12.8	13			
24					0	13.2	12.4	12.8	12.8			
25					0	13.2	12.4	12.6	12.4			
26					0	13.2	12.6	12.6	12.3			
27					0	13.2	12.8	12.8	12.3			
28					0	12.8	12.8	12.8	12.1			
29					0	13.3	12.8	12.8	11.6			
30					0	13.2	12.8	12.8	0			
31					0		12.8	12.8				
COUNT					**23**	**30**	**31**	**31**	**30**	**16**		
MAX					**0**	**14.1**	**13.7**	**12.8**	**13**	**0**		
MIN					**0**	**0**	**12.1**	**12.4**	**0**	**0**		
AVG					**0**	**10.51**	**12.9**	**12.68**	**12.13**	**0**		

A-F	**0**	**626**	**793**	**780**	**722**	**0**

S: Stock water E: Estimated R: Return

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2016

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01				0	0	12	11	12.9	16	0		
02				0	0	12	10.2	12.7	16	0		
03				0	0	12	10	11.7	15.8	0		
04				0	0	12	11.4	11.9	15.8	0		
05				0	0	12	11.1	12.8	15.7	0		
06				0	0	12	12.4	12.9	15.4	0		
07				0	0	12	12.6	14	15	0		
08				0	0	12.3	12.9	14.8	6.1	0		
09				0	0	12	12.4	14.5	0	0		
10				0	0	12	12.2	13.2	0	0		
11				0	0	12.6	12.2	12.6	0	0		
12				0	0	12.2	12.1	14.9	0	0		
13				0	9.1	12	12.1	14.1	0	0		
14				0	15.1	11.6	14.7	14.1	0	0		
15				0	14.9	12.3	12.7	13.5	0	0		
16				0	15.9	12	12	15.3	0	0		
17				0	16.5	12	12.1	14.9	0	0		
18				0	13	12	12.1	14.8	0	0		
19				0	10.4	11.3	12.5	15	0	0		
20				0	10.4	12	10.4	14.7	0	0		
21				0	12.6	12	10.2	14.5	0	0.2		
22				0	13.2	12	11.8	14.6	0			
23				0	13.1	12	12	20.8	0			
24				0	13.3	12	11.8	14.4	0			
25				0	12	12	11.8	14.3	0			
26				0	10.9	12	12.6	14.3	0			
27				0	9.5	12.2	13.5	15	0			
28				0	13.6	11.4	14.4	17.4	0			
29				0	11.9	12	13.5	17	0			
30				0	12	11.5	13.1	15.9	0			
31					11.7		12.9	15.9				
COUNT				**30**	**31**	**30**	**31**	**31**	**30**	**21**		
MAX				**0**	**16.5**	**12.6**	**14.7**	**20.8**	**16**	**0.2**		
MIN				**0**	**0**	**11.3**	**10**	**11.7**	**0**	**0**		
AVG				**0**	**7.71**	**11.98**	**12.15**	**14.5**	**3.86**	**0.01**		

A-F	**0**	**474**	**713**	**747**	**891**	**230**	**0**

S: Stock water E: Estimated R: Return

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2015

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01				0	12.4	14.1	0	0	0	0	0	
02				0	12.4	8.3	0	0	0	0	0	
03				0	12.5	0	0	0	0	0	0	
04				0	12.3	0	0	0	0	0	0	
05				0	12.4	0	0	0	0	0	0	
06				3.6	12.3	0	0	0	0	0	0	
07				5.3	12.4	0	0	0	0	0	0	
08				7	12.2	0	0	0	0	0	0	
09				8.6	12	0	0	0	0	0	0	
10				10.3	12	0	0	0	0	0	0	
11				12	12	0	0	0	0	0	0	
12				13.7	12	0	0	0	0	0	0	
13				15.4	11.8	0	0	0	0	0	0	
14				15.2	11.7	0	0	0	0	0	0	
15				15.4	11.7	0	0	0	0	0	0	
16				14	11.2	0	0	0	0	0	0	
17				13.3	10.7	0	0	0	0	0	0	
18				11.3	10.5	0	0	0	0	0	0	
19				9.7	10.4	0	0	0	0	0	0	
20				9.7	10.5	0	0	0	0	0	0	
21				10	10.8	0	0	0	0	0	0	
22				11.1	10.8	0	0	0	0	0	0	
23				12	10.9	0	0	0	0	0	0	
24				12.2	12.1	0	0	0	0	0	0	
25				12.6	12.9	0	0	0	0	0	0	
26				12.6	13.2	0	0	0	0	0	0	
27				12.4	13.6	0	0	0	0	0	0	
28				12.4	14.3	0	0	0	0	0	0	
29				12.1	14.2	0	0	0	0	0	0	
30			0	12.4	14.3	0	0	0	0	0	0	
31			0		14.1		0	0		0		
COUNT			**2**	**30**	**31**	**30**	**31**	**31**	**30**	**31**	**30**	
MAX			**0**	**15.4**	**14.3**	**14.1**	**0**	**0**	**0**	**0**	**0**	
MIN			**0**	**0**	**10.4**	**0**	**0**	**0**	**0**	**0**	**0**	
AVG			**0**	**9.48**	**12.15**	**0.75**	**0**	**0**	**0**	**0**	**0**	

A-F	**0**	**564**	**747**	**44**	**0**	**0**	**0**	**0**	**0**

S: Stock water E: Estimated R: Return

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2014

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01						13	13.5	0	0	0		
02					17.7	13	13.5	0	0	0		
03					17.1	13.2	13.5	0	0	0		
04					18	13.2	13.5	0	0	0		
05					17.9	13.2	13.5	0	0	0		
06					14.8	13.2	13.5	0	0	0		
07					13.2	13.3	13.6	0	0	0		
08					13	13.2	13.7	0	0	0		
09					13	13.3	13.7	0	0	0		
10					13.1	13.2	13.7	0	0	0		
11					13.4	13.2	13.7	0	0	0		
12					13.6	13.2	13.7	0	0	0		
13					13.3	13.3	13.7	0	0	0		
14					13.5	13.4	13.9	0	0	0		
15					13.5	13.3	13.7	0	0	0		
16					13.3	13.2	14	0	0	0		
17					13.3	13.4	14.1	0	0	0		
18					13.1	13.6	14	0	0	0		
19					13.2	13.7	13.8	0	0	0		
20					13	13.6	13.8	0	0	0		
21					13.4	13.8	13.8	0	0	0		
22					13.1	13.5	13.7	0	0	0		
23					12.9	13.4	13.9	0	0			
24					13	13.4	14	0	0			
25					13.2	13.5	14	0	0			
26					13.3	13.6	14.1	0	0			
27					13.3	13.7	14.3	0	0			
28					13.4	13.8	14.3	0	0			
29					13.5	13.7	14.1	0	0			
30					13.3	13.6	7.6	0	0			
31					13		0	0				
COUNT					**30**	**30**	**31**	**31**	**30**	**22**		
MAX					**18**	**13.8**	**14.3**	**0**	**0**	**0**		
MIN					**12.9**	**13**	**0**	**0**	**0**	**0**		
AVG					**13.88**	**13.39**	**13.16**	**0**	**0**	**0**		

A-F	**826**	**796**	**810**	**0**	**0**	**0**

S: Stock water E: Estimated R: Return

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2013

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01					0	11.7	11.8	12.5	12.6	0		
02					0	11.9	12.1	12.4	12.4	0		
03					0	11.9	12.4	12.5	12.4	0		
04					0	12	12.4	12.5	12.4	0		
05					0	12.1	12.2	12.7	12.7	0		
06					0	11.8	12.3	12.8	12.8	0		
07					0	11.9	12.4	12.8	12.8	0		
08					0	12	12.4	12.8	12.8	0		
09					0	12	12.5	12.6	12.8	0		
10					10.3	11.9	12.4	12.7	12.8	0		
11					16.4	11.8	12.6	12.7	12.6	0		
12					18.1	11.8	12.4	12.5	12.6	0		
13					19.2	11.9	12.4	12.4	12.7	0		
14					19.1	11.8	12.4	12.4	12.9	0		
15					16.4	11.8	12.4	12.4	12.9	0		
16					13.2	11.8	12.4	12.4	6.5			
17					12.8	12	12.1	12.5	0			
18					12.8	12	12.1	12.7	0			
19					12.3	12	12.1	12.7	0			
20					12.2	12	12	12.5	0			
21					12	12	12.1	12.4	0			
22					12.1	12	12.2	12.4	0			
23					12.1	12	12.1	12.4	0			
24					12	12	12.4	12.5	0			
25					12	12	12.4	12.6	0			
26					12	11.9	12.4	12.4	0			
27					12	12	12.4	12.4	0			
28					11.8	12.3	12.4	12.4	0			
29				0	11.8	11.9	12.4	12.5	0			
30				0	11.7	11.9	12.4	12.6	0			
31					11.9		12.4	12.6				
COUNT				**2**	**31**	**30**	**31**	**31**	**30**	**15**		
MAX				**0**	**19.2**	**12.3**	**12.6**	**12.8**	**12.9**	**0**		
MIN				**0**	**0**	**11.7**	**11.8**	**12.4**	**0**	**0**		
AVG				**0**	**9.49**	**11.94**	**12.3**	**12.54**	**6.56**	**0**		

