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# FINAL DRAINAGE REPORT FOR RETREAT AT TIMBERRIDGE FILING NO. 3

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Job No. 1185.30

PCD Project No. SF-22-041



# FINAL DRAINAGE REPORT FOR RETREAT AT TIMBERRIDGE FILING NO. 3

#### **ENGINEER'S STATEMENT:**

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the County for drainage reports and said report is in conformity with the applicable master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors, or omissions on my part in preparing this report.

Marc A. Whorton (	Colorado P.E. #37155	Date
	•	aply with all of the requirements specified in this
Business Name:	TIMBERRIDGE DEVELOPM	IENT GROUP, LLC
Ву:		
Title:		
Address:	2138 Flying Horse Club D	<u>rive</u>
	Colorado Springs, CO 809	921
	•	e Drainage Criteria Manual, Volumes 1 and 2, El Paso evelopment Code as amended.
Joshua Palmer, P.E County Engineer, /	ECM Administrator	Date
Conditions:		



# FINAL DRAINAGE REPORT FOR RETREAT AT TIMBERRIDGE FILING NO. 3

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### FINAL DRAINAGE REPORT FOR RETREAT AT TIMBERRIDGE FILING NO. 3

#### **PURPOSE**

The purpose of this Final Drainage Report is to address on-site and off-site drainage patterns and identify specific drainage improvements and facilities required to minimize impacts to the adjacent properties.

#### **GENERAL DESCRIPTION**

The Retreat at TimberRidge Filing No. 3 is 44.578-acre site located in portions sections 21, 22, 27 and 28, township 12 south, range 65 west of the sixth principal meridian. The site is bounded on the north by Arroya Lane, to the south by existing Retreat at TimberRidge development, to the east by Sterling Ranch property (zoned for future urban development), and to the west by existing Retreat at TimberRidge development and Vollmer Road. The site is in the upper portion of the Sand Creek Drainage Basin. Both large lot rural single family residential and urban single family residential are proposed in this Filing.

The average soil condition reflects Hydrologic Group "B" (Pring coarse sandy loam) as determined by the "Web Soil Survey of El Paso County Area," prepared by the Natural Resources Conservation Service (see map in Appendix).

#### **EXISTING DRAINAGE CONDITIONS**

The Retreat at TimberRidge Filing No. 3 property is located in the upper portion of the Sand Creek drainage basin on the south edge of Black Forest. Nearly the entire site, other than the Sand Creek corridor, is mainly covered with native grasses with few or no pine trees. The Sand Creek channel bisects the site in a north-south direction. A wetlands delineation was prepared by CORE Consultants, Inc., which included the entire TimberRidge property and submitted along with Filing No. 1 and 2. (See Appendix) This document reflects some wetlands throughout the Sand Creek channel. Any effect on these wetlands within jurisdictional waters will be described later in this report along with the appropriate permitting.



Portions of this site have been previously studied in the "Sand Creek Drainage Basin Planning Study" (DBPS) prepared by Kiowa Engineering Corporation, March 1996. The portion of Sand Creek that traverses the site is defined as Reach SC-9 in the DBPS. 1000+ acres north of this property is tributary to this reach of the channel. (See Off-site Drainage Map in Appendix) According to the DBPS, this reach of Sand Creek all contained within the channel has the following flow characteristics:  $Q_{10} = 630$  cfs  $Q_{100} = 2170$  cfs. However, the 100 yr. flow recognized by FEMA in the LOMR 08-08-0541P with effective date of July 23, 2009, equals nearly  $Q_{100} = 2600$ cfs. Also, Sterling Ranch has finalized their MDDP which includes modeling of this property as well as the large acreage north up to the top of the Sand Creek Basin. The MDDP suggests developed flows within Sand Creek that are significantly lower than both the DBPS and FEMA currently show. These flows are as follows: At Arroya Lane crossing (SR MDDP DP-77)  $Q_{10} = 581$ cfs  $Q_{100}$  = 1468 cfs and TimberRidge south property line (SR MDDP DP-71)  $Q_{10}$  = 630 cfs  $Q_{100}$  = 1638 cfs. Even with the County approval of the MDDP and these adjusted flows, a CLOMR/LOMR will be required to be prepared, submitted and approved by FEMA prior to utilizing these flows in any Final Drainage Reports within this development. Based on the anticipated 12-18-month timing of the CLOMR/LOMR process, this development will continue to utilize the much larger FEMA recognized flows for all proposed channel improvements through this property. This is how Filings No. 1 and 2 were developed.

The majority of these off-site flows enter the property at the north end of the site conveying flows from the northwest (Black Forest area) and the off-site stock ponds to the north (both tributary to hundreds of acres of property in Black Forest). There are multiple existing culvert crossings of Vollmer Rd. just north of Arroya Lane to facilitate these historic flow patterns. The following are the few key culverts that directly feed the Sand Creek channel north of Arroya Lane: Approximately 1,000 feet north of Arroya Lane, an existing 36" CMP crosses Vollmer Road (Basin SC-1 on Off-site Drainage Map). A small basin and natural ravine just west of Vollmer feeds this facility. From a recent field visit, this small facility seems to be in good working condition, however, not labeled in the DBPS. Another 700 feet+ north along Vollmer a much larger basin exists west of the roadway. This off-site basin is approximately 350+ acres northwest of Vollmer



Road (Basin SC-2 on Off-site Drainage Map). As shown within the DBPS, this existing crossing is a 60" CMP with some very dense and tall vegetation at both the entrance and exit of this facility. But, based on a recent field visit this facility seems to be in good working condition. The DBPS depicts this facility and recommends an additional 60" CMP at this location. However, there are no signs of erosion or over topping the road at this location at this time based on the current development within the tributary area to this facility. Based on the existing surrounding topography and roadway configuration, the 100 yr. historic flows at this location would appear to spill over the roadway and continue in their historic drainage pattern downstream within the upper reach of Sand Creek.

The following descriptions represent the pre-development flow design points for the property excluding the major off-site flows within Sand Creek just described:

Basin EX-7 ( $Q_5$  = 7 cfs,  $Q_{100}$  = 37 cfs) consists of a 27.6 Ac. off-site basin west of Vollmer Road (not a part of this development) that drains under Vollmer into the TimberRidge property via an existing 48" CMP culvert and then within a natural ravine that routes the off-site flow towards the existing dual 30" RCP culverts crossing Aspen Valley within TimberRidge Filing 2 at EX DP-1.

Basin EX-1 ( $Q_5$  = 1.4 cfs,  $Q_{100}$  = 8 cfs) consists of a 4.0 Ac. on-site basin that sheet flows towards the natural ravine traversing his area and then ultimately off-site through Basin EX-9 towards the existing dual 30" RCP culverts.

Basin EX-2 ( $Q_5 = 1$  cfs,  $Q_{100} = 5$  cfs) consists of 2.8 Ac. on-site basin that sheet flows towards the proposed extension of Aspen Valley Rd. It is then conveyed south via sideroad ditch through Basin EX-9 towards the existing dual 30" RCP culverts.

Basin EX-9 ( $Q_5$  = 4 cfs,  $Q_{100}$  = 14 cfs) consists of a 6.0 Ac. off-site basin within TimberRidge Filing 2 containing the natural ravine that has been improved with permanent rock check dams. The combined flows from the basins described above are then routed towards the existing dual 30"



RCP culverts at Design Point **EX DP-1 (Q**<sub>5</sub> = **11 cfs, Q**<sub>100</sub> = **55 cfs).** Per the TimberRidge Filing 2 approved final drainage report (DP-1) the flows were determined to be ( $Q_5 = 12$  cfs,  $Q_{100} = 57$  cfs) at this location.

Basin EX-3 ( $Q_5$  = 3 cfs,  $Q_{100}$  = 20 cfs) consists of both on and off-site property containing 11.1 Ac. that sheet flows in a southwesterly direction towards Design Point EX DP-2 ( $Q_5$  = 3 cfs,  $Q_{100}$  = 20 cfs). At this location a 30" RCP storm pipe exists and collects these flows. Per the TimberRidge Filing 1 approved final drainage report (DP-18) the flows were determined to be ( $Q_5$  = 6 cfs,  $Q_{100}$  = 30 cfs) at this location.

**Basin EX-4 (Q**<sub>5</sub> = **5 cfs, Q**<sub>100</sub> = **35 cfs)** consists of 23.4 Ac. that is mostly comprised of tributary area off-site within the Sterling Ranch Master Plan. This basin sheet flows in a southwesterly direction towards a natural ravine and then directly into an existing stock pond. A 24" RCP storm system was constructed with TimberRidge Filing 1 routing the release from this facility directly into Sand Creek, as currently taking place. Per the TimberRidge Filing 1 approved final drainage report (DP-21) the flows were determined to be ( $Q_5 = 5$  cfs,  $Q_{100} = 35$  cfs) at this location.

Basin EX-5 ( $Q_5$  = 4 cfs,  $Q_{100}$  = 21 cfs) consists of 11.7 Ac. of the remaining portion of the yet undeveloped TimberRidge property along with off-site future Sterling Ranch property. This entire area sheet flows in a southwesterly direction towards the secondary emergency access road up to Arroya Lane. An existing temporary sediment basin currently captures this off-site tributary area.

Basin EX-6 ( $Q_5$  = 4 cfs,  $Q_{100}$  = 14 cfs) consists of 5.7 Ac. of the remaining portion of the yet undeveloped TimberRidge property along with some rear yards of TimberRidge Filing 2 lots. This area sheet flows in a westerly direction towards the secondary emergency access road. An existing temporary sediment basin currently captures this tributary area.



Basin EX-8 ( $Q_5$  = 4 cfs,  $Q_{100}$  = 27 cfs) consists of 14.8 Ac. of the remaining portion of the yet undeveloped TimberRidge property adjacent to Sand Creek. This entire area sheet flows directly into Sand Creek.

Add Basin EX-10

#### PROPOSED DRAINAGE CONDITIONS

Proposed development within the Retreat at TimberRidge Filing No. 3 will consist of a variety of different residential lot sizes ranging from 1.0 – 2.5-acre large rural lots to 12,000 SF min. urban lots. The rural lots will have paved streets and roadside ditches while the urban lots paved streets with County standard curb, gutter and sidewalk. Development of the urban lots proposed will consist of overlot grading for the planned roadways and lots. Development of rural lots proposed within the site will be limited to roadways and building pads, conserving the natural feature areas. Individual home sites on these lots are to be left generally in their natural condition with minimal disturbance to existing conditions per individual lot construction. Per the El Paso County ECM, Section I.7.1.B.5, rural lots of 2.5 ac. and larger are not required to provide Water Quality Capture Volume (WQCV). However, based on the current County/Urban Drainage stormwater quality standards, a WQCV component is automatically built into the UD Detention spreadsheet utilized in the detention basin design. Thus, the proposed facilities within both the rural and urban portions of this development will provide WQCV along with an Excess Urban Runoff Volume (EURV) in the lower portion of the facility storage volume with an outlet control device. Frequent and infrequent inflows are released at rates approximating undeveloped conditions. This concept provides some mitigation of increased runoff volume by releasing a portion of the increased runoff at a low rate over an extended period of time, up to 72 hours. This means that frequent storms, smaller than the 2-year event, will be reduced to very low flows near or below the sediment carrying threshold value for downstream drainage ways. Also, by incorporating an outlet structure that limits the 100-year runoff to the undeveloped condition rate, the discharge hydrograph for storms between the 2 year and the 100-year event will approximate the hydrograph for the undeveloped conditions and will help effectively mitigate the effects of development. As reasonably possible, WQCV will be provided for all new roads and urban lots.



The following describes how this development proposes to handle both the off-site and on-site drainage conditions:

As mentioned previously, the majority of the off-site flows are already within the Sand Creek channel prior to entering the property. However, the few off-site basins that must travel through the proposed site development areas prior to entering Sand Creek have been accounted for.

#### The following represent the basins east of Sand Creek:

**Design Point 1** ( $Q_5 = 6$  **cfs**,  $Q_{100} = 30$  **cfs**) represents developed flows from Basin A (14.2 Ac.). This basin develops flows from both on and off-site area that sheet flows in a southwesterly direction towards Hawks Hill Ct. and the existing 30" RCP storm at Design Point. At this location both the 5-yr. and 100-yr. flows will be completely captured and routed further downstream. Currently this storm system day lights into a natural ravine just west of Hawks Hill Ct. and ultimately into Sand Creek. With the construction of this Filing, a portion of this storm pipe will be removed west of the roadway, a Type II storm manhole will be installed to route these flows further east in Hawks Hill Ct. and then south down Antelope Ravine Dr. These developed flows enter the existing storm system within Filing 1 and are then treated by FSD Pond 2 at the south end of Filing 1. These developed flows remain consistent as accounted for in the previous Filing 1 approved drainage report at Design Point 18 ( $Q_5 = 6$  cfs,  $Q_{100} = 30$  cfs). The emergency overflow route for this sump condition will be over the crown of the roadway and then conveyed directly to Sand Creek via a side lot swale on Lot 32 within a drainage esmt.

Design Point 2 ( $Q_5$  = 2 cfs,  $Q_{100}$  = 4 cfs) represents developed flows from Basin C (1.4 Ac.). This basin develops flows from the front yards that then travel as curb and gutter flow down to Design Point 2. At this location a 5' Type R Sump Inlet will be installed to completely capture both the 5-yr. and 100-yr. developed flows. An 18" RCP storm pipe will then convey these flows further downstream. These developed flows enter the existing storm system within Filing 1 and are then treated by FSD Pond 2 at the south end of Filing 1.



**Design Point 3** ( $Q_5$  = 5 cfs,  $Q_{100}$  = 21 cfs) represents developed flows from Basins B (8.8 Ac.), D (0.20 Ac.) and OS-2 (0.47 Ac.). These basins develop flows in a westerly direction towards Antelope Ravine Dr. and then south as curb and gutter flow down to Design Point 3. At this location a 15' Type R Sump Inlet will be installed to completely capture both the 5-yr. and 100-yr. developed flows. A 30" RCP storm pipe will then convey these flows further downstream. These developed flows enter the existing storm system within Filing 1 and are then treated by FSD Pond 2 at the south end of Filing 1. The temporary sediment basin within lots 15-17 will be removed along with the completion of the overlot grading for these lots. The emergency overflow at this sump condition will pond up 1.0' and then spill over the high point in Antelope Ravine and travel south within the roadway.

**Design Point 4** ( $Q_5$  = 2 cfs,  $Q_{100}$  = 8 cfs) represents developed flows from Basins F (0.80 Ac.) and OS-7 (1.4 Ac.). These basins develop flows that travel as curb and gutter flow down Antelope Ravine Dr., off-site into Filing No. 1 to Design Point 4. At this location an existing 10' Type R At-Grade, installed with Filing 1 construction captures the following: ( $Q_5$  = 2.0 cfs,  $Q_{100}$  = 6.3 cfs) captured and ( $Q_5$  = 0.0 cfs,  $Q_{100}$  = 1.7 cfs) flow-by. These developed flows and capture percentage remain consistent as accounted for in the previous Filing 1 approved drainage report at Design Point 6 ( $Q_5$  = 2 cfs,  $Q_{100}$  = 8 cfs). An 18" RCP storm pipe will then convey these flows to connect to the existing 36" RCP outfall storm system tributary to FSD Pond 2 as described in the Filing 1 approved report. These flows are then formerly treated within this existing facility.

**Design Point 5** ( $Q_5$  = **5 cfs**,  $Q_{100}$  = **17 cfs**) represents developed flows from Basins E (2.3 Ac.) and OS-3 (4.2 Ac.). These basins develop flows that travel as curb and gutter flow down Antelope Ravine Dr. and Elk Antler Dr. within Filing No. 1 to Design Point 5. At this location an existing 15' Type R At-Grade, installed with Filing 1 construction captures the following: ( $Q_5$  = 5.0 cfs,  $Q_{100}$  = 12.7 cfs) captured and ( $Q_5$  = 0.0 cfs,  $Q_{100}$  = 4.3 cfs) flow-by. These developed flows and capture percentage remain consistent as accounted for in the previous Filing 1 approved



drainage report at Design Point 5 ( $Q_5 = 5$  cfs,  $Q_{100} = 17$  cfs). A 24" RCP storm pipe will then convey these flows to connect to the existing 36" RCP outfall storm system tributary to FSD Pond 2 as described in the Filing 1 approved report. Again, these flows are then formerly treated within this existing facility.

**Design Point 6 (Qs = 5 cfs, Q**<sub>100</sub> = **35 cfs)** represents pre-developed flows from off-site Basin OS-1 (24.1 Ac.) within the future northern portion of the Sterling Ranch property. These flows currently sheet flow into a natural ravine that then conveys them to a proposed sediment basin to be constructed on the Sterling Ranch property within a drainage esmt. acquired from the Sterling Ranch ownership group. This facility will replace the existing on-site stock pond within lot 12, which will in turn be removed and filled in. The proposed drainage esmt. will be acquired prior to the removal of the existing stock pond and any grading taking place off-site. An existing 24" RCP storm system that daylights directly into Sand Creek (previously constructed with Filing 1) will be extended up to the new off-site sediment basin and be utilized for the outlet. Upon development within this portion of the Sterling Ranch property, this sediment basin will be replaced with a permanent FSD facility to be held to these release rates. The TimberRidge Development will be responsible for the ownership and maintenance of the 24" RCP storm system and off-site sediment basin until this development within this portion of Sterling Ranch takes place.

Design Point 7 ( $Q_5$  = 2 cfs,  $Q_{100}$  = 6 cfs) represents flows from Basin OS-4 (3.3 Ac.). This basin is made up of Arroya Lane development (Rural Collector) along with off-site pre-developed flows from property to the north of Arroya Lane. Flows from this basin sheet flow into the sideroad ditch along the north side of Arroya Lane and then westerly towards Design Point 7. At this location an 18" RCP storm pipe will be installed to completely capture both the 5-yr. and 100-yr. developed flows. These flows are then conveyed further west where they combine with other developed flows within Arroya Lane and then towards Design Point 10 for stormwater quality treatment in a proposed Rain Garden 1.



Design Point 8 ( $Q_5 = 1$  cfs,  $Q_{100} = 3$  cfs) represents developed flows from Basin N (0.55 Ac.). This stretch of Arroya Lane is proposed as an urban section with curb and gutter in order to collect the impervious roadway area efficiently. The developed flows travel via curb and gutter towards Design Point 8 where a 5' Type R Sump Inlet will be installed to completely capture both the 5-yr. and 100-yr. developed flows. An 18" RCP and then 24" RCP storm pipe will then convey these flows further downstream towards the Rain Garden 1 planned at Design Point 10.

Design Point 9 ( $Q_5 = 2$  cfs,  $Q_{100} = 3$  cfs) represents developed flows from Basin M (0.54 Ac.). This stretch of Arroya Lane is also proposed as an urban section with curb and gutter. The developed flows travel via curb and gutter towards Design Point 9 where a 5' Type R Sump Inlet will be installed to completely capture both the 5-yr. and 100-yr. developed flows. An 18" RCP and then 24" RCP storm pipe will then convey these flows further downstream towards the Rain Garden 1 planned at Design Point 10.

**Design Point 10 (Q**<sub>5</sub> = **4 cfs, Q**<sub>100</sub> = **11 cfs)** represents the total developed flows routed to the proposed Rain Garden 1. This includes **Basin O (Q**<sub>5</sub> = **0.3 cfs, Q**<sub>100</sub> = **2 cfs),** a 0.61 Ac. basin that sheet flows directly into the Rain Garden 1 on the south side of Arroya Lane.

The following represents the proposed Rain Garden 1 design:

(See MHFD-Detention Design Sheets in Appendix)

Total Tributary acreage: 5.0 Ac. (Basins: OS-4, N, M and O)

0.042 Ac.-ft. WQCV required

0.238 Ac.-ft. 100-yr. Storage

0.279 Ac.-ft. Total

Total In-flow:  $Q_5 = 4 \text{ cfs}, \quad Q_{100} = 11 \text{ cfs}$ 

Pond Design Release:  $Q_5 = 1.7 \text{ cfs}, Q_{100} = 7.0 \text{ cfs}$ 

(Ownership and maintenance by the Retreat at TimberRidge Metro District 2)



**Basin OS-5** ( $Q_5 = 9$  cfs,  $Q_{100} = 61$  cfs) represents a large off-site basin that is partially owned by the TimberRidge Development Group. This mostly undeveloped basin is planned to be future 2.5 Ac. to 5.0 Ac. Rural residential lots. This basin currently sheet flows in a westerly direction and crosses the adjacent property owner's driveway just prior to directly entering Sand Creek. With the development of Arroya Lane and the culvert crossing proposed at Sand Creek, a short portion of this driveway will need to be relocated. In an effort to alleviate possible drainage concerns for this new driveway location and the off-site flows currently generated from Basin OS-5, an off-site sediment basin is proposed within the TimberRidge Development Group property to dissipate these flows. The location of this facility will likely be the permanent location of a future FSD Pond required for the development of this large parcel north of Arroya Lane. A 30" RCP storm stub is proposed to outlet this temporary sediment Basin along with the permanent future facility. This storm system will daylight directly into Sand Creek at the proposed culvert crossing.

**Basin H1** ( $Q_5 = 2$  cfs,  $Q_{100} = 8$  cfs) is 2.5 Ac. and represents developed flows from the west half of Hawks Hill Ct. and the proposed rural lot 4. These developed flows will continue to sheet flow in a southwesterly direction and directly into Sand Creek. The construction of Hawks Hill Ct. cuts off the majority of the pre-development area tributary to this portion of Sand Creek. Given the nature of this large rural lot, the minimal unconnected impervious area anticipated at the rear of this single 2.5 Ac. lot and the sizeable receiving pervious area, the WQCV reduction = 100% with 0 untreated WQCV within this basin. (See UD-BMP Runoff Reduction Sheet – Ver. 3.07 in Appendix)

**Basin H2** ( $Q_5 = 3$  cfs,  $Q_{100} = 11$  cfs) is 3.1 Ac. and represents developed flows from the west half of Hawks Hill Ct. and the proposed rural lots 31-33. These developed flows will continue to sheet flow in a southwesterly direction and directly into Sand Creek. The construction of Hawks Hill Ct. cuts off the majority of the pre-development area tributary to this portion of Sand Creek. Given the nature of these large rural lots, the minimal unconnected impervious area



anticipated at the rear of the lots and the sizeable receiving pervious area, the WQCV reduction = 100% with 0 untreated WQCV within this basin. (See UD-BMP Runoff Reduction Sheet – Ver. 3.07 in Appendix)

**Basin G** ( $Q_5 = 1$  cfs,  $Q_{100} = 5$  cfs) is 2.2 Ac. and represents developed flows from the rear yards of proposed lots 22-30. These developed flows will continue to sheet flow in a westerly direction and directly into Sand Creek. Given the minimal unconnected impervious area introduced and sizeable receiving pervious areas per lot (25' min. rear yard setback), the WQCV reduction = 100% with 0 untreated WQCV within this basin. (See UD-BMP Runoff Reduction Sheet – Ver. 3.07 in Appendix)

**Basin I** ( $Q_5 = 0.8 \text{ cfs}$ ,  $Q_{100} = 4 \text{ cfs}$ ) is 2.6 Ac. and represents the proposed tract containing the existing Sand Creek channel. No development is proposed in this basin other than the extension of the trail system along the west side of the creek corridor and the associated grading for the culvert crossing of Arroya Lane. These flows will continue to sheet flow to the base of the creek bed and be conveyed south within the natural channel. The trail within this basin falls under the WQ exclusion 1.7.1.B.9.

