Austin Bluffs Tank Replacement and Pump Station Construction Drainage Variance Request Letter

BLACK & VEATCH PROJECT NO. 422060
BLACK & VEATCH FILE NO. 30.3000

PREPARED FOR

ON BEHALF OF

18 MARCH 2026

Table of Contents

LIST OF FIGURES

Signature Page

<u>Engineer's Statement:</u>

This letter is requesting a drainage variance for the Austin Bluffs Tank Replacement and Pump Station Construction and is correct to the best of my knowledge and belief. Said report and plan has been prepared in accordance with the City of Colorado Springs Drainage Criteria Manual and is in conformity with the master plan of the drainage basin. I understand that the City of Colorado Springs does not and will not assume liability for drainage facilities designed by others. I accept responsibility for any liability caused by any negligent acts, errors, or omissions on my part in preparing this report.

SIGNATURE (Affix Seal):

____

Colorado P.E. No: 51238, Date: March 18, 2026

1.0 Project Statements

1.1 Project Description

The Austin Bluffs Tank Replacement and Pump Station project includes replacement of an existing 5-million-gallon steel water storage tank with a new 5-million-gallon ANSI/AWWA D110 Type III wire-and-strand-wound, circular, prestressed concrete water tank and associated yard piping. The concrete water tank will be similar in appearance and color to the existing tank. A new 32 million gallon per day pumping station will be constructed west of the new 5-million-gallon tank. The pumping station architectural design will incorporate materials and finishes that complement the surrounding environment, maintain visual consistency with nearby structures and comply with the Hillside Overlay requirements. Landscaping and site improvements will be designed to blend with existing terrain and vegetation, minimizing visual impact. The project will be executed in two phases, it is anticipated that the tank will be replaced in 2026, and the pump station will be constructed in 2027 through 2028. All infrastructure on the site will be owned by Colorado Springs Utilities.

In order to comply with grading and drainage requirements outlined in the Colorado Springs Stormwater Enterprise (SWENT) 4-Step Process discussed in the Colorado Springs Drainage Criteria Manual Chapter 8, an extended detention basin (EDB) is proposed southeast of the pump station building and southwest of the 5-million-gallon tank. Due to the site having over 50% type D soils, volume reduction is not required per step 1 of the 4-step process outlined in Chapter 8, section 4.2 of the Drainage Criteria Manual (DCM), the runoff reduction tab of the MHFD-BMP will be completed in compliance with step 1 in the Final Drainage Report. The EDB will provide slow release (greater than required 40 hour drain down time) for the entire water quality capture volume (WQCV) to comply with step 2 of the 4-step process in Chapter 8, section 5.0 of the DCM. The EDB will drain through a proposed 24” RCP connecting to an existing 30” CMP with eventual outfall at an existing stable channel beyond the Colorado Springs Utilities property in Open Space. This channel was verified as stable with a site visit and can handle existing flows. The EDB will reduce post-development peak flows to be less than the pre-development condition that exists on site currently for storm events up to the 100-yr storm in accordance with the Redevelopment clause of the DCM in Chapter 8, section 2.4.2 that states “Detention is required for all redevelopment projects that disturb 1 acre or more unless the downstream existing drainage system can be shown to have adequate capacity for anticipated flows, including flows from the redevelopment site.” Because peak flows will be reduced up to the 100-yr event and the downstream channel is currently stabilized with heavy vegetation including mature trees, no improvements to the existing channel are necessary to comply with step 3 in Chapter 8, section 6.0. This project will comply with all site specific source control required by Chapter 8, section 7.0 of the DCM to meet criteria for step 4.

1.2 Project Justification

The Austin Bluffs Tank Replacement and Pump Station Project is part of the planned Austin Bluffs Tank, Pump Station and Pipeline Improvements Master Project within the Colorado Springs Utilities Pump Station Facility Plan to improve reliability and reduce vulnerabilities in water deliveries for Utilities water customers. The existing 5-million-gallon steel water storage tank has reached the end of its useful life, having been in service for over 60 years. Continued operation of this aging infrastructure poses increased risk, maintenance challenges, and potential service disruptions. Replacing the steel tank with a new 5-million-gallon concrete water storage tank will enhance durability, reduce long-term maintenance costs, and improve system resiliency. Additionally, the construction of a new 32-million-gallon-per-day pumping station will provide the capacity required to support regional growth and ensure adequate water supply during peak demand periods or emergency outages. This project is necessary to ensure the health, safety, and welfare of the public.

1.3 Project Variance Request Statement

A variance request for several modifications to the standard design for an EDB described within the DCM is requested. The following variances result from restrictions inherent to the existing site conditions, property boundaries, and the location and layout of Colorado Springs Utilities water infrastructure that is necessary for the health, safety, and welfare of the public. This letter seeks to lay out and justify variances related to the following criteria:

Section 3.0: Emergency Overflow

An emergency overflow/spillway as described in Chapter 13 – Storage, section 5.12 of the DCM cannot be constructed at the site. Emergency spillways are typically designed to safely convey flows greater than the 100-yr storm. The project alternative seeks to be able to safely convey all flows that occur up to 159% of the 500-yr storm, equal to the maximum capacity of the outlet discharge pipe, through a separate overflow weir structure connected to the outlet discharge pipe.

Section 4.0: EDB outlet structure and outflow pipe elevation

The typical outlet structure detail provided in the Drainage Criteria Manual shows the outlet pipe with an invert roughly equal to the invert of the micropool. The project variance seeks to set the outlet pipe at the highest elevation to where the basin can be completely drained so that the basin volume can be maximized with a fixed outlet pipe elevation. This change in the outlet structure will not affect the performance of the EDB.

Section 5.0: EDB side slope, bottom slopes and low-flow channel

DCM Chapter 13, paragraph 5.4, sentence 1 states that "All excavated or embankment slopes from the pond bottom to the 100-year water surface elevation should be no steeper than 4 feet horizontally to 1 foot vertically (4H:1V) for stability when soils are saturated..." A 4H:1V sloped-basin is not practical for use on the site, and a 3H:1V-side sloped basin is proposed instead.

DCM Chapter 13, section 5.7 states that "For grassed detention facilities, the pond bottom shall be sloped at least 4 percent for the first 25 feet and at least 1 percent thereafter to drain toward the low-flow channel or outlet, measured perpendicular to the low-flow channel." The bottom of the basin cannot be sloped at these higher rates and have a traditional low-flow channel without posing flood risks to critical infrastructure based on existing site conditions. Interior slopes of 0.5% are requested to be used with a fully-vegetated lining instead of the presence of a concrete low-flow channel due to low input flows.

Section 6.0: EDB freeboard requirements

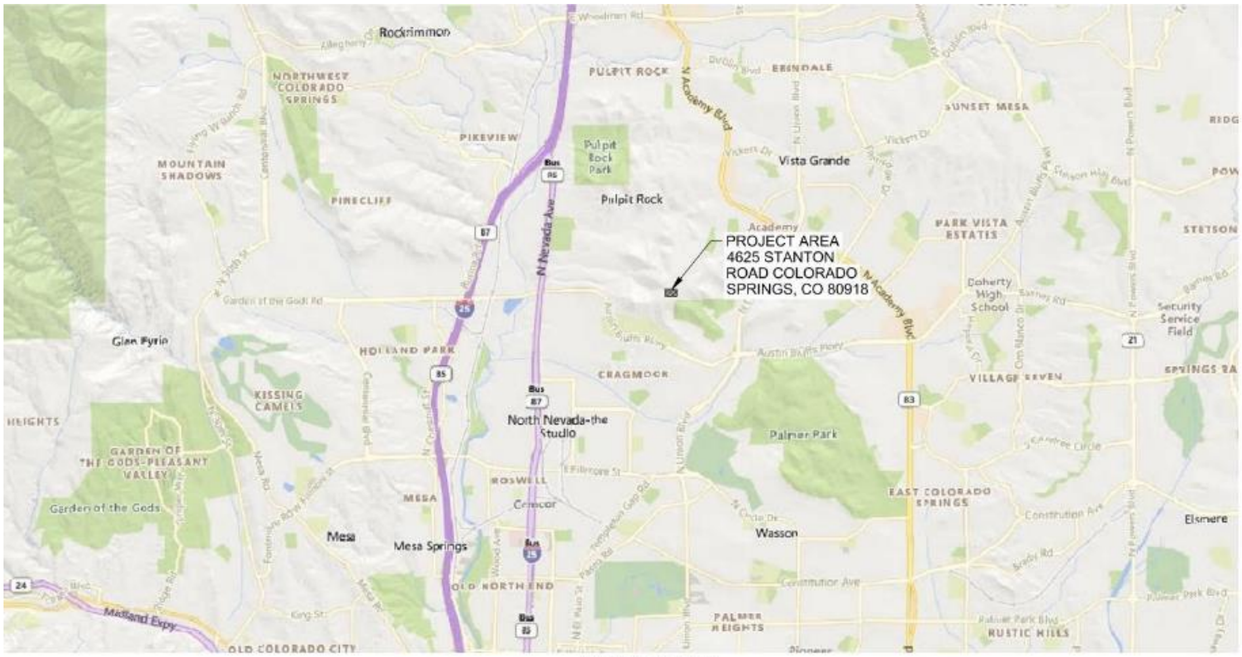
The proposed EDB will not have 1.0' of freeboard between the 100-yr storm maximum water level and the top of the basin due to existing site constraints, the elevation of the existing 30" discharge CMP on the site, and the need for the basin to have enough volume to meet 4-step water quality and detention requirements associated with downstream channel stability. The basin will have 0.78' (9.4") of freeboard instead, along with an additional 9" of freeboard between the top of the basin and the first floor elevation (i.e. parking lot storage). The planned freeboard provides an adequate margin of safety to prevent flooding on the site.

The variances requested in this letter are necessary for the proposed basin to meet the 4-step process as described in Section 1.1. None of the variances described in this letter will result in a change in peak flows and water quality in Fountain Creek.

2.0 Site Description

2.1 Vicinity Map

The project area is located approximately 1.8 miles east of the intersection between Interstate 25 and Garden of the Gods Road in Colorado Springs, Colorado.

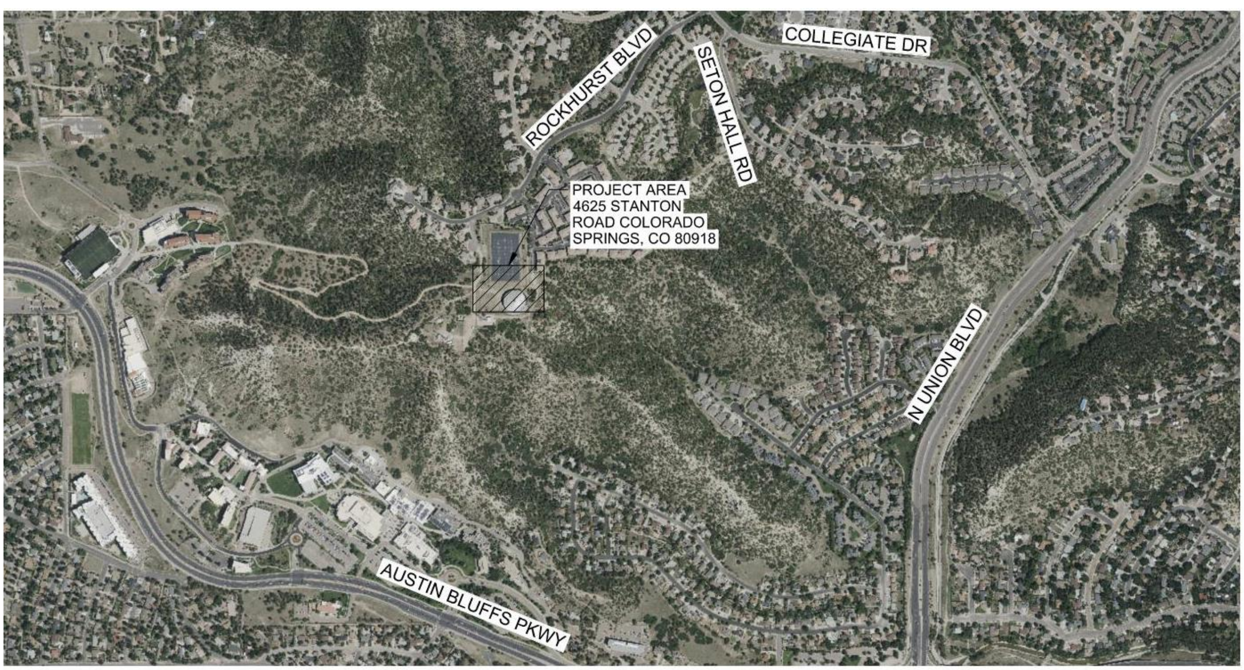


VICINITY MAP

Figure 2-1 – Vicinity Map

2.2 Area Map

The project is in the vicinity of 4625 Stanton Road, Colorado Springs, Colorado, 80918. The proposed enumerations project address is 4535 Marconi Heights, Colorado Springs, Colorado, 80918.