A-F	**0**	**583**	**710**	**757**	**771**	**391**	**0**

S: Stock water E: Estimated R: Return

OFFICE OF THE WATER MASTER

TRUCKEE RIVER SYSTEM

DAILY FLOW RECORD

Orr D

Year: 2012

Date	JAN (cfs)	FEB (cfs)	MAR (cfs)	APR (cfs)	MAY (cfs)	JUN (cfs)	JUL (cfs)	AUG (cfs)	SEP (cfs)	OCT (cfs)	NOV (cfs)	DEC (cfs)
01						12	14.5	14.8	14.1	0		
02					0	12	14.5	15	13.6	0		
03					0	12	14.5	14.9	13.7	0		
04					0	12	14.5	15	13.8	0		
05					0	12	14.5	15	13.8	0		
06					0	12	14.7	15.2	13.7	0		
07					0	12	14.7	15	13.7	0		
08					0	12	14.8	15	13.7	0		
09					0	12.1	14.6	15	13.7	0		
10					0	12.2	14.5	15	13.7	0		
11					14.3 R	12	14.5	15	13.7	0		
12					15.4 R	12	14.5	15	9.2	0		
13					17.9 R	12.3	14.6	15	0	0		
14					17.7 R	12.3	14.7	15	0	0		
15					14.3	12.1	14.7	15	0	0		
16					13.7	12.2	14.9	15	0	0		
17					13.2	12.1	14.7	15	0	0		
18					12.9	12	14.9	15	0	0		
19					12.8	12.3	15	15	0	0		
20					12.8	12.4	15.4	15	0	0		
21					12.8	12.3	15	15	0	0		
22					12.5	14	14.8	14.9	0	0		
23					12.8	14.5	15	15	0	0		
24					12.5	14.5	15	15	0	0		
25					12.5	14.5	15	14	0	0		
26					12.4	14.6	15	13.7	0	0		
27					12.1	14.5	14.9	13.7	0	0		
28					12.2	14.5	15	13.8	0	0		
29					12	14.6	15	13.7	0	0		
30					12.1	14.5	15	14.1	0	0		
31					12.1		15	14.1		0		
COUNT					**30**	**30**	**31**	**31**	**30**	**31**		
MAX					**17.9**	**14.6**	**15.4**	**15.2**	**14.1**	**0**		
MIN					**0**	**12**	**14.5**	**13.7**	**0**	**0**		
AVG					**9.37**	**12.82**	**14.79**	**14.74**	**5.35**	**0**		

A-F	**557**	**763**	**908**	**904**	**318**	**0**

S: Stock water E: Estimated R: Return

7.0 APPENDIX C - ORR DITCH EARTHEN CHANNEL Flow Conditions

This appendix includes the upstream existing earthen channel cross section during the four previously selected flow conditions. Using the Flow Master V8i computer analysis program analyzing depths and velocities in the channel were important for future calculations when sizing the inlet structres.

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Sheet: 1 of 1

Date:

Project:

Project No.:

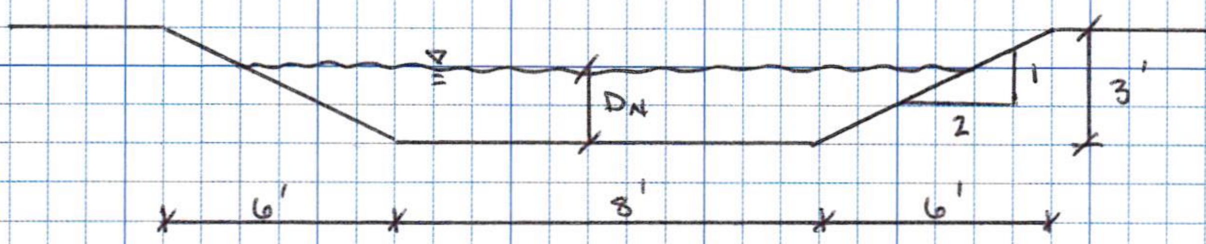
Subject: EXISTING ORR DITCH CROSS SECTION SUMMARY

Prepared by:

Checked by:

EARTHEN TRAPEZOIDAL CHANNEL (UPSTREAM)

* SUMMARY OF FLOW CONDITIONS FROM PROVIDED FLOWMASTER DATA, AND SITE INVESTIGATION



$S_o = 0.1\%$

FLOW : Q
CHANNEL SLOPE : S_o
NORMAL DEPTH : D_N

AVERAGE FLOW CONDITION :	Q_{AVG} = 15 cfs	(HISTORICAL FLOW DATA)
	D_N = 0.83 ft.	(FLOW MASTER V8i)
FLUSHING FLOW CONDITION :	Q_{FLUSH} = 40 cfs	(HISTORICAL FLOW DATA)
	D_N = 1.43 ft.	(FLOWMASTER V8i)
MAXIMUM FLOW CONDITION :	Q_{MAX} = 51 cfs	(HISTORICAL FLOW DATA)
	D_N = 1.64 ft.	(FLOWMASTER V8i)
ORR DITCH CAPACITY FLOW CONDITION :	Q_{CAP} = 160 cfs	(FLOWMASTER V8i)
	D_N = 3 ft.	(MAX CHANNEL DEPTH)

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Worksheet for Orr Ditch Earthen Channel - Average Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.020	
Channel Slope	0.10710	%
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	8.00	ft
Discharge	15.00	ft³/s

Results

Normal Depth	0.83	ft
Flow Area	7.97	ft²
Wetted Perimeter	11.69	ft
Hydraulic Radius	0.68	ft
Top Width	11.30	ft
Critical Depth	0.46	ft
Critical Slope	0.00804	ft/ft
Velocity	1.88	ft/s
Velocity Head	0.06	ft
Specific Energy	0.88	ft
Froude Number	0.40	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.83	ft
Critical Depth	0.46	ft
Channel Slope	0.10710	%

Worksheet for Orr Ditch Earthen Channel - Average Flow Condition

GVF Output Data		
Critical Slope	0.00804	ft/ft

Worksheet for Orr Ditch Earthen Channel - Flushing Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.020	
Channel Slope	0.10710	%
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	8.00	ft
Discharge	40.00	ft³/s

Results

Normal Depth	1.44	ft
Flow Area	15.61	ft²
Wetted Perimeter	14.42	ft
Hydraulic Radius	1.08	ft
Top Width	13.74	ft
Critical Depth	0.85	ft
Critical Slope	0.00679	ft/ft
Velocity	2.56	ft/s
Velocity Head	0.10	ft
Specific Energy	1.54	ft
Froude Number	0.42	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.44	ft
Critical Depth	0.85	ft
Channel Slope	0.10710	%

Worksheet for Orr Ditch Earthen Channel - Flushing Flow Condition

GVF Output Data

Critical Slope	0.00679	ft/ft

Worksheet for Orr Ditch Earthen Channel - Maximum Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.020	
Channel Slope	0.10710	%
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	8.00	ft
Discharge	51.00	ft^3/s

Results

Normal Depth	1.64	ft
Flow Area	18.51	ft^2
Wetted Perimeter	15.34	ft
Hydraulic Radius	1.21	ft
Top Width	14.56	ft
Critical Depth	0.99	ft
Critical Slope	0.00654	ft/ft
Velocity	2.76	ft/s
Velocity Head	0.12	ft
Specific Energy	1.76	ft
Froude Number	0.43	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.64	ft
Critical Depth	0.99	ft
Channel Slope	0.10710	%

Worksheet for Orr Ditch Earthen Channel - Maximum Flow Condition

GVF Output Data

Critical Slope	0.00654	ft/ft

Worksheet for Orr Ditch Earthen Channel - Capacity Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.020	
Channel Slope	0.10710	%
Normal Depth	3.00	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	8.00	ft

Results

Discharge	160.00	ft³/s
Flow Area	42.00	ft²
Wetted Perimeter	21.42	ft
Hydraulic Radius	1.96	ft
Top Width	20.00	ft
Critical Depth	1.95	ft
Critical Slope	0.00553	ft/ft
Velocity	3.81	ft/s
Velocity Head	0.23	ft
Specific Energy	3.23	ft
Froude Number	0.46	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	3.00	ft
Critical Depth	1.95	ft
Channel Slope	0.10710	%

Worksheet for Orr Ditch Earthen Channel - Capacity Flow Condition

GVF Output Data

Critical Slope	0.00553	ft/ft

8.0 APPENDIX D - SIPHON INLET CHANNEL Flow Conditions (FlowMaster V8i)

This appendix includes the proposed upstream concrete rectangular channel cross section during the three previously selected flow conditions. Using the Flow Master V8i computer analysis program, the cross sections and flow depths were computed and used in calculations sizing the inlet structure and weirs.

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Sheet: 1 of 1

Date:

Project:

Project No.:

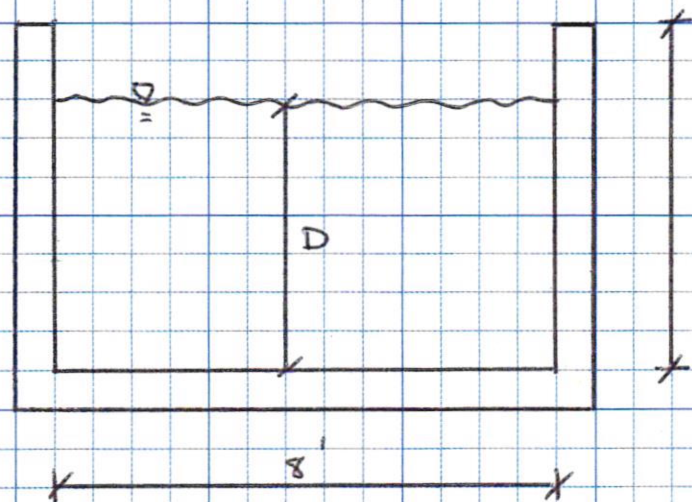
Subject: PROPOSED ORR DITCH CROSS SECTION SUMMARY

Prepared by:

Checked by:

RECTANGULAR CONCRETE CHANNEL

* SUMMARY OF FLOW CONDITIONS FROM PROVIDED FLOWMASTER DATA.