#### The following represent the basins and design points west of Sand Creek:

**Basin J** ( $Q_5 = 2$  cfs,  $Q_{100} = 6$  cfs) represents a 2.5 ac. portion of the proposed rural lot 3 and the adjacent Aspen Valley roadway extension that will continue to sheet flow in a southeasterly direction towards Sand Creek. Given the size of this lot (rural residential large lot - 2.5 ac. min.) stormwater quality is not required for the lot area but only the adjacent paved roadway extension. Based on the impervious area of this minimal roadway extension (0.25 ac.), this basin seems to meet the requirements for exclusions I.7.1.B.5 & I.7.1.C.1 of the ECM. (See drainage map for exclusion area)



Design Point 11 ( $Q_5$  = 3 cfs,  $Q_{100}$  = 11 cfs) represents construction of the north half of Arroya Lane and off-site pre-developed flows from Basin OS-6 ( $Q_5$  = 3 cfs,  $Q_{100}$  = 11 cfs), a 5.9 Ac. basin that sheet flows in a southeasterly direction towards Arroya Lane. The majority of this off-site area is within the RR-5 zone and planned to be a single future rural residential lot. Given the size of this basin and future land use (rural residential large lot - 2.5 ac. min.) stormwater quality is not required for the lot area but only the adjacent paved roadway extension. Based on the impervious area of the adjacent roadway improvements for Vollmer and Arroya Lane (0.70 ac.), this basin-seems to meet the requirements for exclusions I.7.1.B.5 & I.7.1.C.1 of the ECM. (See drainage map for exclusion area) At Design Point 11, these developed flows will be collected by a 24" RCP culvert and then routed directly to Sand Creek with a rip-rap dissipator at the outfall location. (See Appendix)

As mentioned previously, Basin EX-7 ( $Q_5 = 7$  cfs,  $Q_{100} = 37$  cfs) consists of the (27.6 Ac.) off-site basin west of Vollmer Road (not a part of this development) that drains under Vollmer into the TimberRidge property via an existing 48" CMP culvert. This facility will remain. These flows are then combined with the flows from basins L, K and EX-9. Basins L ( $Q_5 = 2$  cfs,  $Q_{100} = 9$  cfs) and K ( $Q_5 = 2$  cfs,  $Q_{100} = 8$  cfs) consist of (3.9 Ac. and 3.3 Ac. respectively) on-site developed flows from proposed lots 1 and 2 adjacent to Vollmer Rd. and Arroya Lane. These flows travel as natural channel and sideroad ditch flows directly into Basin EX-9. Basin EX-9 ( $Q_5 = 4$  cfs,  $Q_{100} = 4$  cfs) consists of (6.0 Ac.) made up of two existing rural lots within Filing 2 and the natural ravine. Along with Filing 2 construction, permanent rock check dams have been installed down this natural ravine tributary to the dual existing 30" RCP culverts at **Design Point 12** ( $Q_5 = 12$  cfs,  $Q_{100} = 57$  cfs). This is compared to the anticipated developed flows within the Filing 2 report at DP-1 of ( $Q_5 = 12$  cfs,  $Q_{100} = 57$  cfs). Thus, these facilities will continue to adequately handle the developed flows. As described in the Filing No. 2 FDR, these combined developed flows are then conveyed via storm system to FSD Pond 3 for stormwater quality treatment.



#### **DETENTION / STORMWATER QUALITY FACILITES**

As required, storm water quality measures will be utilized in order to reduce the amount of sediment, debris and pollutants that are allowed to enter Sand Creek. These features include but are not limited to Rain Gardens, temporary sediment basins, runoff reduction RPA's and two existing FSD's. Site Planning and design techniques for the large lot, rural areas should help limit impervious area, minimize directly impervious area, lengthen time of travel and increase infiltration in order to decrease the rate and volume of stormwater runoff. Urban areas that require detention will provide a Water Quality Capture Volume (WQCV) and Excess Urban Runoff Volume (EURV) in the lower portion of the facility storage volume that will release the more frequent storms at a slower rate to help minimize the effects of development of the property. The proposed SWQ facilities are to be private facilities with ownership and maintenance by the TimberRidge Metropolitan District 2. After completion of construction and upon the Board of County Commissioners acceptance, the Sand Creek channel and proposed structural improvements will be owned and maintained by the El Paso County along with all drainage facilities within the public Right of Way. The remaining portion of the channel (non-structural) will be maintained by the Retreat at TimberRidge Metro District 2. Access ramps are provided in multiple locations and constructed along with the channel improvements and are accessed from the proposed regional trail along the west side of the channel.

#### SAND CREEK CHANNEL IMPROVEMENTS

As stated in the Sand Creek DBPS, this Reach SC-9 is recommended as a floodplain preservation design concept. Given the fact of the current requirements for detention/SWQ facilities planned for the property with designed release at or below pre-development flows, the existing Sand Creek drainageway is expected to remain stable. From the HEC-RAS model provided in the appendix, channel velocities through this portion of the reach range from 1.2-8.5 ft/s and the shear stress range from 0.3-15.1 lb/sq ft. Although, based on the findings from the CORE Consultants, Inc. Impact Identification Report, no significant erosion or channel degradation through this property currently exists at this time. Specifically located grade control structures (See Appendix) were specified in the DBPS through this reach in order to slow the channel velocity



to the DBPS recommended 7 feet per second and to prevent localized and long-term stream degradation affecting channel linings and overbanks. The allowable velocity and shear stress will vary depending upon the existing riparian vegetation/wetlands found within the channel and overbank floodplain terrace areas. The following general criteria taken from the Mile High Flood Control District (Table 8-3) has been utilized for this channel design with adjustments made for site specific riparian vegetation through this reach (See HEC-RAS Modeling Section below):

Open Channels Chapter 8

Table 8-3. Design parameters for naturalized channels

Design Parameter	Design Value
Maximum 100-year depth outside of bankfull channel	5 ft
Roughness values	Per Table 8-5
Maximum 5-year velocity, main channel (within bankfull channel width) (ft/s)	5 ft/s
Maximum 100-year velocity, main channel (within bankfull channel width) (ft/s)	7 ft/s
Froude No., 5-year, main channel (within bankfull channel width)	0.7
Froude No., 100-year, main channel (within bankfull channel width)	0.8
Maximum shear stress, 100-year, main channel (within bankfull channel width)	1.2 lb/sf
Minimum bankfull capacity of bankfull channel (based on future development conditions)	70% of 2-year discharge of 10% of 100-yr discharge, whichever is greater
Minimum bankfull channel geometry	Per Table 8-2
Minimum bankfull channel width/depth ratio (Equation 8-3)	9
Minimum entrenchment ratio (Equation 8-4)	3
Maximum longitudinal slope of low flow channel (assuming unlined, unvegetated low flow channel)	0.2 percent
Bankfull channel sinuosity (Equation 8-5)	1.1 to 1.3
Maximum overbank side slope	4(H):1(V)
Maximum bankfull side slope	2.5(H):1(V)
Minimum radius of curvature	2.5 times top width

(See page 20 below for allowable shear stress adjustments)

January 2016

A HEC-RAS hydraulic analysis for this portion of Reach SC-9 has been provided in order to determine any necessary channel improvements for the proposed Filing No. 3 development. A separate wetland impact report along with the Section 404 permitting, prepared by Bristlecone Ecology, LLC, will be developed based on these proposed channel improvements and submitted directly to the U.S. Army Corps of Engineers with necessary consult with U.S. Fish and Wildlife for their review and approval. This report was submitted separately to El Paso County Staff and can be found in same file number SF-22-041.

#### **HEC-RAS MODELING**

HEC-RAS ver. 6.3.1 was used to perform an updated one-dimensional, steady flow hydraulic model of the upper portion of Reach SC-9 from approximately 400 feet north of Arroya Lane down to the Retreat at TimberRidge Filing 2 boundary within Sand Creek. This model defined the stream centerline, overbanks, cross-sections and manning's n values. The stream centerline follows the channel thalweg to define the reach network. Cross-section topography data was obtained by using the generated surface from the 2-ft. flown contours utilized for all site design. This data was then utilized within the AutoCAD containing three-dimensional coordinates for the stream centerline, cross-sections, reach stations, overbank stations and reach lengths. Different Manning's n values were applied across the various channel cross-sections to reflect the changes in vegetative cover within the channel and overbanks. The selected Manning's n values for the channel and overbanks were determined using Tables 10-1 and 10-2 from the DCM and Table 3 from the USGS Guide for selecting Manning's Roughness Coefficients based on numerous site visits in an effort to photograph and document each cross-section. (See Appendix) The following table summarizes the selected Manning's n values:

Table 1 Manning's n Values

Feature	Manning's n Value
Main Channel	0.03 - 0.10
Overbank Floodplain Terraces	0.12 - 0.16



Steady flow data was entered starting just north of Arroya Lane, channel station 14+00.86 down to approximately 300 feet south of the Filing 2 boundary, channel station 1+02.08 all within the Sand Creek DBPS segment 171. Steady flow data corresponding to recurrence intervals of 100 Yr. for the FEMA, DBPS and Sterling Ranch MDDP conditions was entered. The models were run in subcritical mode to evaluate hydraulic conditions. Boundary conditions for the entire reach were based on normal depth calculations for the upstream and downstream channel slopes. The following table summarizes the flows used in the models:

Table 2 Model Flow Values

Flood Event / Location	Flow Value (cfs)	
DBPS Segment 171		
FEMA 100 Yr.	2600	
DBPS 100 Yr.	2170	
SR MDDP 100 yr.	1468	
DBPS 10 yr.	630	
SR MDDP 10 yr.	581	

Per the approved DBPS, the anticipated developed flows just upstream of Arroya Lane are  $Q_{10}$  = 630 cfs and  $Q_{100}$  = 2170 cfs as depicted within DBPS segment no. 171. Per the approved Sterling Ranch MDDP, the flows are are  $Q_{10}$  = 581 cfs and  $Q_{100}$  = 1468 cfs. As discussed earlier, the FEMA FIS flows appear to be significantly higher than both those presented in the DBPS and the Sterling Ranch MDDP. However, we have continued to utilize the significantly larger flows as determined by the FEMA FIS (2600 cfs) in the channel improvement designs. The proposed public roadway crossing of Sand Creek at Arroya Lane will be constructed with this Filing No. 3 and consists of a single cell Bridgecor two radius arch (31'-11" Span x 11'-8" Rise) with concrete headwalls and wingwalls to facilitate the conveyance of the 100 yr. flow.

Based on site visits during September of 2022, the entire Sand Creek drainage corridor through the Retreat at TimberRidge development was walked and photographed for documentation purposes and aide in the HEC-RAS modeling. (See Appendix) As discovered in the field and documented in the photos taken along the corridor, this reach of the Sand Creek channel appears very stable with no signs of erosion within the main channel or channel overbanks. This is mainly due to the significant vegetal cover throughout the reach. The classification of the vegetal cover seems to have a range from Retardance Class A-C as defined by HEC-15 chart (See Appendix) This type of vegetation retardance significantly increases the allowable shear stress within the channel while reducing the velocity. The following table defines the retardance level based on the vegetation class:

Table 3

Vegetal Retardance Curve Index by SCS Retardance Class

SCS Retardance Class	Retardance Curve Index	
А	10.0	
В	7.64	
С	5.60	
D	4.44	
E	2.88	

Based on this information, the maximum allowable sheer stress is found by the flowing equation:

T = 0.75Curve Index

Thus, the allowable range of shear stress for this reach of Sand Creek equals 4.2 – 7.5 (lb/ft²). With the proposed channel improvements/structures, all sections within this reach fall within the shear stress range above.

Referencing the HEC-RAS model calculations in the Appendix shows that only one station section showed velocity outside the recommended 5.0 - 7.0 ft./sec. per Table 8.3 and shear stress exceeding the limit above. This station is immediately downstream of the proposed culvert crossing (Sta: 9+21.93). Improvements proposed to help stabilize this area include proposed Riprap dissipation and a check structure installed at approximately Station 8+35. Thus, the channel improvements proposed within Filing No. 3 consist of upstream bank stabilization to facilitate



the conveyance of flow into the culvert from Sta: 12+25 to 10+75, rip-rap dissipation immediately downstream of the culvert and installation of an additional check structure located at station 8+35.

The DBPS recommended to provide selective rip-rap channel stabilization located at culvert crossings, pipe outlets and outside bends of the channel. Based on the mean channel slope of 1.58% for this stretch and maximum allowable velocity criteria of 7.0 fps, Type L Rip-Rap stabilization will be provided at the select location upstream of the culvert crossing. (See Appendix for d<sub>50</sub> calculation and tables describing slope, velocity, shear, Froude No., etc.) The existing channel slope throughout this reach ranges from 0.7% to 2.5%. Per the HEC-RAS model, the proposed channel velocities downstream of the culvert range from 2.9 ft./sec. to 4.9 ft./sec. After improvements, all stations are within the allowable velocity of 7.0 ft./sec. and shear stress is below 5.0 based on the SCS Retardance Index above. The proposed model calculations also show only one station with Froude No. over the 0.8 criteria. However, at this specific area, the proposed check structure and rip-rap are planned.

The DBPS does not depict any structures along this stretch of channel. However, one additional check structure is being planned to further limit degradation and help control the elevation of the channel invert. The check structures are designed to be sheet piling with a concrete cap per Urban Drainage Vol. 2 Figures 9-27 thru 9-28. The intent of this structure is to hold grade so if the stream wants to flatten its equilibrium slope, the incision is limited. Thus, the plan is for this structure to eventually become drop structures as dictated by future channel characteristics.

A public trail/ access road along the west side of Sand Creek is planned and will allow for maintenance access to associated channel improvements. (See channel plans for exact ramp locations and details)



#### DRAINAGE CRITERIA

Hydrologic calculations were performed using the City of Colorado Springs/El Paso County Drainage Criteria Manual, as revised in November 1991 and October 1994 with County adopted Chapter 6 and Section 3.2.1 of Chapter 13 of the City of Colorado Springs/El Paso County Drainage Criteria Manual as revised in May 2014. Individual on-site developed basin design used for detention/SWQ basin sizing, inlet sizing and storm system routing was calculated using the Rational Method. Runoff Coefficients are based on the imperviousness of the particular land use and the hydrologic soil type in accordance with Table 6-6. The average rainfall intensity, by recurrence interval found in the Intensity-Duration-Frequency (IDF) curves in Figure 6-5. (See Appendix)

The City of Colorado Springs/El Paso County DCM requires the Four Step Process for receiving water protection that focuses on reducing runoff volumes, treating the water quality capture volume (WQCV), stabilizing drainage ways, and implementing long-term source controls. The Four Step Process pertains to management of smaller, frequently occurring storm events, as opposed to larger storms for which drainage and flood control infrastructure are sized. Implementation of these four steps helps to achieve storm water permit requirements.

This site adheres to this **Four Step Process** as follows:

1. Employ Runoff Reduction Practices: Proposed rural lot impervious area (roof tops, patios, etc.) will sheet flow across lengthy landscape/natural areas within the large lots and proposed urban lot impervious areas (roof tops, patios, etc.) will sheet flow across landscaped yards and through open space areas to slow runoff and increase time of concentration prior to being conveyed to the proposed public streets or detention facilities. This will minimize directly connected impervious areas within the project site.



Reference the Water Quality Treatment Plan Map in the Appendix for the following:

Area qualifies for exclusion E – large lot single famil	y 4.9 ac.
Area qualifies for exclusion H – stream stabilization	2.6 ac.
Area qualifies for 20% exclusion (ECM I.7.1.C.1)	5.9 ac.
Area treated in existing permanent Pond 3	40.8 ac.
Area treated in existing permanent Pond 2	33.77 ac.
Areas treated in proposed Rain Garden Facility	5.0 ac.
Area of runoff reduction water quality treatment	5.9 ac.
(Reference Runoff Reduction Calculations in Appen	dix for these areas)

Filing No. 3 Total platted area	44.35 ac.
Off-site area treated in temporary sediment basins	69.9 ac.

- 2. Stabilize Drainageways: After developed flows utilize the runoff reduction practices through the front and rear yards, developed flows will travel via roadside ditches in the large lot, rural portions of the development, curb and gutter within the public streets in the urban portions of the development and eventually public storm systems. These collected flows are then routed directly to existing extended detention basins (full-spectrum facilities). Where developed flows are not able to be routed to public street, sheet flows will travel across landscaped rear yards and then through undeveloped property prior to entering Sand Creek. The Sand Creek channel corridor will be protected with various channel improvements as recommended in the Sand Creek DBPS and proposed with this Filing in order to reduce velocities to erosive levels.
- 3. Provide Water Quality Capture Volume (WQCV): Runoff from this development will be treated through capture and slow release of the WQCV and excess urban runoff volume (EURV) in the existing Full-Spectrum permanent Extended Detention Basins designed per current El Paso County drainage criteria (constructed with Filings 1 & 2). For the 5.3 ac. that is not able to be captured and routed to one of the permanent EDB's, Runoff Reduction practices are required and provided in the 25'-35' rear setbacks of the lots



within these specific basins. Reference Runoff Reduction Calculations in Appendix for these areas that show a 100% WQCV Reduction and meets El Paso County standards.

4. Consider need for Industrial and Commercial BMPs: No industrial or commercial uses are proposed within this development. However, a site-specific storm water quality and erosion control plan and narrative has been submitted along with the grading and erosion control plan. Details such as site-specific sediment and erosion control construction BMP's as well as temporary and permanent BMP's were detailed in this plan and narrative to protect receiving waters. Multiple temporary BMP's are proposed based on specific phasing of the overall development. BMP's will be constructed and maintained as the development has been graded and erosion control methods employed.

#### FLOODPLAIN STATEMENT

Portions of this site are located within a floodplain as determined by the Flood Insurance Rate Maps (F.I.R.M.) Map Number 08041C 0535G with effective date of December 7, 2018 and the previously mentioned LOMR 08-08-0541P with an effective date of July 23, 2009. (See Appendix).

#### **DRAINAGE AND BRIDGE FEES**

This site lies entirely within the Sand Creek Drainage Basin boundaries.

The fees are calculated using the following impervious acreage method approved by El Paso County. The Retreat at TimberRidge Filing No. 3 has a total area of 44.35 acres with the following different land uses proposed:

2.60 Ac.	Sand Creek Drainage corridor (Tract E)
1.34 Ac.	Open Space/Trail Tracts (Tracts A, B, C, D, F, G, H and I)
19.41 Ac.	2.5 Ac. lots (Rural Lots 1-6 incl. adj. ROW)
12.95 Ac.	1.0 Ac. lots (Rural Lots 7-14 and 31-33 incl. adj. ROW)
8.05 Ac.	0.5 Ac. avg. Lots (Urban Lots 15-30 incl adj. ROW)

44.35 Ac. Total



The percent imperviousness for this subdivision is calculated as follows:

#### **Fees for Sand Creek Drainage Corridor**

(Per El Paso County Percent Impervious Chart: 2%)

2.60 Ac.  $\times$  2% = **0.052 Impervious Ac.** 

#### Fees for Open Space/Trail Tracts

(Per El Paso County Percent Impervious Chart: 7%)

1.34 Ac. x 7% = 0.09 Impervious Ac.

#### Fees for 2.5 Ac. lots

(Per El Paso County Percent Impervious Chart: 11% with 25% fee reduction for

2.5 ac. lots planned – ECM 3.10.2a) – Reduction for Drainage Fees only

19.41 Ac. x 11% x 75% = **1.60 Impervious Ac.** (Drainage Fees)

19.41 Ac. x 11% = **2.14 Impervious Ac.** (Bridge Fees)

#### Fees for 1.0 Ac. lots

(Per El Paso County Percent Impervious Chart: 30%)

12.95 Ac.  $\times$  20% = **2.59 Impervious Ac.** 

#### Fees for 1/2 Ac. lots

(Per El Paso County Percent Impervious Chart: 25%)

8.05 Ac.  $\times$  25% = **2.01** Impervious Ac.

**Total Impervious Acreage:** 6.34 Imp. Ac. (Drainage Fees)

**Total Impervious Acreage:** 6.88 Imp. Ac. (Bridge Fees)



The following calculations are based on the 2022 Sand Creek drainage/bridge fees:

#### **ESTIMATED FEE TOTALS:**

**Bridge Fees** 

\$ 8,923.00 x 6.88 Impervious Ac. = \$ **61,390.24** 

**Drainage Fees** 

\$ 21,814.00 x 6.34 Impervious Ac. = **\$ 138,300.76** 

However, per the ECM 3.10.5.a, this development requests a reduction of drainage fees based on the on-site regional channel improvements for this stretch of Sand Creek Reach SC-9 as shown in the DBPS. The following facilities within the Sand Creek Drainage Basin seem to meet the criteria for this reduction:

#### **Proposed Filing No. 3 Sand Creek Channel Improvements:**

	Sheet Pile Check Structure w/ Conc. Cap	\$45,000 EA x 1 =	\$ 45,000
	Selective Bank Stabilization (Buried Rip-Rap)	\$100/LF x 270 LF =	\$ 27,000
	Selective Bank Stabilization (Grading & Reveg.)	=	\$ 80,000
	Sub-Total	=	\$152,000
	10 % Engineering		\$ 15,200
Total			\$167,200

Classic Consulting Engineers & Surveyors cannot and does not guarantee that the construction cost will not vary from these opinions of probable construction costs. These opinions represent our best judgment as design professionals familiar with the construction industry and this development in particular.

(Exact facility costs provided upon construction and acceptance by County. Any credits may be used within the Sand Creek basin)



In order to ascertain the current drainage fee versus reimbursable drainage facility obligations, the Sand Creek Drainage Basin Planning Study (Kiowa Engineering Corporations, dated March 1996) was consulted.

For the TimberRidge Filing No. 3, the reimbursable drainage facilities identified in the DBPS are as follows:

<u>Item</u>	<u>Location</u>	<u>on</u>	<u>1992 Cost</u>	2021 Cost (adjusted)
	Check Structure	std. 890+50	\$14,400	\$40,103.91
		(reach SC-9)		
	Check Structure	std. 907+100	\$15,300	\$42,610.00
		(reach SC-9)		
		Total Reimbur	sable / Improvements	\$82,713.91
		10% Engineer	ing	\$ 8,271.39
		5% Contingen	су	\$ 4,135.70
		Total		\$95,121.00

Until actual construction costs are vetted or additional improvements are approved in a DBPS amendment, the \$95,121.00 can offset the total drainage fees shown above as \$138,300.76. Thus, the total current drainage fees owed for Filing No. 3 is \$43,179.76.