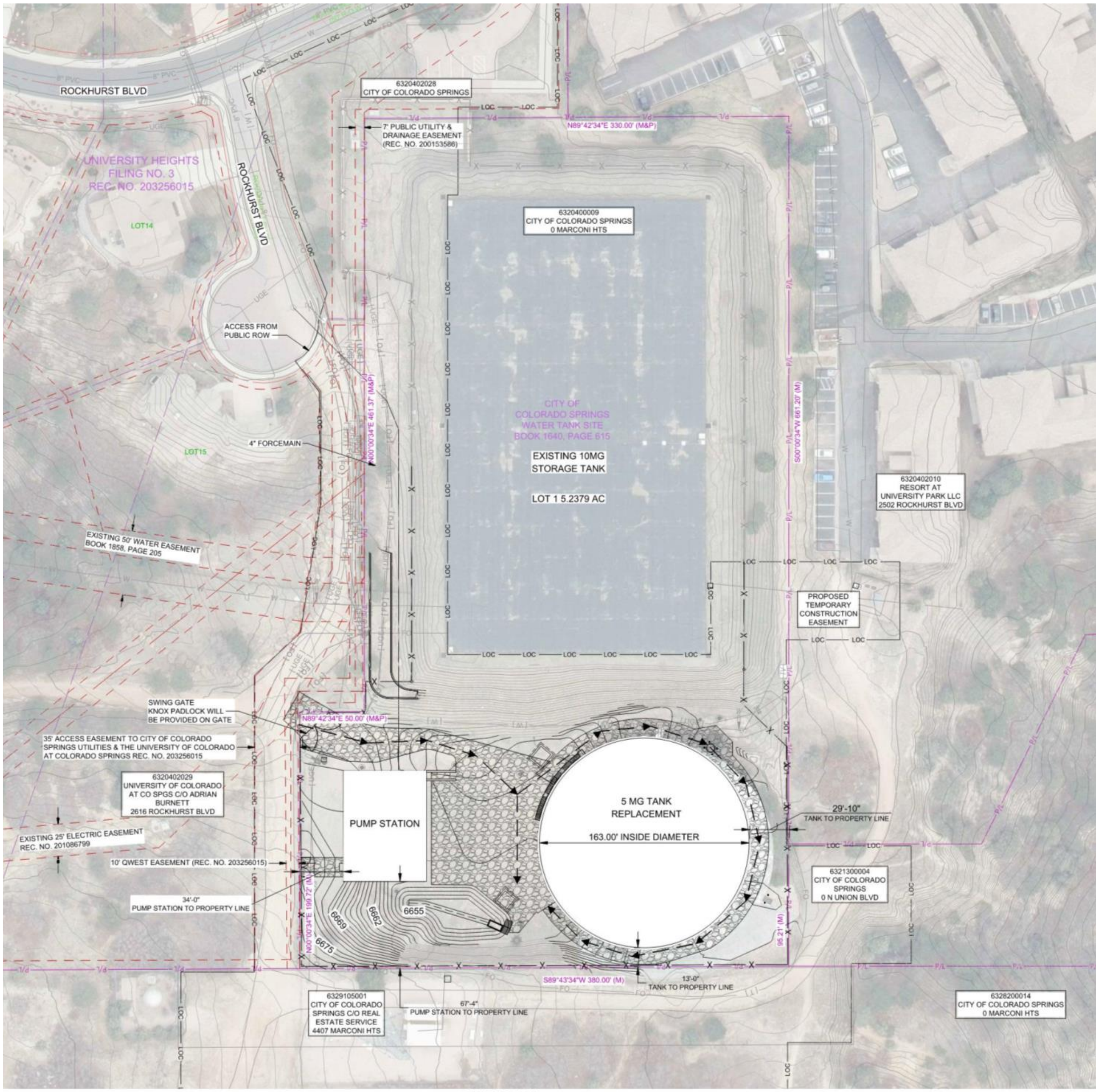


AREA MAP

Figure 2-2 – Area Map

2.3 Site Area Description

The site where the proposed variances are requested is a 5.2379 acre site located on parcels 6320400009 and 6320400004 within the City of Colorado Springs. The area drained by the proposed basin PCM is an approximately 2.45 acre site located generally south and east of the proposed pump station and is marked as Drainage Basin DA-1. This enlarged area viewpoint in Figure 2-3 covers roughly the southern one-third of the 5.2379 acre site. The name of the plat for the site is Austin Bluffs Pump Station & Tank Site.



SITE PLAN

Figure 2-3 –Site Area

2.4 Proposed PCM Extended Drainage Basin

The proposed drainage basin is located on a roughly 3,000 sq. foot area south and southeast of the pump station. It has a minimum elevation of 6653.25' and a maximum elevation of 6657.50', with a corresponding outlet structure that is approximately 16' by 6'. The main purpose of the planned extended drainage basin is designed to detain the WQCV over a greater-than 40-hr drain down time, meeting the water quality section of Step 2. **Additional storage reduces the 100-yr peak flow leaving the site from 10.0 cfs to 7.8 cfs**. The table below presents the outflows from the site before and after the planned improvements. The existing site flows were calculated using the rational method standards approved for use in Chapter 6, Section 3.0, and are presented in Appendix C, Rational Calculation for the Pre-Construction Condition.

Table 2-1 – Comparison of Stormwater Flowrates from the Austin Bluffs Site Pre- and Post-Construction

Design Storm	Pre-Construction Flows from Site (cfs)	Post-Construction Flows from Site (cfs)
2-year	2.0	0.20
5-year	3.3	0.63
10-year	4.8	1.0
25-year	6.7	3.7
50-year	8.3	5.7
100-year	**10.0**	**7.8**

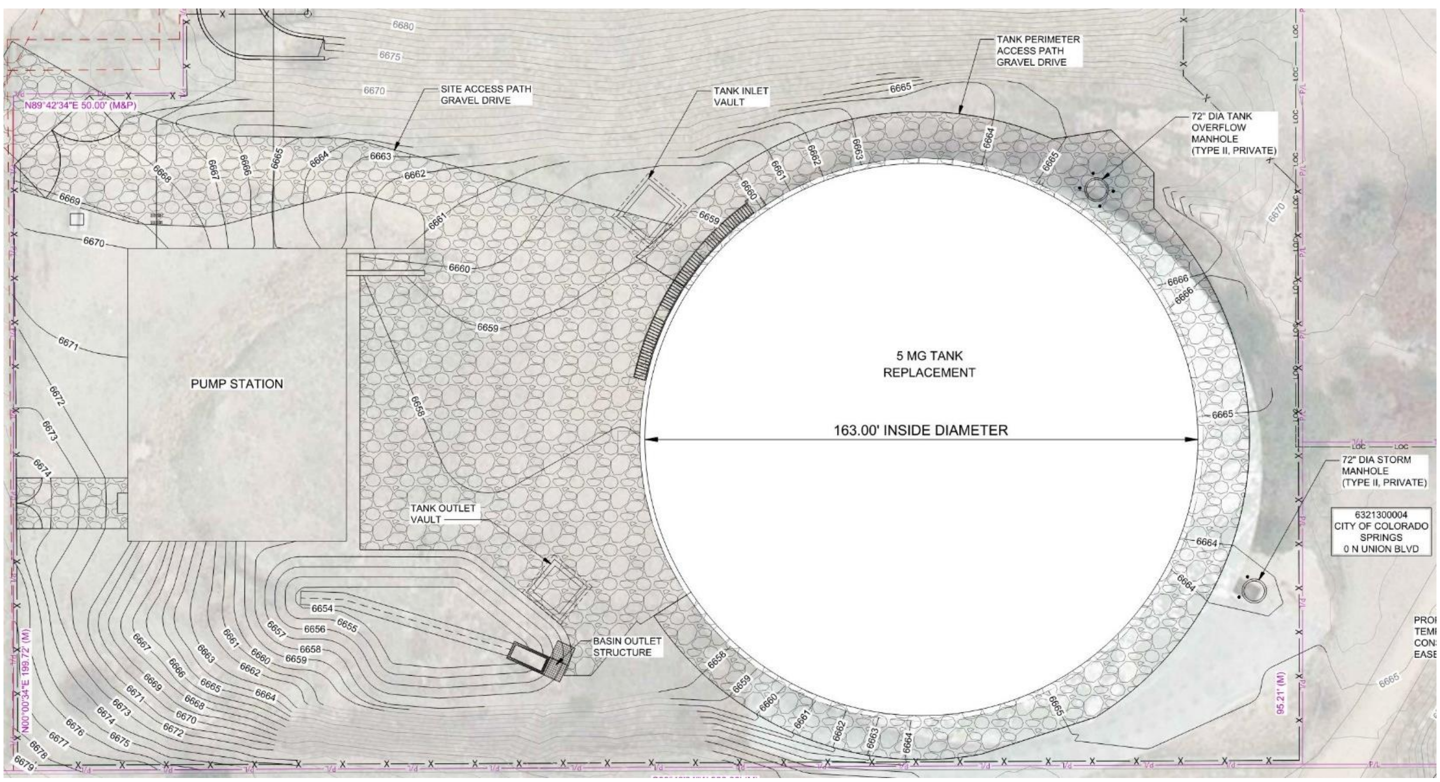


Figure 2-4 – Enlarged Site Plan with Proposed Drainage Basin PCM, Outlet Structure, and Low-Flow Channel

3.0 Emergency Overflow

An emergency overflow/spillway as described in Chapter 13 – Storage, section 5.12 of the DCM is not able to be constructed at the site. Emergency spillways are typically designed to safely convey the 100-yr storm. The project alternatively proposes to include a separate emergency overflow structure connected to the standard overflow structure that will allow for the safe conveyance of up to 159% of the 500-yr storm. A 30" wall pipe will connect the emergency overflow inlet hydraulically to the normal inlet and common 24" discharge pipe.

3.1 Emergency Overflow Variance

Instead of having large storm events discharge downstream through a hardened low point on an EDB embankment, all normal flows will be directed to a sloped overflow weir structure and proposed 24" RCP outlet, existing 30" CMP and ultimately outfall to an existing downstream channel. If the standard outlet structure gets clogged, the water level in the EDB will rise until it reaches the inlet grating on the emergency overflow structure. Flows from the emergency overflow will pass through the 24" RCP discharge pipe. The 24" RCP inlet piping will be sized for flows exceeding the 500-yr storm to allow for conveyance of excess flow. The 500-year storm can be conveyed through the standard outlet structure. The emergency overflow inlet would only be used for storms beyond this level, or if there is an issue with the standard inlet. A variance for the project to use a separate emergency overflow structure that is hydraulically connected to the normal overflow structure in lieu of a standard embankment overflow is requested. See Figure 3-1 for a plan view of the EDB showing the location of the emergency overflow structure to the north of the standard basin outlet structure.

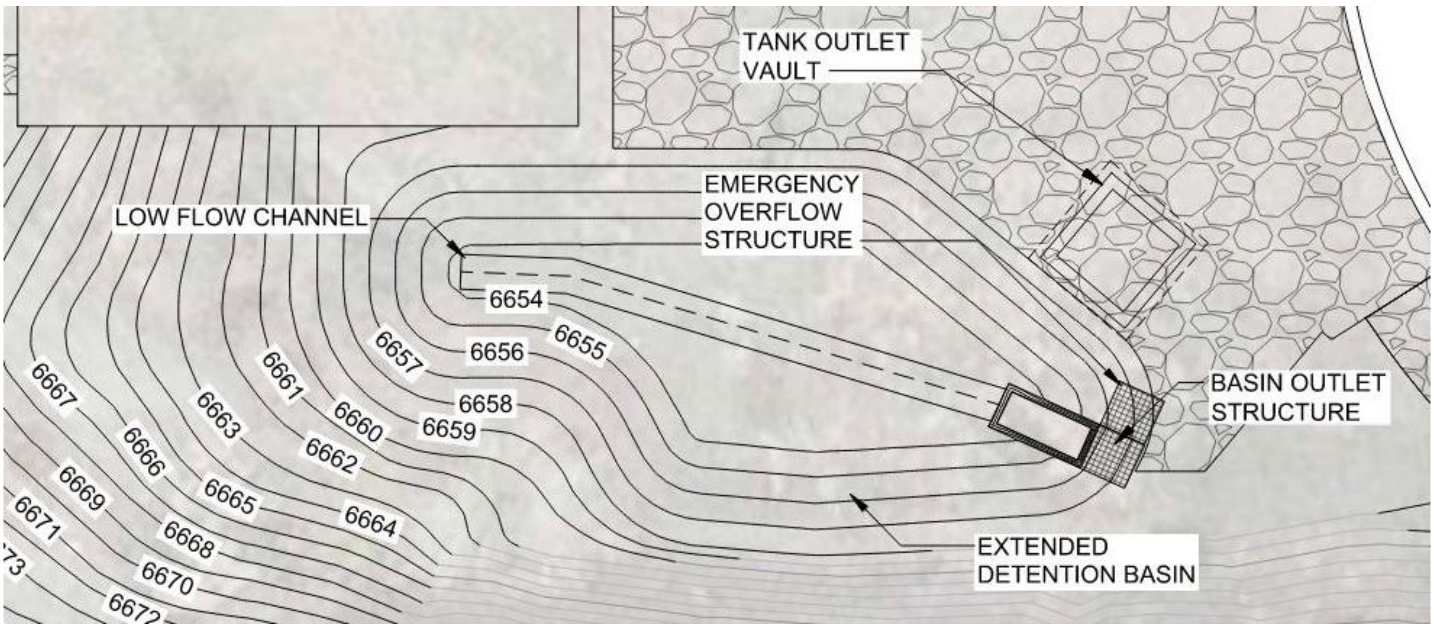


Figure 3-1 – Plan view of the EDB Showing Important Structures, Including the Emergency Overflow Structure

3.2 Emergency Overflow Justification for Variance

The project site, located near the top of Austin Bluffs, has limited usable area for the use of PCMs mentioned within the DCM due to the redevelopment of an existing site. Physical site constraints connected to the site location at the top of Austin Bluffs combined with existing infrastructure on the site, and the need for vehicle access and maintenance areas for operations and maintenance of the Colorado Spring Utilities water storage and pumping facilities, constrain design of PCMs to the exact standards mentioned in the DCM.

The site and drainage area is generally located within a "bowl" shaped area that was originally cut in the 1960s when a 10MG underground reservoir and the original 5MG steel tank were constructed onsite. This excavated "bowl" shape only allows for gravity flow of stormwater out of the site through the existing culvert and proposed storm sewer piping connecting the PCM to the outfall. Additionally, the proposed basin will not have a traditional EDB embankment (and associated risk of catastrophic failure) with the lowest elevation onsite being located within the basin. A traditional emergency spillway will not be able to function because there is a lack of fall to direct flows to stormwater infrastructure beyond the basin, and lack of space due to required Colorado Springs Utilities water critical infrastructure onsite. New and existing stormwater infrastructure is sized to convey an equal flow through the proposed normal and emergency outlet structures equivalent to an emergency spillway.

The existing site infrastructure includes a 30" CMP culvert southeast of the tank, which will be connected to the EDB outlet structure by approximately 220' of new 24" RCP. The new 24" pipe is sized to be able convey the 500-yr storm of 13.9 cfs (See Appendix E, MHFD Calculations), and will able to safely convey additional flow without surcharging up to 16.04 cfs (See Appendix F, UD-Sewer Calculation, 22 cfs). Given the proposed top of basin elevation of EL 6657.5', the proposed system will be able to convey up to 22 cfs without overtopping the basin in an extreme situation. Given the site's inability to have an emergency spillway, and the exceptional nature of not only reaching flows beyond the 500-yr storm, but also flows in excess of 159% of the 500-yr storm, the proposed overflow solution is reasonable. The proposed emergency overflow structure will be able to convey these higher flows without basin overtopping. The only path for surface flow to leave the site by gravity and flow through an acceptable drainage course is through the proposed 24" RCP and existing 30" CMP. The completion of a second, gravity-fed way for water to leave the site is not possible. A separate emergency overflow structure will allow for redundancy in the event of a large storm or blockage of the normal inlet structure while meeting site constraints.