FLOW: Q

CHANNEL FLOW DEPTH: D

AVERAGE FLOW CONDITION:

Q_{AVG} = 15 cfs (HISTORICAL FLOW DATA)

D = 0.72 ft. (FLOWMASTER V8i)

FLUSHING FLOW CONDITION:

Q_{FLUSH} = 40 cfs (HISTORICAL FLOW DATA)

D = 1.37 ft. (FLOWMASTER V8i)

MAXIMUM FLOW CONDITION:

Q_{MAX} = 51 cfs (HISTORICAL FLOW DATA)

D = 1.61 ft. (FLOWMASTER V8i)

CAPACITY FLOW CONDITION:

Q_{CAP} = 160 cfs (FLOWMASTER V8i)

D = 3.61 ft. (FLOWMASTER V8i)

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Worksheet for Siphon Inlet Channel - Average Flow Condition

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.013	
Channel Slope		0.10000	%
Bottom Width		8.00	ft
Discharge		15.00	ft³/s

Results

Normal Depth		0.72	ft
Flow Area		5.76	ft²
Wetted Perimeter		9.44	ft
Hydraulic Radius		0.61	ft
Top Width		8.00	ft
Critical Depth		0.48	ft
Critical Slope		0.00366	ft/ft
Velocity		2.60	ft/s
Velocity Head		0.11	ft
Specific Energy		0.83	ft
Froude Number		0.54	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		0.00	ft
Length		0.00	ft
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	ft
Profile Description			
Profile Headloss		0.00	ft
Downstream Velocity		Infinity	ft/s
Upstream Velocity		Infinity	ft/s
Normal Depth		0.72	ft
Critical Depth		0.48	ft
Channel Slope		0.10000	%
Critical Slope		0.00366	ft/ft

Worksheet for Siphon Inlet Channel - Flushing Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.10000	%
Bottom Width	8.00	ft
Discharge	40.00	ft³/s

Results

Normal Depth	1.37	ft
Flow Area	10.93	ft²
Wetted Perimeter	10.73	ft
Hydraulic Radius	1.02	ft
Top Width	8.00	ft
Critical Depth	0.92	ft
Critical Slope	0.00334	ft/ft
Velocity	3.66	ft/s
Velocity Head	0.21	ft
Specific Energy	1.57	ft
Froude Number	0.55	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.37	ft
Critical Depth	0.92	ft
Channel Slope	0.10000	%
Critical Slope	0.00334	ft/ft

Worksheet for Siphon Inlet Channel - Maximum Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.10000	%
Bottom Width	8.00	ft
Discharge	51.00	ft³/s

Results

Normal Depth	1.61	ft
Flow Area	12.87	ft²
Wetted Perimeter	11.22	ft
Hydraulic Radius	1.15	ft
Top Width	8.00	ft
Critical Depth	1.08	ft
Critical Slope	0.00330	ft/ft
Velocity	3.96	ft/s
Velocity Head	0.24	ft
Specific Energy	1.85	ft
Froude Number	0.55	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.61	ft
Critical Depth	1.08	ft
Channel Slope	0.10000	%
Critical Slope	0.00330	ft/ft

Worksheet for Siphon Inlet Channel - Capacity Flow Condition

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.10000	%
Bottom Width	8.00	ft
Discharge	160.00	ft³/s

Results

Normal Depth	3.61	ft
Flow Area	28.88	ft²
Wetted Perimeter	15.22	ft
Hydraulic Radius	1.90	ft
Top Width	8.00	ft
Critical Depth	2.32	ft
Critical Slope	0.00342	ft/ft
Velocity	5.54	ft/s
Velocity Head	0.48	ft
Specific Energy	4.09	ft
Froude Number	0.51	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	3.61	ft
Critical Depth	2.32	ft
Channel Slope	0.10000	%
Critical Slope	0.00342	ft/ft

9.0 Appendix E - Hand Calculations – Siphon Inlet Structure Design

This appendix includes the proposed upstream concrete rectangular channel cross section during the three previously selected flow conditions. Using the Flow Master V8i computer analysis program, the cross sections and flow depths were computed and used in calculations sizing the inlet structure and weirs.

Sheet: 1 of 7

Date:

Project:

Project No.:

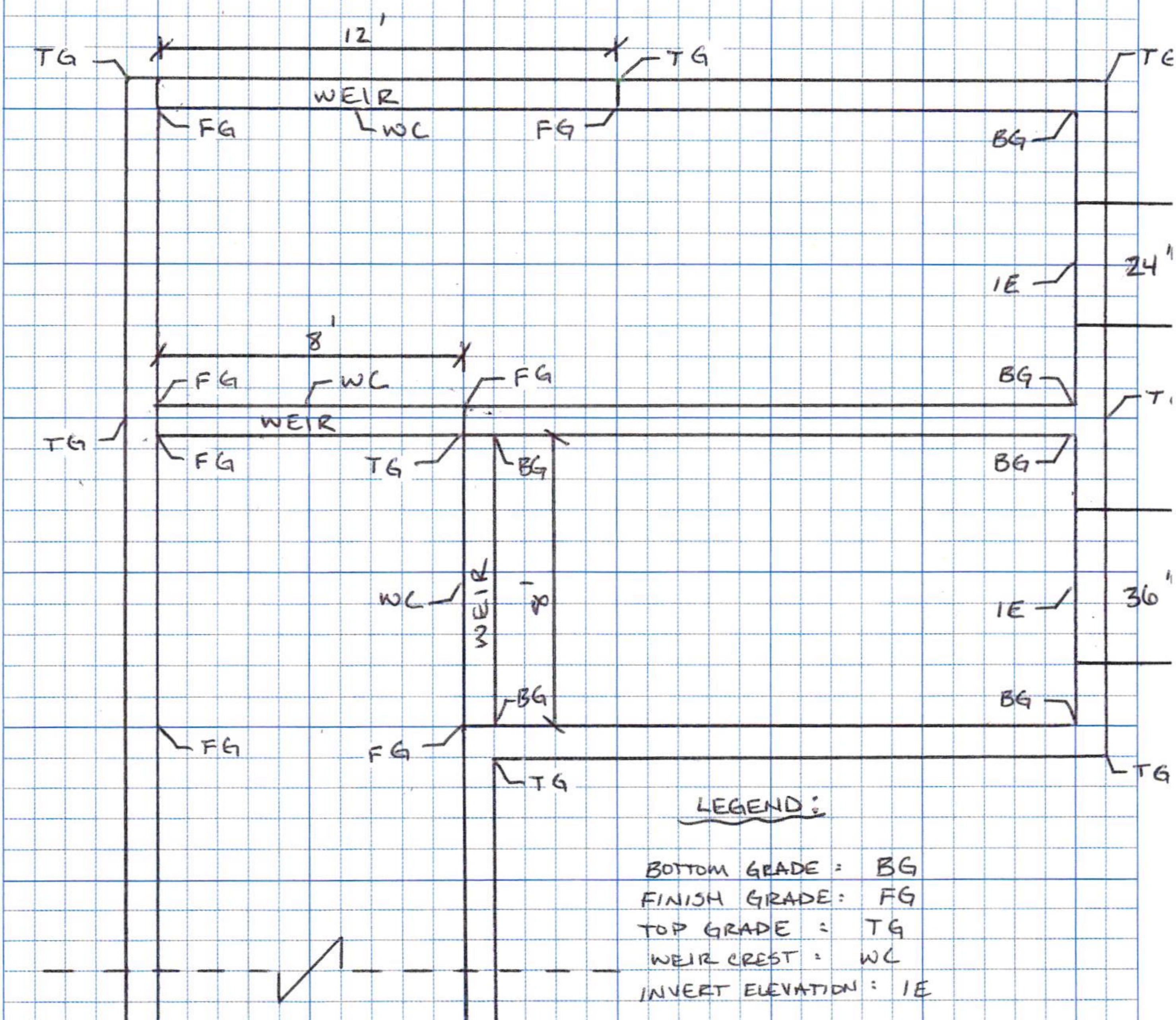
Subject: SIPHON INLET STRUCTURE DESIGN - OVERVIEW

Prepared by:

Checked by:

* SYSTEM FLOW CONDITION USAGE:

Q_{AVG} → 24" SYSTEM ONLY
Q_{FLUSH} → 24" AND 36" SYSTEM
Q_{MAX} → 24" AND 36" SYSTEM
Q_{CAP} → 24", 36" AND OVERFLOW



LEGEND:

BOTTOM GRADE : BG
FINISH GRADE : FG
TOP GRADE : TG
WEIR CREST : WC
INVERT ELEVATION : IE

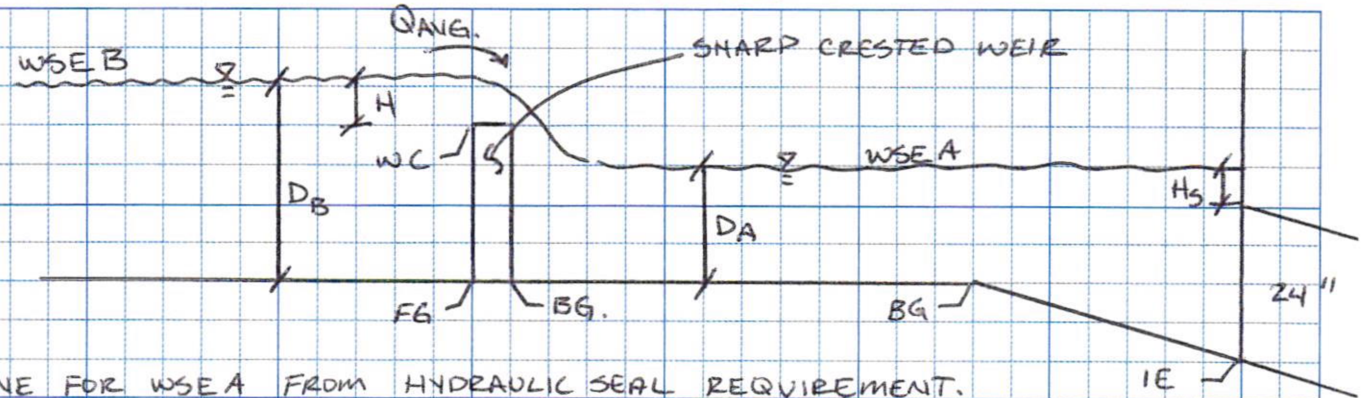
WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project: Project No.:

Subject: SIPHON INLET STRUCTURE DESIGN - 24" SYSTEM

Prepared by: Checked by:



* SOLVE FOR WSEA FROM HYDRAULIC SEAL REQUIREMENT.

$WSEA = IE + D_{PIPE} + H_S \rightarrow 4513.5 + 2' + 0.4'$

$\boxed{WSEA = 4515.9}$ (MINIMUM WSE FOR EFFICIENCY)

* SOLVE FOR BG FROM WSEA AND DEPTH OF FLOW DURING QAVG.

$BG = WSE\,A - D_A$ * DURING QAVG CONDITION.

$BG = 4515.9 - 0.72' \rightarrow \boxed{BG = 4515.18}$

* SET $FG = BG \rightarrow \boxed{FG = 4515.18}$

* SOLVE FOR WEIR CREST ELEVATION (FROM USBR pg 184)

$WC = WSEA + 1.5(D_c) + 1'$ $D_c(Q_{AVG}) = 0.48\ ft$ (FLOWMASTER V8i APPENDIX D

$WC = 4515.9 + 1.5(0.48) + 1'$

$\boxed{WC = 4517.62}$

* SOLVE FOR THE HEIGHT OF WATER FLOWING OVER THE WEIR.