As it pertains to Bridge fees, the Sand Creek DBPS indicates, "Bridges are defined as those structures conveying at least 1500 cubic feet per second, having a flow area of at least 200 square feet, or having a span of 20 feet or greater." The DBPS reflects the Arroya Lane crossing as "new 6' x 12" W CBC – 10 Year capacity". Similar to the Poco crossing, a full 100-yr conveyance has been provided on Arroya Lane due to the roadways anticipated easterly extension and connection to future communities. The Arroya Lane crossing meets all of the



three thresholds established in the DBPS to define it as a bridge (essentially the same as the Poco crossing). The Poco crossing (constructed with Filing No. 1) exceeded \$1,000,000 in costs and therefor since similarly sized, the Arroya crossing will be similar. While bridge fees were already paid in Filing No. 1 and Filing No. 2, we'd respectfully request that bridge fees not be collected for Filing No. 3 based on the required construction of the Arroya Lane bridge crossing and anticipated costs well exceeding the bridge costs shown above.

In summary, we feel that the following drainage and bridge fees should be collected with prior to the recording of Retreat at TimberRidge Filing No. 3:

#### **ADJUSTED FEE TOTALS:**

#### **Bridge Fees**

\$61,390.24 (offset by Arroya bridge construction) = No Bridge fees due

#### **Drainage Fees**

\$ 138,300.76 – 95,121.00 (Improvements in DBPS) = **\$ 43,179.76** 



#### **SUMMARY**

The proposed Retreat at TimberRidge Filing No. 3 is within the Sand Creek Drainage Basin. Recommendations are made within this report concerning necessary improvements that will be required as a result of development of this property. The points of storm water release from the proposed site are required to be at or below the calculated historic flow quantities. The development of the proposed site does not significantly impact any downstream facility or property to an extent greater than that which currently exists in the pre-development conditions. All drainage facilities within this report were sized according to the Drainage Criteria Manuals and the full-spectrum storm water quality requirements.

PREPARED BY:

Classic Consulting Engineers & Surveyors, LLC

Marc A. Whorton, P.E.

Project Manager

maw/118530/FDR Fil. 3.doc



#### REFERENCES

- 1. City of Colorado Springs/County of El Paso Drainage Criteria Manual as revised in November 1991 and October 1994 with County adopted Chapter 6 and Section 3.2.1 of Chapter 13 of the City of Colorado Springs/El Paso County Drainage Criteria Manual as revised in May 2014.
- 2. "Urban Storm Drainage Criteria Manual Volume 1, 2 & 3" Urban Drainage and Flood Control District, dated January 2016.
- 3. "Final Drainage Report for Forest Gate Subdivision" Law & Mariotti Consultants, Inc. dated October 2004.
- 4. "Sand Creek Drainage Basin Planning Study," Kiowa Engineering Corporation, dated March 1996.
- 5. "Master Development Drainage Plan for The Retreat at TimberRidge", Classic Consulting, approved March 2018.
- 6. "Preliminary Drainage Report for The Retreat at TimberRidge Preliminary Plan South of Arroya Lane", Classic Consulting, approved October 2018.
- 7. "2018 Sterling Ranch MDDP", M&S Civil Consultants, Inc., June 2018
- 8. "Final Drainage Report for Retreat at TimberRidge Filing No. 1", Classic Consulting, approved November, 2020.
- 9. "Final Drainage Report for Retreat at TimberRidge Filing No. 2", Classic Consulting, approved September, 2022.



#### **APPENDIX**



#### **VICINITY MAP**



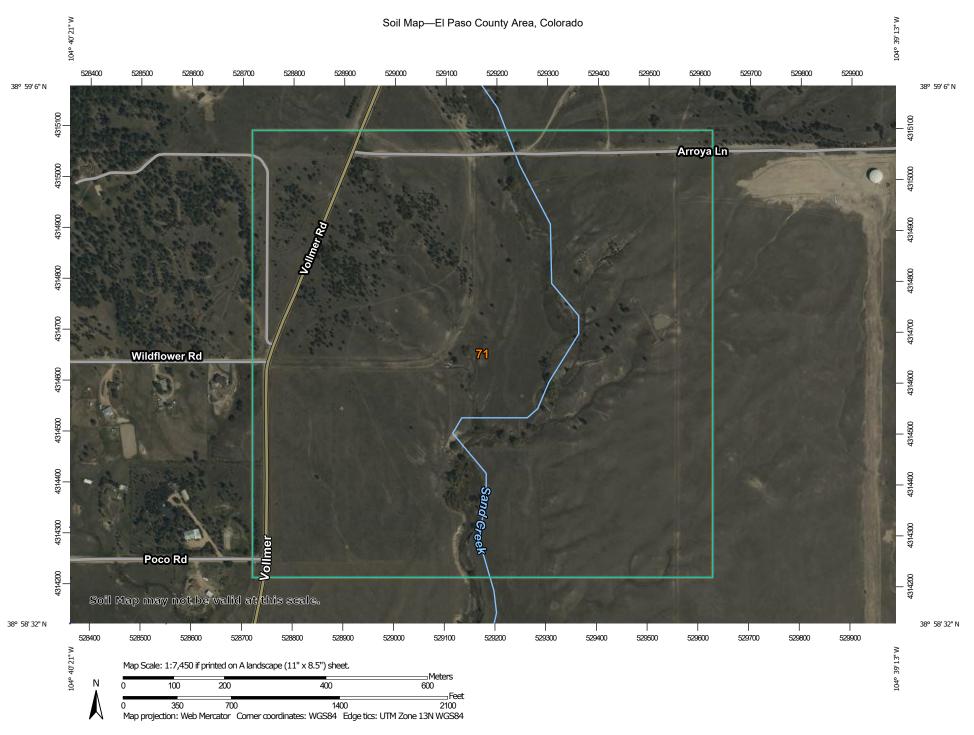
# RETREAT AT TIMBERRIDGE FILING NO. 3

# **VICINITY MAP ARROYA LANE** FILING NO. 3 FILING NO. 3 3 LOTS 3 LOTS (2.5 AC.) (2.5 AC.) (FILING NO. 2) VOLLMER ROAD FILING NO. 3 **27 URBAN LOTS** (FILING NO. 1) (FILING NO. 2) **POCO ROAD** (FILI VG NO. 1) STERLING RANCH **PROPERTY**

STERLING RANCH PROPERTY

**SOILS MAP (S.C.S SURVEY)** 





#### MAP LEGEND

#### Area of Interest (AOI)

Area of Interest (AOI)

#### Soils

Soil Map Unit Polygons



Soil Map Unit Points

#### Special Point Features

(o) Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Candfill

Lava Flow

Marsh or swamp

Walsh or swall

Mine or Quarry

Miscellaneous Water

Perennial Water

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

#### CLIAD

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot
 Other

Special Line Features

#### Water Features

Δ

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 19, Aug 31, 2021

Soil map units are labeled (as space allows) for map scales 1:50.000 or larger.

Date(s) aerial images were photographed: Sep 11, 2018—Oct 20, 2018

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.

# **Map Unit Legend**

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
71	Pring coarse sandy loam, 3 to 8 percent slopes	197.8	100.0%
Totals for Area of Interest		197.8	100.0%

# El Paso County Area, Colorado

# 71—Pring coarse sandy loam, 3 to 8 percent slopes

# **Map Unit Setting**

National map unit symbol: 369k Elevation: 6,800 to 7,600 feet

Farmland classification: Not prime farmland

## **Map Unit Composition**

Pring and similar soils: 85 percent

Estimates are based on observations, descriptions, and transects of

the mapunit.

# **Description of Pring**

## Setting

Landform: Hills

Landform position (three-dimensional): Side slope

Down-slope shape: Linear Across-slope shape: Linear

Parent material: Arkosic alluvium derived from sedimentary rock

## Typical profile

A - 0 to 14 inches: coarse sandy loam
C - 14 to 60 inches: gravelly sandy loam

#### **Properties and qualities**

Slope: 3 to 8 percent

Depth to restrictive feature: More than 80 inches

Drainage class: Well drained

Runoff class: Low

Capacity of the most limiting layer to transmit water (Ksat): High

(2.00 to 6.00 in/hr)

Depth to water table: More than 80 inches

Frequency of flooding: None Frequency of ponding: None

Available water supply, 0 to 60 inches: Low (about 6.0 inches)

## Interpretive groups

Land capability classification (irrigated): None specified

Land capability classification (nonirrigated): 3e

Hydrologic Soil Group: B

Ecological site: R048AY222CO - Loamy Park

Hydric soil rating: No

#### **Minor Components**

## **Pleasant**

Percent of map unit: Landform: Depressions Hydric soil rating: Yes

## Other soils

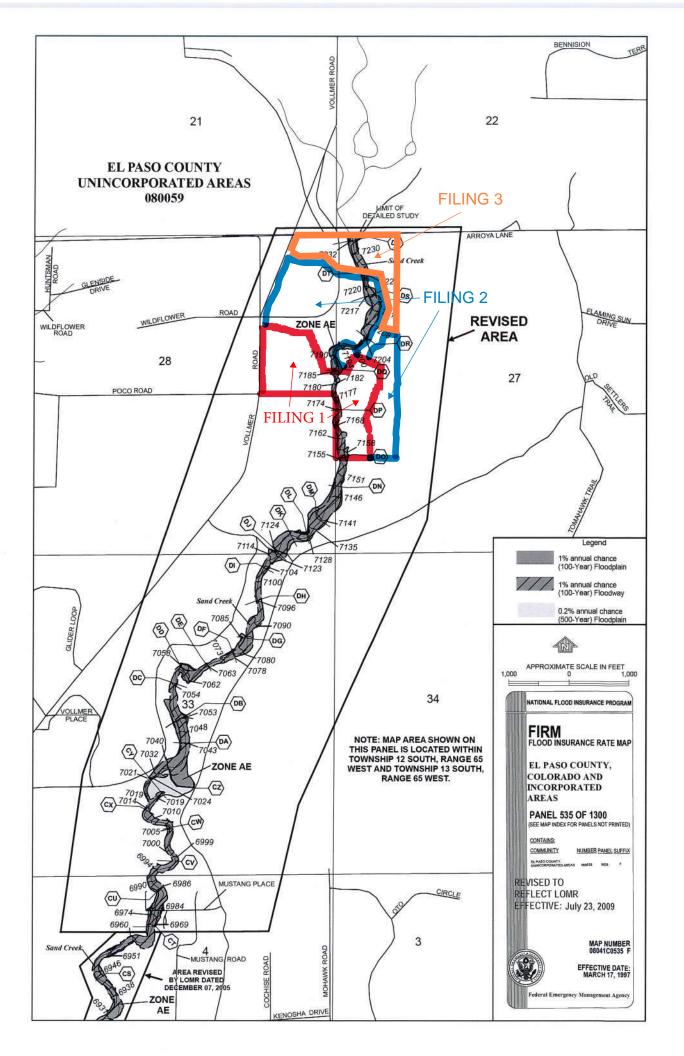
Percent of map unit: Hydric soil rating: No

# **Data Source Information**

Soil Survey Area: El Paso County Area, Colorado Survey Area Data: Version 19, Aug 31, 2021

F.E.M.A. MAP / LOMR (08-08-0541P)





#### NOTES TO USERS

map is for use in administering the National Flood Insurance Program. It doe necessarily identify all areas subject to fooding, particularly from local dranegerose of small size. The community map repository should be consulted to sible updated or additional flood hazard information. his map is for use in ac

obtain more detailed information in areas where Base Flood Elevations (BFE o obtain more destaled information in areas where Base Flood Elevations (BFEs) didor floodways have been determined, users are encouraged to causal the Floor folders and Floodway Data and/or Summary of Siliwater Elevations tables contained thinhi the Flood Insurance Study (FIS) report that accompanies this FIRM. Users rould be aware that BFEs shown on the FIRM represent rounded whole-foot sevations. These BFEs are intended for flood insurance rating purposes only and round to be used as the sole source of flood elevation information. Accordingly ood elevation data presented in the FIS report should be utilized in conjunction with e FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.1 North American Vertical Datum of 1988 (NAVD88). Users of this FIRMs should be ware that coastal food elevations are also provided in the Simmary of Sithware Elevations table in the Flood Insurance Study report for this presdiction. Elevations shown in the Summary of Silkwart Elevations table should be used for construction and/or Roodplain management purposes when they are higher than the elevation theorem on this Ellipse.

ertain areas not in Special Flood Hazard Areas may be protected by flood control ructures. Refer to section 2.4 "Flood Protection Measures" of the Flood Insurance tudy report for information on flood control structures for this jurisdiction.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988 (NAVD88). These flood elevations must be compared to structure and ground elevations referenced to the same verifical datum. For information region conversion between the Nabonal Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, with the Nabional Geodetic Survey website at http://www.ngs.nosa.gov/ or contact the National Geodetic Survey website at http://www.ngs.nosa.gov/ or contact the National Geodetic Survey at the following address:

NGS Information Ser NOAA, N/NGS12 ational Geodetic Survey SMC-3, #9202

o obtain current elevation, description, and/or location information for bench mark hown on this map, please contact the Information Services Branch of the Nation leodelic Survey at (301) 713-3242 or visit its website at http://www.ngs.ndaa.gov/.

Base Map information shown on this FIRM was provided in digital format by El Pasc County, Colorado Springs Ultilities, and Anderson Consulting Engineers, Inc. These data are current as of 2008.

This map reflects more detailed and up-to-date stream channel configurations and floodplain delineations than those shown on the previous FIRM for this jurisdiction. The floodplain and floodways that were transferred from the previous FIRM so this jurisdiction. The floodplain and floodways that were transferred from the previous FIRM so that previous FIRM so that the state of the flood insurance. So a result, the Flood floodway Date tables in the Flood insurance. So a flood floodway Date tables in the Flood insurance. Set distances that differ from what is shown on this map. The profile baselines depicted and infloodway Date is shown on this map. The profile baselines depicted and infloodway Date Tables If applicable, in the FIS report. As a result, the profile baselines and evoluties depicted and infloodway Date Tables If applicable, in the FIS report. As a result, the profile baselines are given deviate significationly from the new base map channel representation and may appear outside of the floodplain.

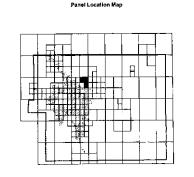
reporate limits shown on this map are based on the best data available at the tir

Please refer to the separately printed Map Index for an overview map of the county showing the layout of map panels, community map repository addresses; and a usting of Communities table containing National Flood insurance Program dates for each community as well as a listing of the panels on which each community is

Contact FEMA Map Service Center (MSC) via the FEMA Map information exchange (FMIX) 1-877-335-2677 for information on available products associated with things. FIRM. Available products may include previously issued Leltiers of Map Chanton. Plood insurance Study Report, and/or digital versions of this map. The MSC may also be reached by Fax at 1-800-358-9620 and its website at http://www.mac.fema.gov/.

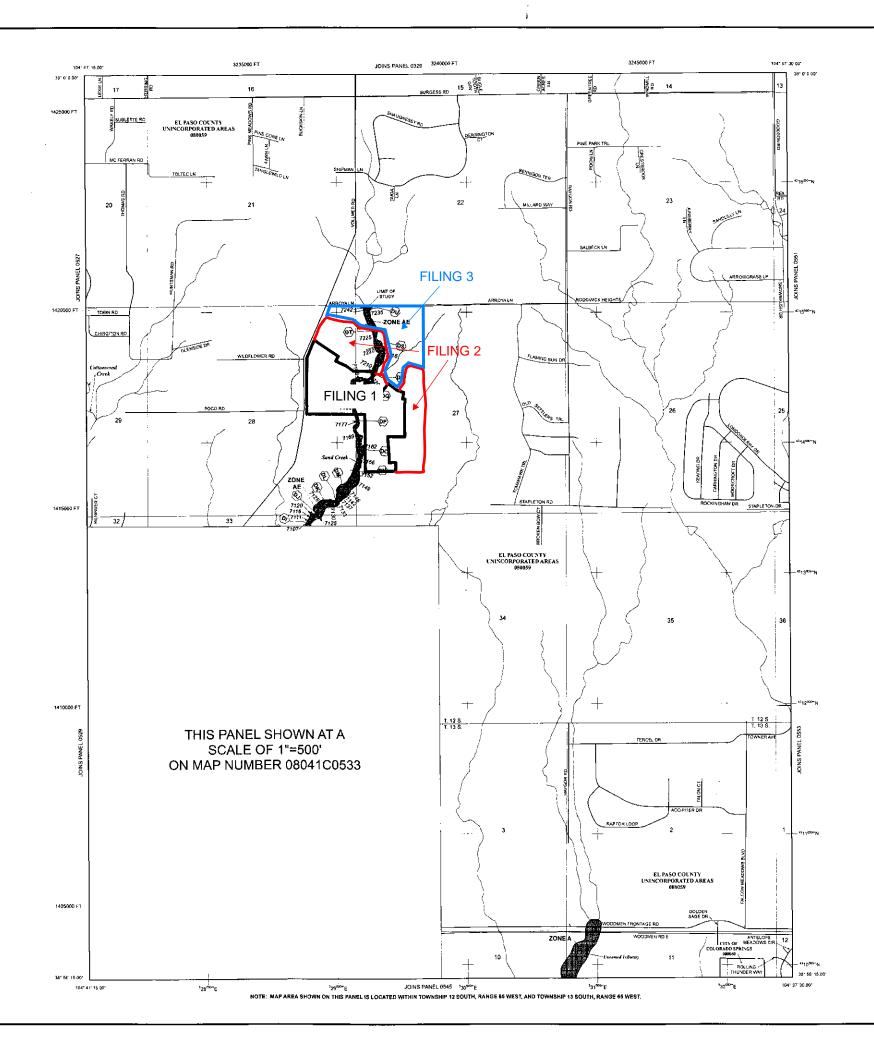
If you have questions about this map or questions concurning the National Floo-insurance Program in general, please call 1-877-FEMA MAP (1-877-338-2627) of visit the FEMA website at http://www.fema.gov/business/nfip

# Flooding Source REFER TO SECTION 3.3 OF THE EL PASO COUNTY FLOOD INSURANCE STUDY FOR STREAM BY STREAM VERTICAL DATUM CONVERSION INFORMATION



This Digital Flood Insurance Rate Map (DFIRM) was produced through a Cooperating Technical Partner (CTP) agreement between the State of Colorado Water Conservation Board (CWCB) and the Faderal Emergency Management Agency (FEM).





#### LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAS) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1%, chance of being equaled or excessed in any given year. The Special Flood Heard Area is the area subject to flooding by the 1% annual chance flood, Amas of Special Flood Heard floods 2 from J. A.E. A.M., A.A. A.R. A.P. N. Y. A.D. V. The Base Flood Beatlot is the water-sufface Beatlot of the 1% entire chance flood in the 1% entire chance flood.

ZONE A No Base Hood Elevations determined.

ZONE AE Base Froot Behalising determined

ZONE AF OF OR OBJAIN OF 1 to 3 feet (usually areas of ponding); Base Floot Elevations determined

Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocibes also determined.

Special Proof Herard Area Formerly protected from the 1% annual chance food by a flood control system that was subsequently decertified. Zone AR indicates that the former food control system is being restored to provide protection from the 1% annual chance or greater flood.

Area to be protected from 1% annual chance flood by a Federal flood protection system under construction: no Base Fixed Elevations determined.

Coasta flood zone with velocity hazard (wave action); no Base Flood Bevalions determined Coasta floor zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream prissing acjecent floodplain areas that must be kept free of encrosomment so that the 1% annual chance flood can be carried without substantial increases in flood beights.

OTHER FLOOD AREAS

Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less then 1 floot or with drawage areas less than 1 square male; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE D Areas in which flood nazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally rocated within or adjacent to Special Pood (

Floodplain boundary Zone D Boundary

CBRS and OPA boundar

Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, Flood depths or flood velocities

— 513 → Base Flood Elevation line and value; elevation in feet\* (EL 987) Base Flood Elevation value where uniform within zone; elevation in feet\*

\* Referenced to the North American Vertical Datum of 1988 (NAVID 88)

(<u>A</u>)——(<u>A</u>)

97 07 30.00° 32 27 30.00°

1000-meter Universal Transverse Mercator grid ticks, zone 13

5000-foot gnd toks: Colorado State Plane co-system, central zone (FIPSZONE 0502). Lampert Conformat Conk, Projection.

DX5510

\_ M1.5

MAP REPOS TORIES

Refer to Map Repositories I st on Map Index

EFFECTIVE DATE(S) DF REVISION(S) TO THIS PANEL DECEMBER 7, 2016 - to update corporate 1-the, to change Base Flood Elevations and Sceoul Popol Mazzan Aires, to update near Dimera; is add neads such road names, and to scorporate previously sessed tables of Map Revision.

For community map revision Existory poor to countywide mapping, refer to the Community Map History Table located in the Floor Insurance Study report for this jurisdiction.

To determine if flood insurance is evaluable in this community, contact your insurance agent or call the National Flood Insurance Program at 2-800-638-6620.

MAP SCALE 1" = 1000" METERS

PANEL 0535G

FLOOD INSURANCE RATE MAP EL PASO COUNTY, COLORADO

PANEL 535 OF 1300 (SEE MAP INDEX FOR FIRM PANEL LAYOUT

AND INCORPORATED AREAS

COMMUNITY NUMBER PANEL SUFFIX

COLORADO SPRINOS, CITY OF DIAGRE 6255 E. MASO COUNTY 86056 6535

NATIONALIEKOODHINSK



MAP REVISED **DECEMBER 7, 2018** Federal Emergency Management Agency Page 1 of 4 Issue Date: March 6, 2009 Effective Date: July 23, 2009 Case No.: 08-08-0541P LOMR-APP



# Federal Emergency Management Agency

Washington, D.C. 20472

# LETTER OF MAP REVISION **DETERMINATION DOCUMENT**

	COMMUNITY AND REVISION	INFORMATION	PROJECT DESCRIPTION	BASIS OF REQUEST				
COMMUNITY	Co	so County blorado porated Areas)	NO PROJECT	HYDRAULIC ANALYSIS NEW TOPOGRAPHIC DATA				
COMMUNITY NO.: 080059  Sand Creek Letter of Map Revision.								
IDENTIFIER	Sand Creek Letter of Map Re Mustang Place to Arroya Lar		APPROXIMATE LATITUDE & LONGITUDE: 38.971, -104.668 SOURCE: USGS QUADRANGLE DATUM: NAD 27					
ANNOTATED MAPPING ENCLOSURES			ANNOTATED STUDY ENCLOSURES					
TYPE: FIRM*	NO.: 08041C0535 F	DATE: March 17, 1997	DATE OF EFFECTIVE FLOOD INSU PROFILE(S): 204P(a), 204P(b), 20 FLOODWAY DATA TABLE: 5					

# FLOODING SOURCE(S) & REVISED REACH(ES)

Sand Creek - from approximately 360 feet downstream of Mustang Place to just downstream of Arroya Lane

	SUMMARY OF REV	ISIONS		
Flooding Source	Effective Flooding	Revised Flooding	Increases	Decreases
Sand Creek	Zone A	Zone AE	YES	YES
	No BFEs*	BFEs	YES	NONE
	No Floodway	Floodway	YES	NONE

BFEs - Base Flood Elevations

#### DETERMINATION

This document provides the determination from the Department of Homeland Security's Federal Emergency Management Agency (FEMA) regarding a request for a Letter of Map Revision (LOMR) for the area described above. Using the information submitted, we have determined that a revision to the flood hazards depicted in the Flood Insurance Study (FIS) report and/or National Flood Insurance Program (NFIP) map is warranted. This document revises the effective NFIP map, as indicated in the attached documentation. Please use the enclosed annotated map panels revised by this LOMR for floodplain management purposes and for all flood insurance policies and renewals in your community.