4.0 EDB Outlet Structure

The typical outlet structure detail provided in the Drainage Criteria Manual shows the outlet pipe with an invert roughly equal to the invert of the micropool. The project variance seeks to set the outlet pipe at the highest elevation to where the basin can be completely drained. This change in the outlet structure will not affect the performance of the EDB.

4.1 EDB Outlet Structure Variance

DCM Chapter 13, Figure 13-10 shows the typical detail for an EDB outlet structure. The outlet pipe invert elevation is shown as relatively equal to the invert of the micropool.

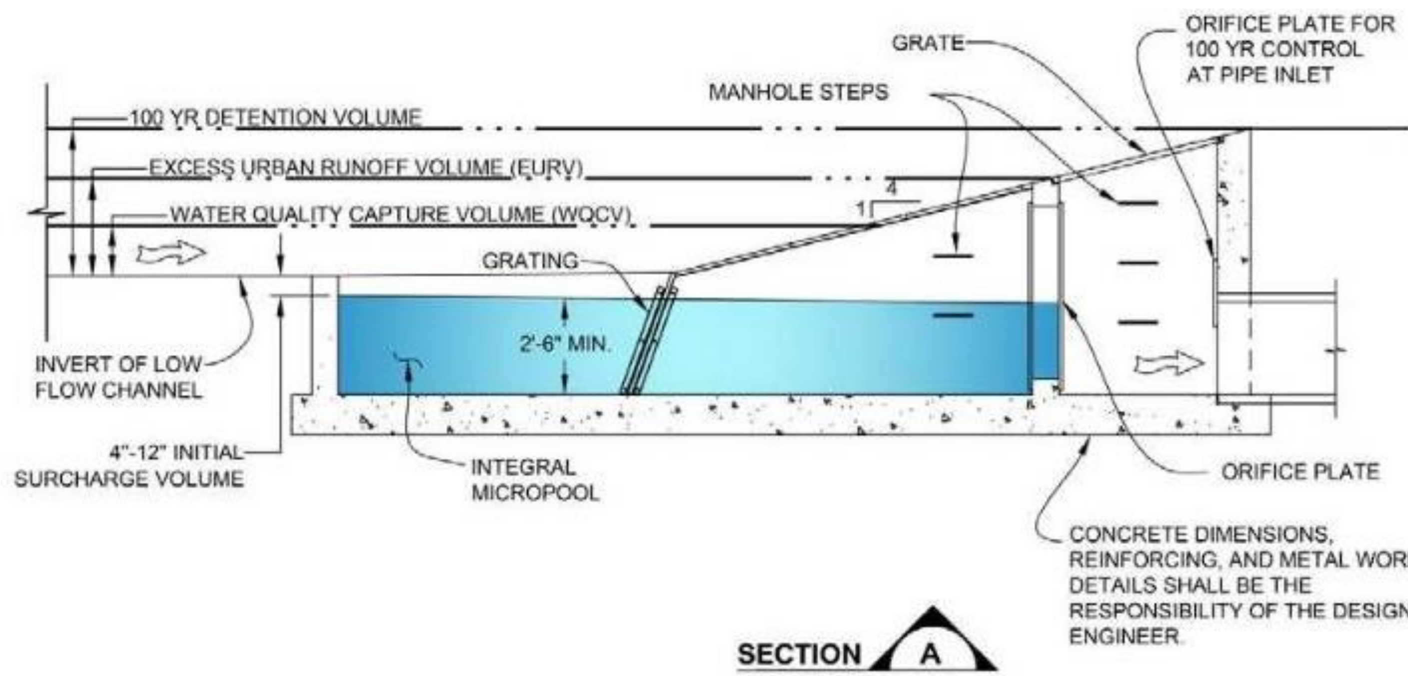


Figure 4-1 – DCM Chapter 13, Figure 13-10: Concept for Outlet Structure with Parallel Wingwalls

This arrangement is not possible with the current site conditions because of the constraints placed on the site by the existing outlet pipe. If the outlet pipe were to be equal to the level shown in the Figure, the depth of the basin would be less than 2’ and would not be able to perform as intended and risk flooding of critical infrastructure necessary for life safety, health and welfare of the public. The outlet pipe will be raised over 2 feet in the proposed basin and still allow the basin to drain completely. A plan and section view of the new outlet structure that will allow the EDB to perform as expected is shown below in Figure 4-2 through 4-5. A 30” wall pipe will connect the emergency overflow inlet to the normal inlet and 24” stormwater pipe.