$Q = 3.33(L_w - 0.2H)(H)^{3/2}$ (USBR pg 275)

$15\ cfs = 3.33(8' - 0.2(H))(H)^{3/2} \rightarrow \boxed{H = 0.69\ ft}$

Subject: SIPHON INLET STRUCTURE DESIGN - 24" SYSTEM

Prepared by:

Checked by:

* SOLVE FOR WSEB FROM THE WEIR CREST ELEVATION AND HEIGHT OF WATER OVER THE WEIR.

$$WSEB = WC + H \rightarrow 4517.62 + 0.69'$$

$$\boxed{WSEB = 4518.31}$$

* SOLVE FOR THE DEPTH OF FLOW BEFORE THE WEIR D_B

$$D_B = WSEB - FG \rightarrow 4518.31 - 4515.18$$

$$\boxed{D_B = 3.13 \text{ ft.}}$$

* SET WEIR CREST HEIGHT IN 36" SYSTEM TO WSEB IN 24" SYSTEM. ANY MORE FLOW THAN QAVG WILL BE DIVERTED INTO THE 36" SYSTEM.

WC @ 36" SYSTEM = WSEB

$\boxed{\text{WC @ 36" SYSTEM} = 4518.31}$

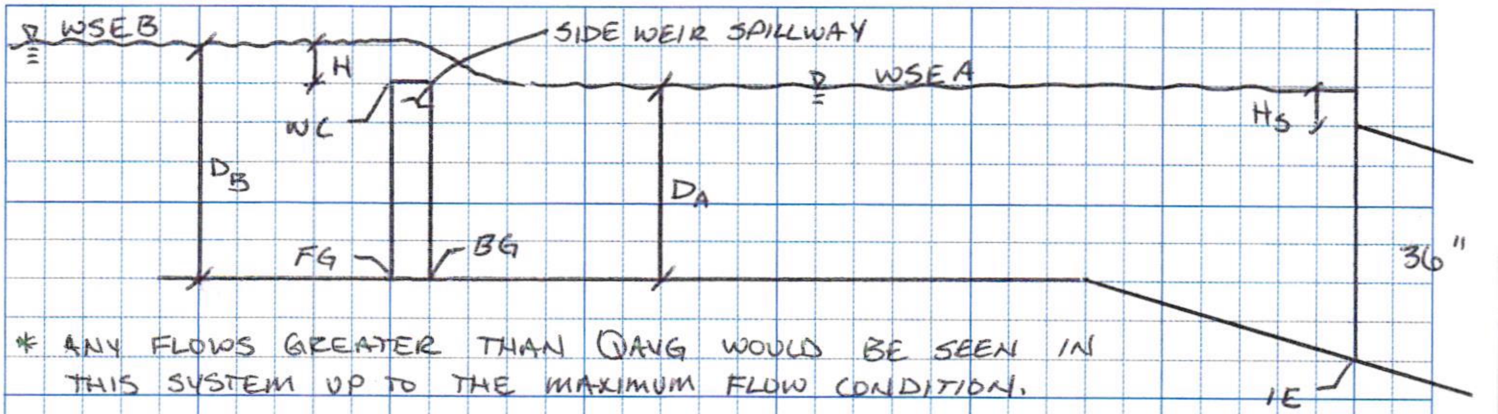
WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project: Project No.:

Subject: SIPHON INLET STRUCTURE DESIGN - 36" SYSTEM

Prepared by: Checked by:



* ANY FLOWS GREATER THAN QAVG WOULD BE SEEN IN THIS SYSTEM UP TO THE MAXIMUM FLOW CONDITION.

* SOLVE FOR WC FROM 24" SYSTEM HIGHEST WSEB @ QAVG. (APPENDIX E, PG 2)

WC = WSEB @ QAVG. → $\boxed{WC = 4518.31}$

* SOLVE FOR WSEA FROM HYDRAULIC SEAL REQUIREMENT.

$WSEA = IE + D_{PIPE} + H_S \rightarrow 4513.5 + 3' + 0.4'$

$\boxed{WSEA = 4516.9}$

* SOLVE FOR BG FROM WSEA AND DEPTH OF FLOW DURING QMAX
15 cfs IS TAKEN BY 24" SYSTEM, MUST BE SUBTRACTED FROM QMAX.

$BG = WSEA - D_A\,(Q_{MAX} - Q_{AVG})$

$Q_{MAX} - Q_{AVG} = 36\text{ cfs}$
$D_A = 1.27\text{ ft.}$
(FLOWMASTER V8; APPENDIX D)

$BG = 4516.9 - 1.27$

$\boxed{BG = 4515.63}$

* FG IS EQUAL TO 24" SYSTEM FG. (APPENDIX E, pg 2)

$\boxed{FG = 4515.18}$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project: Project No.:

Subject: SIPHON INLET STRUCTURE DESIGN - 36" SYSTEM

Prepared by: Checked by:

* SOLVE FOR THE HEIGHT OF WATER FLOWING OVER THE SIDE WEIR SPILLWAY.

$$Q = 3.33 L_w H^{3/2} \quad \text{(CHOW: EQN 14-16)}$$

$$36 \text{ cfs} = 3.33(8)(H)^{3/2} \longrightarrow \boxed{H = 1.22 \text{ ft.}}$$

* SOLVE FOR WSEB FROM THE WEIR CREST ELEVATION AND HEIGHT OF WATER OVER THE SIDE WEIR.

$$WSEB = WC + H \rightarrow 4518.31 + 1.22$$

$$\boxed{WSEB = 4519.53}$$

* SOLVE FOR THE DEPTH OF FLOW BEFORE THE WEIR, D_B

$$D_B = WSEB - FG \rightarrow 4519.53 - 4515.18$$

$$\boxed{D_B = 4.35 \text{ ft.}}$$

* SET WEIR CREST HEIGHT IN OVERFLOW SPILLWAY SYSTEM TO WSEB IN 36" SYSTEM. ANY MORE FLOW THAN QMAX WILL BE DIVERTED TO THE OVERFLOW SPILLWAY SYSTEM.

WC @ OVERFLOW SPILLWAY = WSE B

$\boxed{\text{WC @ OVERFLOW SPILLWAY} = 4519.53}$

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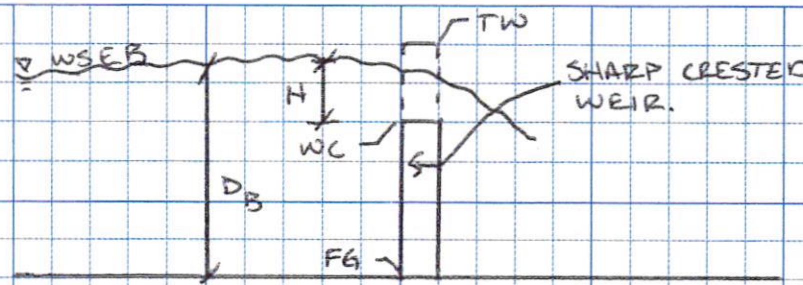
Subject: SIPHON INLET STRUCTURE DESIGN - OVERFLOW SPILLWAY.

Prepared by: Checked by:



* PREVIOUSLY CALCULATED INFORMATION

$\boxed{FG = 4515.18}$ (APPENDIX E pg. 2)

$\boxed{WC = 4519.53}$ (APPENDIX E pg. 5)

* SOLVE FOR THE HEIGHT OF WATER FLOWING OVER THE OVERFLOW SPILLWAY WEIR, THIS IS ANY FLOW GREATER THAN QMAX. THE MAXIMUM FLOW IS EQUAL TO QCAP MINUS THE FLOWS TAKEN BY THE 24" AND 36" SYSTEMS.

$$Q = Q_{CAP} - Q_{MAX} \rightarrow Q = 160 - 51 \rightarrow \boxed{Q = 109 \text{ cfs}}$$

$$Q = 3.33\left(L_w - 0.2H\right)(H)^{3/2} \quad \text{(USBR pg 275)}$$

$$109 \text{ cfs} = 3.33\left(12 - 0.2(H)\right)(H)^{3/2}$$

$$\boxed{H = 1.99 \text{ ft}}$$

* SOLVE FOR WSEB FROM THE WEIR CREST ELEVATION AND HEIGHT OF WATER FLOWING OVER THE OVERFLOW SPILLWAY WEIR.

$$WSEB = WC + H \rightarrow 4519.53 + 1.99$$

$$\boxed{WSEB = 4521.52}$$

WOOD RODGERS
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Date:

Project: Project No.:

Subject: SIPHON INLET STRUCTURE DESIGN - OVERFLOW SPILLWAY.

Prepared by: Checked by:

* SOLVE FOR THE TOP OF GRADE ELEVATION. FROM THE CAPACITY FLOW WSE AND ADDITIONAL FREE BOARD.

TG = WSEB + FREE BOARD → 4521.52 + 0.5'

$\boxed{TG = 4522.02}$

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10.0 APPENDIX F – Siphon Inlet – Rip Rap Design

This appendix includes the calculations for the rip rap design at the inlet transition from earthen to concrete channel and at the overflow spillway. The design is based on size of rock or class of rip rap and the overall flow velocities through the rip rap.

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Project: Project No.:

Subject: SIPHON INLET - RIP RAP DESIGN.

Prepared by: Checked by:

HYDRODYNAMIC RIP RAP SIZING. (USDA, NRCS: CH. 2)

* SOLVE FOR HYDRODYNAMIC FORCE, D_{50} ROCK SIZE.

$$D_{50} = 14.2 (SF)(D_{MAX})(S_e / K_1)$$

- SAFETY FACTOR (SF)
- MAX FLOW DEPTH (D_{MAX})
- CHANNEL SLOPE (S_e)
- BANK ANGLE MODIFICATION FACTOR (K_1)

→ FROM:
- ANGLE OF REPOSE CHART (Φ)
- BANK ANGLE (θ)

* SEE NEXT PAGE FOR EXCEL SOLUTION.