This determination is based on the flood data presently available. The enclosed documents provide additional information regarding this determination. If you have any questions about this document, please contact the FEMA Map Assistance Center toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMR Depot, 3601 Eisenhower Avenue, Alexandria, VA 22304. Additional Information about the NFIP is available on our website at http://www.fema.gov/nfip.

> David N. Bascom, Program Specialist Engineering Management Branch Mitigation Directorate

Enclosures reflect changes to flooding sources affected by this revision. \* FIRM - Flood Insurance Rate Map; \*\* FBFM - Flood Boundary and Floodway Map; \*\*\* FHBM - Flood Hazard Boundary Map



# Federal Emergency Management Agency

Washington, D.C. 20472

# LETTER OF MAP REVISION DETERMINATION DOCUMENT (CONTINUED)

# **COMMUNITY INFORMATION**

#### APPLICABLE NFIP REGULATIONS/COMMUNITY OBLIGATION

We have made this determination pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (P.L. 93-234) and in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, P.L. 90-448), 42 U.S.C. 4001-4128, and 44 CFR Part 65. Pursuant to Section 1361 of the National Flood Insurance Act of 1968, as amended, communities participating in the NFIP are required to adopt and enforce floodplain management regulations that meet or exceed NFIP criteria. These criteria, including adoption of the FIS report and FIRM, and the modifications made by this LOMR, are the minimum requirements for continued NFIP participation and do not supersede more stringent State/Commonwealth or local requirements to which the regulations apply.

We provide the floodway designation to your community as a tool to regulate floodplain development. Therefore, the floodway revision we have described in this letter, while acceptable to us, must also be acceptable to your community and adopted by appropriate community action, as specified in Paragraph 60.3(d) of the NFIP regulations.

#### **COMMUNITY REMINDERS**

We based this determination on the 1-percent-annual-chance flood discharges computed in the FIS for your community without considering subsequent changes in watershed characteristics that could increase flood discharges. Future development of projects upstream could cause increased flood discharges, which could cause increased flood hazards. A comprehensive restudy of your community's flood hazards would consider the cumulative effects of development on flood discharges subsequent to the publication of the FIS report for your community and could, therefore, establish greater flood hazards in this area.

Your community must regulate all proposed floodplain development and ensure that permits required by Federal and/or State/Commonwealth law have been obtained. State/Commonwealth or community officials, based on knowledge of local conditions and in the interest of safety, may set higher standards for construction or may limit development in floodplain areas. If your State/Commonwealth or community has adopted more restrictive or comprehensive floodplain management criteria, those criteria take precedence over the minimum NFIP requirements.

We will not print and distribute this LOMR to primary users, such as local insurance agents or mortgage lenders; instead, the community will serve as a repository for the new data. We encourage you to disseminate the information in this LOMR by preparing a news release for publication in your community's newspaper that describes the revision and explains how your community will provide the data and help interpret the NFIP maps. In that way, interested persons, such as property owners, insurance agents, and mortgage lenders, can benefit from the information.

This determination is based on the flood data presently available. The enclosed documents provide additional information regarding this determination. If you have any questions about this document, please contact the FEMA Map Assistance Center toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMR Depot, 3601 Eisenhower Avenue, Alexandria, VA 22304. Additional Information about the NFIP is available on our website at http://www.fema.gov/nfip.

David N. Bascom, Program Specialist Engineering Management Branch Mitigation Directorate

112553 10.3.1.08080541

102-I-A-C



# Federal Emergency Management Agency

Washington, D.C. 20472

# LETTER OF MAP REVISION DETERMINATION DOCUMENT (CONTINUED)

We have designated a Consultation Coordination Officer (CCO) to assist your community. The CCO will be the primary liaison between your community and FEMA. For information regarding your CCO, please contact:

Ms. Jeanine D. Petterson
Director, Mitigation Division
Federal Emergency Management Agency, Region VIII
Denver Federal Center, Building 710
P.O. Box 25267
Denver, CO 80225-0267
(303) 235-4830

#### STATUS OF THE COMMUNITY NFIP MAPS

We will not physically revise and republish the FIRM and FIS report for your community to reflect the modifications made by this LOMR at this time. When changes to the previously cited FIRM panel(s) and FIS report warrant physical revision and republication in the future, we will incorporate the modifications made by this LOMR at that time.

This determination is based on the flood data presently available. The enclosed documents provide additional information regarding this determination. If you have any questions about this document, please contact the FEMA Map Assistance Center toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMR Depot, 3601 Eisenhower Avenue, Alexandria, VA 22304. Additional Information about the NFIP is available on our website at http://www.fema.gov/nfip.

David N. Bascom, Program Specialist Engineering Management Branch Mitigation Directorate

112553 10.3.1.08080541

102-I-A-C



# Federal Emergency Management Agency

Washington, D.C. 20472

# LETTER OF MAP REVISION DETERMINATION DOCUMENT (CONTINUED)

# **PUBLIC NOTIFICATION OF REVISION**

#### **PUBLIC NOTIFICATION**

FLOODING SOURCE	LOCATION OF REFERENCED ELEVATION	BFE (FEET	NGVD 29)	MAP PANEL
		EFFECTIVE	REVISED	NUMBER(S)
Sand Creek	Just upstream of Mustang Place	None	6,984	08041C0535 F
	Just downstream of Arroya Lane	None	7,238	08041C0535 F

Within 90 days of the second publication in the local newspaper, a citizen may request that we reconsider this determination. Any request for reconsideration must be based on scientific or technical data. Therefore, this letter will be effective only after the 90-day appeal period has elapsed and we have resolved any appeals that we receive during this appeal period. Until this LOMR is effective, the revised BFEs presented in this LOMR may be changed.

A notice of changes will be published in the *Federal Register*. A short notice also will be published in your local newspaper on or about the dates listed below. Please refer to FEMA's website at https://www.floodmaps.fema.gov/fhm/Scripts/bfe\_main.asp for a more detailed description of proposed BFE changes, which will be posted within a week of the date of this letter.

LOCAL NEWSPAPER

Name: El Paso County News

Dates: 03/18/09 03/25/09

This determination is based on the flood data presently available. The enclosed documents provide additional information regarding this determination. If you have any questions about this document, please contact the FEMA Map Assistance Center toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMR Depot, 3601 Eisenhower Avenue, Alexandria, VA 22304. Additional Information about the NFIP is available on our website at http://www.fema.gov/nfip.

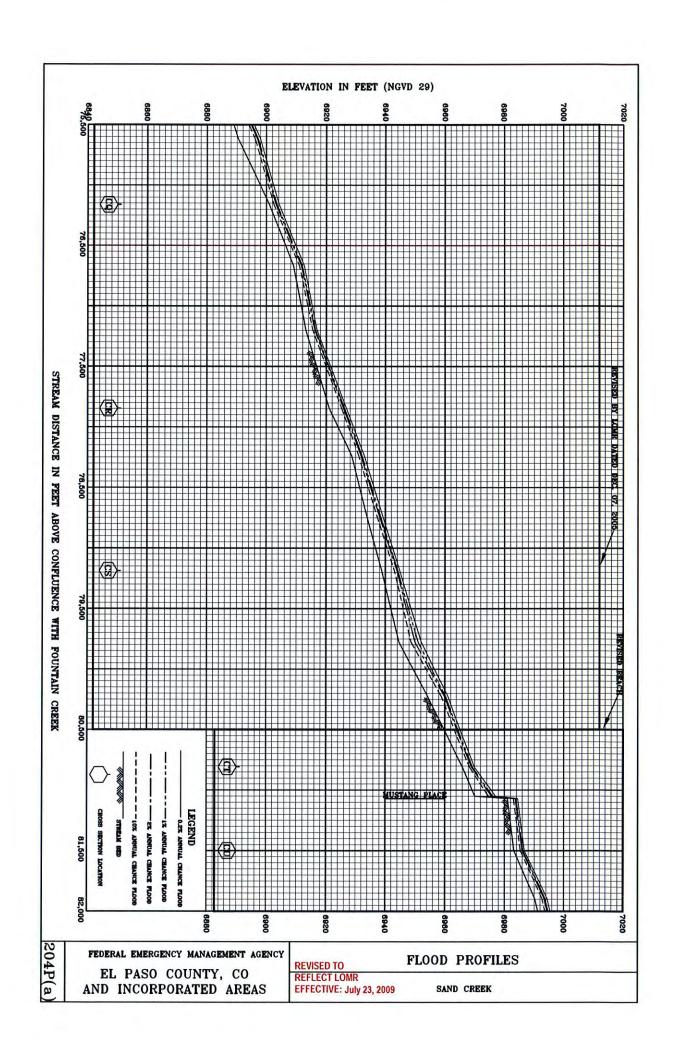
David N. Bascom, Program Specialist Engineering Management Branch Mitigation Directorate

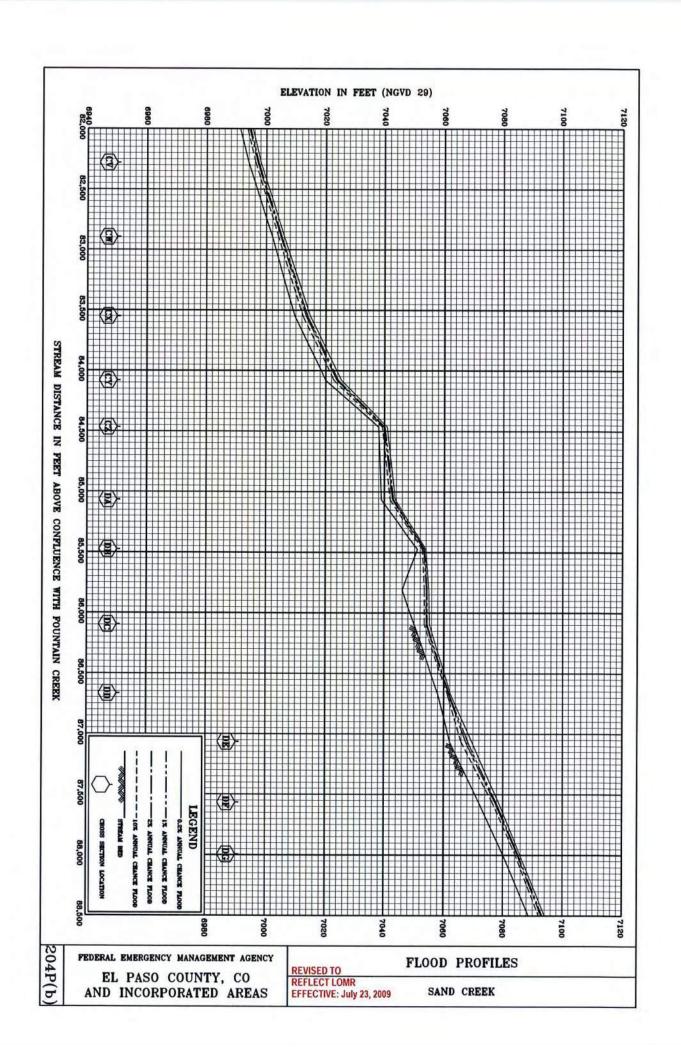
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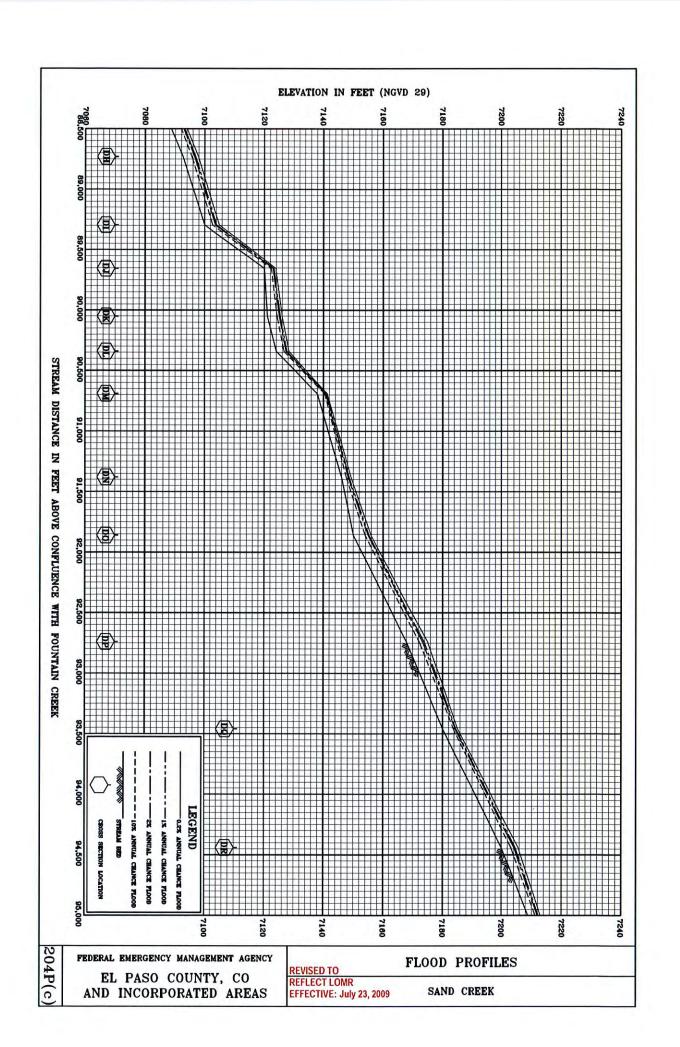
102-I-A-C

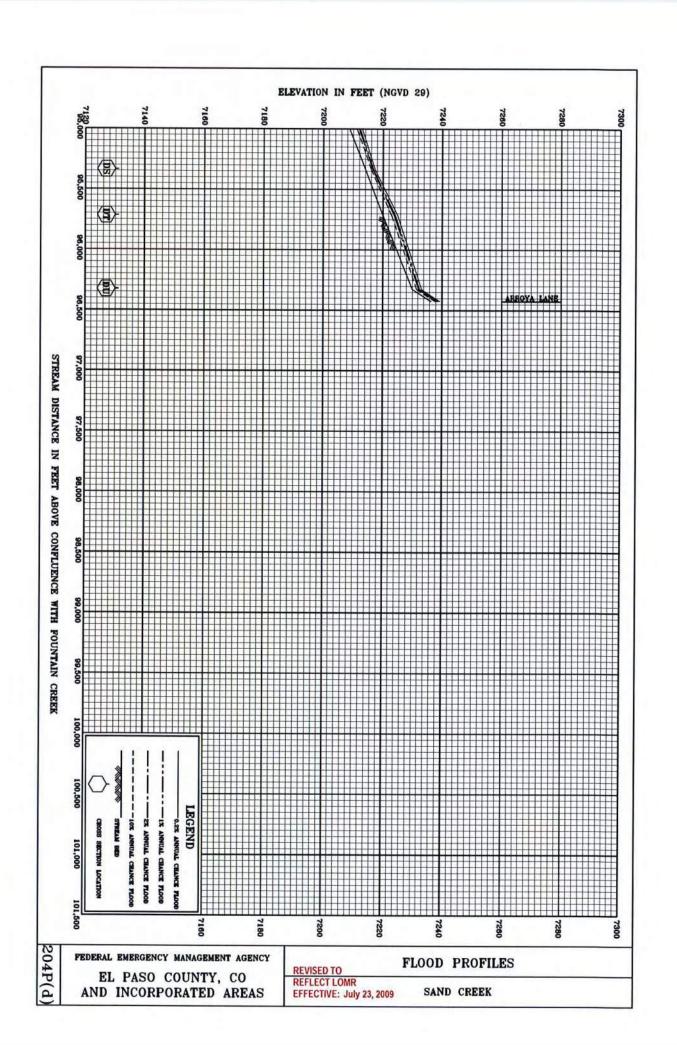
reek  'd)  65,292  66,092  41  66,247  90  67,647  50  68,297  69,147  50  70,157  70,577  205  70,577  205  70,807  70,807  71,162  71,162  73,644  73,644  73,644  75,142	SECTION AREA (SQUARE FEET)	ATTICAL TO		LICHLIM	HILLM	
65,292 164 66,092 41 66,247 90 67,647 50 68,297 65 69,147 50 70,157 50 70,577 205 70,627 180 70,727 210 70,807 195 71,162 90 71,977 226 73,052 174	N	VELOCITY (FEET PER SECOND)	REGULATORY	FLOODWAY FEET	MAY FLOODWAY FEET (NGVD)	
65,292 164 66,092 41 66,247 90 67,647 50 68,297 65 69,147 50 70,157 50 70,577 205 70,627 180 70,627 180 70,727 210 70,807 195 71,162 90 71,977 226 71,977 226 71,977 226 71,977 226 73,644 237	N					
66,092 41 66,247 90 67,647 50 68,297 65 69,147 50 70,157 50 70,577 205 70,627 180 70,807 195 71,162 90 71,977 226 71,977 226 73,052 174 73,052 174		6.1	6,748.7	6,748.7	6,749.4	
66,247 90 67,647 50 68,297 65 69,147 50 70,157 50 70,577 205 70,627 180 70,727 205 70,807 195 71,162 90 71,162 90 71,977 226 71,977 226 71,977 226 73,052 174	223	11.7	6,761.2	6,761.2	6,762.2	
67,647 50 68,297 65 69,147 50 70,157 50 70,627 180 70,727 205 70,727 210 71,162 90 71,162 90 71,977 226 71,977 226 71,977 226 73,052 174 73,052 174	270	9.6	6,773.6	6,773.6	6,773.7	
68,297 65 69,147 50 70,157 50 70,627 205 70,627 210 70,807 195 71,162 90 71,977 226 71,977 226 71,977 226 71,977 226 71,977 226 73,052 174	218	11.9	6,782.6	6,782.6	6,783.3	
69,147 50 70,157 50 70,577 205 70,627 180 70,727 210 70,807 195 71,162 90 71,977 226 71,977 226 73,052 174 73,052 174	284	8.8	6,793.9	6,793.9	6,794.4	0
CG 70,157 50 CH 70,577 205 CI 70,627 180 CJ 70,727 210 CK 70,807 195 CL 71,162 90 CM 71,977 226 CM 71,977 226 CM 73,052 174 CO 73,644 237 CP 75,142 172	213	11.7	6,804.5	6,804.5	6,804.5	0
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70,627 180 70,727 210 70,807 195 71,162 90 71,977 226 73,052 174 73,052 174 73,644 237 75,142 172	347	7.2	6,823.9	6,823.9	6,824.5	0
70,727 210 70,807 195 71,162 90 71,977 226 73,052 174 73,644 237 75,142 172	267	4.6	6,826.7	-	6,827.7	1
70,807 195 71,162 90 71,977 226 73,052 174 73,644 237 75,142 172	340	7.3	6,831.1	•	831	0
71,162 90 71,977 226 73,052 174 73,644 237 75,142 172	334	7.5	6,832.5		832	0.0
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73,052 174 73,644 237 75,142 172	503	5.2	6,847.4	6,847.4	6,848.3	0.9
73,644 237 75,142 172	328	7.9	6,861.1	6,861.1	6,861.2	0
75,142 172	364	7.1	6,870.2	6,870.2	6,870.2	0
	324	8.0	6,888.5	6,888.5	6,888.7	0
76,161 109	283	9.2	6,903.5		6,903.7	0.2
77,846 100	272	9.6	6,926.1	926	6,926.7	0
187 117	287	9.1	944.	,944	944	0
80,808 142	310	8.4	6,969.2	6,969.2	6,969.2	0
CU 81,501 120 3	342	7.6	6,986.1	6,986.1	6,986.5	0
CV 82,281 124 2	295	8.8	6,997.4	6,997.4	6,997.4	0.0
CW 82,897 64 2	3	11.0	7,005.3	7,005.3	7,006.1	0.8
83,517 90	266	8.6	7,013.9	7,013.9	7,013.9	0.0
84,087 70	244	10.7	7,024.3	7,024.3	7,024.3	0.0
,473 160	322	8.1		7,040.2	0	0
		REVISED	D T0			
		REFLE(	REFLECT LOMR			
Feet Above Confluence With Fountain Creek	>	EFFECT	EFFECTIVE: July 23, 2009	600		
FEDERAL EMERGENCY MANAGEMENT AGENCY			H	FLOODWAY DATA	DATA	
AND INCORPORATED AREAS				SAND CREEK	    *	

FLOODING S	CROSS SECTION	Sand Creek	(cont'd)	DA	DB	DC	DD	DE	DF	DG	DH	DI	DC	DK	DL	DM	ON	00	DP	ŏ O	DR	DS	DI	ממ		l Feet Above Confluence	FEDERAL EME	AND IN
SOURCE	DISTANCE <sup>1</sup>			85,073	85,483	86,103	86,673	87,073	87,573	-	88,738	,30	89,663	90,058	90,348	869'06	91,388	91,868	7	93,468	44	95,343	5,72	96,333		With	FEDERAL EMERGENCY MANAGEMENT AGENCY	EL PASO COUNIT, CO AND INCORPORATED AREAS
	WIDTH (FEET)			139	170	100	197	83	86	135	89	74	143	140	102	300	120	105	65	117	81	100	77	06		Fountain	EMENT AGEN	r, co
FLOODWAY	SECTION AREA (SQUARE FEET)			456	328	274	434	270	325	304	263	249	309	426	276	398	292	313	239	288	260	274	252	266		Creek	ICY	
	MEAN VELOCITY (FEET PER SECOND)			5.7	7.9			9.6	8.0	8.6	6.6	10.4	8.4	6.1	9.4	6.5	8.9	8.3	10.9	0.6	10.0	9.5	10.3	8.6	REVISED TO	EFFECTIVE: July 23, 2009		
×	REGULATORY			7,043.0	7,053.4	7,054.4	7,061.7	990	7,077.7	7,085.1	7,096.9	7,104.1	7,123.2	7,125.1	7,127.6	7,141.0	7,148.5	7,155.2	7,173.8	7,184.6	7,204.5	7,216.8	7,224.2	7,232.5		7 23, 2009	료	
BASE FI WATER SURFACE	WITHOUT FLOODWAY FEET			7,043.0	7,053.4	054	190	990	077	7,085.1	7,096.9	7,104.1	7,123.2	7,125.1	7,127.6	7,141.0	7,148.5	7,155.2	7,173.8	7,184.6	7,204.5	7,216.8	7,224.2	7,232.5			FLOODWAY DATA	SAND CREEK
FLOOD CE ELEVATION	NOT WITH WAY FLOODWAY FEET (NGVD)			7,043.1	053.		062.	99	777	985	7,096.9	7,104.3	123	12	127	7,141.0	148	155	7,173.8	7,184.6	7,204.6	217	7,224.3	7,233.0			DATA	<u> </u>
7	INCREASE			0.1		0.0	0.3	0.1	0.2	0.0	0.0	0.2	0.0	0.1			0.1		0.0			4.0	0.1	0.5				