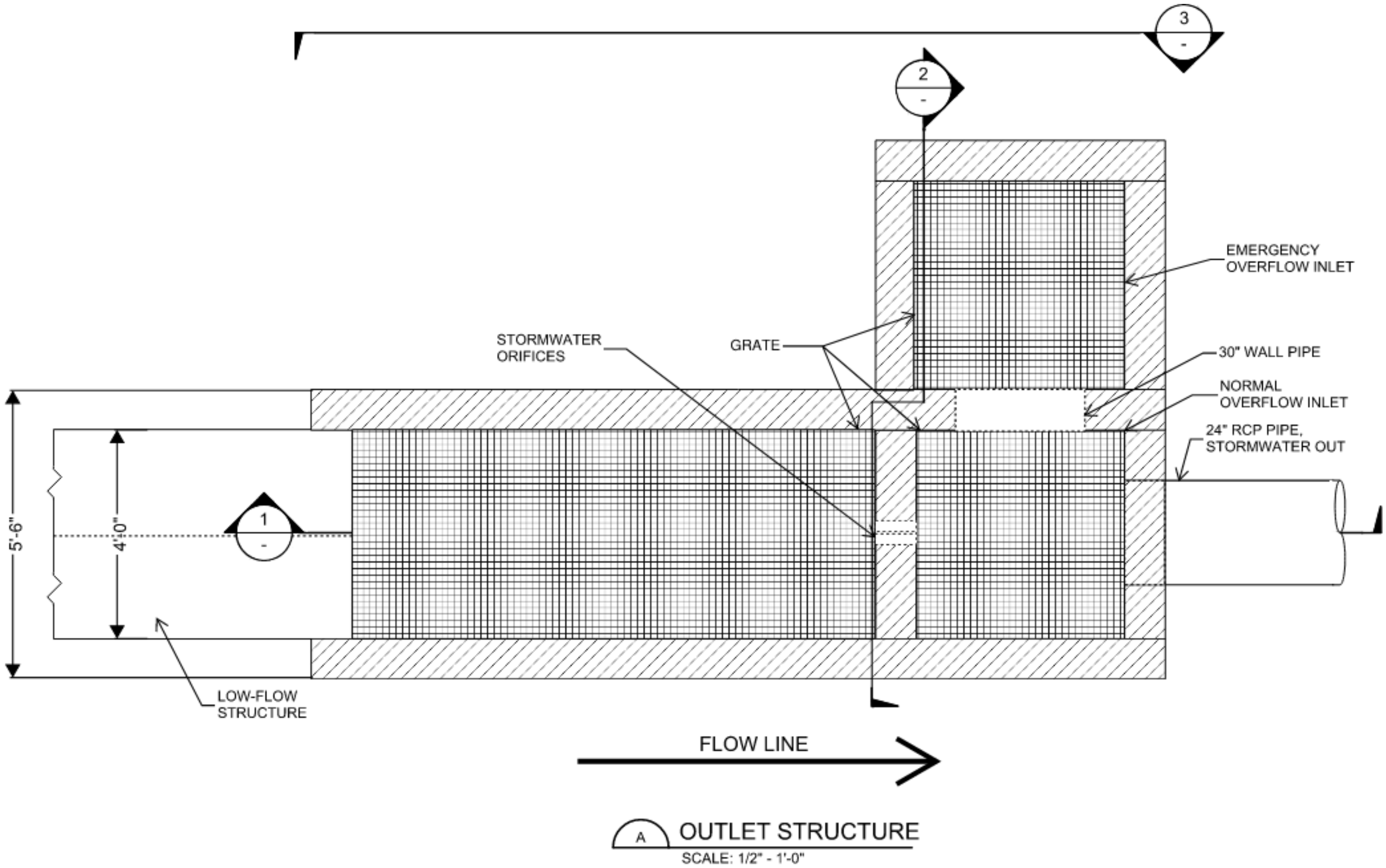


Figure 4-2 – Proposed Outlet Structure Plan

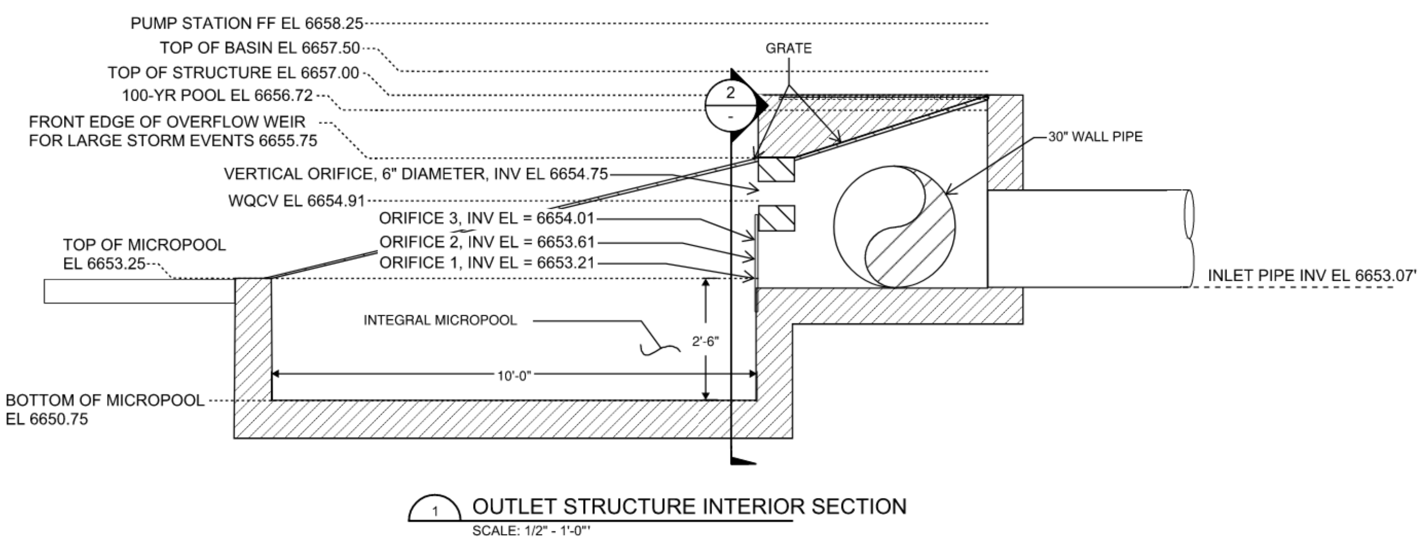


Figure 4-3 – Proposed Outlet Structure Interior Section

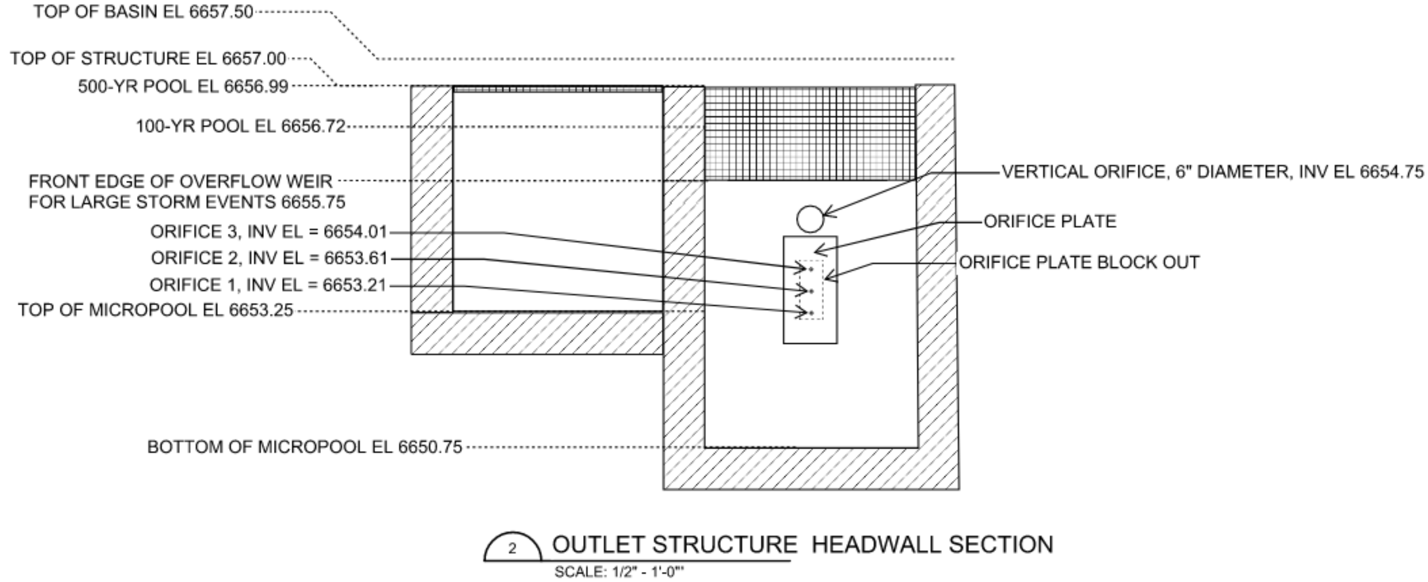


Figure 4-4 – Proposed Outlet Structure Headwall Section

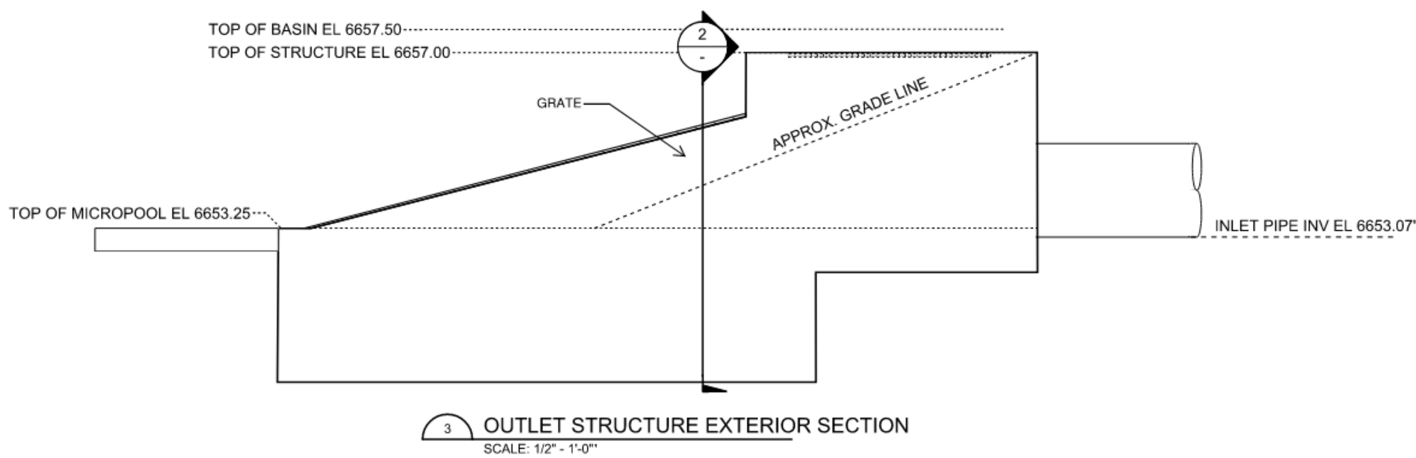


Figure 4-5 – Proposed Outlet Structure Exterior Section

4.2 EDB Outlet Structure Justification for Variance

It is not possible to make the outlet pipe deeper than currently proposed, as the basin depth is constrained by the existing culvert installed on the site in the 1960s. The minimum outlet pipe elevation at the basin must be 6653.07' or higher to ensure 0.5% slope in the 24" outlet storm pipe draining the proposed basin in line with Chapter 9, section 7.1.3 of the DCM. If the outlet pipe were to match the depth of the micropool, the basin depth would be less than 2' deep. The basin would not meet the planned WQCV and detention requirements that are necessary to meet the 4-step process without encroaching on critical Colorado Springs Utilities Water Infrastructure necessary for the health, safety and welfare of the public. The proposed basin and 24" storm line location maintains as much of the existing slopes on the site as possible. The site location is within the Hillside Overlay zone of Colorado Springs, and the use of large amounts of cut and fill along with steep final slopes in this area necessary to lower the existing outlet culvert is discouraged in the Hillside Overlay according to the City of Colorado Springs City Code 7.2.610.D.

Raising the basin outlet pipe to more closely match the invert of the lowest orifice instead of the invert of the micropool is the only solution. The purpose of a micropool is to “create a permanent pool of water on which debris will float.” (DCM Chapter 13, section 5.10, paragraph 2.) The micropool is therefore not drained by the outlet pipe. The invert of the outlet pipe can be raised sufficiently to drain the flow from the bottom-most hole in the proposed orifice plate. The flow leaving the bottom-most orifice plate hole is 0.08 cfs per the MHFD spreadsheet. Using the UD sewer model of the outlet pipes at 0.08 cfs (See Appendix E, UD-Sewer Calculation, 0.08 cfs), the HGL of the water on the upstream side of the 24” inlet pipe must be at least 6653.18’. The planned centroid elevation of the lowest orifice hole is 6653.25’, and so the basin will be able to drain completely using a pipeline elevation of 6653.07’. The larger basin depth resulting from this solution is the only way to allow the EDB to meet the 4-step process requirements.

5.0 EDB Side Slopes, Interior Slopes, and Low Flow Channel

DCM Chapter 13, paragraph 5.4, sentence 1 states that “All excavated or embankment slopes from the pond bottom to the 100-year water surface elevation should be no steeper than 4 feet horizontally to 1 foot vertically (4H:1V) for stability when soils are saturated...” A 4H:1V sloped-basin is not practical for use on the site, and a 3H:1V-side sloped basin is proposed instead.

DCM Chapter 13, section 5.7 states that “For grassed detention facilities, the pond bottom shall be sloped at least 4 percent for the first 25 feet and at least 1 percent thereafter to drain toward the low-flow channel or outlet, measured perpendicular to the low-flow channel.” The bottom of the basin cannot be sloped at these higher rates and have a traditional low-flow channel without posing flood risks to critical infrastructure based on existing site conditions. Interior slopes of 0.5% are requested to be used with a V-type concrete low-flow channel.

5.1 EDB Side and Interior Slope Variance

The proposed EDB will utilize 3H:1V basin side slopes and 0.50% interior slopes. A schematic basin with the proposed slopes are shown in Figure 5-1. The usage of these slopes will allow for the capture of the WQCV and of the additional flows generated from redevelopment of the site up to the 100-yr event in accordance with the redevelopment clause in Chapter 8 paragraph 2.4.2. The proposed basin meets the size and water quality requirements for the site, while also minimizing earthwork and extensive slope rehabilitation.

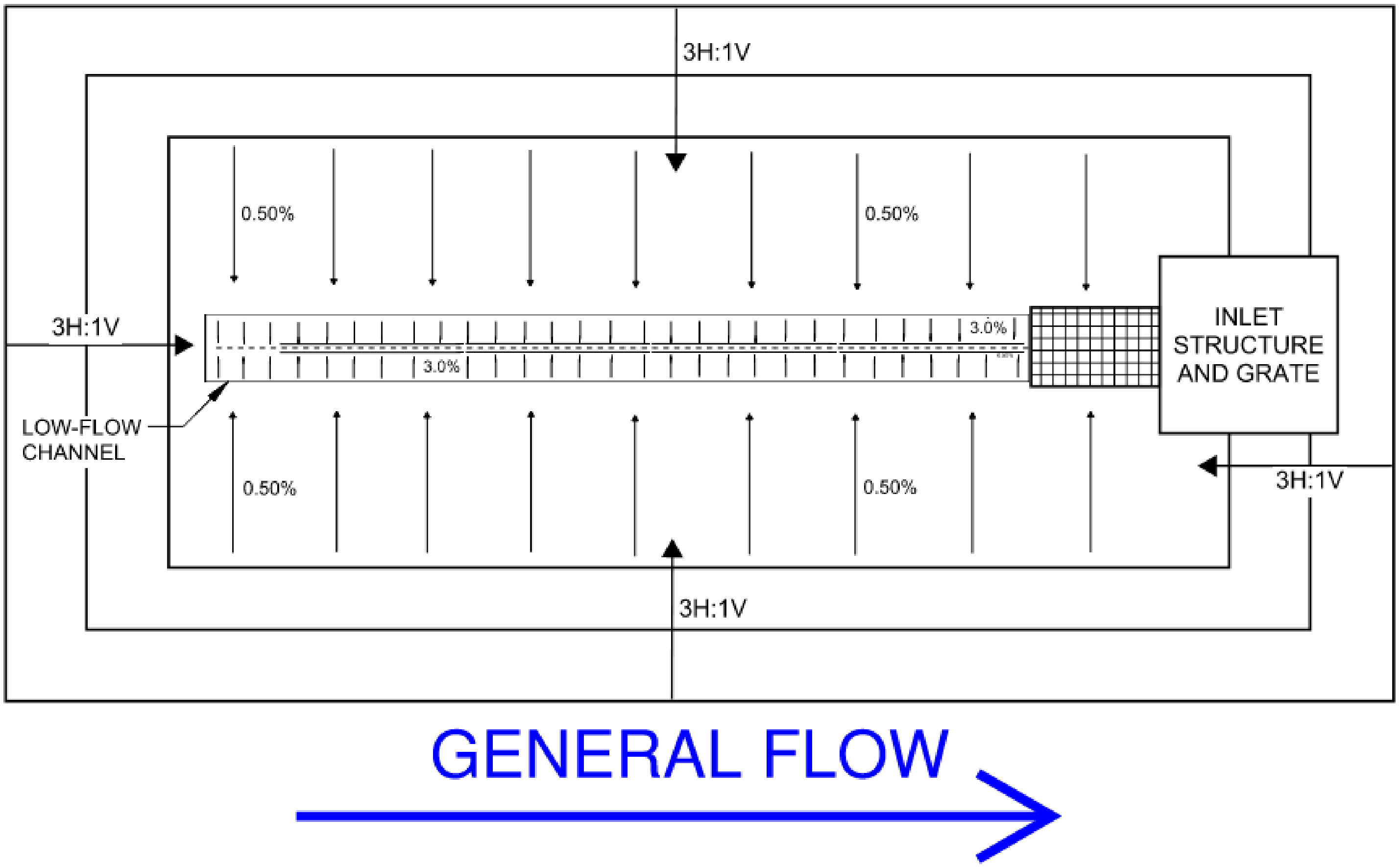


Figure 5-1 – Schematic of the Proposed Slopes to be Used in the Austin Bluffs Site Stormwater Basin

5.2 EDB Side and Interior Slope Justification for Variance

DCM Chapter 13, paragraph 5.4, sentence 2 states that "Steeper slopes, up to 3H:1V, may be allowed when the site is constrained." The site of the proposed improvements is constrained. The project site, located near the top of Austin Bluffs, has limited flat space available for the use of PCMs mentioned within the DCM. Physical site constraints connected to the site location at the top of Austin Bluffs combined with existing infrastructure on the site, and the need for vehicle access and maintenance area for operations and maintenance of the critical Colorado Spring Utilities facilities, constrain traditional PCMs to be unable to meet the exact standards mention in the DCM.

A 4H:1V-sloped basin with bottom-slopes of 1 and 4% that fits within the site and does not impact critical Colorado Springs Utilities infrastructure would risk flooding to critical infrastructure and is therefore unsuitable. Expansion of the surface area of the basin to the north would block access to the pump station electrical room access door or the pump station overhead maintenance door, both of which are critical life safety and operations and maintenance accessways for necessary infrastructure for the health, safety and welfare of the public. Expanding the basin to the west by completing a large cut into the existing slope south of the pump station is also prohibitive. The use of large amounts of cut and fill in this area to produce a new area that is both large and flat enough within the space provided for a basin with the recommended slopes is not possible under the Hillside Overlay section of the City of Colorado Springs City Code 7.2.610.D., as slopes greater than 2:1 are prohibited per the City of Colorado Springs Hillside Design Manual, and the height and concentration of retaining walls is also restricted. The elevation of the outlet pipe is also fixed in place and shallow. It is not possible to make the outlet pipe deeper than currently proposed, as the basin depth is constrained by the existing culvert installed on the site in the 1960s. The minimum outlet pipe elevation at the basin must be 6653.07' or higher to ensure 0.5% slope in the 24" outlet storm pipe in line with Chapter 9, section 7.1.3 of the DCM.

In Chapter 13, section 5.6, paragraph 1 states "Detention ponds collect both wet and dry weather flows from the upstream basins, including excess irrigation water that can keep pond bottoms wet and difficult to maintain. Therefore, all grassed-bottom detention ponds shall include a low-flow channel sized to convey a minimum of 1 percent of the 100-year peak inflow." Given the vertical elevation constraints related to the site, a traditional low-flow channel as shown in DCM is not possible. 1% of the peak 100-year flowrate for the site is approximately 0.1 cfs (See Appendix E, MHFD Calculations). This value is 2.4% of the 4.21cfs flow capacity of the standard 6" deep, 2' wide concrete-lined channel sloped at 0.5% shown in Figure 13-6 of the DCM (See Appendix D, Manning's Calculations for Low Flow Channels). The typical designs in the DCM are not sized for the basin type and site specific constraints, and is not appropriate given the vertical space available for stormwater infrastructure on the Austin Bluffs site is extremely limited. An alternate design for a low-flow channel, to accomplish the same effect, is proposed. This includes a concrete v-type gutter that is 4' wide with wings sloped at 3%, and a longitudinal slope of 0.5%. This proposed low-flow channel will be able to convey 0.101 cfs (See Appendix D, Manning's Calculations for Low-Flow Channels).

Incorporating the designs in the DCM would lead to the basin not meeting the planned requirements that are necessary to meet the 4-step process for this site as described in Section 1.1. The depth of the basin would have to be reduced over the same basin surface area, and the basin surface area cannot be expanded to compensate without impacting critical water infrastructure necessary for the safety, health and welfare of the public, or existing property boundaries.

The interior slopes and the slope of the bottom of the basin not within the low-flow channel are proposed to be 0.50%, as this is a generally accepted value for the minimum slope to easily convey the flow of water. This value is referenced several times in the DCM as it relates to other conveyance structures that are sloped and convey flow, such as stormwater pipes (Chapter 9, Section 7.1.3), and for low-flow channels themselves (Chapter 13 Section 5.6).

6.0 EDB Freeboard

6.1 EDB Freeboard Variance

The proposed EDB is not able to meet the 1.0' (12") of freeboard between the 100-yr water surface elevation and the top of the basin. The freeboard between the 100-yr storm event and the top of the basin is 0.78' (9.4"). The proposed freeboard is limited, 2.6" less than the criteria. The top of the proposed basin is optimized to the greatest extent possible and set at the highest feasible elevation based on the fixed finished floor elevation at the pump station (EL 6658.25') required for hydraulics and pump maintenance and minimum parking area slopes. In the unlikely event that the EDB is overtopped the parking lot will have the capacity to store the temporary water .

6.2 EDB Freeboard Justification for Variance

DCM chapter 13, section 5.5 states "The minimum required freeboard for detention facilities is 1.0 foot above the computed water surface elevation when the emergency spillway is conveying its design flow, except as defined in Section 6.0, Parking Lot Detention." As already discussed in section 3.0 of this letter, the planned PCM basin is not planned to have a traditional emergency spillway and embankment, as existing site constraints restricting the depth of the basin. Additional freeboard/depth above the designed amounts cannot be added. There are only two ways that depth could be added, and neither can be achieved: 1. Expanding the basin footprint, 2. Increasing the depth of the basin. Expanding the basin is not possible, as there are either steep slopes associated with the original 1960s construction, or required Colorado Springs Utilities infrastructure constricting the area. The depth of the basin cannot be increased due to Hillside Overlay requirements, which seek to limit the amount of fill and the height of buildings within the zone according to the

City of Colorado Springs City Code 7.2.610.D. The proposed basin is designed to be as large as possible and achieve the required performance to meet criteria given the site conditions. The 100-yr pool elevation of the proposed basin is EL 6656.72', and the top of the proposed basin is 6657.50'. This is 0.78', and is less than what is stated in DCM chapter 13, section 5.5 (See MHFD Calculation in Appendix E, MHFD Calculations). Table 6-1 shows the heights of various important elevations within the basin design, such as the micropool elevation, the WQCV elevation, and the 100-year storm.