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Proposed Channel Riprap Configuration Sizing

B. Design Relationships

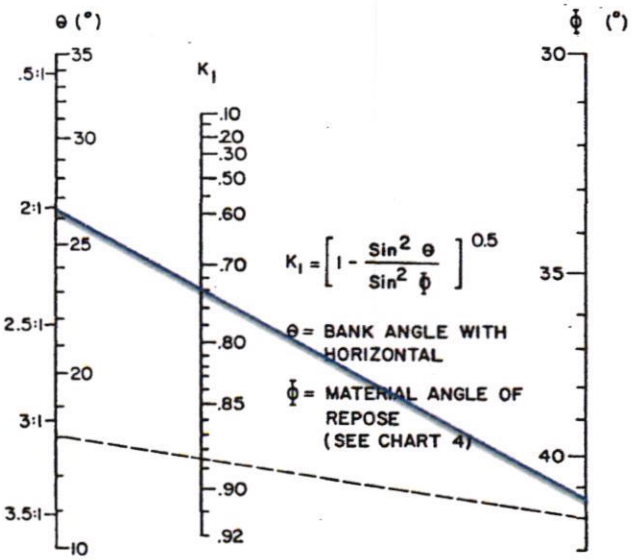
The hydrodynamic force of water flowing in a channel is known as the tractive force. The basic premise underlying riprap design based on tractive force theory is that the flow-induced tractive force should not exceed the permissible or critical shear stress of the riprap. Assuming a specific gravity of 2.50, equation 2-2 can be used to determine D_{50} of the riprap by the tractive stress method (reference 14, page 30).

$$D_{50} = 14.2\ SF\ dmax\ Se/K1 \qquad (2\text{-}2)$$

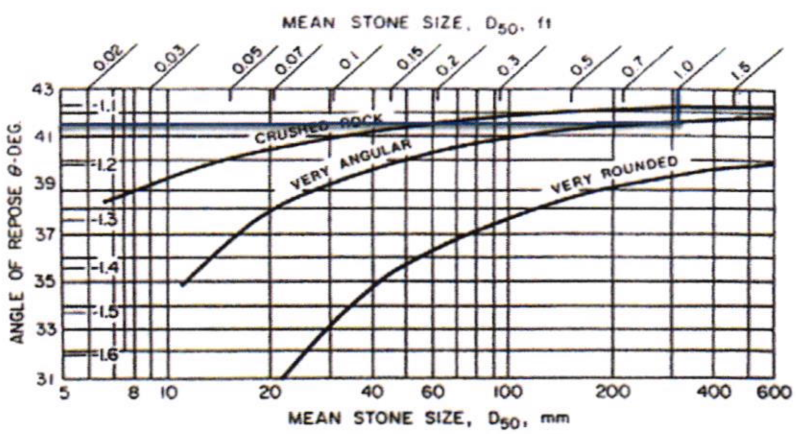
where,
SF = stability factor
dmax = maximum section depth, feet
Se = average energy grade line slope, ft/ft
D_{50} = median riprap size in feet
K1 = bank angle modification factor (see eq'n 2-3 or Figure 2-4)

15

** United States Department of Agriculture - USDA
** Natural Resources Conservation Services - NRCS Chapter 2



Angles of Repose of Riprap Stones (FHWA)



INLET - Channel

SF	2	Channel Width	8.00 ft
D_{MAX}	3 ft	d_{50}=	1.18 ft
S_e	1 %	Riprap Size=	USE CLASS 400
K1	0.72	Depth=	1.50 ft

Riprap Size= USE CLASS 400
Depth= 1.50 ft

11.0 APPENDIX G - HAND CALCULATIONS – Inverted Siphon Design

This appendix includes the Calculations for the inverted siphon under the three flow condition. The average flow has been contained within the 24" system and the remaining flow up to the maximum flow condition of 51 cfs has been contained within the 24" and 36" system. Flows higher than the maximum operational flow conditions will be released over the overflow spillway.

This design is taken solely from the USBR pg 24-38. The calculations for this system solve for multiple parameters of the system to maintain hydraulic efficiencies. These parameters are:

- Existing Conditions Considerations
- Tie-in Locations/Elevations
- Inlet Structure Hydraulic Properties/Requirements
- Weir Structure Sizing/Calculations
- Inlet Hydraulics
- System Head Loss Analysis
- Overall System Operating Head
- Low Point Drain/Outlet Protection

Date:

Project:

Project No.:

Subject: INVERTED SIPHON DESIGN.

Prepared by:

Checked by:

* ALL KNOWN DESIGN / LAYOUT INFORMATION.

FLOW CONDITIONS :

AVERAGE FLOW CONDITION $Q_{AVG} = 15$ cfs

FLUSHING FLOW CONDITION $Q_{FLUSH} = 40$ cfs

MAXIMUM FLOW CONDITION $Q_{MAX} = 51$ cfs.

SIPHON START INVERT = 4513.5

SIPHON END INVERT = 4508.0

SIPHON LENGTH : 1079 ft.

* EXISTING ORR DITCH EARTHEN TRAPEZOIDAL CHANNEL TRANSITION TO PROPOSED REINFORCED CONCRETE RECTANGULAR CHANNEL

* SLOPE INTO SIPHON $S = 0.1\%$

Subject: INVERTED SIPHON DESIGN - 24" SYSTEM HYDRAULIC PROPERTIES.

Prepared by:

Checked by:

* SOLVE FOR FRICTION SLOPE OF PIPE. (USBR pg 24-38)

$$S_F = \left[\frac{1}{(2.2\ R^{4/3})}\right](n^2)(V^2)$$

HYDRAULIC RADIUS : $R = A_{PIPE} / P_{WET} = \frac{\pi (r)^2}{\pi D} = \frac{3.14\ ft^2}{6.28\ ft} = \boxed{0.5\ ft}$

WETTED PERIMETER : $P_{WET} = \pi D_{PIPE} = \boxed{6.28\ ft}$

VELOCITY IN PIPE : $V = Q/A = \boxed{4.77\ fps}$ (CHOW EQ: 1-1)

MANNING'S : n = STEEL PIPE $\boxed{n = 0.012}$ (FLOWMASTER V8i)

VELOCITY HEAD : $h_v = \frac{V^2}{2g} = \frac{4.77^2}{2(32.2)} = \boxed{0.354\ ft.}$

$$S_F = \left[\frac{1}{(2.2(0.5)^{4/3})}\right](0.012)^2(4.77)^2$$

$$\boxed{S_F = 0.00375\ ft/ft}$$

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Date:

Project: Project No.:

Subject: INVERTED SIPHON DESIGN - 24" SYSTEM

Prepared by: Checked by:

* FOLLOW DESIGN PROCEDURES OUTLINED IN (USBR pg 24-38)

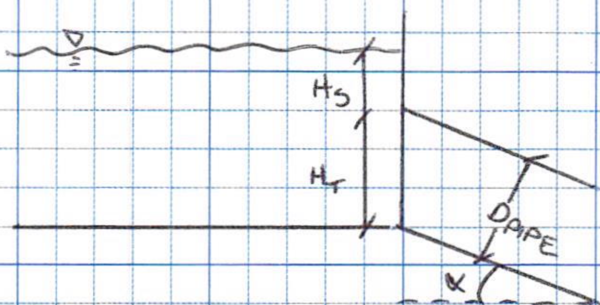
* SOLVE FOR SYSTEM FREEBOARD AT INVERTED SIPHON ENTRANCE.

PER USBR: NORMAL FREE BOARD $F_N = 1.0$ ft.

ADDITIONAL FREEBOARD = 0.5 ft. (F_N) = 0.5 ft.

TOTAL FREEBOARD = $F_N + 0.5(F_N) = \boxed{1.5 \text{ ft}}$

* SOLVE FOR INLET VERTICAL TRANSITION AT ENTRANCE.



HYDRAULIC SEAL = H_S
VERTICAL TRANSITION = H_T
PIPE DIAMETER = D_{PIPE}
PIPE SLOPE ANGE = $\alpha°$

$H_T = D_{PIPE} / \cos(\alpha)$ (USBR pg. 33)

$H_T = \dfrac{(24''/12)}{\cos(7\%)} \rightarrow \boxed{H_T = 2.015 \text{ ft.}}$

* SOLVE FOR HYDRAULIC SEAL H_S

$H_S = h_v(1.5)$ (USBR pg. 33)

$H_S = (0.354)(1.5)$

$\boxed{H_S = 0.53 \text{ ft.}}$

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Project:

Project No.:

Subject: INVERTED SIPHON DESIGN - 24" SYSTEM

Prepared by:

Checked by:

* SOLVE FOR REQUIRED VERTICAL DROP (USBR pg. 33)

$\rightarrow H_T + H_S = 2.015 + 0.53 = \boxed{2.546 \text{ ft.}}$

* SOLVE FOR MAXIMUM INLET STRUCTURE DROP P_I

$P_I = 0.75(D_{PIPE}) = 1.5 \text{ ft.}$

$\boxed{P_I = 1.5 \text{ ft.}}$

* SOLVE FOR THE DROP REQUIRED IN THE APPROACH CHANNEL

DROP = VERTICAL DROP - MAX DROP

$\rightarrow 2.546 - 1.5 \text{ ft} = \boxed{1.046 \text{ ft.}}$

* SOLVE FOR MAXIMUM OUTLET STRUCTURE DROP P_O

$P_O = 0.5(D_{PIPE}) \rightarrow 0.5(2)$

$\boxed{P_O = 1.0 \text{ ft.}}$

* P_I AND P_O ARE USED FOR HEADLOSS ANALYSIS, SEE NEXT PAGE.

* VERIFY OUTLET SUBMERGENCE PER (USBR pg. 33)

OUTLET SUBMERGENCE: O.S. $< H_T/6$

$O.S. = (D_{CRIT} + P_O) - H_T$

$= (0.48 + 1.0) - 2.015$

$= -0.54$

$D_{CRIT} = 0.48$ (FLOWMASTER V8i)

$H_T/6 = 2.015/6 = 0.336$

$-0.54 < 0.336$ $\boxed{\text{OK}}$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Sheet: 5 of 10

Date:

Project:

Project No.:

Subject: INVERTED SIPHON DESIGN - 24" SYSTEM HEADLOSS ANALYSIS

Prepared by:

Checked by:

* SOLVE FOR TOTAL AVAILABLE HEAD IN THE SYSTEM (H_A)

$$H_A = IE(\text{ENTRANCE}) - IE(\text{EXIT})$$

$$H_A = 4513.5 - 4508 \rightarrow \boxed{H_A = 5.5 \text{ ft.}}$$

* SOLVE FOR TOTAL HEAD LOSS IN THE SYSTEM (H_L) WITH A 10% FACTOR OF SAFETY PER (USBR pg 34).

$$H_L = (H_i + H_f + H_b + H_o)1.1 \quad (\text{USBR pg } 34)$$

INLET HEADLOSS (H_i)

$$H_i = (0.4)(h_v) = 0.4(0.354) \rightarrow \boxed{H_i = 0.141}$$

SIPHON FRICTION LOSS (H_f)

$$H_f = (S_F)(L) = (0.0037)(1079) \rightarrow \boxed{H_f = 3.99}$$

SIPHON BEND LOSS (H_b)

$$\boxed{H_b = 0} \quad \text{PER } (\text{USBR pg } 34)$$

OUTLET HEAD LOSS (H_o)

$$H_o = 0.7(h_v) \rightarrow \boxed{H_o = 0.248}$$

TOTAL HEAD LOSS $H_L = (H_i + H_f + H_b + H_o)1.1$

$$\boxed{H_L = 4.82}$$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project: Project No.:

Subject: INVERTED SIPHON DESIGN - 36" SYSTEM HYDRAULIC PROPERTIES.