RECOMMENDATIONS PER SAND CREEK DBPS



8061-20608 Colorado Springs, Colorado PRELIMINARY DESIGN PLANS 419 W. Bijou Street SAND CREEK DRAINAGE STUDY 55 Kiowa Engineering Corporation IS A MASTER PLANNING SHEET PRELIMINARY AND CONCEPTUAL IT SHOULD NOT BE USED FOR PURPOSES. SELECTIVE RIPRAP L GRADE CONTROL CHANNEL IMPROVEMENTS TO SENSE WITH THE EXISTING FLOODPLIN ON THE LOW FLOW FLOW FOR SOFT SHALL BE PRESENTED WITH THE SOFT SHALL BE PRESENTED WITH THE SHALL BE SOFT SHALL BE SOFT SHALL BE SOFT SHALL BE SOFT SHALL SHALL BE SOFT SHALL SHALL BE SOFT SHALL SHALL BE SOFT SHALL SH 2170 THIS DRAWING IS REPRESENTING P ENGINEERING. I STA 933 + 80 56 MATCH SHT SHEET 54 STA 905 +00

# VI. DEVELOPMENT OF ALTERNATIVES AND RECOMMENDED PLAN

The concepts which are available for handling stormwater mnoff within the Sand Creek basin have been presented and discussed in detail in the Sand Creek Drainage Basin Planning Study Development of Alternatives Report and the draft East Fork Sand Creek Drainage Basin Planning Study. The process of combining the various channel treatment options, detention schemes and roadway crossing structures into a contiguous plan for all of the reaches is presented in this chapter of the report. As a result of the evaluation of the flood control, environmental, open space, operations and maintenance, and implementation concerns within the Sand Creek basin, the following concepts were identified as having sufficient feasibility to warrant further evaluation and review:

Channel Concepts:

Floodplain Preservation Channelization, 10- or 100-year Selective Improvements

Detention: Regional detention systems

Channel Concepts: The channel concepts listed above have been evaluated with respect to the parameters listed in the previous chapter. A concept's feasibility depends upon its impact, positive or negative, upon the evaluation parameters. The floodplain preservation concept would involve the regulation of the floodplain preservation concept would involve the regulation of the floodplain limits, generally as depicted on the effective City of Colorado Springs and El Paso County Flood Insurance Rate Maps. Regulation of the floodplain so that future encroachments are minimized and the floodprooffing of structures which are currently within the 100-year floodplain would presumably be the methods used to address the flood hazard concerns along Sand Creek. In the upper reaches of Sand Creek, the ownership or easements associated with the 100-year floodplain (or greater limits to allow for an erosion buffer zone) would be a primary issue in regards to implementation of such a concept. Detention in the upper reaches of the basin Sand Creek basin and in the East Fork Sand Creek basin will maintain the 100-year floodplain at existing limits within the lower reaches of Sand Creek. The "do-nothing" concept is feasible wherever

the existing drainageway improvements are of adequate capacity to convey flood flows. Channelization would involve the lining of the Creek into a more confined flow area, and could be done for either the 100-year or 10-year flood discharges. Several typical channel concepts have been presented. The primary bank lining material would probably be riprap. Grade control and/or drop structures would be required in a channelization concept so that the flood velocities could be controlled to a level requiring medium to heavy riprap. Soil cement offers an alternative to riprap and concrete for the construction of drops or grade control structures. Revegetation would occur wherever the native vegetation was disturbed by the channel construction. Willows at the toe of the riprap banks would be a minimum replacement. Selective linings would involve the construction of grade controls, drop structures, bank linings, storm sewer outlet control structures selectively sited to resist stream erosion or to reduce potential flooding damages. Areas of future concern such as at the outside bends of the creek, or at the outlets of bridges or culverts which will cross the drainageway would be subject to selective improvements.

versus regional detention. During the evaluation process, it was determined that the onsite detention concept has a low feasibility relative to a regional concept. This is because, (1) onsite detention has a unpredictable impact upon lowering peak discharges from urbanized areas to historic conditions (reference, Urbonas and Glidden, "Effect of Detention on Flows in Major Drainageways" ASCE Water Forum '81, 1981), (2) an onsite concept has little impact upon maintaining or enhancing water quality, (3) the number of onsite detention basins, their locations and size cannot be accurately determined in the undeveloped portions of the basin at this time, and (4) onsite detention would present a substantial maintenance responsibility to the jurisdictions involved. For these reasons the onsite detention concept was eliminated and regional detention basin concepts were developed. In the analysis of the channel concepts, regional detention facilities were assumed to be in place.

# Channel Alternatives

Presented on Table VI-1 is a marrix of channel alternatives which were evaluated. All reaches of Sand Creek and the East Fork of Sand Creek had at least three alternatives analyzed. Presented on Tables VI-2 through VI-6 are comparative evaluations of the floodplain preservation (do-nothing), channelization and selective lining concepts, for the mainstem Sand Creek basin, by reach. The purpose of the evaluation process was to identify the relative advantages and disadvantages of each concept within each reach.

100-year peak discharge to levels. This will allow for the channel improvements to be constructed within the existing right-of way.

Reaches SC-5 and SC-6: A selective channel improvement concept has been recommended for these reaches. Detention in Reach SC-8 of the basin will maintain flows to historic peak discharge levels, however the low flows will increase in frequency and volume. For this reason it has been recommended to provide riprap channel linings at selective locations to at least the 10-year water surface and install grade controls. This will prevent the long-term degradation of the invert. A residual 100-year floodplain will remain and will offer opportunities for habitat replacement and open space preservation. Land adjacent to the drainageway is currently undeveloped or unplatted at this time which makes the feasibility of implementing this concept greater in comparison to the urbanized reaches of the creek.

Reaches SC-7 and SC-8: A selective improvement concept involving the localized lining of channel banks and grade control construction has been recommended for these reaches. The feasibility of this concept stems from the fact that flows will be treduced because of detention. Numerous individual rural ownerships cross the drainageway, however no habitable structures lie within the 100-year floodplain. Because of this, the economic feasibility of channelization concepts is low. Non-structural measures can be used to limit encroachments into floodprone areas. Additionally, the City of Colorado Springs Comprehensive plan recommends that the floodplains be maintained as open space. Potential habitat disturbances can be avoided with a selective plan, or simply replaced as part of the particular construction activity which caused the disturbance.

Reach SC-9: A floodplain preservation concept has been recommended for this reach. Little increase in urbanization is anticipated in this reach, and for this reason the existing drainageway is expected to remain stable. Localized improvements may be necessary to limit erosion caused by flow concentrations at culverts or storm sewers. Private ownership of the drainageway is anticipated to continue which lower the feasibility of channel concepts which require permanent right-of-ways or easements for construction and maintenance.

Reaches WF-1 through WF-3: A 100-year channel concept has been recommended for these reaches primarily because of the potential for flooding damages. Several roadway crossings are in need of replacement because of the flood hazard the constrictions create. Some open space enhancement potential exists for this concept since these reaches have been degraded visually by debris accumulation, bank sloughing and sedimentation. Little opportunity exists for widening the drainageway because the

# Development of the Recommended Plan

Presented on Table VI-7 is a matrix representing the recommended plan for each major drainageway reach. The selection of a recommended channel treatment scheme has been based upon the qualitative and quantitative information presented in the Sand Creek Drainage Basin Planning Study Development of Alternatives report and the draft East Fork Sand Creek Drainage Basin Planning Study. Contained within the Technical Addendum to the Sand Creek Drainage Basin Planning Study Development of Alternatives report, is the alternative hydrologic, hydraulic and conceptual cost data used in the evaluation and comparison of each of the alternatives within the mainstem Sand Creek basin.

# Discussion of Recommended Plan

The recommendation of a particular channel treatment or detention scheme has been based upon the qualitative and quantitative data presented. For each reach the flood hazard, environmental, cost, operations and maintenance and open space aspects of the drainageway were weighed for each alternative concept.

Reach SC-I: For this reach a 10-year channel section was recommended for further evaluation. With the implementation of regional detention in the upper basin, the 100-year floodplain will generally be confined within the existing banks, excepting at roadway crossings lacking 100-year capacity. It is recommended that a 10-year low flow channel be constructed within the invert of the existing channel through the construction of benches and sand bars. As urbanization continues towards the full development scenario, the base flow and annual flows will increase in volume and frequency. For this reason, the low flow area must be stabilized to protect the existing channel banks from undermining and subsequent bank sloughing. The benched areas offer an opportunity for habitat replacement and enhancement. At some locations within this reach, a residual 100-year floodplain will remain which will have to be regulated. The residual 100-year floodplain offers some potential for open space preservation and enhancement. This is particularly true in the portion of the reach downstream of Hancock Expressway.

Reaches SC-2 through SC-4: A 100-year channel concept has been recommended primarily because of the potential for flooding damages which exists in these reaches. Habitat disturbed by the construction of channel linings and grade control structures could be replaced along the channel toes and on the overbanks. The replacement of the Waynoka Road crossing will reduce the potential for flood damages in areas adjacent to these roadways. The detention within the upper reaches will limit the

The results of the preliminary design analysis are summarized in this section. The alternative improvements have been quantitatively and qualitatively evaluated, and presented to the City of Colorado Springs and other interested agencies and individuals. Field review of specific areas of concern have been conducted in order to refine the channel treatments suggested for use along Sand Creek, East Fork Sand Creek and their major tributaries. The preliminary plan for the recommended alternative is shown on the drawings contained at the rear of this report.

# Criteria

The City of Colorado Springs, El Paso County Drainage Criteria Manual was used in the development of the typical sections and plans for the major drainageways within the Basin. The City/County manual was supplemented by various criteria manuals with more specific application. These were:

- "Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soils," prepared by Simons, Li & Associates, Inc., 1981.
- Urban Storm Drainage Criteria Manual, Volumes I, II, and III, prepared by the Urban Drainage and Flood Control District.

Various design plans for roadway and channel improvement projects, either proposed or already constructed were reviewed in order to prepare the preliminary design plans. Specifically, the project design plans for the Las Vegas Street and Galley Road bridge replacement projects were reviewed and the improvements incorporated in the preliminary design. The proposed Sand Creek Stabilization Project, AT&SF Railroad to Hancock Expressway and the proposed Sand Creek Stabilization Project at Fountain Boulevard design plans have been reviewed and incorporated into the preliminary design plan and profiles.

# Ivdrology

Presented on Table VII-1 is selected hydrologic data to be used for the sizing of major drainageway improvements within the Basin. Peak flow rates for the 10- and 100-year frequency incorporating and the selected detention alternatives for the Sand Creek and East Fork Sand Creek Basin are summarized for key points along the major drainageways.

Contained within the The technical addenda of this report contains a complete listing of peak discharges for all the sub-basins, stream segments and design points shown on Exhibit 1.

The sizing the drainageway improvements for the tributaries will need to be verified during the final design and layout of the proposed drainageway facilities. Land development activities may alter the location of design points along the tributaries, and therefore slight alteration in a sub-basin's length, slope and area may occur. The methods outlined in the City/County Drainage Criteria Manual should be applied during final design analysis. The rational method should be used to check the peak flow rates for all tributary drainageways and storm sewers draining areas less than 100 acres in size.

# hannels

The recommended channel sections for each reach of drainageway has been outlined in Section VI of this report. In general, the banks of Sand Creek channel, from the confluence with Fountain Creek to the proposed Sand Creek Detention Basin No. 2 are to be lined, or in some cases relined, with riprap to either a 10-year or 100-year flow depth, as shown on the preliminary design plans. Above the Sand Creek Detention Basin No. 2, selectively located riprap bank protection, such as at outside bends, at bridge or culvert outlets, and at confluences with side tributaries have been recommended. In conjunction with the selective improvement measures, and the 10-year low flow concept, the 100-year floodplain should be preserved and regulated. Wherever existing bank linings were judged to be adequate, no improvements have been recommended at this time.

For the West Fork Sand Creek, 100-year riprap bank linings have been recommended in order to address the 100-year flooding hazard which exists at numerous locations along the West Fork. The final design improvements shown in the Palmer Park Bridge Replacement project drawings have been incorporated into the preliminary design plans. In the uppermost reaches of the West Fork, a short segment of rectangular concrete channel has been recommended because of right-of-way constraints.

For the Center Tributary of Sand Creek, 100-year riprap lined channels have been recommended from the confluence with East Fork to Platte Avenue. Above Platte Avenue, the existing concrete channels have adequate capacity except where the drainageway channel has yet to be improved. The final design plans for the US 24 Bypass Project, Phase II have been incorporated into the plans. As part of the bypass construction, it is proposed to line the Center Tributary using riprap. The location of the proposed roadway, new crossings, drops and channel as shown on the Phase II Bypass plans have been reflected on the preliminary design drawings.

For the East Fork Sand Creek drainageway, riprap lined channel banks have been recommended for the majority of the reaches. This is mainly because of the high level of development predicted for the basin in the area known as the Banning-Lewis Ranch development. Open space to accommodate the 100-year floodplains should be allowed for as the East Fork Sand Creek drainageways develop. This is consistent with the Banning-Lewis Ranch master development plan which was approved at the time of annexation of this property. Above Woodmen Road, selective channel lining improvements and grade control structures have been recommended.

For the most part the side tributaries have been recommended to be lined with riprap, however there are some locations in the upper basin which have been proposed to be grasslined. The location of the side drainageways should be considered approximate and may very likely be modified in the future because of land development.

The primary criteria used when sizing the proposed channel sections has been velocity. For all riprap lined channels, the average design velocity should be no greater than 9 feet per second. This criteria allows for the use of Type H riprap within the main flow area of the drainageway. For the case of a 10-year channel with an overall floodplain section, limiting the main channel velocity to 9 feet per second will result in overbank velocities in the five feet per second range. At this level of overbank velocity, native vegetation will be able to withstand the crosive forces which might result in a 100-year flow event. Velocities approaching 10 feet per second could occur at constrictions such as at roadway crossings and at culvert outlets.

# Drop Structures and Check Structures

Drop and check structures have been sited along Sand Creek in order to slow the channel velocity to the recommended 7 feet per second, and to prevent localized and long-term stream degradation from affecting channel linings and overbanks. In the reaches to be selectively lined, drops and check structures will protect the native vegetation from the detrimental effects of stream invert headcutting. Several types of structures could be considered for the Sand Creek Basin. For channel bottom widths in excess of fifty feet, soil cement or sheet piling drops/checks are feasible. For channels narrower than this, reinforced concrete structures are probably the best alternative. A maximum drop height of three feet is recommended. The methodology recommended for use when designing vertical structures is contained with Volume II of the Urban Storm Drainage Criteria Manual.

# Detention

The recommended plan calls for the construction of six regional detention basins within the Sand Creek basin, and six regional basins within the East Fork Sand Creek basin. The

purpose of the Sand Creek detention basins is to limit peak discharges at Powers Boulevard to existing development condition levels. The detention basins in the upper portions of the Sand Creek basin will keep the majority of the existing channel sections and bridges below Powers Boulevard with adequate flow capacity in the future development condition. The detention basins within the East Fork Sand Creek basin have been sized to maintain the flow outfalling from the Banning-Lewis Ranch property at existing levels. This in turn will help to reduce flow to the mainstern of Sand Creek basin have been designed to accommodate the 100-year future condition volume without overtopping the overflow spillway. Sand Creek Basin Nos. 2 and 6, and East Fork Sand Creek Basin Nos. 1, 2, and 3 will be classified as jurisdictional structures, and their design and operation would be subject to State Engineer's office criteria. Sand Creek basins number 1 and 3 should be designed so as to take advantage of the adjacent roadway embankments, and therefore classifying as incidental storage and not subject State Engineer's regulations.

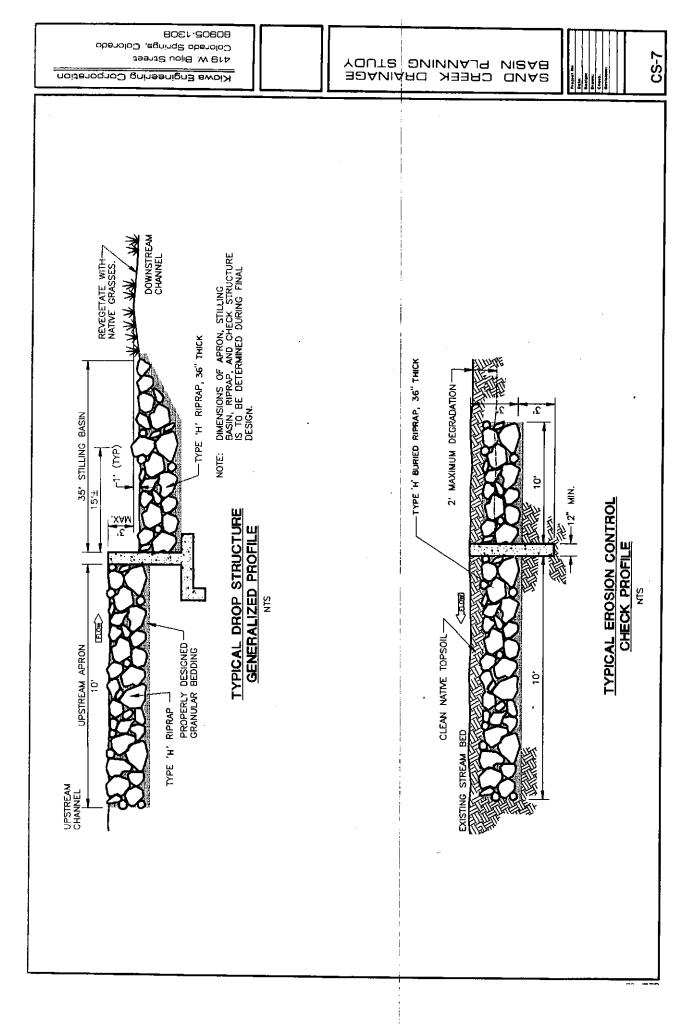
At Stetson Hills Boulevard, the roadway embankment has created a 2 acre open water wetland which was identified during the environmental review of the basin. It is recommended that this wetland be preserved. Accordingly, an outlet control structure will have to be constructed to pass the 100-year discharge to the downstream channel without overtopping the roadway. No floodwater storage or routing has been accounted for in the hydrology modelling at this roadway for the selected detention plan.

For the East Fork Sand Creek detention basin numbers 2, and 3, the existing embankment and outlet structure act to maintain a permanent pool at this time. It is recommended that the design of these detention basins be directed at maintaining the permanent pool when the flood control storage is to be added. The existence of a permanent pool may enhance the water quality aspects of these basins, and offer the opportunity of open space development conducive with open water.

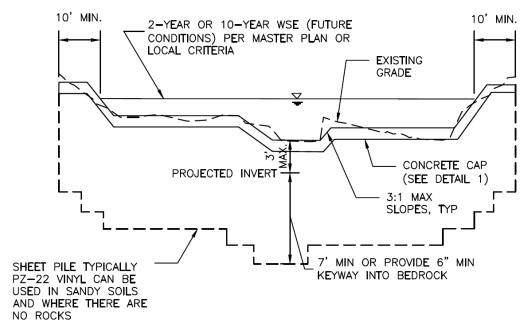
# Water Quality

Improvement of urban stormwater quality has become and important issue in drainage basin planning. Many pollutants are naturally associated with sediments that enter sensitive receiving waters. The pollutants are naturally occurring compounds that are carried to the drainageways in storm runoff. Other pollutants are the result of urbanization such as lawn chemicals, oil and grease, pet feces, lawn clippings and other items. Many pollutants can be limited by programs such as erosion control at construction sites, educational programs to inform the public as to the proper use of lawn chemicals, oil recycling programs and street sweeping programs. Even with these programs in place, erosion along the drainageways can generate large quantities of sediment that can settle out along the downstream channel bottoms.

8061.30608 Typical Channel Sections Colorado Springa, Colorado **CS-3** 419 W. Bijou Street BASIN PLANNING STUDY Kiowa Engineering Corporation Existing Vegetation to be Preserved and/or Replaced 18-inch Thick Type L Buried Riprap Toe Protection Floodplain Width varies 100-year W.S. Regulate Existing Floodplain & Limit Encroachments 2-foot Thick Type M Riprap with 12-inches of Native Sand Bedding at Outside Bends Natural Slope

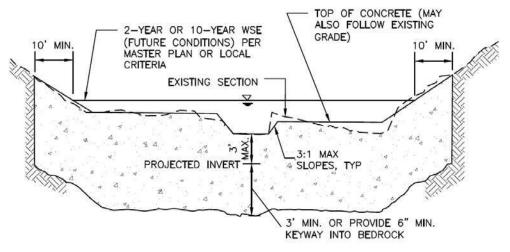


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NOTE: THE STRUCTURE MAY BE COVERED WITH 6" OF SOIL OUTSIDE OF THE LOW FLOW AREA.

# SHEET PILE CHECK



NOTES: 1. TRENCH IN UNDISTURBED SOIL. FORM TOP 6" OF CHECK. DO NOT OVER EXCAVATE TO FORM WALLS OR CONSTRUCT A FOOTING.

2. THE STRUCTURE MAY BE COVERED WITH 6" OF SOIL OUTSIDE OF THE LOW FLOW AREA.

3. VIBRATE CONCRETE INTO TRENCH.



Figure 9-27. Check structure details (Part 2 of 3)

Chapter 9 Hydraulic Structures

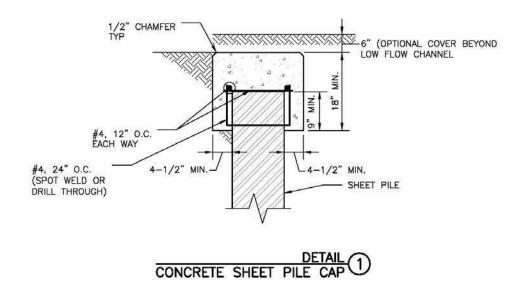


Figure 9-28. Check structure details (Part 3 of 3)

# **HYDROLOGIC CALCULATIONS**



For Colorado Springs and much of the Fountain Creek watershed, the 1-hour depths are fairly uniform and are summarized in Table 6-2. Depending on the location of the project, rainfall depths may be calculated using the described method and the NOAA Atlas maps shown in Figures 6-6 through 6-17.

Return	1-Hour	6-Hour	24-Hour
Period	Depth	Depth	Depth
2	1.19	1.70	2.10
5	1.50	2.10	2.70
10	1.75	2.40	3.20
25	2.00	2.90	3.60
50	2.25	3.20	4.20
100	2.52	3.50	4.60

Table 6-2. Rainfall Depths for Colorado Springs

Where Z = 6.840 ft/100

These depths can be applied to the design storms or converted to intensities (inches/hour) for the Rational Method as described below. However, as the basin area increases, it is unlikely that the reported point rainfalls will occur uniformly over the entire basin. To account for this characteristic of rain storms an adjustment factor, the Depth Area Reduction Factor (DARF) is applied. This adjustment to rainfall depth and its effect on design storms is also described below. The UDFCD UD-Rain spreadsheet, available on UDFCD's website, also provides tools to calculate point rainfall depths and Intensity-Duration-Frequency curves<sup>2</sup> and should produce similar depth calculation results.