Table 6-1 – EDB Elevations

Description	Elevation
Top of Basin	6657.50'
500-YR STORM	6656.99'
100-YR STORM	6656.72'
WQCV	6654.91'
6" VERT ORIFICE INVERT	6654.75'
ORIFICE PLATE INVERTS	VARIES 6653.21' TO 6654.05'
TOP OF MICROPOOL	6653.25'
BOTTOM OF MICROPOOL	6650.75

See Table 6-2 for the freeboards between the maximum water surface elevation of various storm events and the top of the EDB, and the freeboards between the maximum water surface elevation of various storm events, and the first floor elevation of the pump station.

Table 6-2 – Freeboards of Various Design Storms to the Proposed Top of Basin and First Floor of Pump Station

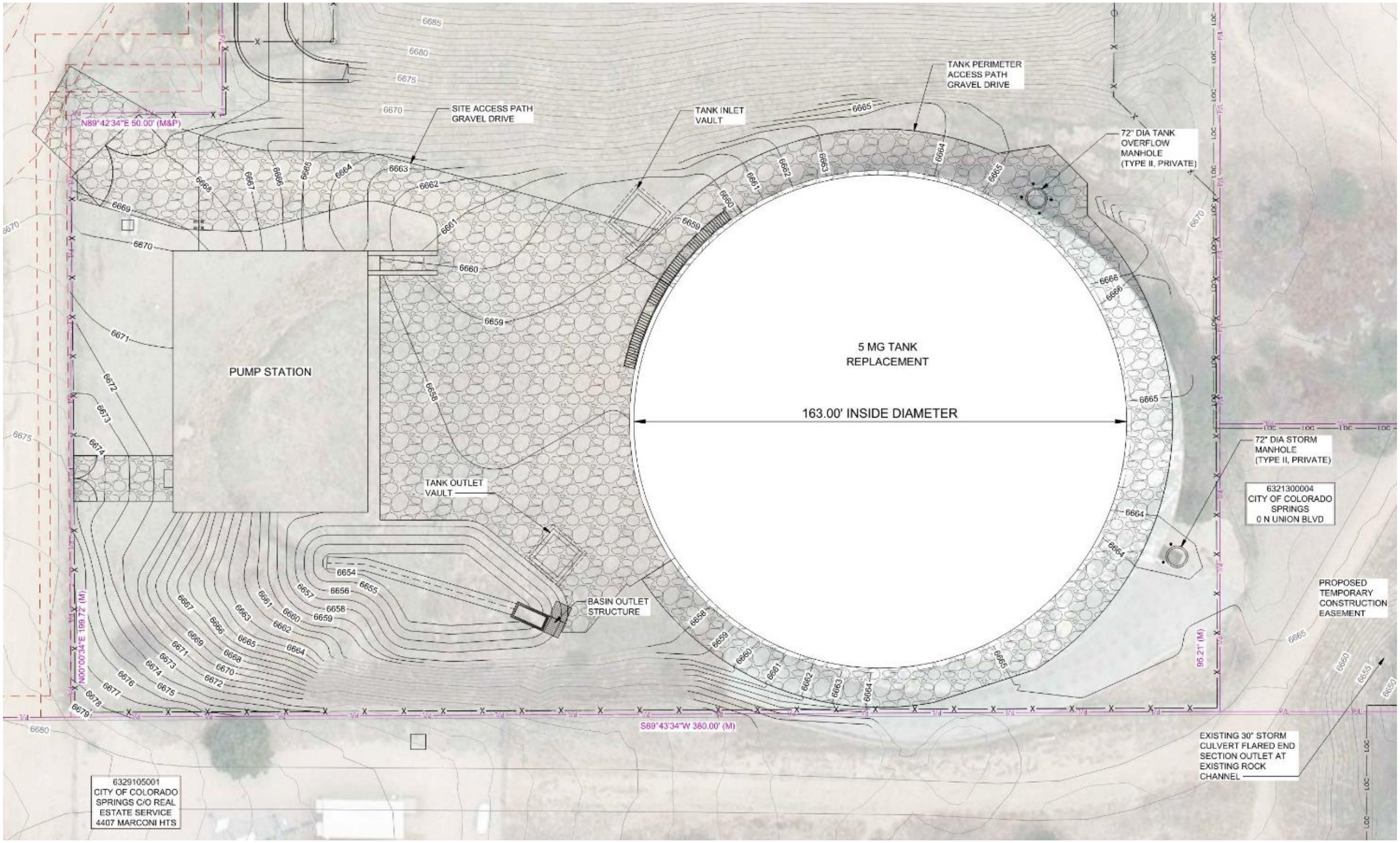
Design Storm	Freeboard Available Between the WSE of a given storm event to the Top of the Basin (6657.50').
5-yr	1.85'
50-yr	0.92'
100-yr	0.78'
500-yr	0.51'

The 100-year event is proposed with a freeboard of 0.78' to the top of the EDB. The freeboard provides the EDB with an adequate margin of safety to prevent flooding and would not cause any negative downstream impacts because overflow would be entirely contained onsite.

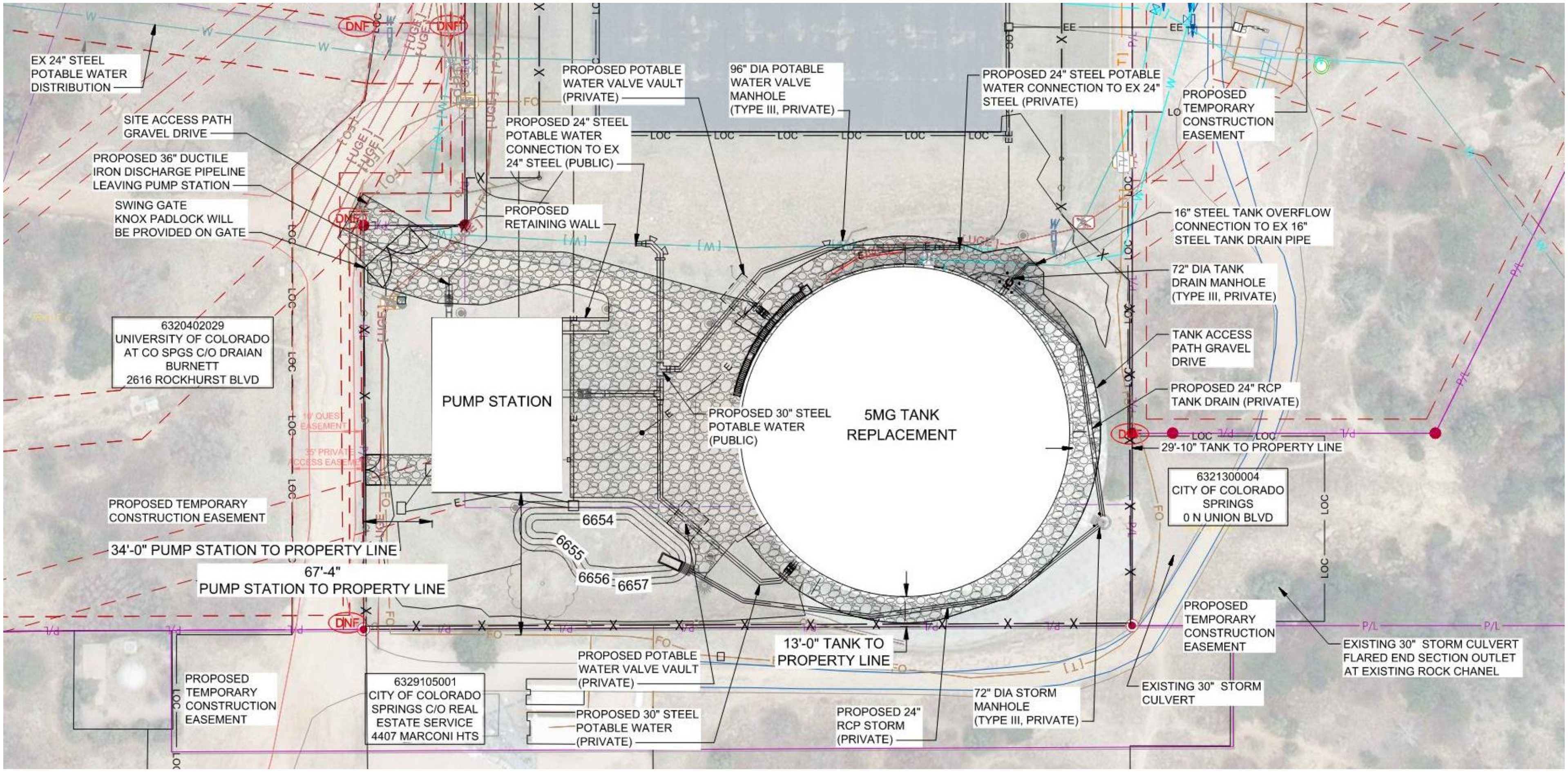
7.0 References

1. City of Colorado Springs. (2021, January). *Drainage Criteria Manual: Volume I (Rev. January 2021)*. https://coloradosprings.gov/document/current-dcmvolume1.pdf
2. City of Colorado Springs. (2020, December). *Drainage Criteria Manual: Volume II* (*Rev. December 2020*). https://coloradosprings.gov/document/current-dcmvolume2.pdf
3. City of Colorado Springs. (2025, July). *DCM Chapter 8: 4-Step Process and Detention* [PDF]. https://coloradosprings.gov/document/dcm-chapter-8-4-step-process-and-detentionfinal.pdf
4. City of Colorado Springs. (2001–2025). City Code of Colorado Springs, Colorado (Code current through Ord. 25 94, passed Nov. 25, 2025). American Legal Publishing. https://codelibrary.amlegal.com/codes/coloradospringsco/latest/coloradosprings_co/0-0-0-1Appendix

8.0 Appendix A, Enlarged Site Map, Grading



Appendix B, Site Map, Utility Plan



Appendix C, Rational Calculation for the Pre-Construction Condition

Standard Form SF-1. Time of Concentration

PROJECT Austin Bluffs Pump Station and Tank Reconstruction
PREPARED BY Ashley Peters

Sub-Basin Data			Initial/Overland Time (t_i)			Travel Time (t_t)				t_c Check (Urbanized Basins)		Final t_c $t_c=t_i+t_t$ minutes	Remarks
DESIGN	C_5	AREA (A) acres	LENGTH (L) feet	SLOPE (S) %	t_i minutes	LENGTH (L) feet	SLOPE (S) %	Velocity (V) ft/sec	t_t minutes	TOTAL LENGTH (L) feet	$t_c=(L/180)+10$ minutes		
Pre-Development	0.34	2.45	72.15	7.08	1.34	106.41	9.74	3.1	0.57	340.69	11.89	4.23	Use 10 minute minimum
						29.02	44.80	4.8	0.10				
						133.11	2.13	1.0	2.22				
Post-Development	0.44	2.45	72.15	7.08	1.17	63.20	8.48	2.8	0.38	204.19	11.13	1.85	Use 10 minute minimum
						68.84	26.87	3.7	0.31				

Runoff Coefficient Table

Land Use	Pre-Development Area (SF)	2-yr Runoff Coefficient	5-yr Runoff Coefficient	10-yr Runoff Coefficient	25-yr Runoff Coefficient	50-yr Runoff Coefficient	100-yr Runoff Coefficient
Roofs	21,635	0.73	0.75	0.77	0.80	0.82	0.83
Drive and Walks	4,289	0.89	0.90	0.92	0.94	0.95	0.96
Gravel	7,897	0.60	0.63	0.66	0.70	0.72	0.74
Meadow	72,818	0.04	0.15	0.25	0.37	0.44	0.50
Total:	106,639	0.26	0.34	0.41	0.50	0.56	0.60

Standard Form SF-2. Storm Drainage System Design (Rational Method Procedure)

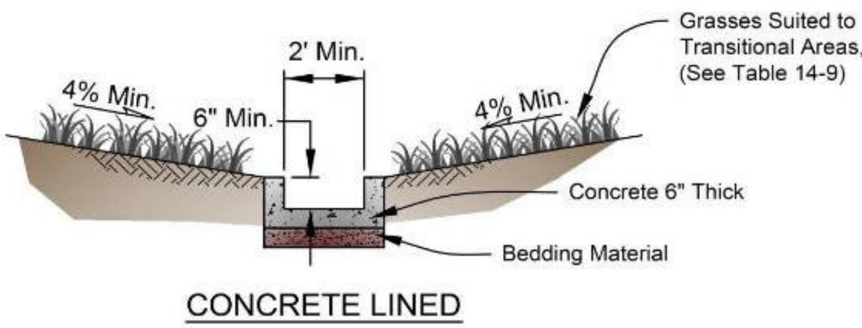
CALCULATED BY: Ashley Peters
DATE: 2/5/2026
CHECKED BY:

STREET	DESIGN POINT	DIRECT RUNOFF						
		DESIGN AREA	AREA (A)	RUNOFF COEFF. (C)	t_c	CA	I	Q
			ac		min		in/hr	cfs
N/A	Pre-Development 2-yr	DA-1	2.45	0.26	10	0.63	3.2	2.00
N/A	Pre-Development 5-yr	DA-1	2.45	0.34	10	0.83	4.0	3.30
N/A	Pre-Development 10-yr	DA-1	2.45	0.41	10	1.01	4.7	4.75
N/A	Pre-Development 25-yr	DA-1	2.45	0.50	10	1.24	5.4	6.67
N/A	Pre-Development 50-yr	DA-1	2.45	0.56	10	1.37	6.1	8.34
N/A	Pre-Development 100-yr	DA-1	2.45	0.60	10	1.48	6.8	10.04

Appendix D, Manning’s Calculations for Low Flow Channels

Manning's Calculation for the Typical Concrete-Lined Low Flow Channel Detail
Expressed in Figure 13-6 of The City of Colorado Springs Drainage Criteria Manual