Prepared by: Checked by:

* SOLVE FOR FRICTION SLOPE OF PIPE (USBR pg 24-38)

$$S_f = \left[\frac{1}{(2.2\ R^{4/3})}\right](n^2)(V^2)$$

HYDRAULIC RADIUS: $R = A/P_W = \frac{\pi r^2}{\pi D} = \boxed{7.065\ ft^2}$

WETTED PERIMETER: $P_W = \pi D_{PIPE} = \boxed{9.42\ ft.}$

VELOCITY IN PIPE: $V = Q/A = 51.15/7.065 = \boxed{5.095\ fps}$ (CHOW EQ: 1-1)

MANNING'S: n = STEEL PIPE $\boxed{n = 0.012}$ (FLOW MASTER V8i)

VELOCITY HEAD: $h_v = \frac{V^2}{2g} = (5.095)^2 / 2(32.2) \rightarrow \boxed{h_v = 0.403}$

$$S_F = \left[\frac{1}{2.2(7.065)^{4/3}}\right](.012)^2(5.095)^2$$

$$\boxed{S_F = 0.00013\ ft/ft.}$$

Date:

Project:

Project No.:

Subject: INVERTED SIPHON DESIGN - 36" SYSTEM

Prepared by:

Checked by:

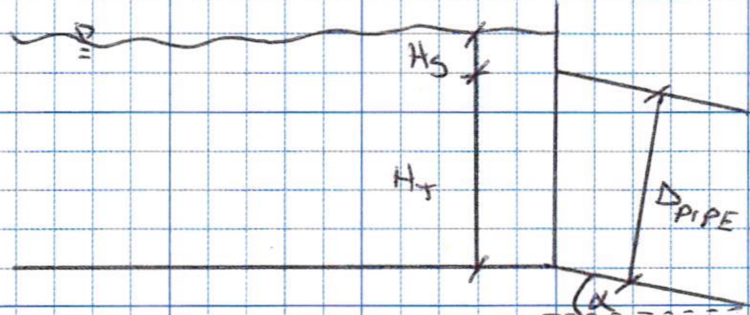
* FOLLOW DESIGN PROCEDURES OUTLINED IN (USBR pg 24-38)

* SOLVE FOR SYSTEM FREEBOARD AT INVERTED SIPHON ENTRANCE.

PER USBR: NORMAL FREEBOARD $F_N = 1.0$ ft.
ADDITIONAL FREE BOARD = 0.5 ft.

TOTAL FREE BOARD = 1.0 + 0.5 ft = $\boxed{1.5 \text{ ft.}}$

* SOLVE FOR INLET VERTICAL TRANSITION AT ENTRANCE.



HYDRAULIC SEAL: H_S
VERTICAL TRANSITION: H_T
PIPE DIAMETER: D_{PIPE}
PIPE ANGLE = $\alpha°$

$H_T = D_{PIPE} / \cos(\alpha°)$ (USBR pg 33)

$$H_T = \frac{36''/12}{\cos(7°)} \rightarrow \boxed{H_T = 3.022 \text{ ft.}}$$

* SOLVE FOR HYDRAULIC SEAL H_S

$$H_S = h_v(1.5) \rightarrow (0.403)(1.5)$$

$$\boxed{H_S = 0.604}$$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project:

Project No.:

Subject: INVERTED SIPHON DESIGN - 36" SYSTEM

Prepared by:

Checked by:

* SOLVE FOR REQUIRED VERTICAL DROP (USBR pg 33)

$$\rightarrow H_T + H_S = 3.022 + 0.604 = \boxed{3.626}$$

* SOLVE FOR MAXIMUM INLET STRUCTURE DROP P_I

$$P_I = 0.75(D_{PIPE}) \rightarrow \boxed{P_I = 2.25}$$

* SOLVE FOR THE DROP REQUIRED IN THE APPROACH CHANNEL

DROP = VERTICAL DROP − MAX DROP

$$\rightarrow 3.626 - 2.25 = \boxed{1.37\text{ ft.}}$$

* SOLVE FOR MAXIMUM OUTLET STRUCTURE DROP P_O

$$P_O = 0.5(D_{PIPE}) \longrightarrow \boxed{P_O = 1.5\text{ ft}}$$

* P_I AND P_O ARE USED FOR HEADLOSS ANALYSIS, SEE NEXT PAGE.

* OUTLET SUBMERGENCE CHECK. (USBR pg. 33)

$$O.S. < H_T/6 \qquad \frac{H_T}{6} = \frac{3.022}{6} = 0.504$$

$$O.S. = (D_{CRIT} + P_O) - H_T \qquad D_{CRIT} = 1.08 \text{ (FLOWMASTER V8i)}$$

$$O.S = (1.08 + 1.5) - 3.022 \rightarrow O.S = -0.442$$

$$-0.442 < 0.504 \quad \boxed{OK}$$

Subject: INVERTED SIPHON DESIGN – 36" SYSTEM HEADLOSS ANALYSIS

Prepared by: Checked by:

* SOLVE FOR TOTAL AVAILABLE HEAD IN THE SYSTEM (H_A)

$$H_A = IE\,(\text{ENTRANCE}) - IE\,(\text{EXIT}) \rightarrow 4513.5 - 4508$$

$$\boxed{H_A = 5.5 \text{ ft}}$$

* SOLVE FOR THE TOTAL HEADLOSS IN THE SYSTEM (H_L) WITH A 10% FACTOR OF SAFETY, PER (USBR pg 34).

$$H_L = (H_i + H_f + H_b + H_o)\,1.1 \qquad (\text{USBR pg } 34)$$

INLET HEADLOSS (H_i)

$$H_i = 0.4\,(h_v) \rightarrow (0.4)(0.403)$$

$$\boxed{H_i = 0.161 \text{ ft}}$$

SIPHON FRICTION HEADLOSS (H_f)

$$H_f = (S_f)(L) = (0.00013)(1079)$$

$$\boxed{H_f = 0.140 \text{ ft}}$$

SIPHON BEND LOSS (H_b)

$$H_b = 0 \qquad (\text{PER USBR pg } 34)$$

OUTLET HEADLOSS (H_o)

$$H_o = 0.7\,(h_v) = (0.7)(.403) \rightarrow \boxed{H_o = 0.28 \text{ ft}}$$

$$\text{TOTAL HEAD LOSS } (H_L) = (H_i + H_f + H_b + H_o)\,1.1$$

$$\boxed{H_L = 0.64 \text{ ft.}}$$

WOOD RODGERS
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Date:

Project:

Project No.:

Subject: INVERTED SIPHON DESIGN - LOW POINT DRAIN - BAFFLED OUTLET

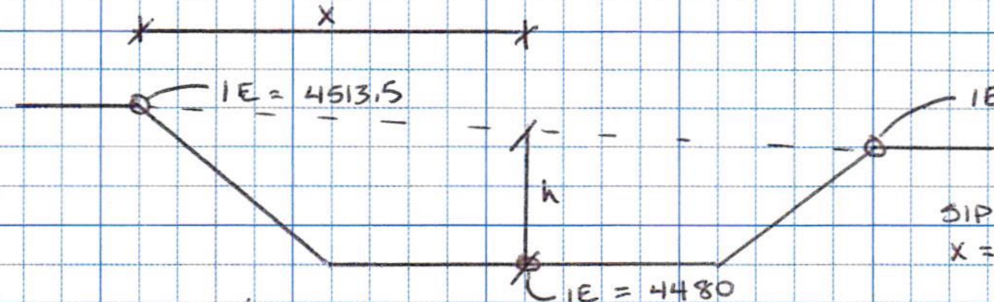
Prepared by:

Checked by:

* SOLVE FOR THE DIMENSIONS OF THE BAFFLED OUTLET STRUCTURE AND SIZE THE LOW POINT DRAIN PIPE.

→ SELECT A 12" DRAIN PIPE. $A = \pi r^2 \rightarrow A = 0.785\ ft^2$

* PER USBR pg 309, SOLVE FOR THE THEORETICAL VELOCITY AT THE LOW POINT.



SIPHON LENGTH = 1079 ft.
X = 610 ft.

$$h = 4513.5 - \left(\frac{4513.5 - 4508}{1079}\right)(610) - 4480$$

$$\boxed{h = 30.4\ ft.}$$

$$V = \sqrt{2gh} \rightarrow \sqrt{2(32.2)(30.4)} \rightarrow \boxed{V = 44.25\ fps}$$

* SOLVE FOR THE FROUDE NUMBER FROM FIGURE 6-10 IN USBR pg. 310

$$F = V/\sqrt{gd} = 44.25/\sqrt{(32.2)(\sqrt{.785})}$$

$d = \sqrt{A}$ (ASSUMED SQUARE FLOW PER USBR

$$\boxed{F = 8.28}$$

FROM FIGURE 6-10, $F = 8.28 \rightarrow \boxed{W/d = 9.5}$

* SOLVE FOR THE WIDTH OF THE BAFFLED OUTLET, W

$$W = d \times W/d = (\sqrt{.785})(9.5) \rightarrow \boxed{W = 8.42\ ft.}$$

12.0 APPENDIX H - HAND CALCULATIONS – Siphon Outlet Structure Design

This appendix includes the Calculations for the inverted siphon Outlet Structure during the three flow conditions. The average flow has been contained within the 24" system and the remaining flow up to the maximum flow condition of 51 cfs has been contained within the 24" and 36" system.