# 2.2 Design Storms

Design storms are used as input into rainfall/runoff models and provide a representation of the typical temporal distribution of rainfall events when the creation or routing of runoff hydrographs is required. It has long been observed that rainstorms in the Front Range of Colorado tend to occur as either short-duration, high-intensity, localized, convective thunderstorms (cloud bursts) or longer-duration, lower-intensity, broader, frontal (general) storms. The significance of these two types of events is primarily determined by the size of the drainage basin being studied. Thunderstorms can create high rates of runoff within a relatively small area, quickly, but their influence may not be significant very far downstream. Frontal storms may not create high rates of runoff within smaller drainage basins due to their lower intensity, but tend to produce larger flood flows that can be hazardous over a broader area and extend further downstream.

■ Thunderstorms: Based on the extensive evaluation of rain storms completed in the Carlton study (Carlton 2011), it was determined that typical thunderstorms have a duration of about 2 hours. The study evaluated over 300,000 storm cells using gage-adjusted NEXRAD data, collected over a 14-year period (1994 to 2008). Storms lasting longer than 3 hours were rarely found. Therefore, the results of the Carlton study have been used to define the shorter duration design storms.

To determine the temporal distribution of thunderstorms, 22 gage-adjusted NEXRAD storm cells were studied in detail. Through a process described in a technical memorandum prepared by the City of Colorado Springs (City of Colorado Springs 2012), the results of this analysis were interpreted and normalized to the 1-hour rainfall depth to create the distribution shown in Table 6-3 with a 5 minute time interval for drainage basins up to 1 square mile in size. This distribution represents the rainfall

Table 6-6. Runoff Coefficients for Rational Method

(Source: UDFCD 2001)

Land Use or Surface	Percent						Runoff Co	efficients					
Characteristics	Impervious	2-у	ear	5-y	ear	10-1	/ear	25-	year	50-1	/ear	100-	γear
		HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D
Business													1100 000
Commercial Areas	95	0.79	0.80	0.81	0.82	0.83	0.84	0.85	0.87	0.87	0.88	0.88	0.89
Neighborhood Areas	70	0.45	0.49	0.49	0.53	0.53	0.57	0.58	0.62	0.60	0.65	0.62	0.68
Residential				_									
1/8 Acre or less	65	0.41	0.45	0.45	0.49	0.49	0.54	0.54	0.59	0.57	0.62	0.59	0.65
1/4 Acre	40	0.23	0.28	0.30	0.35	0.36	0.42	0.42	0.50	0.46	0.54	0.50	0.58
1/3 Acre	30	0.18	0.22	0.25	0.30	0.32	0.38	0.39	D.47	0.43	0.52	0.47	0.57
1/2 Acre	25	0.15	0.20	0.22	0.28	0.30	0.36	0.37	0:46	0.41	0.51	0.46	0.56
1 Acre	20	0.12	0.17	0.20	0.26	0.27	0.34	0.35	0.44	0.40	0.50	0.44	0.55
Industrial	<del> </del>				_							-	
Light Areas	80	0.57	0.60	0.59	0.63	0.63	0.66	0.66	0.70	0.68	0.72	0.70	0.74
Heavy Areas	90	0.71	0.73	0.73	0.75	0.75	0.77	0.78	0.80	0.80	0.82	0.81	0.83
Parks and Cemeteries	7	0.05	0.09	0.12	0.19	0.20	0.29	0.30	0.40	0.34	0.46	0.39	0.52
Playgrounds	13	0.07	0.13	0.16	0.23	0.24	0.31	0.32	0.42	0.37	D.48	0.41	0.54
Railroad Yard Areas	40	0.23	0.28	0.30	0.35	0.36	0.42	0.42	0.50	0.46	0.54	0.50	0.58
Undeveloped Areas				-	_								
Historic Flow Analysis Greenbelts, Agriculture	2	0.03	0.05	0.09	0.16	0.17	0.26	0.26	0.38	0.31	0.45	0.36	0.51
Pasture/Meadow	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50
Forest	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50
Exposed Rock	100	0.89	0.89	0.90	0.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Offsite Flow Analysis (when	i –	- 1					0.52	0.5+	0.57	0.55	0.55	0.50	0.50
landuse is undefined)	45	0.26	0.31	0.32	0.37	0.38	0.44	0.44	0.51	0.48	0.55	0.51	0.59
	<u>                                       </u>												
Streets	ļi		_								'`]		
Paved	100	0.89	0.89	0.90	D.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Gravel	80	0.57	0.60	0.59	0.63	0.63	0.66	0.66	0.70	0.68	0.72	0.70	0.74
Drive and Walks	100	0.89	0.89	0.90	0.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Roofs	90	0.71	0.73	0.73	0.75	0.75	0.77	0.78	0.80	0.80	0.82	0.81	0.83
Lawns	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50

# 3.2 Time of Concentration

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time required for water to flow from the hydraulically most remote part of the drainage area under consideration to the design point. However, in practice, the time of concentration can be an empirical value that results in reasonable and acceptable peak flow calculations.

For urban areas, the time of concentration  $(t_c)$  consists of an initial time or overland flow time  $(t_i)$  plus the travel time  $(t_i)$  in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time  $(t_i)$  plus the time of travel in a concentrated form, such as a swale or drainageway. The travel portion  $(t_i)$  of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway. Initial time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. The time of concentration is represented by Equation 6-7 for both urban and non-urban areas.

Table 6-10. NRCS Curve Numbers for Frontal Storms & Thunderstorms for Developed Conditions (ARCII)

		Hydrologic	.		Pre-Devel	opment CN	<u> </u>
Fully Developed Urban Areas (vegetation established) <sup>1</sup>	Treatment	Condition	% I	HSG A	HSG B	HSG C	HSG D
Open space (lawns, parks, golf courses, cemeteries, etc.):							
Poor condition (grass cover < 50%)				68	79	86	89
Fair condition (grass cover 50% to 75%)				49	69	79	84
Good condition (grass cover > 75%)				39	61	74	80
Impervious areas:							
Paved parking lots, roofs, driveways, etc. (excluding right-of-way				98	98	98	98
Streets and roads:							
Paved; curbs and storm sewers (excluding right-of-way)				98	98	98	98
Paved; open ditches (Including right-of-way)				83	89	92	93
Gravel (Including right-of-way)				76	85	89	91
Dirt (including right-of-way)				72	82	87	89
Western desert urban areas:							
Natural desert landscaping (pervious areas only)				63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert				96	96	96	96
shrub with 1- to 2-inch sand or gravel mulch and basin borders)							
Urban districts:			0				
Commercial and business			85	89	92	94	95
Industrial Decide the business (as also as a second			72	81	88	91	93
Residential districts by average lot size:							
1/8 acre or less (town houses)			65	77	85	90	92
1/4 acre			38	61	75	83	87
1/3 acre			30	57	72	81	86
1/2 acre			25	54	70	80	85
2 acres			20	51	68	79	84
Zacies			12	45	65	77	82
Developing Urban Areas <sup>1</sup>	Treatment <sup>2</sup>	Hydrologic	% I	HSG A	HSG B	HSG C	HSG D
		Condition <sup>3</sup>					_
Newly graded areas (pervious areas only, no vegetation)	*****			77	. 86	91	94
Cultivated Agricultural Lands <sup>1</sup>	Treatment	Hydrologic Condition	% I	HSG A	HSG B	HSG C	HSG D
	Bare soil			77	86	91	94
Fallow	Crop residue	Poor		76	85	90	93
	cover (CR)	Good		74	83	88	90
	Straight row	Poor		72	81	88	91
	(SR)	Good		67	78	85	89
	SR+CR	Poor		71	80	87	90
		Good	+	64	75	82	85
	Contoured (C)	Poor		70	79	84	88
Row crops	,,,	Good		65	75	82	86
	C+CR	Poor		69	78	83	87
		Good		64	74	81	85
	Contoured &	Poor		66	74	80	82
	terraced (C&T)	Good		62	71	78	81
	C&T+ CR	Poor		65	73	79	81
		Good		61	70	77	80
	SR	Poor		65	76	84	88
		Good		63	75	83	87
	SR + CR	Poor		64	75	83	86
		Good		60	72	80	84
	С	Poor		63	74	82	85
Small grain	-	Good		61	73	81	84
	C + CR Poor	Poor		62	73	81	84
		Good		60	72	80	83
!	C&T	Poor		61	72	79	82
		Good		59	70	78 79	81
	C&T+ CR	Poor Good		60 58	71 69	78 77	81 80

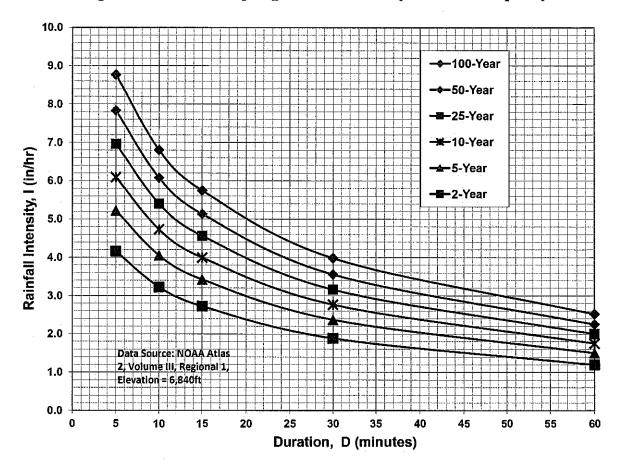


Figure 6-5. Colorado Springs Rainfall Intensity Duration Frequency

**IDF** Equations

 $I_{100} = -2.52 \ln(D) + 12.735$ 

 $I_{50} = -2.25 \ln(D) + 11.375$ 

 $I_{25} = -2.00 \ln(D) + 10.111$ 

 $I_{10} = -1.75 \ln(D) + 8.847$ 

 $I_5 = -1.50 \ln(D) + 7.583$ 

 $I_2 = -1.19 ln(D) + 6.035$ 

Note: Values calculated by equations may not precisely duplicate values read from figure. JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30

DATE: 08/25/22

CALCULATED BY: MAW

# FINAL DRAINAGE REPORT ~ BASIN RUNOFF COEFFICIENT SUMMARY

		IMP	ERVIOUS A	REA / STRE	ETS	LANI	DSCAPE/DI	EVELOPED A	AREAS	١	WEIGHTED			WEIGHTED (	CA	
BASIN	TOTAL AREA (AC)	AREA (AC)	C(2)	C(5)	C(100)	AREA (AC)	C(2)	C(5)	C(100)	C(2)	C(5)	C(100)	CA(2)	CA(5)	CA(100)	EFFECTIVE IMPERVIOUS (%)
EX-1	4.0	0.10	0.89	0.90	0.96	3.9	0.03	0.09	0.36	0.05	0.11	0.38	0.21	0.44	1.50	4.2%
EX-2	2.8	0.10	0.89	0.90	0.96	2.7	0.03	0.09	0.36	0.06	0.12	0.38	0.17	0.33	1.07	4.8%
EX-3	11.1	0.20	0.89	0.90	0.96	10.9	0.03	0.09	0.36	0.05	0.10	0.37	0.51	1.16	4.12	3.4%
EX-4	23.4	0.00	0.57	0.59	0.70	23.4	0.03	0.09	0.36	0.03	0.09	0.36	0.70	2.11	8.42	2.0%
EX-5	11.7	1.50	0.18	0.25	0.47	10.2	0.03	0.09	0.36	0.05	0.11	0.37	0.58	1.29	4.38	5.6%
EX-6	5.7	1.50	0.23	0.30	0.50	4.2	0.15	0.22	0.46	0.17	0.24	0.47	0.98	1.37	2.68	28.9%
EX-7	27.6	0.00	0.89	0.90	0.96	27.6	0.05	0.12	0.39	0.05	0.12	0.39	1.38	3.31	10.76	7.0%
EX-8	14.8	0.00	0.89	0.90	0.96	14.8	0.03	0.09	0.36	0.03	0.09	0.36	0.44	1.33	5.33	2.0%
EX-9	6.0	0.60	0.89	0.90	0.96	5.4	0.06	0.14	0.40	0.14	0.22	0.46	0.86	1.30	2.74	18.9%
EX-10	0.61	0.10	0.89	0.90	0.96	0.51	0.06	0.14	0.40	0.20	0.26	0.49	0.12	0.16	0.30	16.4%
OS-1	24.1	0.00	0.89	0.90	0.96	24.1	0.03	0.09	0.36	0.03	0.09	0.36	0.72	2.17	8.68	2.0%
OS-2	0.47	0.00	0.89	0.90	0.96	0.47	0.15	0.22	0.46	0.15	0.22	0.46	0.07	0.10	0.22	25.0%
OS-3	4.2	0.00	0.89	0.90	0.96	4.2	0.18	0.25	0.47	0.18	0.25	0.47	0.76	1.05	1.97	30.0%
OS-4	3.3	0.25	0.89	0.90	0.96	3.1	0.05	0.12	0.39	0.11	0.18	0.43	0.38	0.59	1.43	13.3%
OS-5	45.8	0.00	0.89	0.90	0.96	45.8	0.03	0.09	0.36	0.03	0.09	0.36	1.37	4.12	16.49	2.0%
OS-6	5.9	0.50	0.89	0.90	0.96	5.4	0.03	0.09	0.36	0.10	0.16	0.41	0.61	0.94	2.42	14.0%
OS-7	1.4	0.60	0.89	0.90	0.96	0.8	0.06	0.14	0.40	0.42	0.47	0.64	0.58	0.65	0.90	55.7%
Α	14.2	0.50	0.89	0.90	0.96	13.70	0.05	0.12	0.39	0.08	0.15	0.41	1.13	2.09	5.82	10.1%
В	8.8	0.00	0.89	0.90	0.96	8.80	0.12	0.20	0.44	0.12	0.20	0.44	1.06	1.76	3.87	20.0%
С	1.4	0.20	0.89	0.90	0.96	1.20	0.18	0.25	0.47	0.28	0.34	0.54	0.39	0.48	0.76	38.6%
D	0.2	0.00	0.89	0.90	0.96	0.20	0.23	0.30	0.50	0.23	0.30	0.50	0.05	0.06	0.10	40.0%
<u> </u>	2.3	0.00	0.89	0.90	0.96	2.30	0.15	0.22	0.46	0.15	0.22	0.46	0.35	0.51	1.06	25.0%
F	0.8	0.20	0.89	0.90	0.96	0.60	0.18	0.25	0.47	0.36	0.41	0.59	0.29	0.33	0.47	45.0%
G	2.2	0.00	0.89	0.90	0.96	2.20	0.18	0.25	0.47	0.18	0.25	0.47	0.40	0.55	1.03	30.0%
H1	2.5	0.20	0.89	0.90	0.96	2.30	0.05	0.12	0.39	0.12	0.18	0.44	0.29	0.46	1.09	14.0%
H2	3.1	0.30	0.89	0.90	0.96	2.80	0.07	0.16	0.41	0.15	0.23	0.46	0.46	0.72	1.44	15.5%
<u> </u>	2.6	0.00	0.89	0.90	0.96	2.60	0.05	0.12	0.39	0.05	0.12	0.39	0.13	0.31	1.01	7.0%
J1	2.4	0.26 0.00	0.89 0.89	0.90 0.90	0.96	2.14	0.05	0.12	0.39 0.39	0.14	0.20 0.12	0.45 0.39	0.34 0.02	0.49 0.05	1.08	16.5%
J2 K	0.4 3.30	0.00	0.89	0.90	0.96 0.96	0.40 2.75	0.05 0.05	0.12 0.12	0.39	0.05 0.19	0.12	0.39	0.02	0.05	0.16 1.60	7.0% 24.2%
I I	3.90	0.55	0.89	0.90	0.96	3.70	0.05	0.12	0.39	0.19	0.25	0.49	0.63	0.63	1.67	15.1%
L M	0.54	0.20	0.89	0.90	0.96	0.16	0.05	0.14	0.40	0.10	0.16	0.43	0.40	0.76	0.43	65.1%
N	0.55	0.30	0.89	0.90	0.96	0.10	0.05	0.12	0.39	0.04	0.55	0.79	0.34	0.30	0.43	52.7%
0	0.55	0.00	0.89	0.90	0.96	0.23	0.05	0.12	0.39	0.05	0.33	0.70	0.28	0.07	0.39	7.0%

JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30

DATE: 08/25/22

CALC'D BY:  $\overline{MAW}$ 

Return Period	1-Hour Depth
2	1.19
5	1.50
10	1.75
25	2.00
50	2.25
100	2.52

$$t_i = \frac{0.395 \big(1.1 - C_5 \big) \sqrt{L}}{S^{0.33}} \qquad \qquad V = C_v S_w^{-0.5} \qquad \text{Tc=L/V}$$

Table 6-7.	Conveyance	Coefficient, C,
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Type of Land Surface	$C_v$
Heavy meadow	2.5
Tillage/field L	5
Riprap (not buried)* $I_c = \frac{1}{180} + 10$	6.5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

For buried riprap, select C<sub>v</sub> value based on type of vegetative cover.

# FINAL DRAINAGE REPORT ~ BASIN RUNOFF SUMMARY

	WEIGHTED			D OVERLAND					STREET / CHANNEL FLOW				Tc INTENSITY			TOTAL FLOWS		
BASIN	CA(2)	CA(5)	CA(100)	C(5)	Length (ft)	Height (ft)	Tc (min)	Length (ft)	Slope (%)	Velocity (fps)	Tc (min)	TOTAL (min)	l(2) (in/hr)	I(5) (in/hr)	I(100) (in/hr)	Q(2) (cfs)	Q(5) (cfs)	Q(100) (cfs)
EX-1	0.21	0.44	1.50	0.09	280	12	18.9					18.9	2.54	3.18	5.33	0.5	1.4	8
EX-2	0.17	0.33	1.07	0.09	260	8	20.3					20.3	2.45	3.07	5.15	0.4	1.0	5
EX-3	0.51	1.16	4.12	0.09	300	11	20.6	160	2.0%	1.4	1.9	22.5	2.33	2.92	4.89	1	3	20
EX-4	0.70	2.11	8.42	0.09	300	16	18.2	1300	3.5%	1.9	11.6	29.8	2.00	2.49	4.18	1	5	35
EX-5	0.58	1.29	4.38	0.09	300	14	19.0	550	3.0%	1.7	5.3	24.3	2.24	2.80	4.70	1	4	21
EX-6	0.98	1.37	2.68	0.22	300	10	18.5					18.5	2.56	3.21	5.38	2	4	14
EX-7	1.38	3.31	10.76	0.12	300	12	19.4	1300	2.2%	1.0	21.0	40.4	1.63	2.03	3.41	2	7	37
EX-8	0.44	1.33	5.33	0.09	300	10	21.2					21.2	2.40	3.00	5.04	1	4	27
EX-9	0.86	1.30	2.74	0.14	230	15	14.2	450	2.0%	1.4	5.3	19.5	2.50	3.13	5.25	2	4	14
EX-10	0.12	0.16	0.30	0.14	220	4	21.1					21.1	2.41	3.01	5.05	0.3	0.5	1.5

JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30

DATE: 08/25/22

CALC'D BY:  $\overline{MAW}$ 

Return Period	1-Hour Depth
2	1.19
5	1.50
10	1.75
25	2.00
50	2.25
100	2.52

$$t_i = \frac{0.395 (1.1 - C_5) \sqrt{L}}{S^{0.33}} \qquad \qquad V = C_v S_w^{-0.5} \qquad \text{Tc=L/V}$$

# Table 6-7. Conveyance Coefficient, Cv

Type of Land Surface	$C_v$
Heavy meadow	2.5
Tillage/field L	5
Riprap (not buried)* $I_c = \frac{180}{180} + 10$	6.5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

For buried riprap, select C<sub>v</sub> value based on type of vegetative cover.

# FINAL DRAINAGE REPORT ~ BASIN RUNOFF SUMMARY

	WEIGHTED			OVERLAND				STREET / CHANNEL FLOW				Tc	INTENSITY			TOTAL FLOWS		
BASIN	CA(2)	CA(5)	CA(100)	C(5)	Length (ft)	Height (ft)	Tc (min)	Length (ft)	Slope (%)	Velocity (fps)	Tc (min)	TOTAL (min)	l(2) (in/hr)	l(5) (in/hr)	I(100) (in/hr)	Q(2) (cfs)	Q(5) (cfs)	Q(100) (cfs)
OS-1	0.72	2.17	8.68	0.09	300	16	18.2	1300	3.5%	1.9	11.6	29.8	2.00	2.49	4.18	1	5	35
OS-2	0.07	0.10	0.22	0.22	220	11	13.9					13.9	2.91	3.64	6.11	0.2	0.4	1.3
OS-3	0.76	1.05	1.97	0.25	100	2	12.2	550	2.8%	1.7	5.5	17.7	2.62	3.27	5.50	2	3	11
OS-4	0.38	0.59	1.43	0.12	300	10	20.6	700	5.0%	2.2	5.2	25.8	2.17	2.71	4.54	1	2	6
OS-5	1.37	4.12	16.49	0.09	300	20	16.9	2000	3.0%	1.7	19.2	36.1	1.77	2.20	3.70	2	9	61
OS-6	0.61	0.94	2.42	0.09	300	10	21.2	300	2.0%	1.4	3.5	24.8	2.22	2.77	4.65	1	3	11
OS-7	0.58	0.65	0.90	0.14	100	2	13.8	250	2.0%	1.4	2.9	16.7	2.68	3.36	5.63	2	2	5
А	1.13	2.09	5.82	0.12	300	13	18.9	100	2.0%	1.4	1.2	20.1	2.47	3.08	5.18	3	6	30
В	1.06	1.76	3.87	0.20	230	3	22.6	700	2.0%	1.4	8.2	30.8	1.96	2.44	4.10	2	4	16
С	0.39	0.48	0.76	0.25	50	1	8.6	700	2.0%	1.4	8.2	16.9	2.67	3.34	5.61	1	2	4
D	0.05	0.06	0.10	0.30	50	1	8.1					8.1	3.54	4.44	7.46	0.2	0.3	0.7
Е	0.35	0.51	1.06	0.22	100	5	9.3	200	5.5%	4.7	0.7	10.1	3.29	4.12	6.92	1	2	7
F	0.29	0.33	0.47	0.25	60	2	8.0	200	5.5%	4.7	0.7	8.7	3.46	4.34	7.28	1.0	1.4	3
G	0.40	0.55	1.03	0.25	300	9	18.5	600	1.5%	1.2	8.2	26.7	2.13	2.66	4.46	1	1	5

JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30 DATE: 08/25/22

 $\overline{MAW}$ 

CALC'D BY:

Return	1-Hour
Period	Depth
2	1.19
5	1.50
10	1.75
25	2.00
50	2.25
100	2.52

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}}$$
  $V = C_v S_w^{0.5}$  Tc=L/V

Type of Land Surface	$C_v$
Heavy meadow	2.5
Tillage/field L	5
Riprap (not buried)* $I_c = \frac{1}{180} + 10$	6.5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

For buried riprap, select C<sub>v</sub> value based on type of vegetative cover.