Figure 13-6. Typical Low-flow Channel Details



Manning's Calculation for the Typical Concrete-Lined Low Flow Channel Detail
Expressed in Figure 13-6 of The City of Colorado Springs Drainage Criteria Manual

Manning's Equation for Flow:

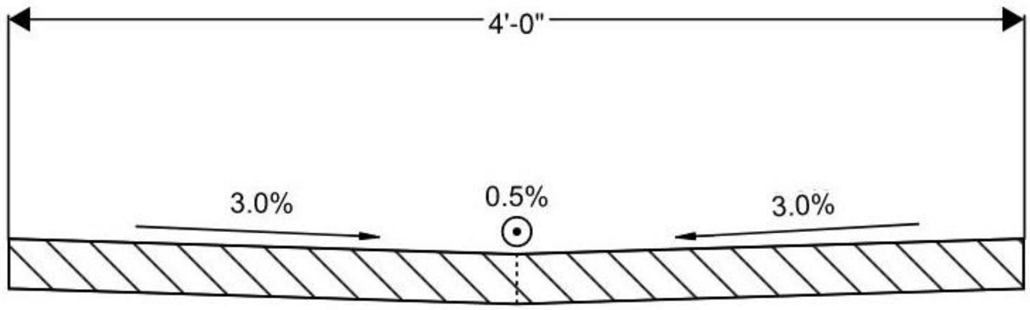
$$Q = A\,(k_n / n)\,R_h^{2/3} * S^{1/2}$$

Where
Q = Volumetric Flowrate (cfs)
A = Cross-Sectional Area of Flow (2 ft. wide by 0.5 ft. tall)
k_n = English Units Coefficient (1.4859 $ft.^{1/3}$/s)
n = Manning's Coefficient (0.012 for concrete)
R_h = Hydraulic Radius (Cross-Sectional Area of Flow / Wetted Perimeter
(2 ft. wide by 0.5 ft. tall divided by 2 feet + 2*0.5 feet for a full channel)
S = Slope/ gradient of the structure (0.5% or 0.005 in Chapter 13, section 5.6)

$$Q = (2\ ft_n * 0.5\ ft.)(1.4859\ ft.^{1/3}/s)/(0.012) * (2\ ft. * 0.5\ ft. / [2*0.5\ ft. + 2\ ft.])^{2/3} * 0.005^{1/2}$$

Q = 4.21 cfs

Manning's Calculation for the Proposed Concrete-Lined Low Flow Channel Detail



Manning's Calculation for the proposed Concrete-Lined Low Flow Channel in lieu of using the channel in the detail Expressed in Figure 13-6 of The City of Colorado Springs Drainage Criteria Manual

Manning's Equation for Flow:

$$Q = A\,(k_n / n)\,R_h^{2/3} * S^{1/2}$$

Where
Q = Volumetric Flowrate (cfs)
A = Cross-Sectional Area of Flow (1/2)*(4 ft. wide by (2 ft*0.03ft))
k_n = English Units Coefficient (1.4859 $ft.^{1/3}$/s)
n = Manning's Coefficient (0.012 for concrete)
R_h = Hydraulic Radius (Cross-Sectional Area of Flow / Wetted Perimeter
(1/2)*(4 ft. wide by (2 ft*0.03ft tall)) divided by 4 feet
S = Slope/ gradient of the structure (0.5% or 0.005 in Chapter 13, section 5.6)

$$Q = (1/2)*(4\text{ ft.}*(2\text{ ft}*0.03\text{ft}))(1.4859\text{ ft.}^{1/3}/\text{s})/(0.012) * [(1/2)*(4\text{ ft.}*(2\text{ ft}*0.03\text{ft})) / 4\text{ ft.}]^{2/3} * 0.005^{1/2}$$

Q = 0.101 cfs

The proposed design meets the low-flow channel minimum flowrate expressed in Chapter 13, section 5.6 of the DCM

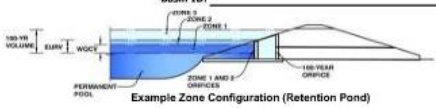
Appendix E, MHFD Calculations

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

MHFD-Detention, Version 4.07 (June 2025)

Project:

Basin ID:



Example Zone Configuration (Retention Pond)

Watershed Information

Selected SCM Type =	**EDB**		**Note: L / W Ratio < 1**
Watershed Area =	2.45	acres	**L / W Ratio = 0.82**
Watershed Length =	295	ft	
Watershed Length to Centroid =	100	ft	
Watershed Slope =	0.060	ft/ft	
Watershed Imperviousness =	45.13%	percent	
Percentage Hydrologic Soil Group A =	0.0%	percent	
Percentage Hydrologic Soil Group B =	0.0%	percent	
Percentage Hydrologic Soil Groups C/D =	100.0%	percent	
Target WQCV Drain Time =	40.0	hours	
Location for 1-hr Rainfall Depths =	Denver - Capitol Building		

After providing required inputs above including 1-hour rainfall depths, click 'Run CUHP' to generate runoff hydrographs using the embedded Colorado Urban Hydrograph Procedure.

			Optional User Overrides	
Water Quality Capture Volume (WQCV) =	0.039	acre-feet		acre-feet
Excess Urban Runoff Volume (EURV) =	0.104	acre-feet		acre-feet
2-yr Runoff Volume (P1 = 0.92 in.) =	0.069	acre-feet	0.92	inches
5-yr Runoff Volume (P1 = 1.19 in.) =	0.106	acre-feet	1.19	inches
10-yr Runoff Volume (P1 = 1.45 in.) =	0.149	acre-feet	1.45	inches
25-yr Runoff Volume (P1 = 1.84 in.) =	0.226	acre-feet	1.84	inches
50-yr Runoff Volume (P1 = 2.17 in.) =	0.286	acre-feet	2.17	inches
100-yr Runoff Volume (P1 = 2.53 in.) =	0.359	acre-feet	2.53	inches
500-yr Runoff Volume (P1 = 3.48 in.) =	0.536	acre-feet	3.48	inches
Approximate 2-yr Detention Volume =	0.070	acre-feet		
Approximate 5-yr Detention Volume =	0.110	acre-feet		
Approximate 10-yr Detention Volume =	0.131	acre-feet		
Approximate 25-yr Detention Volume =	0.159	acre-feet		
Approximate 50-yr Detention Volume =	0.173	acre-feet		
Approximate 100-yr Detention Volume =	0.208	acre-feet		

Define Zones and Basin Geometry

Zone 1 Volume (WQCV) =	0.039	acre-feet	
Zone 2 Volume (2-year - Zone 1) =	0.031	acre-feet	**Total detention volume is less than 100-year volume.**
Select Zone 3 Storage Volume (Optional) =		acre-feet	
Total Detention Basin Volume =	0.070	acre-feet	
Initial Surcharge Volume (ISV) =	user	ft^3	
Initial Surcharge Depth (ISD) =	user	ft	
Total Available Detention Depth (H_{total}) =	user	ft	
Depth of Trickle Channel (H_{TC}) =	user	ft	
Slope of Trickle Channel (S_{TC}) =	user	ft/ft	
Slopes of Main Basin Sides (S_{main}) =	user	H:V	
Basin Length-to-Width Ratio ($R_{L/W}$) =	user		

Initial Surcharge Area (A_{ISV}) =	user	ft^2
Surcharge Volume Length (L_{ISV}) =	user	ft
Surcharge Volume Width (W_{ISV}) =	user	ft
Depth of Basin Floor (H_{FLOOR}) =	user	ft
Length of Basin Floor (L_{FLOOR}) =	user	ft
Width of Basin Floor (W_{FLOOR}) =	user	ft
Area of Basin Floor (A_{FLOOR}) =	user	ft^2
Volume of Basin Floor (V_{FLOOR}) =	user	ft^3
Depth of Main Basin (H_{MAIN}) =	user	ft
Length of Main Basin (L_{MAIN}) =	user	ft
Width of Main Basin (W_{MAIN}) =	user	ft
Area of Main Basin (A_{MAIN}) =	user	ft^2
Volume of Main Basin (V_{MAIN}) =	user	ft^3
Calculated Total Basin Volume (V_{total}) =	**user**	acre-feet

Depth Increment = ft

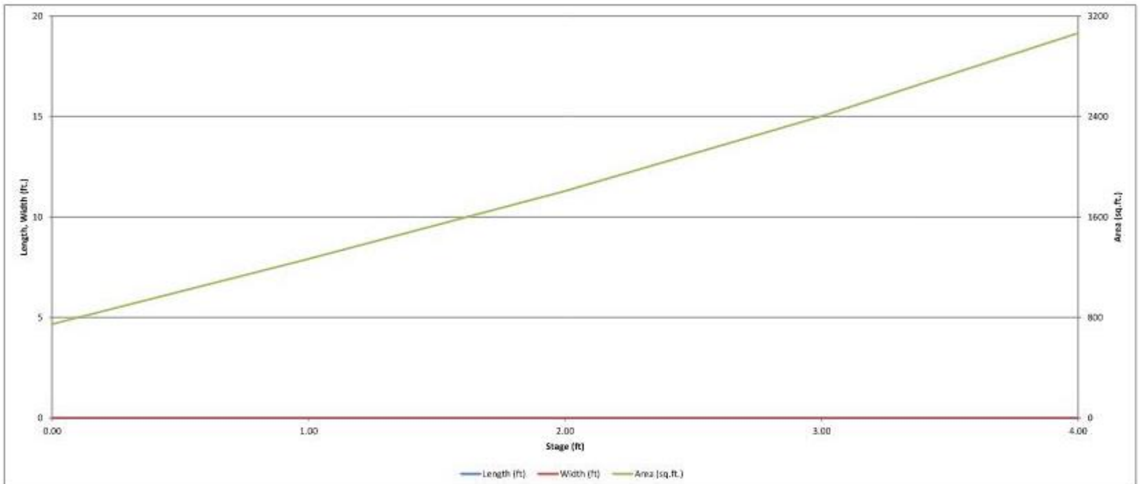
Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	748	0.017		
	--	1.00	--	--	--	1,267	0.029	1,007	0.023
	--	2.00	--	--	--	1,807	0.041	2,544	0.058
	--	3.00	--	--	--	2,404	0.055	4,650	0.107
	--	4.00	--	--	--	3,065	0.070	7,384	0.170

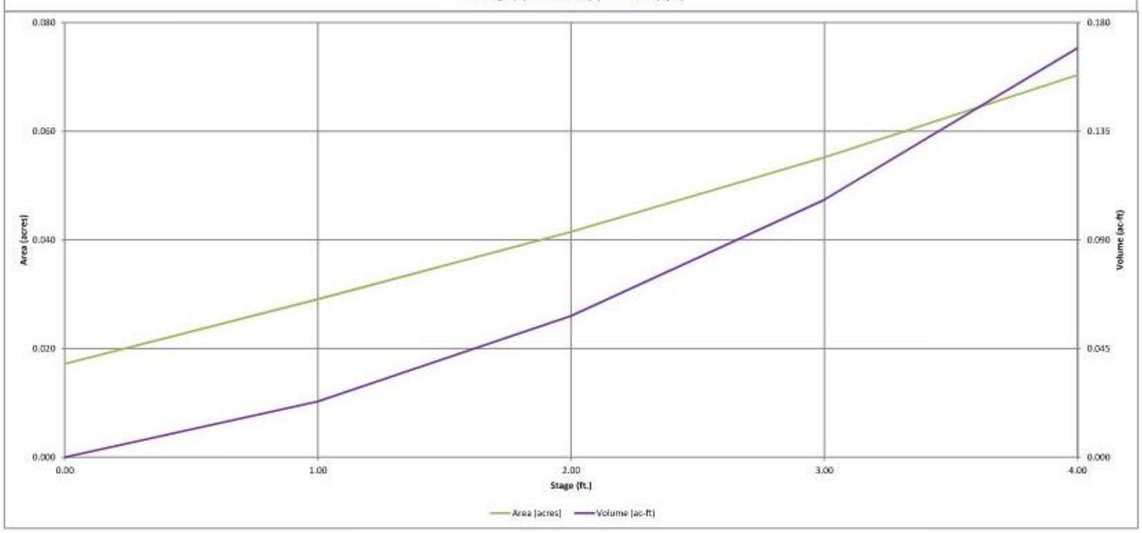
HFD-Detention_v4.07_2026-02-04, Basin

2/5/2026, 1:54 PM

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

MHFD-Detention, Version 4.07 (June 2025)





MHFD-Detention_v4.07_2026-02-04, Basin

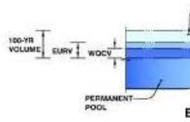
2/5/2026, 1:54 PM

DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.07 (June 2025)

Project:

Basin ID:



Example Zone Configuration (Retention Pond)

	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	1.51	0.039	Orifice Plate
Zone 2 (2-year)	2.28	0.031	Circular Orifice
Zone 3			
Total (all zones)		0.070	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration SCM)

Underdrain Orifice Invert Depth =	N/A	ft (distance below the filtration media surface)
Underdrain Orifice Diameter =	N/A	inches

Calculated Parameters for Underdrain

Underdrain Orifice Area =	N/A	ft^2
Underdrain Orifice Centroid =	N/A	feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation SCM)

Centroid of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	1.66	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	N/A	inches
Orifice Plate: Orifice Area per Row =	0.23	sq. inches (diameter = 1/2 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	1.597E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	0.40	0.80					
Orifice Area (sq. inches)	0.23	0.23	0.23					

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

	Zone 2 Circular	Not Selected	
Invert of Vertical Orifice =	1.50		ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	2.50		ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter =	6.00		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Circular	Not Selected	
Vertical Orifice Area =	0.20		ft^2
Vertical Orifice Centroid =	0.25		feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

	Not Selected	Not Selected	
Overflow Weir Front Edge Height, Ho =	2.50		ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00		feet
Overflow Weir Grate Slope =	3.00		H:V
Horiz. Length of Weir Sides =	4.50		feet
Overflow Grate Type =	Close Mesh Grate		
Debris Clogging % =	0%		%

Calculated Parameters for Overflow Weir

	Not Selected	Not Selected	
Height of Grate Upper Edge, H_t =	4.00		feet
Overflow Weir Slope Length =	4.74		feet
Grate Open Area / 100-yr Orifice Area =			
Overflow Grate Open Area w/o Debris =	15.01		ft^2
Overflow Grate Open Area w/ Debris =	15.01		ft^2