The calculations for this system solve for multiple parameters of the system to maintain hydraulic efficiencies. These parameters are:

- Outlet Location/Elevations
- Outlet Structure Hydraulic Properties/Requirements
- Weir Structure Sizing/Calculations
- Outlet Hydraulics
- Tie-in Location/Elevation

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Sheet: 1 of 7

Date:

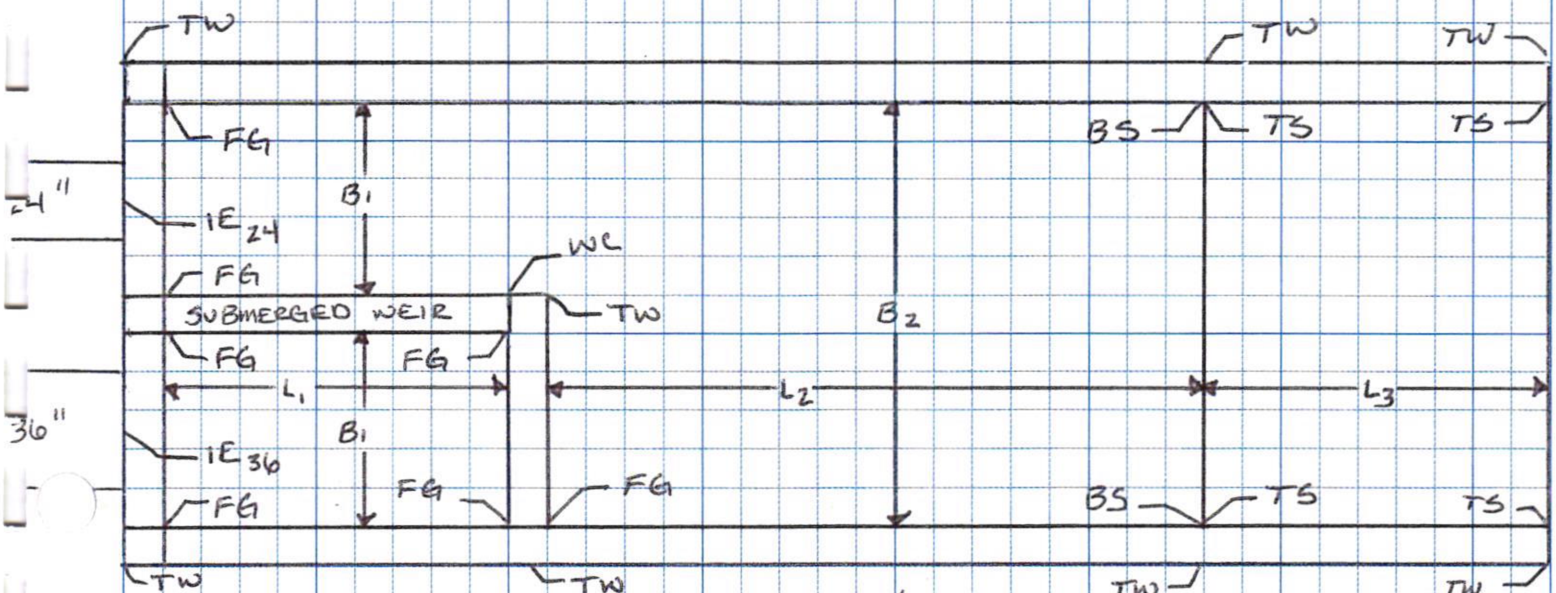
Project:

Project No.:

Subject: OUTLET STRUCTURE OVERVIEW

Prepared by:

Checked by:



IE_{24} = 4508.0

IE_{36} = 4508.0

WC = 4509.61

TW = 4512.0

BS = 4508.0

FG = 4508.0

TS = 4509.17

B_1 = 8.0'

B_2 = 16.67'

L_1 = 12'

L_2 = 20'

L_3 = 10'

IE = INVERT ELEVATION

WC = WEIR CREST

TW = TOP OF WALL

BS = BOTTOM STEP

TS = TOP OF STEP

FG = FINISH GRADE

B = WIDTH OF CHANNEL

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Date:

Project: Project No.:

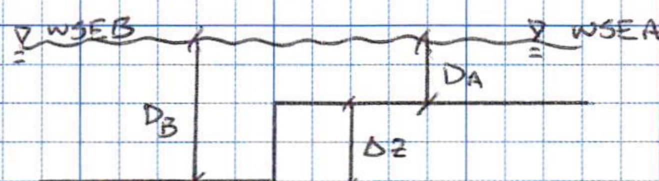
Subject: OUTLET STRUCTURE DESIGN - 24" SYSTEM

Prepared by: Checked by:

BOTTOM STEP = BS = 4508
TOP STEP = TS = 4509.1
INVERT ELEVATION = IE = 450

* WSEB = WSEA

* SOLVE FOR WSEA BY ANALYZING BOTTOM STEP AS A BROAD CRESTED WEIR



* WEIR LENGTH L_w

$L_w = 16.67$ ft.

FLOW PER UNIT LENGTH (q)
FLOW OVER WEIR (Q)
WEIR LENGTH (L_w)

$$q = Q/L_w \quad \text{(CHOW EQ 4-17)}$$

$$q = 3.09\,(D_A)^{1.5} \quad \text{(CHOW EQ 4-17)}$$

* BOTTOM STEP ANALYZED AS A BROAD CRESTED WEIR SOLVES WSEB, WSEA, DB, DA ON THE NEXT PAGE FOR ALL THREE FLOW CONDITIONS.

* MAXIMUM OUTLET SUBMERGENCE WSE B (USBR pg 29)

$$WSEB\,(\text{MAX ALLOWED}) = IE + D_{PIPE} + D.S.$$

$$4508 + 2' + 2.01/6 = \boxed{4510.34}$$

Subject: OUTLET STRUCTURE DESIGN - 24" SYSTEM

Prepared by:

Checked by:

* SOLVE FOR WSEB DURING AVERAGE FLOW CONDITION. Q = 15 cfs

$q = Q/L = 15\text{ cfs}/16.67\text{ ft} \rightarrow \boxed{q = 0.899\text{ cfs/ft.}}$ (CHOW EQ 4-17)

$q = 3.09(D_A)^{1.5} \rightarrow \boxed{D_A = 0.44\text{ ft.}}$ (CHOW EQ 4-17)

$WSEA = TS + D_A = 4509.17 + 0.44\text{ ft.} \rightarrow \boxed{WSEA = 4509.61}$

$\boxed{WSEA = WSEB}$

* CHECK OUTLET SUBMERGENCE

WSEB @ QAVG < WSEB (MAX ALLOWED)

$4509.61 < 4510.34 \rightarrow \boxed{OK}$

* SOLVE FOR WSEB DURING FLUSHING FLOW CONDITION Q = 40 cfs

$q = Q/L = 40\text{ cfs}/16.67\text{ ft} \rightarrow \boxed{q = 2.40\text{ cfs/ft.}}$ (CHOW EQ 4-17)

$q = 3.09(D_A)^{1.5} \rightarrow \boxed{D_A = 0.84\text{ ft}}$ (CHOW EQ 4-17)

$WSEA = TS + D_A \rightarrow 4509.17 + 0.84\text{ ft.} \rightarrow \boxed{WSEA = 4510.01}$

$\boxed{WSEA = WSEB}$

* CHECK OUTLET SUBMERGENCE

WSEB @ QFLUSH < WSEB (MAX ALLOWED)

$4510.01 < 4510.34 \rightarrow \boxed{OK}$

Date:

Project: Project No.:

Subject: OUTLET STRUCTURE DESIGN - 24" SYSTEM

Prepared by: Checked by:

* SOLVE FOR WSEB DURING MAXIMUM FLOW CONDITION Q = 51 cfs

$q = Q/L = 51\text{ cfs}/16.67\text{ ft.} \rightarrow \boxed{q = 3.06\text{ cfs/ft}}$ (CHOW EQ 4-17)

$q = 3.09(D_A)^{1.5} \rightarrow \boxed{D_A = 0.99\text{ ft.}}$ (CHOW EQ 4-17)

$WSEA = TS + D_A \rightarrow 4509.17 + 0.99\text{ ft.}$

$\boxed{WSEA = 4510.16}$

$\boxed{WSEA = WSEB}$

* CHECK OUTLET SUBMERGENCE

WSEB @ Q_{MAX} < WSEB (MAX ALLOWED)

$4510.16 < 4510.34 \rightarrow \boxed{OK}$

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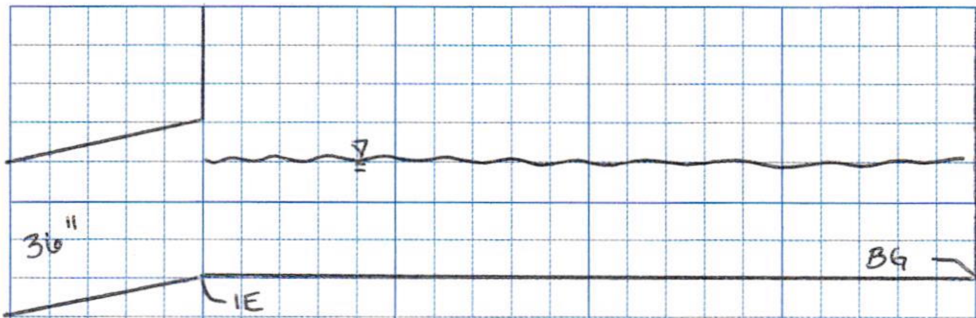
Date:

Project: Project No.:

Subject: OUTLET STRUCTURE DESIGN - 36" SYSTEM

Prepared by: Checked by:



INVERT ELEVATION : IE = 4508
BOTTOM GRADE = BG = 4508

AVERAGE FLOW CONDITION : Q_{AVG} = 15 cfs (NOT SEEN IN THIS SYSTEM)

FLUSHING FLOW CONDITION : Q_{FLUSH} = 40 cfs (25 cfs SEEN IN THIS SYSTEM)

MAXIMUM FLOW CONDITION : Q_{MAX} = 51 cfs (36 cfs SEEN IN THIS SYSTEM)

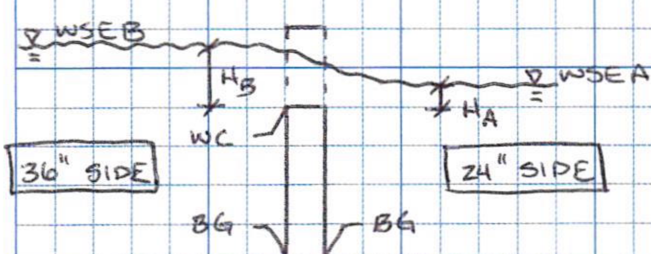
* SET THE WEIR CREST TO THE AVERAGE FLOW CONDITION WSE.

WC = WSE @ Q_{AVG} → $\boxed{WC = 4509.61}$

* 36" SYSTEM WEIR ANALYZED AS A SUBMERGED WEIR UTILIZING USDA DESIGN NOTE NO. 15 AS REFERENCE.

* WEIR LENGTH = 12 ft.