# FINAL DRAINAGE REPORT ~ BASIN RUNOFF SUMMARY

		WEIGHTE	)		OVER	LAND		STRE	ET / CH	IANNEL	FLOW	Тс	IN	NTENSIT	Υ	ТОТ	AL FLO	)WS
BASIN	CA(2)	CA(5)	CA(100)	C(5)	Length (ft)	Height (ft)	Tc (min)	Length (ft)	Slope	Velocity (fps)	Tc (min)	TOTAL (min)	l(2) (in/hr)	l(5) (in/hr)	I(100) (in/hr)	Q(2) (cfs)	Q(5) (cfs)	Q(100) (cfs)
H1	0.29	0.46	1.09	0.12	100	8	8.9					8.9	3.43	4.30	7.22	1	2	8
H2	0.46	0.72	1.44	0.16	100	12	7.5					7.5	3.64	4.57	7.67	2	3	11
I	0.13	0.31	1.01	0.12	300	4	27.9					27.9	2.08	2.59	4.35	0.3	0.8	4
J1	0.34	0.49	1.08	0.12	300	12	19.4					19.4	2.51	3.14	5.26	1	2	6
J2	0.02	0.05	0.16	0.12	160	4	16.5					16.5	2.70	3.37	5.66	0.1	0.2	0.9
К	0.63	0.83	1.60	0.12	300	8	22.2					22.2	2.35	2.93	4.93	1	2	8
L	0.40	0.70	1.67	0.14	300	12	19.0					19.0	2.53	3.17	5.32	1	2	9
М	0.34	0.36	0.43	0.12	15	0.3	5.5	350	1.5%	2.4	2.4	7.8	3.59	4.50	7.55	1	2	3
N	0.28	0.30	0.39	0.12	15	0.3	5.5	350	1.5%	2.4	2.4	7.8	3.59	4.50	7.55	1	1	3
0	0.03	0.07	0.24	0.12	100	6	9.8					9.8	3.32	4.16	6.98	0.1	0.3	2

JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30
DATE: 08/25/22

CALCULATED BY: MAW

# FINAL DRAINAGE REPORT ~ SURFACE ROUTING SUMMARY

					Inten	sity	FI	ow	
Design Point(s)	Contributing Basins	Equivalent CA(5)	Equivalent CA(100)	Maximum Tc	I(5)	I(100)	Q(5)	Q(100)	Inlet Size
EX DP-1	Basin EX-1, EX-2, EX-7 and EX-9 (40.4 AC.)	5.38	16.07	40.4	2.03	3.41	11	55	EX. DUAL 30" RCP CULVERTS
EX DP-2	Basin EX-3 (11.1 AC.)	1.16	4.12	22.5	2.92	4.89	3	20	EX. 30" RCP STORM SEWER
EX DP-3	Basin EX-4 (24.7 AC.)	2.11	8.42	29.8	2.49	4.18	5	35	EX. STOCK POND
1	Basin A (14.2 AC.)	2.09	5.82	20.1	3.08	5.18	6	30	EX. 30" RCP CULVERT
2	Basin C (1.4 AC.)	0.48	0.76	16.9	3.34	5.61	2	4	5' TYPE R SUMP INLET
3	Basins B, OS-2 and D (9.47 AC.)	1.92	4.19	30.8	2.44	4.10	5	21	15' TYPE R SUMP INLET
4	Basins F and OS-7 (2.2 AC.)	0.98	1.37	16.7	3.36	5.63	2	8	EX. 10' TYPE R AT-GRADE INLET
5	Basins E and OS-3 (6.5 AC.)	1.56	3.03	17.7	3.27	5.50	5	17	EX. 15' TYPE R AT-GRADE INLET
6	Basin OS-1 (24.1 AC.)	2.17	8.68	29.8	2.49	4.18	5	35	PROP. TEMP. SEDIMENT BASIN
7	Basin OS-4 (3.3 AC.)	0.59	1.43	25.8	2.71	4.54	2	6	PROP. TYPE C CDOT INLET
8	Basin N (0.55 AC.)	0.30	0.39	7.8	4.50	7.55	1	3	5' TYPE R SUMP INLET
9	Basin M (0.54 AC.)	0.36	0.43	7.8	4.50	7.55	2	3	5' TYPE R SUMP INLET

JOB NAME: RETREAT AT TIMBERRIDGE FILING NO. 3

JOB NUMBER: 1185.30
DATE: 08/25/22

CALCULATED BY: MAW

# FINAL DRAINAGE REPORT ~ SURFACE ROUTING SUMMARY

					Inten	sity	Fle	ow	
Design Point(s)	I Contribution Rasins	Equivalent CA(5)	Equivalent CA(100)	Maximum Tc	I(5)	I(100)	Q(5)	Q(100)	Inlet Size
10	Basins OS-4, N, M, O (5.0 AC.)	1.33	2.48	26.1	2.69	4.51	4	11	PROP. RAIN GARDEN 1
11	Basin OS-6 (5.9 AC.)	0.94	2.42	24.8	2.77	4.65	3	11	PROP. TYPE C CDOT INLET
12	Basins EX-7, K, L and EX-9 (40.8 AC.)	6.13	16.77	40.4	2.03	3.41	12	5/	EX. DUAL 30" RCP CULVERTS

JOB NAME: <u>RETREAT AT TIMBERRIDGE FILING NO. 3</u>

JOB NUMBER: 1185.30 DATE: 08/25/22

CALCULATED BY: MAW

# FINAL DRAINAGE REPORT ~ PIPE ROUTING SUMMARY

					Inten	sity	Fle	ow	
Pipe Run	Contributing Basins	Equivalent CA(5)	Equivalent CA(100)	Maximum Tc	I(5)	I(100)	Q(5)	Q(100)	Pipe Size*
1	DP-1	2.09	5.82	20.1	3.08	5.18	6	30	30" RCP
2	DP-2	0.48	0.76	16.9	3.34	5.61	2	4	18" RCP
3	DP-3	1.92	4.19	30.8	2.44	4.10	5	21	30" RCP
4	PR-1, PR-2 and PR-3	4.50	10.77	20.9	3.03	5.08	14	55	36" RCP
5	DP-7	0.59	1.43	25.8	2.71	4.54	2	6	18" RCP
6	DP-8	0.30	0.39	7.8	4.50	7.55	1	3	18" RCP
7	DP-9	0.36	0.43	7.8	4.50	7.55	2	3	18" RCP
8	PR-6 and PR-7	0.66	0.81	7.8	4.50	7.55	3	6	24" RCP
9	PR-5, PR-8	1.25	2.24	26.0	2.69	4.52	3	10	24" RCP

<sup>\*</sup> PIPES ARE LISTED AT MAXIMUM SIZE REQUIRED TO ACCOMMODATE Q100 FLOWS AT MINIMUM GRADE. REFER TO INDIVIDUAL PIPE SHEETS FOR HYDRAULIC INFORMATION.

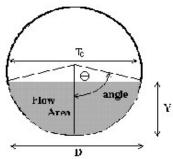
# **EFFECTIVE IMPERVIOUSNESS - RAIN GARDEN 1**

Basin	Acreage	lmp.%		
OS-4	3.30	13.3%		
M	0.54	65.1%		
N	0.55	52.7%		
0	0.61	7.0%		
Total	5.0	22.4%		

# **EFFECTIVE IMPERVIOUSNESS - RAIN GARDEN 2**

Basin	Acreage	lmp.%
OS-6	5.9	14.0%
Total	5.9	14.0%

# CIRCULAR CONDUIT FLOW (Normal & Critical Depth Computation) MHFD-Culvert, Version 4.00 (May 2020) Project: RETREAT AT TIMBERRIDGE FILING NO. 3 Pipe ID: DP-1 (Exist. 30" RCP Pipe)

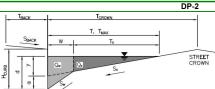


	*		
Design Information (Input)			
Pipe Invert Slope	So =	0.0100	ft/ft
Pipe Manning's n-value	n =	0.0130	
Pipe Diameter	D =	30.00	inches
Design discharge	Q =	30.00	cfs
Full-Flow Capacity (Calculated)			_
Full-flow area	Af =	4.91	sq ft
Full-flow wetted perimeter	Pf =	7.85	ft
Half Central Angle	Theta =	3.14	radians
Full-flow capacity	Qf =	41.13	cfs
Calculation of Normal Flow Condition			_
Half Central Angle (0 <theta<3.14)< td=""><td>Theta =</td><td>1.84</td><td>radians</td></theta<3.14)<>	Theta =	1.84	radians
Flow area	An =	3.28	sq ft
Top width	Tn =	2.41	ft
Wetted perimeter	Pn =	4.60	ft
Flow depth	Yn =	1.58	ft
Flow velocity	Vn =	9.14	fps
Discharge	Qn =	30.00	cfs
Percent of Full Flow	Flow =	72.9%	of full flow
Normal Depth Froude Number	Fr <sub>n</sub> =	1.38	supercritical
Calculation of Critical Flow Condition			
	Theta-c =	2.09	radians
Half Central Angle (0 <theta-c<3.14) area<="" critical="" flow="" td=""><td>Ac =</td><td>3.93</td><td>sq ft</td></theta-c<3.14)>	Ac =	3.93	sq ft
Critical flow area Critical top width	AC = Tc =	2.17	sq it
•	Yc =	1.87	It
Critical flow depth			<b></b>
Critical flow velocity	Vc =	7.63 1.00	fps
Critical Depth Froude Number	$Fr_c = $	1.00	

# ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)
RETREAT AT TIMBERRIDGE FILING NO. 3

Project: Inlet ID:

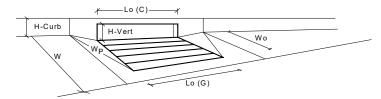


### Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Behind Curb T<sub>BACK</sub> = Side Slope Behind Curb (leave blank for no conveyance credit behind curb) $S_{\text{BACK}}$ 0.020 Manning's Roughness Behind Curb (typically between 0.012 and 0.020) 0.013 Height of Curb at Gutter Flow Line H<sub>CURB</sub> : 6.00 inches Distance from Curb Face to Street Crown T<sub>CROWN</sub> 17.0 Gutter Width w : 2.00 Street Transverse Slope S<sub>X</sub> = 0.020 ft/ft S<sub>W</sub> Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) ft/ft 0.083 Street Longitudinal Slope - Enter 0 for sump condition So 0.000 ft/ft Manning's Roughness for Street Section (typically between 0.012 and 0.020) n<sub>STREET</sub> = 0.016 Minor Storm Major Storm Max. Allowable Spread for Minor & Major Storm 17.0 17.0 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm 6.0 12.0 Check boxes are not applicable in SUMP conditions MINOR STORM Allowable Capacity is based on Depth Criterion Major Storm Minor Storm MAJOR STORM Allowable Capacity is based on Depth Criterion SUMP SUMP cfs

118530 UD-Inlet v4.05, DP-2 11/4/2022, 1:54 PM

# **INLET IN A SUMP OR SAG LOCATION**

Version 4.05 Released March 2017



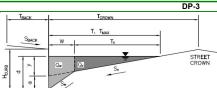
Design Information (Input)	CDOT Type R Curb Opening   ▼	1	MINOR	MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type F	R Curb Opening	
Local Depression (additional to cont	inuous gutter depression 'a' from above)	a <sub>local</sub> =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curt	Opening)	No =	1	1	
Water Depth at Flowline (outside of	local depression)	Ponding Depth =	6.0	12.0	inches
Grate Information			MINOR	MAJOR	Override Depths
Length of a Unit Grate		L₀ (G) =	N/A	N/A	feet
Width of a Unit Grate		W <sub>o</sub> =	N/A	N/A	feet
Area Opening Ratio for a Grate (typi	ical values 0.15-0.90)	A <sub>ratio</sub> =	N/A	N/A	
Clogging Factor for a Single Grate (	typical value 0.50 - 0.70)	$C_f(G) =$	N/A	N/A	
Grate Weir Coefficient (typical value	2.15 - 3.60)	C <sub>w</sub> (G) =	N/A	N/A	
Grate Orifice Coefficient (typical val	ue 0.60 - 0.80)	C <sub>o</sub> (G) =	N/A	N/A	
Curb Opening Information			MINOR	MAJOR	_
Length of a Unit Curb Opening		L <sub>o</sub> (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in I	nches	H <sub>vert</sub> =	6.00	6.00	inches
Height of Curb Orifice Throat in Inch	nes	H <sub>throat</sub> =	6.00	6.00	inches
Angle of Throat (see USDCM Figure	e ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typi	cally the gutter width of 2 feet)	W <sub>p</sub> =	2.00	2.00	feet
Clogging Factor for a Single Curb O	pening (typical value 0.10)	$C_f(C) =$	0.10	0.10	
Curb Opening Weir Coefficient (typi	cal value 2.3-3.7)	C <sub>w</sub> (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (ty	pical value 0.60 - 0.70)	C <sub>o</sub> (C) =	0.67	0.67	
Low Head Performance Reduction	n (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth		d <sub>Grate</sub> =	N/A	N/A	ft
Depth for Curb Opening Weir Equat	tion	d <sub>Curb</sub> =	0.33	0.83	ft
Combination Inlet Performance Red	luction Factor for Long Inlets	RF <sub>Combination</sub> =	0.77	1.00	
Curb Opening Performance Reducti	ion Factor for Long Inlets	RF <sub>Curb</sub> =	1.00	1.00	
Grated Inlet Performance Reduction	Factor for Long Inlets	RF <sub>Grate</sub> =	N/A	N/A	
		_	MINOR	MAJOR	_
Total Inlet Interception Cap	acity (assumes clogged condition)	$Q_a =$	5.4	12.3	cfs
Inlet Capacity IS GOOD for Minor	and Major Storms(>Q PEAK)	Q <sub>PEAK REQUIRED</sub> =	2.0	4.0	cfs

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# ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)
RETREAT AT TIMBERRIDGE FILING NO. 3

Project: Inlet ID:

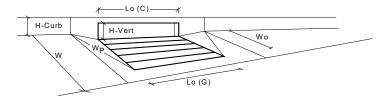


### Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Behind Curb T<sub>BACK</sub> = Side Slope Behind Curb (leave blank for no conveyance credit behind curb) $S_{\text{BACK}}$ 0.020 ft/ft Manning's Roughness Behind Curb (typically between 0.012 and 0.020) 0.013 Height of Curb at Gutter Flow Line H<sub>CURB</sub> : 6.00 inches Distance from Curb Face to Street Crown T<sub>CROWN</sub> 17.0 Gutter Width w : 2.00 Street Transverse Slope S<sub>X</sub> = 0.020 ft/ft S<sub>W</sub> Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) ft/ft 0.083 Street Longitudinal Slope - Enter 0 for sump condition So 0.000 ft/ft Manning's Roughness for Street Section (typically between 0.012 and 0.020) n<sub>STREET</sub> = 0.016 Minor Storm Major Storm Max. Allowable Spread for Minor & Major Storm 17.0 17.0 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm 6.0 12.0 Check boxes are not applicable in SUMP conditions MINOR STORM Allowable Capacity is based on Depth Criterion Major Storm Minor Storm MAJOR STORM Allowable Capacity is based on Depth Criterion SUMP SUMP cfs

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# INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type R	Curb Opening	
Local Depression (additional to conti	nuous gutter depression 'a' from above)	a <sub>local</sub> =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb	Opening)	No =	1	1	
Water Depth at Flowline (outside of I	ocal depression)	Ponding Depth =	6.0	12.0	inches
Grate Information			MINOR	MAJOR	Override Depths
Length of a Unit Grate		L <sub>0</sub> (G) =	N/A	N/A	feet
Width of a Unit Grate		W <sub>o</sub> =	N/A	N/A	feet
Area Opening Ratio for a Grate (typic	cal values 0.15-0.90)	A <sub>ratio</sub> =	N/A	N/A	
Clogging Factor for a Single Grate (t	ypical value 0.50 - 0.70)	C <sub>f</sub> (G) =	N/A	N/A	
Grate Weir Coefficient (typical value	2.15 - 3.60)	C <sub>w</sub> (G) =	N/A	N/A	
Grate Orifice Coefficient (typical valu	e 0.60 - 0.80)	C <sub>o</sub> (G) =	N/A	N/A	
Curb Opening Information		_	MINOR	MAJOR	
Length of a Unit Curb Opening		L <sub>o</sub> (C) =	15.00	15.00	feet
Height of Vertical Curb Opening in In	ches	H <sub>vert</sub> =	6.00	6.00	inches
Height of Curb Orifice Throat in Inche	es	H <sub>throat</sub> =	6.00	6.00	inches
Angle of Throat (see USDCM Figure	ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typic	cally the gutter width of 2 feet)	W <sub>p</sub> =	2.00	2.00	feet
Clogging Factor for a Single Curb Op	pening (typical value 0.10)	$C_f(C) =$	0.10	0.10	
Curb Opening Weir Coefficient (typic	cal value 2.3-3.7)	C <sub>w</sub> (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (type	oical value 0.60 - 0.70)	C <sub>o</sub> (C) =	0.67	0.67	
Low Head Performance Reduction	(Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth		d <sub>Grate</sub> =	N/A	N/A	ft
Depth for Curb Opening Weir Equati	on	d <sub>Curb</sub> =	0.33	0.83	ft
Combination Inlet Performance Redu	uction Factor for Long Inlets	RF <sub>Combination</sub> =	0.57	1.00	
Curb Opening Performance Reduction	on Factor for Long Inlets	RF <sub>Curb</sub> =	0.79	1.00	
Grated Inlet Performance Reduction	Factor for Long Inlets	RF <sub>Grate</sub> =	N/A	N/A	
			MINOR	MAJOR	_
Total Inlet Interception Capa	acity (assumes clogged condition)	$Q_a =$	9.7	39.1	cfs
Inlet Capacity IS GOOD for Minor a	and Major Storms(>Q PEAK)	Q PEAK REQUIRED =	6.0	21.0	cfs

118530 UD-Inlet\_v4.05, DP-3 11/4/2022, 1:55 PM

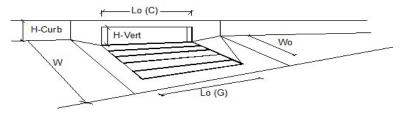
# RETREAT AT TIMBERRIDGE FILING NO. 3 Inlet ID: Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Rehind Curb

MAJOR STORM Allowable Capacity is based on Spread Criterion  Minor storm max. allowable capacity GOOD - greater than the design flow given on s	Q <sub>allow</sub> =	14.2	14.2	cfs
MINOR STORM Allowable Capacity is based on Spread Criterion	_	Minor Storm	Major Storm	_
Tiow Deput at Otteet Crown (leave plank for no)				crieck – yes
Allow Flow Depth at Street Crown (leave blank for no)	UMAX -	0.0	12.0	check = ves
Max. Allowable Spread for Milhor & Major Storm  Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d <sub>MAX</sub> =	6.0	17.0 12.0	inches
Max. Allowable Spread for Minor & Major Storm	T <sub>MAX</sub> =	Minor Storm 17.0	Major Storm	<b>T</b> ft
identified the entire to the entire term (typically between 6.0 in an entire term of the	omee.	0.010		
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n <sub>STREET</sub> =	0.016		
Street Longitudinal Slope - Enter 0 for sump condition	S <sub>O</sub> =	0.003	ft/ft	
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S <sub>W</sub> =	0.020	ft/ft	
Street Transverse Slope	VV - S <sub>X</sub> =	0.020	ft/ft	
Sutter Width	V =	2.00	ft	
Distance from Curb Face to Street Crown	T <sub>CROWN</sub> =	6.00 17.0	ft	
Height of Curb at Gutter Flow Line	H <sub>CURB</sub> =	0.00	inches	
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n <sub>BACK</sub> =	0.013		
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S <sub>BACK</sub> =	0.020	ft/ft	
Maximum Allowable Width for Spread Behind Curb	T <sub>BACK</sub> =	7.5	ft	

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# INLET ON A CONTINUOUS GRADE

Version 4.05 Released March 2017



Design Information (Input)  CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type F	Curb Opening	
Local Depression (additional to continuous gutter depression 'a')	a <sub>LOCAL</sub> =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L <sub>o</sub> =	10.00	10.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W <sub>o</sub> =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C <sub>f</sub> -G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C <sub>f</sub> -C =	0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity'		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	3.0	6.3	cfs
Total Inlet Carry-Over Flow (flow bypassing inlet)	<b>Q</b> <sub>b</sub> =	0.0	1.7	cfs
Capture Percentage = Q <sub>a</sub> /Q <sub>o</sub> =	C% =	100	79	%

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### ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm) (Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread) RETREAT AT TIMBERRIDGE FILING NO. 3 Project: Inlet ID: DP-5 STREET Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Behind Curb T<sub>BACK</sub> = Side Slope Behind Curb (leave blank for no conveyance credit behind curb) $S_{\text{BACK}}$ 0.020 Manning's Roughness Behind Curb (typically between 0.012 and 0.020) 0.013 Height of Curb at Gutter Flow Line H<sub>CURB</sub> : 6.00 inches Distance from Curb Face to Street Crown T<sub>CROWN</sub> 17.0 Gutter Width w : 2.00 Street Transverse Slope S<sub>X</sub> = 0.020 ft/ft S<sub>W</sub> Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) ft/ft 0.083 Street Longitudinal Slope - Enter 0 for sump condition So 0.017 ft/ft Manning's Roughness for Street Section (typically between 0.012 and 0.020) n<sub>STREET</sub> = 0.016 Minor Storm Major Storm Max. Allowable Spread for Minor & Major Storm 17.0 17.0 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm 6.0 12.0 inches

check = yes

Major Storm

14.2

Minor Storm

14.2

Allow Flow Depth at Street Crown (leave blank for no)

MINOR STORM Allowable Capacity is based on Spread Criterion

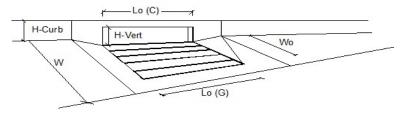
MAJOR STORM Allowable Capacity is based on Spread Criterion

linor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management /ARNING: MAJOR STORM max. allowable capacity is less than the design flow given on sheet 'Inlet Manag

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# INLET ON A CONTINUOUS GRADE

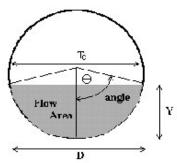
# Version 4.05 Released March 2017



Design Information (Input)  CDOT Type R Curb Opening		MINOR	MAJOR	_
Type of Inlet	Type =	CDOT Type F	Curb Opening	
Local Depression (additional to continuous gutter depression 'a')	a <sub>LOCAL</sub> =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L <sub>0</sub> =	15.00	15.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W <sub>o</sub> =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C <sub>f</sub> -G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C <sub>f</sub> -C =	0.10	0.10	
Street Hydraulics: WARNING: Q > ALLOWABLE Q FOR MAJOR STORM		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	5.0	12.7	cfs
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q <sub>b</sub> =	0.0	4.3	cfs
Capture Percentage = Q <sub>a</sub> /Q <sub>o</sub> =	C% =	100	75	%