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

	Not Selected	Not Selected	
Depth to Invert of Outlet Pipe =			ft (distance below basin bottom at Stage = 0 ft)
Circular Orifice Diameter =			inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Not Selected	Not Selected	
Outlet Orifice Area =			ft^2
Outlet Orifice Centroid =			feet
Half-Central Angle of Restrictor Plate on Pipe =	N/A	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=		ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =		feet
Spillway End Slopes =		H:V
Freeboard above Max Water Surface =		feet

Calculated Parameters for Spillway

Spillway Design Flow Depth=		feet
Stage at Top of Freeboard =		feet
Basin Area at Top of Freeboard =		acres
Basin Volume at Top of Freeboard =		acre-ft

Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	N/A	N/A	0.92	1.19	1.45	1.84	2.17	2.53	3.48
CUHP Runoff Volume (acre-ft) =	0.039	0.104	0.069	0.106	0.149	0.226	0.286	0.359	0.536
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.069	0.106	0.149	0.226	0.286	0.359	0.536
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A	0.2	1.0	1.8	3.5	4.8	6.4	9.8
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A	2.0	3.3	4.8	6.7	8.3	10.0	
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.82	1.35	1.94	2.72	3.40	4.10	4.02
Peak Inflow Q (cfs) =	N/A	N/A	1.7	2.8	3.8	6.2	7.9	9.5	13.9
Peak Outflow Q (cfs) =	0.02	2.42	0.20	0.63	1.00	3.69	5.66	7.77	13.29
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.2	0.2	0.6	0.7	0.8	1.3
Structure Controlling Flow =	Plate	Overflow Weir 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir
Max Velocity through Grate 1 (fps) =	N/A	-0.07	N/A	N/A	-0.1	-0.1	-0.1	-0.1	-0.1
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	50	45	50	45	41	34	30	27	20
Time to Drain 99% of Inflow Volume (hours) =	**56**	55	58	55	53	49	47	44	39
Maximum Ponding Depth (ft) =	1.50	2.95	1.79	2.15	2.56	2.92	3.08	3.22	3.49
Area at Maximum Ponding Depth (acres) =	0.04	0.05	0.04	0.04	0.05	0.05	0.06	0.06	0.06
Maximum Volume Stored (acre-ft) =	0.039	0.104	0.050	0.064	0.083	0.102	0.111	0.119	0.136

MHFD-Detention_v4.07_2026-02-04, Outlet Structure 2/5/2026, 1:55 PM

DETENTION BASIN OUTLET STRUCTURE DESIGN

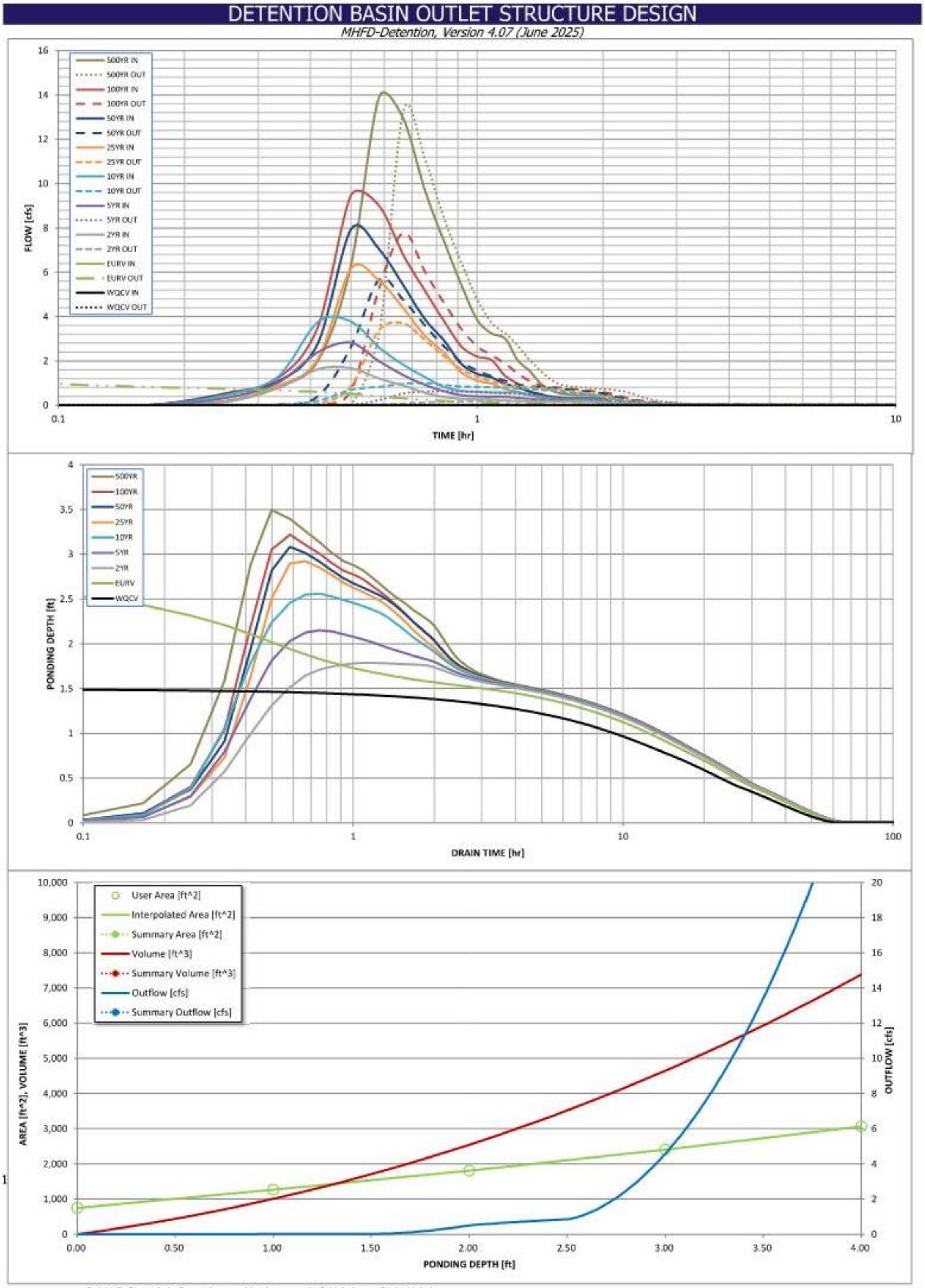
Outflow Hydrograph Workbook Filename:

Inflow Hydrographs

The user can override the calculated inflow hydrographs from this workbook with inflow hydrographs developed in a separate program.

	SOURCE	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP
Time Interval	TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
5.00 min	0:00:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:05:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:10:00	0	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.18
	0:15:00	0	0.00	0.15	0.33	0.46	0.37	0.51	0.54	0.83
	0:20:00	0	0.00	0.74	1.00	1.26	0.86	1.06	1.21	2.00
	0:25:00	0	0.00	1.63	2.40	3.76	2.09	2.80	3.53	6.31
	0:30:00	0	0.00	1.67	2.82	3.77	6.18	7.94	9.46	13.91
	0:35:00	0	0.00	1.21	1.99	2.61	5.64	7.04	9.00	12.94
	0:40:00	0	0.00	0.89	1.37	1.82	4.38	5.41	6.78	9.72
	0:45:00	0	0.00	0.56	0.92	1.28	3.14	3.89	5.19	7.41
	0:50:00	0	0.00	0.37	0.63	0.82	2.37	2.93	3.80	5.43
	0:55:00	0	0.00	0.29	0.46	0.64	1.49	1.86	2.65	3.86
	1:00:00	0	0.00	0.27	0.40	0.60	1.12	1.42	2.20	3.24
	1:05:00	0	0.00	0.26	0.39	0.58	0.97	1.25	2.03	3.01
	1:10:00	0	0.00	0.21	0.38	0.57	0.73	0.96	1.32	2.03
	1:15:00	0	0.00	0.19	0.33	0.58	0.61	0.82	0.97	1.55
	1:20:00	0	0.00	0.18	0.29	0.48	0.47	0.61	0.62	0.98
	1:25:00	0	0.00	0.17	0.26	0.37	0.39	0.50	0.43	0.67
	1:30:00	0	0.00	0.17	0.25	0.31	0.31	0.39	0.34	0.53
	1:35:00	0	0.00	0.17	0.24	0.28	0.26	0.34	0.31	0.48
	1:40:00	0	0.00	0.17	0.20	0.26	0.25	0.31	0.30	0.46
	1:45:00	0	0.00	0.17	0.18	0.26	0.24	0.30	0.30	0.46
	1:50:00	0	0.00	0.17	0.17	0.26	0.24	0.30	0.30	0.46
	1:55:00	0	0.00	0.13	0.17	0.24	0.24	0.30	0.30	0.46
	2:00:00	0	0.00	0.10	0.15	0.21	0.24	0.30	0.30	0.46
	2:05:00	0	0.00	0.05	0.08	0.10	0.12	0.15	0.15	0.23
	2:10:00	0	0.00	0.02	0.04	0.05	0.06	0.07	0.07	0.11
	2:15:00	0	0.00	0.01	0.02	0.02	0.03	0.03	0.03	0.05
	2:20:00	0	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01
	2:25:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:30:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:35:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:40:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:45:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:50:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2:55:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:00:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:05:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:10:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:15:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:20:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:25:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:30:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:35:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:40:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:45:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:50:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:55:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:00:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:05:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:10:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:15:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:20:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:25:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:30:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:35:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:40:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:45:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:50:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:55:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:00:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:05:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:10:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:15:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:20:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:25:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:30:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:35:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:40:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:45:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:50:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:55:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:00:00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

MHFD-Detention_v4.07_2026-02-04, Outlet Structure 2/5/2026, 1:55 PM



S-A-V-D Chart Axis Override	X-axis	Left Y-Axis	Right Y-Axis
minimum bound			
maximum bound			

MHFD-Detention_v4.07_2026-02-04, Outlet Structure

Appendix E, UD-Sewer Calculation, 0.08 cfs

Program: UDSEWER Math Model Interface 2.1.1.4 **Run Date:** 1/29/2026 4:08:08 PM	**UDSewer Results Summary** **Project Title:** New UDSEWER System Module **Project Description:** Default system

System Input Summary

Rainfall Parameters

Rainfall Return Period: 100
Rainfall Calculation Method: Formula

One Hour Depth (in): 2.53
Rainfall Constant "A": 28.5
Rainfall Constant "B": 10
Rainfall Constant "C": 0.786

Rational Method Constraints

Minimum Urban Runoff Coeff.: 0.20
Maximum Rural Overland Len. (ft): 500
Maximum Urban Overland Len. (ft): 300
Used UDFCD Tc. Maximum: Yes

Sizer Constraints

Minimum Sewer Size (in): 12.00
Maximum Depth to Rise Ratio: 0.90
Maximum Flow Velocity (fps): 18.0
Minimum Flow Velocity (fps): 2.0

Backwater Calculations:

Tailwater Elevation (ft): 0.00

Manhole Input Summary:

		Given Flow		Sub Basin Information						
Element Name	**Ground Elevation (ft)**	**Total Known Flow (cfs)**	**Local Contribution (cfs)**	**Drainage Area (Ac.)**	**Runoff Coefficient**	**5yr Coefficient**	**Overland Length (ft)**	**Overland Slope (%)**	**Gutter Length (ft)**	**Gutter Velocity (fps)**
Outfall 1	6650.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

MH 1 SWR 1 - 1	6663.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 2 SWR 2 - 1	6656.00	0.08	0.08	2.45	0.75	0.70	232.00	11.00	0.00	1.00

Manhole Output Summary:

	Local Contribution					Total Design Flow				
Element Name	**Overland Time (min)**	**Gutter Time (min)**	**Basin Tc (min)**	**Intensity (in/hr)**	**Local Contrib (cfs)**	**Coeff. Area**	**Intensity (in/hr)**	**Manhole Tc (min)**	**Peak Flow (cfs)**	**Comment**
Outfall 1	0.00	0.00	0.00	0.00	0.00	1.84	0.04	12450.93	0.08	
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	1.84	0.04	12450.93	0.08	Surface Water Present (Downstream)
MH 2 SWR 2 - 1	0.00	0.00	12450.93	0.04	0.08	1.84	0.04	12450.93	0.08	Used Minimum Tc

Sewer Input Summary:

		Elevation			Loss Coefficients			Given Dimensions		
Element Name	**Sewer Length (ft)**	**Downstream Invert (ft)**	**Slope (%)**	**Upstream Invert (ft)**	**Mannings n**	**Bend Loss**	**Lateral Loss**	**Cross Section**	**Rise (ft or in)**	**Span (ft or in)**
MH 1 SWR 1 - 1	75.40	6650.77	1.4	6651.83	0.024	0.00	0.00	CIRCULAR	30.00 in	30.00 in
MH 2 SWR 2 - 1	220.00	6651.97	0.5	6653.07	0.013	0.71	0.30	CIRCULAR	24.00 in	24.00 in

Sewer Flow Summary:

	Full Flow Capacity		Critical Flow		Normal Flow						
Element Name	**Flow (cfs)**	**Velocity (fps)**	**Depth (in)**	**Velocity (fps)**	**Depth (in)**	**Velocity (fps)**	**Froude Number**	**Flow Condition**	**Flow (cfs)**	**Surcharged Length (ft)**	**Comment**
MH 1 SWR 1 - 1	26.36	5.37	1.09	1.40	1.21	1.20	0.81	Subcritical	0.08	0.00	Velocity is Too Low
MH 2 SWR 2 - 1	16.04	5.11	1.15	1.44	1.22	1.33	0.89	Subcritical	0.08	0.00	Velocity is Too Low

- A Froude number of 0 indicates that pressured flow occurs (adverse slope or undersized pipe).
- If the sewer is not pressurized, full flow represents the maximum gravity flow in the sewer.
- If the sewer is pressurized, full flow represents the pressurized flow conditions.