FLOW PER UNIT LENGTH : q_s
FLOW OVER WEIR : Q (USDA)
WEIR LENGTH : L_W



* MAXIMUM OUTLET SUBMERGENCE WSEB (USBR pg 29)

WSEB (MAX ALLOWED) = IE + D_{PIPE} + 0.S

$4508 + 3' + \frac{3.02}{6}$ → $\boxed{4511.5}$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project:

Project No.:

Subject: OUTLET STRUCTURE DESIGN - 36" SYSTEM

Prepared by:

Checked by:

* SOLVE FOR WSEB DURING FLUSHING FLOW CONDITION $Q = 40$ cfs

$Q = 40 \text{ cfs} - 15 \text{ cfs} \rightarrow Q = 25 \text{ cfs}.$

WSEA (FLUSHING FLOW CONDITION) = 4510.01 (APPENDIX H pg. 2)

$q_s = Q/L_W \rightarrow 25 \text{ cfs}/12 \text{ ft.} \rightarrow \boxed{q_s = 2.08 \text{ cfs/ft.}}$ (USDA DESIGN NOTE 6

$D_A = \text{WSEB (24" FLUSHING FLOW)} - WC \rightarrow 4510.01 - 4509.61$

$\boxed{D_A = 0.40 \text{ ft.}}$

$$\frac{q_s^{2/3}}{D_A} = \frac{(2.08)^{2/3}}{0.40} = 4.07$$ (USDA SHEET 3, ES-207)

$\frac{D_B}{D_A} = 1.9$ (USDA SHEET 2, ES-207)

$D_B = 1.9(0.4) = \boxed{D_B = 0.76 \text{ ft.}}$

WSEB (FLUSHING FLOW) = WC + D_B = 4509.61 + 0.76

$\boxed{\text{WSEB} = 4510.37}$

* CHECK OUTLET SUBMERGENCE

WSEB < WSEB (MAX ALLOWED)

$4510.36 < 4511.5 \rightarrow \boxed{\text{OK}}$

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Date:

Project: Project No.:

Subject: OUTLET STRUCTURE DESIGN - 36" SYSTEM

Prepared by: Checked by:

* SOLVE FOR WSEB DURING MAXIMUM FLOW CONDITION $Q = 51$ cfs

$Q = 51 - 15 \rightarrow Q = 36$ cfs

WSEA (MAXIMUM FLOW CONDITION) = 4510.16 (APPENDIX H pg. 3)

$q_s = Q/L_w \rightarrow 36 \text{ cfs} / 12 \text{ ft} \rightarrow \boxed{q_s = 3.0 \text{ cfs/ft}}$ (USDA DESIGN NOTE 15

D_A = WSEB (24" MAX FLOW) − WC → 4510.16 − 4509.61

$\boxed{D_A = 0.55 \text{ ft.}}$

$$\frac{q_s^{2/3}}{D_A} = \frac{(3.0)^{2/3}}{0.55} = 3.78$$ (USDA SHEET 3, ES - 207)

$$\frac{D_B}{D_A} = 1.82$$ (USDA SHEET 2, ES - 207)

$D_B = (1.82)(0.55) \rightarrow \boxed{D_B = 1.00 \text{ ft.}}$

WSEB (MAX FLOW) = WC + D_B → 4509.61 + 1.0

$\boxed{\text{WSEB} = 4510.61}$

* CHECK OUTLET SUBMERGENCE

WSEB < WSEB (MAX ALLOWED)

4510.61 < 4511.5 → $\boxed{\text{OK}}$

* SOLVE FOR TOP OF WALL ELEVATION. TW

TW = WSEB (MAX ALLOWED) + FB
= 4511.5 + 0.5

$\boxed{\text{TW} = 4512.0}$

13.0 APPENDIX I – Siphon Outlet Channel Flow Velocities

This appendix includes the Calculations for the Outlet Structure system velocities during the three flow conditions. The average flow has been contained within the 24" system and the remaining flow up to the maximum flow condition of 51 cfs has been contained within the 24" and 36" system.

The calculations for these velocities ensure little to no scouring or erosion of the downstream existing channel.

Date:

Project: Project No.:

Subject: OUTLET STRUCTURES DESIGN - VELOCITY CHECK.

Prepared by: Checked by:

* SOLVE FOR THE OUTLET FLOW VELOCITY TO VERIFY VELOCITY IS BETWEEN 1 fps AND 3 fps. THIS FLOW VELOCITY IS SET TO NOT WASH OUT THE DOWNSTREAM EXISTING CHANNEL OR IMPART ANY NEGATIVE HYDRULIC EFFECTS. DURING ALL THREE FLOW CONDITIONS.

* AVERAGE FLOW CONDITION → Q_{AVG} = 15 cfs.

$Q = V/A$ (CHOW EQ 1-1)

CHANNEL WIDTH = 16.67 ft.
FLOW DEPTH = 0.44 ft.
(APPENDIX H pg. 2)

A = WIDTH x DEPTH
= 16.67 x 0.44
A = 7.335 ft²

$15 \text{ cfs} = V/7.335$ → $\boxed{V = 2.04 \text{ fps}}$

* FLUSHING FLOW CONDITION → Q_{FLUSH} = 40 cfs

$Q = V/A$ (CHOW EQ 1-1)

CHANNEL WIDTH = 16.67 ft.
FLOW DEPTH = 0.84 ft.
(APPENDIX H pg. 2)

A = WIDTH x DEPTH
= 16.67 x 0.84
A = 14.003 ft²

$40 \text{ cfs} = V/14.003$ → $\boxed{V = 2.85 \text{ fps}}$

* MAXIMUM FLOW CONDITION → Q_{MAX} = 51 cfs.

$Q = V/A$ (CHOW EQ 1-1)

A = WIDTH x DEPTH
= 16.67 x .99
A = 16.5 ft²

CHANNEL WIDTH = 16.67 ft.
FLOW DEPTH = .99 ft.

$51 \text{ cfs} = V/16.5$ → $\boxed{V = 3.09 \text{ fps}}$

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14.0 APPENDIX J – Siphon Outlet Rip Rap Design

This appendix includes the calculations for the rip rap design at the Outlet transition from concrete to earthen channel. The design is based on size of rock or class of rip rap and the overall flow velocities through the rip rap.

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME

Sheet: 1 of 1

Date:

Project:

Project No.:

Subject: SIPHON OUTLET - RIP RAP DESIGN

Prepared by:

Checked by:

HYDRODYNAMIC RIP RAP SIZING (USDA, NRCS CH. 2)

* SOLVE FOR HYDRODYNAMIC FORCE, D_{50} ROCK SIZE.

$$D_{50} = 14.2\,(SF)\,(D_{MAX})\,\left(\frac{Se}{K_1}\right)$$

- SAFETY FACTOR (SF)
- MAX FLOW DEPTH (D_{MAX})
- CHANNEL SLOPE (Se)
- BANK ANGLE MODIFICATION FACTOR (K_1)

→ FROM:
- ANGLE OF REPOSE CHART (Φ)
- BANK ANGLE (Θ)

* SEE NEXT PAGE FOR EXCEL SOLUTION.

Proposed Channel Riprap Configuration Sizing

B. Design Relationships

The hydrodynamic force of water flowing in a channel is known as the tractive force. The basic premise underlying riprap design based on tractive force theory is that the flow-induced tractive force should not exceed the permissible or critical shear stress of the riprap. Assuming a specific gravity of 2.50, equation 2-2 can be used to determine D_{50} of the riprap by the tractive stress method (reference 14, page 30).

$$D_{50} = 14.2 \text{ SF dmax Se}/K1 \qquad (2\text{-}2)$$

where,
SF = stability factor
dmax = maximum section depth, feet
Se = average energy grade line slope, ft/ft
D_{50} = median riprap size in feet
K1 = bank angle modification factor (see eq'n 2-3 or Figure 2-4)

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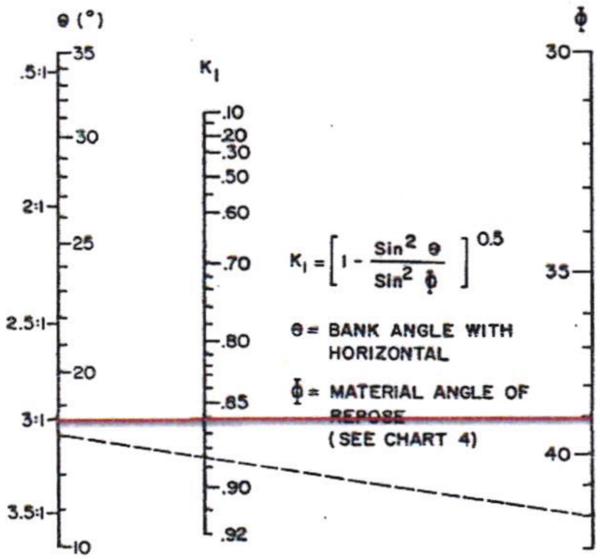
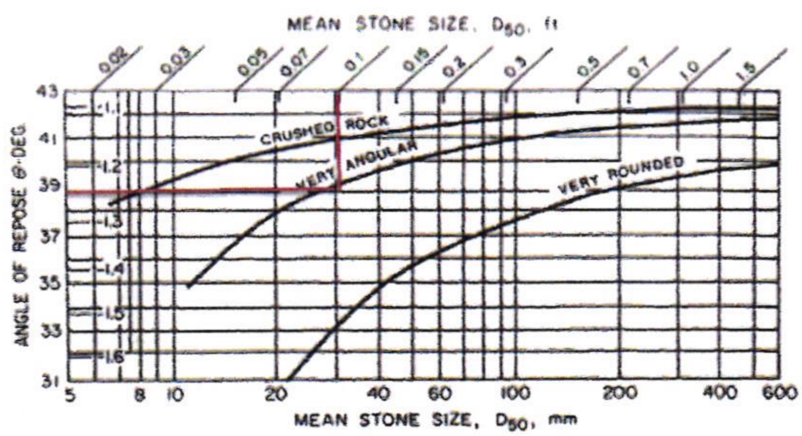
** United States Department of Agriculture - USDA
** Natural Resources Conservation Services - NRCS Chapter 2



Angles of Repose of Riprap Stones (FHWA)



OUTLET - Channel

SF	2
D_{MAX}	3 ft
S_e	0.1 %
K1	0.86

Channel Width	16.00 ft
d_{50}=	0.10 ft
Riprap Size=	USE CLASS 150
Depth=	1.50 ft

Riprap Size= **USE CLASS 150**
Depth= **1.50 ft**