Sewer Sizing Summary:

			Existing		Calculated		Used			
Element Name	Peak Flow (cfs)	Cross Section	Rise	Span	Rise	Span	Rise	Span	Area (ft^2)	Comment
MH 1 SWR 1 - 1	0.08	CIRCULAR	30.00 in	30.00 in	12.00 in	12.00 in	30.00 in	30.00 in	4.91	
MH 2 SWR 2 - 1	0.08	CIRCULAR	24.00 in	24.00 in	12.00 in	12.00 in	24.00 in	24.00 in	3.14	

- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

Grade Line Summary:

Tailwater Elevation (ft): 0.00

	Invert Elev.		Downstream Manhole Losses		HGL		EGL		
Element Name	Downstream (ft)	Upstream (ft)	Bend Loss (ft)	Lateral Loss (ft)	Downstream (ft)	Upstream (ft)	Downstream (ft)	Friction Loss (ft)	Upstream (ft)
MH 1 SWR 1 - 1	6650.77	6651.83	0.00	0.00	6650.87	6651.95	6650.90	1.07	6651.96
MH 2 SWR 2 - 1	6651.97	6653.07	0.00	0.00	6652.07	6653.18	6652.10	1.10	6653.20

- Bend and Lateral losses only apply when there is an outgoing sewer. The system outfall, sewer #0, is not considered a sewer.
- Bend loss = Bend K * V_fi ^ 2/(2*g)
- Lateral loss = V_fo ^ 2/(2*g)- Junction Loss K * V_fi ^ 2/(2*g).
- Friction loss is always Upstream EGL - Downstream EGL.

Excavation Estimate:

The trench side slope is 1.0 ft/ft
The minimum trench width is 2.00 ft

					Downstream			Upstream				
Element Name	Length (ft)	Wall (in)	Bedding (in)	Bottom Width (ft)	Top Width (ft)	Trench Depth (ft)	Cover (ft)	Top Width (ft)	Trench Depth (ft)	Cover (ft)	Volume (cu. yd)	Comment
MH 1 SWR 1 - 1	75.40	3.50	6.00	6.08	0.00	0.80	0.00	21.84	12.46	8.88	199.29	Sewer Too Shallow
MH 2 SWR 2 - 1	220.00	3.00	4.00	5.50	22.06	12.11	9.28	5.50	3.51	0.68	614.23	Sewer Too Shallow

Total earth volume for sewer trenches = 814 cubic yards.

- The trench was estimated to have a bottom width equal to the outer pipe diameter plus 36 inches.
- If the calculated width of the trench bottom is less than the minimum acceptable width, the minimum acceptable width was used.
- The sewer wall thickness is equal to: (equivalent diameter in inches/12)+1 inches
- The sewer bedding thickness is equal to:
  - Four inches for pipes less than 33 inches.
  - Six inches for pipes less than 60 inches.
  - Eight inches for all larger sizes.

Appendix F, UD-Sewer Calculation, 22 cfs

Program: UDSEWER Math Model Interface 2.1.1.4 **Run Date:** 1/29/2026 11:04:36 AM	**UDSewer Results Summary** **Project Title:** New UDSEWER System Module **Project Description:** Default system

System Input Summary

Rainfall Parameters

Rainfall Return Period: 100
Rainfall Calculation Method: Formula

One Hour Depth (in): 2.53
Rainfall Constant "A": 28.5
Rainfall Constant "B": 10
Rainfall Constant "C": 0.786

Rational Method Constraints

Minimum Urban Runoff Coeff.: 0.20
Maximum Rural Overland Len. (ft): 500
Maximum Urban Overland Len. (ft): 300
Used UDFCD Tc. Maximum: Yes

Sizer Constraints

Minimum Sewer Size (in): 12.00
Maximum Depth to Rise Ratio: 0.90
Maximum Flow Velocity (fps): 18.0
Minimum Flow Velocity (fps): 2.0

Backwater Calculations:

Tailwater Elevation (ft): 0.00

Manhole Input Summary:

		Given Flow		Sub Basin Information						
Element Name	**Ground Elevation (ft)**	**Total Known Flow (cfs)**	**Local Contribution (cfs)**	**Drainage Area (Ac.)**	**Runoff Coefficient**	**5yr Coefficient**	**Overland Length (ft)**	**Overland Slope (%)**	**Gutter Length (ft)**	**Gutter Velocity (fps)**
Outfall 1	6650.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

MH 1 SWR 1 - 1	6663.50	22.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 2 SWR 2 - 1	6656.00	22.00	22.00	2.45	0.75	0.70	232.00	11.00	0.00	1.00

Manhole Output Summary:

	Local Contribution					Total Design Flow				
Element Name	**Overland Time (min)**	**Gutter Time (min)**	**Basin Tc (min)**	**Intensity (in/hr)**	**Local Contrib (cfs)**	**Coeff. Area**	**Intensity (in/hr)**	**Manhole Tc (min)**	**Peak Flow (cfs)**	**Comment**
Outfall 1	0.00	0.00	0.00	0.00	0.00	1.84	11.97	-0.18	22.00	
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	1.84	11.97	-0.18	22.00	Surface Water Present (Downstream)
MH 2 SWR 2 - 1	0.00	0.00	5.00	11.97	22.00	1.84	11.97	-0.18	22.00	Surface Water Present (Upstream) Used Minimum Tc

Sewer Input Summary:

		Elevation			Loss Coefficients			Given Dimensions		
Element Name	**Sewer Length (ft)**	**Downstream Invert (ft)**	**Slope (%)**	**Upstream Invert (ft)**	**Mannings n**	**Bend Loss**	**Lateral Loss**	**Cross Section**	**Rise (ft or in)**	**Span (ft or in)**
MH 1 SWR 1 - 1	75.40	6650.77	1.4	6651.83	0.024	0.00	0.00	CIRCULAR	30.00 in	30.00 in
MH 2 SWR 2 - 1	220.00	6651.97	0.5	6653.07	0.013	0.71	0.30	CIRCULAR	24.00 in	24.00 in

Sewer Flow Summary:

	Full Flow Capacity		Critical Flow		Normal Flow						
Element Name	**Flow (cfs)**	**Velocity (fps)**	**Depth (in)**	**Velocity (fps)**	**Depth (in)**	**Velocity (fps)**	**Froude Number**	**Flow Condition**	**Flow (cfs)**	**Surcharged Length (ft)**	**Comment**
MH 1 SWR 1 - 1	26.36	5.37	19.14	6.66	20.95	6.01	0.84	Subcritical	22.00	0.00	
MH 2 SWR 2 - 1	16.04	5.11	24.00	7.00	24.00	7.00	0.00	Pressurized	22.00	220.00	

- A Froude number of 0 indicates that pressured flow occurs (adverse slope or undersized pipe).
- If the sewer is not pressurized, full flow represents the maximum gravity flow in the sewer.
- If the sewer is pressurized, full flow represents the pressurized flow conditions.

Sewer Sizing Summary:

			Existing		Calculated		Used			
Element Name	**Peak Flow (cfs)**	**Cross Section**	**Rise**	**Span**	**Rise**	**Span**	**Rise**	**Span**	**Area (ft^2)**	**Comment**
MH 1 SWR 1 - 1	22.00	CIRCULAR	30.00 in	30.00 in	30.00 in	30.00 in	30.00 in	30.00 in	4.91	
MH 2 SWR 2 - 1	22.00	CIRCULAR	24.00 in	24.00 in	30.00 in	30.00 in	24.00 in	24.00 in	3.14	Existing height is smaller than the suggested height. Existing width is smaller than the suggested width. Exceeds max. Depth/Rise

- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

Grade Line Summary:

Tailwater Elevation (ft): 0.00

	Invert Elev.		Downstream Manhole Losses		HGL		EGL		
Element Name	**Downstream (ft)**	**Upstream (ft)**	**Bend Loss (ft)**	**Lateral Loss (ft)**	**Downstream (ft)**	**Upstream (ft)**	**Downstream (ft)**	**Friction Loss (ft)**	**Upstream (ft)**
MH 1 SWR 1 - 1	6650.77	6651.83	0.00	0.00	6652.37	6653.70	6653.06	1.13	6654.18
MH 2 SWR 2 - 1	6651.97	6653.07	0.54	0.08	6654.32	6656.39	6655.08	2.07	6657.15

- Bend and Lateral losses only apply when there is an outgoing sewer. The system outfall, sewer #0, is not considered a sewer.
- Bend loss = Bend K * V_fi ^ 2/(2*g)
- Lateral loss = V_fo ^ 2/(2*g)- Junction Loss K * V_fi ^ 2/(2*g).
- Friction loss is always Upstream EGL - Downstream EGL.

Excavation Estimate:

The trench side slope is 1.0 ft/ft
The minimum trench width is 2.00 ft

	Downstream	Upstream	

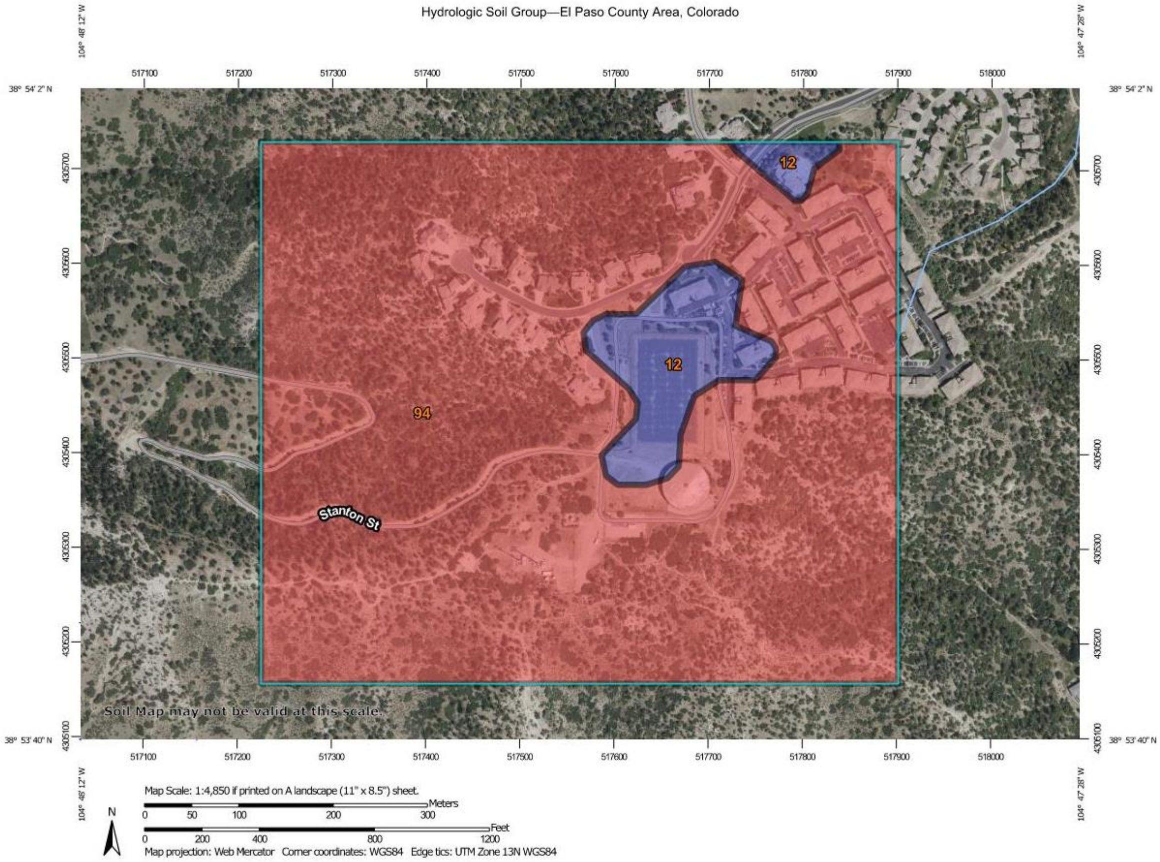
Element Name	Length (ft)	Wall (in)	Bedding (in)	Bottom Width (ft)	Top Width (ft)	Trench Depth (ft)	Cover (ft)	Top Width (ft)	Trench Depth (ft)	Cover (ft)	Volume (cu. yd)	Comment
MH 1 SWR 1 - 1	75.40	3.50	6.00	6.08	0.00	0.80	0.00	21.84	12.46	8.88	199.29	Sewer Too Shallow
MH 2 SWR 2 - 1	220.00	3.00	4.00	5.50	22.06	12.11	9.28	5.50	3.51	0.68	614.23	Sewer Too Shallow

Total earth volume for sewer trenches = 814 cubic yards.

- The trench was estimated to have a bottom width equal to the outer pipe diameter plus 36 inches.
- If the calculated width of the trench bottom is less than the minimum acceptable width, the minimum acceptable width was used.
- The sewer wall thickness is equal to: (equivalent diameter in inches/12)+1 inches
- The sewer bedding thickness is equal to:
  - Four inches for pipes less than 33 inches.
  - Six inches for pipes less than 60 inches.
  - Eight inches for all larger sizes.

Appendix G, NRCS Soils Data

Hydrologic Soil Group—El Paso County Area, Colorado



USDA Natural Resources Conservation Service

Web Soil Survey
National Cooperative Soil Survey

10/1/2025
Page 1 of 4

Hydrologic Soil Group—El Paso County Area, Colorado

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 23, Aug 29, 2025

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jul 23, 2024—Aug 4, 2024

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Appendix H, Appendix I Sheet

APPENDIX I

PERMANENT CONTROL MEASURE IM PLAN ADDENDUM

PROJECT NO./NAME: Austin Bluffs Tank and Pump Station

SUBDIVISION: Austin Bluffs Pump Station and Tank Site

THE FOLLOWING ITEMS DEVIATE FROM CITY PCM STANDARDS BUT HAVE BEEN APPROVED THROUGH THE VARIANCE PROCESS:

1) PCM Basin Side and Bottom Slopes

2) PCM Basin Outlet Structure and Emergency Overflow

3) PCM Freeboard

INSPECTION AND MAINTENANCE OF THE DEVIATIONS ARE DETAILED BELOW:

1) Inspect the outlet structure micropool, orifice plate, inlet grating, and site outfall after significant rain events.

2) Clean the outlet structure micropool, orifice plate, inlet grating, and outfall of trash and other detritus when observed that may inhibit the flow of water out of the PCM.

3) Ensure that water does not pond within the bottom of the basin for extended periods of time and perform re-grading activities if water does not flow out of the basin and into the outlet structure.

4) Inspect the 24” RCP inlet pipe, site storm manhole, and 30” CMP culvert discharge and confirm that it does not have any debris that could inhibit flow.

DESIGN ENGINEER SIGNATURE (Affix Seal):

This sheet is signed and sealed by the person(s) seal and signature indicated on the signature page of this document