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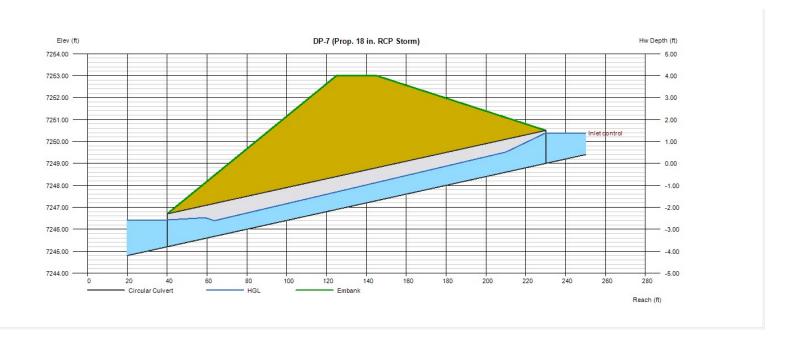
# CIRCULAR CONDUIT FLOW (Normal & Critical Depth Computation) MHFD-Culvert, Version 4.00 (May 2020) Project: RETREAT AT TIMBERRIDGE FILING NO. 3 Pipe ID: DP-7 (Prop. 18" RCP Storm)



Design Information (Input)			_
Pipe Invert Slope	So =	0.0200	ft/ft
Pipe Manning's n-value	n =	0.0130	
Pipe Diameter	D =	18.00	inches
Design discharge	Q =	6.00	cfs
Full-Flow Capacity (Calculated)			
Full-flow area	Af =	1.77	sq ft
Full-flow wetted perimeter	Pf =	4.71	ft
Half Central Angle	Theta =	3.14	radians
Full-flow capacity	Qf =	14.90	cfs
Calculation of Normal Flow Condition			
Half Central Angle (0 <theta<3.14)< td=""><td>Theta =</td><td>1.45</td><td>radians</td></theta<3.14)<>	Theta =	1.45	radians
Flow area	An =	0.75	sq ft
Top width	Tn =	1.49	ft
Wetted perimeter	Pn =	2.18	ft
Flow depth	Yn =	0.66	ft
Flow velocity	Vn =	7.97	fps
Discharge	Qn =	6.00	cfs
Percent of Full Flow	Flow =	40.3%	of full flow
Normal Depth Froude Number	Fr <sub>n</sub> =	1.98	supercritical
Calculation of Critical Flow Condition			
Half Central Angle (0 <theta-c<3.14)< td=""><td>Theta-c =</td><td>1.84</td><td>radians</td></theta-c<3.14)<>	Theta-c =	1.84	radians
Critical flow area	Ac =	1.17	sq ft
Critical top width	Tc =	1.45	ft
Critical flow depth	Yc =	0.95	ft
Critical flow velocity	Vc =	5.11	fps
Critical Depth Froude Number	Fr <sub>c</sub> =	1.00	

# DP-7 (Prop. 18 in. RCP Storm)

Invert Elev Dn (ft)	= 7245.20	Calculations	
Pipe Length (ft)	= 190.00	Qmin (cfs)	= 6.00
Slope (%)	= 2.00	Qmax (cfs)	= 6.00
Invert Elev Up (ft)	= 7249.00	Tailwater Elev (ft)	= (dc+D)/2
Rise (in)	= 18.0		
Shape	= Circular	Highlighted	
Span (in)	= 18.0	Qtotal (cfs)	= 6.00
No. Barrels	= 1	Qpipe (cfs)	= 6.00
n-Value	= 0.013	Qovertop (cfs)	= 0.00
Culvert Type	= Circular Concrete	Veloc Dn (ft/s)	= 3.89
Culvert Entrance	= Groove end projecting (C)	Veloc Up (ft/s)	= 5.12
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.2	HGL Dn (ft)	= 7246.42
		HGL Up (ft)	= 7249.95
Embankment		Hw Elev (ft)	= 7250.39
Top Elevation (ft)	= 7253.00	Hw/D (ft)	= 0.93
Top Width (ft)	= 20.00	Flow Regime	= Inlet Control
Crest Width (ft)	= 20.00		



# ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

RETREAT AT TIMBERRIDGE FILING NO. 3

DP-8

Project: Inlet ID:

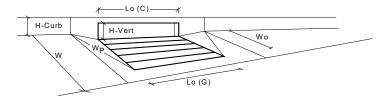
STREET

### Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Behind Curb T<sub>BACK</sub> = 15.0 Side Slope Behind Curb (leave blank for no conveyance credit behind curb) $S_{\text{BACK}}$ 0.020 ft/ft Manning's Roughness Behind Curb (typically between 0.012 and 0.020) 0.013 Height of Curb at Gutter Flow Line H<sub>CURB</sub> : 6.00 inches Distance from Curb Face to Street Crown T<sub>CROWN</sub> 18.0 Gutter Width w : 2.00 Street Transverse Slope S<sub>X</sub> = 0.020 ft/ft S<sub>W</sub> Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) ft/ft 0.083 Street Longitudinal Slope - Enter 0 for sump condition So 0.000 ft/ft Manning's Roughness for Street Section (typically between 0.012 and 0.020) n<sub>STREET</sub> = 0.016 Minor Storm Major Storm Max. Allowable Spread for Minor & Major Storm 12.0 12.0 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm 6.0 6.0 Check boxes are not applicable in SUMP conditions MINOR STORM Allowable Capacity is based on Depth Criterion Minor Storm Major Storm MAJOR STORM Allowable Capacity is based on Depth Criterion SUMP SUMP cfs

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# INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



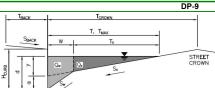
Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type R	Curb Opening	
Local Depression (additional to continuous gutter depression 'a' from above)		a <sub>local</sub> =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb	Opening)	No =	1	1	
Water Depth at Flowline (outside of	local depression)	Ponding Depth =	4.4	6.0	inches
Grate Information			MINOR	MAJOR	Override Depths
Length of a Unit Grate		L <sub>0</sub> (G) =	N/A	N/A	feet
Width of a Unit Grate		W <sub>o</sub> =	N/A	N/A	feet
Area Opening Ratio for a Grate (typi	cal values 0.15-0.90)	A <sub>ratio</sub> =	N/A	N/A	
Clogging Factor for a Single Grate (	typical value 0.50 - 0.70)	C <sub>f</sub> (G) =	N/A	N/A	
Grate Weir Coefficient (typical value	2.15 - 3.60)	C <sub>w</sub> (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value	ue 0.60 - 0.80)	C <sub>o</sub> (G) =	N/A	N/A	
Curb Opening Information		_	MINOR	MAJOR	
Length of a Unit Curb Opening		L <sub>o</sub> (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in In	nches	H <sub>vert</sub> =	6.00	6.00	inches
Height of Curb Orifice Throat in Inch	es	H <sub>throat</sub> =	6.00	6.00	inches
Angle of Throat (see USDCM Figure	ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typi-	cally the gutter width of 2 feet)	W <sub>p</sub> =	2.00	2.00	feet
Clogging Factor for a Single Curb O	pening (typical value 0.10)	$C_f(C) =$	0.10	0.10	
Curb Opening Weir Coefficient (typic	cal value 2.3-3.7)	C <sub>w</sub> (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (ty	pical value 0.60 - 0.70)	C <sub>o</sub> (C) =	0.67	0.67	
Low Head Performance Reduction	ı (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth		d <sub>Grate</sub> =	N/A	N/A	ft
Depth for Curb Opening Weir Equat	ion	d <sub>Curb</sub> =	0.20	0.33	ft
Combination Inlet Performance Red	uction Factor for Long Inlets	RF <sub>Combination</sub> =	0.56	0.77	
Curb Opening Performance Reducti	on Factor for Long Inlets	RF <sub>Curb</sub> =	1.00	1.00	
Grated Inlet Performance Reduction	Factor for Long Inlets	RF <sub>Grate</sub> =	N/A	N/A	
			MINOR	MAJOR	_
Total Inlet Interception Cap	acity (assumes clogged condition)	<b>Q</b> <sub>a</sub> =	2.5	5.4	cfs
Inlet Capacity IS GOOD for Minor	and Major Storms(>Q PEAK)	Q PEAK REQUIRED =	1.0	3.0	cfs

118530 UD-Inlet\_v4.05, DP-8 11/4/2022, 1:55 PM

# ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)
RETREAT AT TIMBERRIDGE FILING NO. 3

Project: Inlet ID:

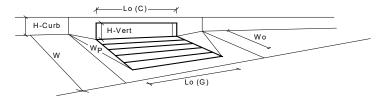


### Gutter Geometry (Enter data in the blue cells) Maximum Allowable Width for Spread Behind Curb T<sub>BACK</sub> = 15.0 Side Slope Behind Curb (leave blank for no conveyance credit behind curb) $S_{\text{BACK}}$ 0.020 ft/ft Manning's Roughness Behind Curb (typically between 0.012 and 0.020) 0.013 Height of Curb at Gutter Flow Line H<sub>CURB</sub> : 6.00 inches Distance from Curb Face to Street Crown T<sub>CROWN</sub> 18.0 Gutter Width w : 2.00 Street Transverse Slope S<sub>X</sub> = 0.020 ft/ft S<sub>W</sub> Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) ft/ft 0.083 Street Longitudinal Slope - Enter 0 for sump condition So 0.000 ft/ft Manning's Roughness for Street Section (typically between 0.012 and 0.020) n<sub>STREET</sub> = 0.016 Minor Storm Major Storm Max. Allowable Spread for Minor & Major Storm 12.0 12.0 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm 6.0 6.0 Check boxes are not applicable in SUMP conditions MINOR STORM Allowable Capacity is based on Depth Criterion Minor Storm Major Storm MAJOR STORM Allowable Capacity is based on Depth Criterion SUMP SUMP cfs

118530 UD-Inlet v4.05, DP-9 11/4/2022, 1:56 PM

# INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)	CDOT Type R Curb Opening   ▼	1	MINOR	MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type F	R Curb Opening	
Local Depression (additional to cont	inuous gutter depression 'a' from above)	a <sub>local</sub> =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curt	Opening)	No =	1	1	
Water Depth at Flowline (outside of	local depression)	Ponding Depth =	4.4	6.0	inches
Grate Information			MINOR	MAJOR	Override Depths
Length of a Unit Grate		L <sub>0</sub> (G) =	N/A	N/A	feet
Width of a Unit Grate		W <sub>o</sub> =	N/A	N/A	feet
Area Opening Ratio for a Grate (typi	ical values 0.15-0.90)	A <sub>ratio</sub> =	N/A	N/A	
Clogging Factor for a Single Grate (	typical value 0.50 - 0.70)	$C_f(G) =$	N/A	N/A	
Grate Weir Coefficient (typical value	2.15 - 3.60)	C <sub>w</sub> (G) =	N/A	N/A	
Grate Orifice Coefficient (typical val	ue 0.60 - 0.80)	C <sub>o</sub> (G) =	N/A	N/A	
Curb Opening Information			MINOR	MAJOR	_
Length of a Unit Curb Opening		L <sub>o</sub> (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in I	nches	H <sub>vert</sub> =	6.00	6.00	inches
Height of Curb Orifice Throat in Inch	nes	H <sub>throat</sub> =	6.00	6.00	inches
Angle of Throat (see USDCM Figure	e ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typi	cally the gutter width of 2 feet)	W <sub>p</sub> =	2.00	2.00	feet
Clogging Factor for a Single Curb O	pening (typical value 0.10)	$C_f(C) =$	0.10	0.10	
Curb Opening Weir Coefficient (typi	cal value 2.3-3.7)	C <sub>w</sub> (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (ty	pical value 0.60 - 0.70)	C <sub>o</sub> (C) =	0.67	0.67	
Low Head Performance Reduction	n (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth		d <sub>Grate</sub> =	N/A	N/A	ft
Depth for Curb Opening Weir Equat	tion	d <sub>Curb</sub> =	0.20	0.33	ft
Combination Inlet Performance Red	luction Factor for Long Inlets	RF <sub>Combination</sub> =	0.56	0.77	
Curb Opening Performance Reducti	ion Factor for Long Inlets	RF <sub>Curb</sub> =	1.00	1.00	
Grated Inlet Performance Reduction	Factor for Long Inlets	RF <sub>Grate</sub> =	N/A	N/A	
		_	MINOR	MAJOR	_
Total Inlet Interception Cap	acity (assumes clogged condition)	$Q_a =$	2.5	5.4	cfs
Inlet Capacity IS GOOD for Minor	and Major Storms(>Q PEAK)	Q <sub>PEAK REQUIRED</sub> =	2.0	3.0	cfs

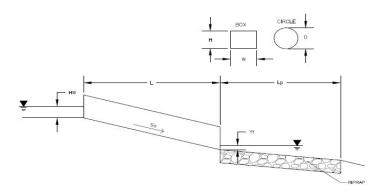
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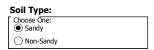
# DETERMINATION OF CULVERT HEADWATER AND OUTLET PROTECTION

MHFD-Culvert, Version 4.00 (May 2020)

Project: RETREAT AT TIMBERRIDGE FILING NO. 3

ID: DP 11 24 in. Outfall





inches

# Supercritical Flow! Using Adjusted Diameter to calculate protection type. 11

OR

Circular	Cuivert:		
	Barrel Diameter i	n	Tr

Design Discharge

nches 24 Inlet Edge Type (Choose from pull-down list) Grooved Edge Projecting

OR: Box Culvert:

Design Information:

......

Barrel Height (Rise) in Feet H (Rise) = Barrel Width (Span) in Feet W (Span) =

Inlet Edge Type (Choose from pull-down list)

Number of Barrels # Barrels = Inlet Elevation Elev IN = 7246 Outlet Elevation OR Slope Elev OUT = 7234 ft Culvert Length 175.97 ft 1 = Manning's Roughness n = 0.013 Bend Loss Coefficient k<sub>b</sub> = n Exit Loss Coefficient k<sub>x</sub> = Tailwater Surface Elevation ft Max Allowable Channel Velocity 5 V = ft/s

## Calculated Results:

Culvert Cross Sectional Area Available

Culvert Normal Depth Culvert Critical Depth Froude Number **Entrance Loss Coefficient** Friction Loss Coefficient

Sum of All Loss Coefficients

A =	3.14	π
Y <sub>n</sub> =	0.58	ft
Y <sub>c</sub> =	1.19	ft
Fr =	3.92	Supercritical!
k <sub>e</sub> =	0.20	
k <sub>e</sub> = k <sub>f</sub> =	2.17	
k. =	3.37	ft

### Headwater:

Inlet Control Headwater Outlet Control Headwater

**Design Headwater Elevation** Headwater/Diameter OR Headwater/Rise Ratio

$HW_{I} =$	1.66	ft
$HW_0 =$	N/A	ft
HW =	7247.66	ft
HW/D =	0.83	

# Outlet Control Headwater Approximation Method Inaccurate for Low Flow - Backwater Calculations Required

# Outlet Protection:

Flow/(Diameter^2.5) Tailwater Surface Height

Tailwater/Diameter Expansion Factor Flow Area at Max Channel Velocity Width of Equivalent Conduit for Multiple Barrels Length of Riprap Protection

Width of Riprap Protection at Downstream End

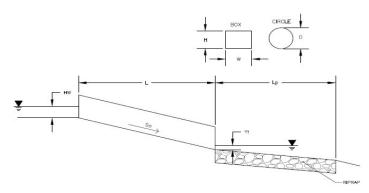
Adjusted Diameter for Supercritical Flow Minimum Theoretical Riprap Size

Nominal Riprap Size MHFD Riprap Type

$Q/D^2.5 =$	1.94	ft <sup>0.5</sup> /s
$Y_t =$	0.80	ft
Yt/D =	0.40	
$1/(2*tan(\Theta)) =$	5.80	
$A_t = W_{eq} =$	2.20	ft <sup>2</sup> ft
$W_{eq} =$	-	ft
L <sub>p</sub> = T =	6	ft
T =	4	ft
		_
Da =	1.29	ft

VL	
6	in
4	in
1.29	ft
	4 6

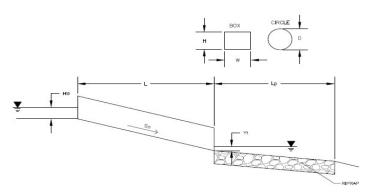
# DETERMINATION OF CULVERT HEADWATER AND OUTLET PROTECTION MHFD-Culvert, Version 4.00 (May 2020) Project: RETREAT AT TIMBERRIDGE FILING NO. 3 ID: 30 in. Future Pond Outfall





	Supercritica	al Flow! Using Adjusted Dia	meter to cale	culate protection type.
Design Infor	mation:			
	Design Discharge	Q =	45	cfs
				<del></del>
Circular Culve	ert:			
	Barrel Diameter in Inches	D =	30	inches
	Inlet Edge Type (Choose from pull-down list)	Grooved	Edge Projectii	ng
<u>OF</u>	<u>t:</u>			
Box Culvert:			OR	
	Barrel Height (Rise) in Feet	H (Rise) =		ft
	Barrel Width (Span) in Feet	W (Span) =		ft
	Inlet Edge Type (Choose from pull-down list)	· · / <u>-</u>		_
	3 // (			
	Number of Barrels	# Barrels =	1	
	Inlet Elevation	Elev IN =	7240.31	T <sub>ft</sub>
	Outlet Elevation OR Slope	Elev OUT =	7235	T <sub>ft</sub>
	Culvert Length	L =	265.77	mt.
	Manning's Roughness	n =	0.013	<b>-</b>
	Bend Loss Coefficient	k <sub>b</sub> =	0	
	Exit Loss Coefficient	k <sub>x</sub> =	1	
	Tailwater Surface Elevation	Y <sub>t, Elevation</sub> =	-	H <sub>ft</sub>
	Max Allowable Channel Velocity	V =	5	ft/s
	Tak Tale Table Shall be Velocity			.43
alculated R	esults:			
alculated is	Culvert Cross Sectional Area Available	A =	4.91	∏ft²
	Culvert Normal Depth	Y <sub>n</sub> =	1.65	⊣'t
	Culvert Critical Depth	Y <sub>c</sub> =	2.23	⊢¦t
	Froude Number	r = -	1.91	Supercritical!
	Entrance Loss Coefficient	k <sub>e</sub> =	0.20	Supercriticals
	Friction Loss Coefficient	k <sub>f</sub> =	2.44	_
	Sum of All Loss Coefficients		3.64	—  ft
	Sulli of All Loss Coefficients	k <sub>s</sub> =	3.04	
leadwater:				
icaawatei.	Inlet Control Headwater	$HW_{T} =$	4.36	∏ft.
	Outlet Control Headwater	HW <sub>0</sub> =	N/A	⊢¦t
	Design Headwater Elevation	HW =	7244.67	⊣ <u>"t</u>
	Headwater/Diameter <u>OR</u> Headwater/Rise Ratio	HW/D =	1.74	<b>⊣"</b>
	Outlet Control Headwater Approximation Metho	· —		Calculations Pequired
Outlet Protect		ou inaccurate for Low Flow	Dackwater	Calculations (Cquired
aciec i roceci	Flow/(Diameter^2.5)	Q/D^2.5 =	4.55	ft <sup>0.5</sup> /s
	Tailwater Surface Height	Ψ <sub>t</sub> = -	1.00	- it /3
	Tailwater/Diameter	Yt/D =	0.40	<b>⊣</b> "
	Expansion Factor	$1/(2*tan(\Theta)) =$	2.90	$\dashv$
	Flow Area at Max Channel Velocity	$A_t = \begin{bmatrix} 1/(2 \cdot tan(O)) - \\ A_t = \end{bmatrix}$	9.00	Ht <sup>2</sup>
	Width of Equivalent Conduit for Multiple Barrels	W <sub>eq</sub> =	-	
	Length of Riprap Protection		19	⊢¦t
	Width of Riprap Protection at Downstream End	L <sub>p</sub> =   T =	10	⊣'t
	width of Riprap Protection at Downstream End	1=_	10	
	Adjusted Diameter for Supercritical Flow	Da =	2.08	□ <sub>ft</sub>
	Minimum Theoretical Riprap Size	_	10	nπ in
	Nominal Riprap Size	d <sub>50</sub> min=	12	—lin
	Norminal Kipi ap Size	d <sub>50</sub> nominal=	12	
	MHFD Riprap Type	Type =	М	

# DETERMINATION OF CULVERT HEADWATER AND OUTLET PROTECTION MHFD-Culvert, Version 4.00 (May 2020) Project: RETREAT AT TIMBERRIDGE FILING NO. 3 ID: Rain Garden 1 Outfall

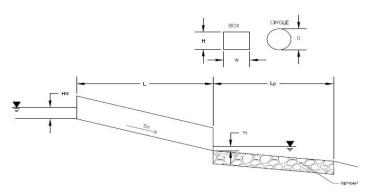




-	Supercritical	Flow! Using Adjusted Dia	ameter to calc	ulate protection type.
Design Infor	mation:	_		
	Design Discharge	Q =	10	cfs
Circular Culve	ert:	_		_
	Barrel Diameter in Inches	D =	24	inches
	Inlet Edge Type (Choose from pull-down list)	Groove	d Edge Projectin	g
<u> </u>	<u>ર:</u>			
Box Culvert:			OR	
	Barrel Height (Rise) in Feet	H (Rise) =		ft
	Barrel Width (Span) in Feet	W (Span) =		ft
	Inlet Edge Type (Choose from pull-down list)			
		_		
	Number of Barrels	# Barrels =	1	
	Inlet Elevation	Elev IN =	7234.25	ft
	Outlet Elevation OR Slope	Elev OUT =	7234	ft
	Culvert Length	L =	25.39	ft
	Manning's Roughness	n =	0.013	
	Bend Loss Coefficient	k <sub>b</sub> =	0	
	Exit Loss Coefficient	k <sub>x</sub> =	1	
	Tailwater Surface Elevation	Y <sub>t, Elevation</sub> =		<b>⊤</b> ft
	Max Allowable Channel Velocity	V =	5	ft/s
	, , , , , , , , , , , , , , , , , , ,	_	-	<b>_</b>
Calculated R	tesults:			
	Culvert Cross Sectional Area Available	A =	3.14	∏ft²
	Culvert Normal Depth	Y <sub>n</sub> =	0.93	⊣ft
	Culvert Critical Depth	Y <sub>c</sub> =	1.13	⊣rt
	Froude Number	Fr =	1.44	Supercritical!
	Entrance Loss Coefficient	k <sub>e</sub> =	0.20	Supercriticals
	Friction Loss Coefficient	k <sub>f</sub> =	0.31	-
	Sum of All Loss Coefficients	k <sub>s</sub> =	1.51	-  <sub>ft</sub>
	Sum of All 2033 Coefficients	N <sub>S</sub> —	1.51	
Headwater:				
	Inlet Control Headwater	HW <sub>T</sub> =	1.63	∏ft
	Outlet Control Headwater	HW <sub>o</sub> =	1.55	
	Design Headwater Elevation	HW =	7235.88	⊣ <del>n</del>
	Headwater/Diameter <u>OR</u> Headwater/Rise Ratio	HW/D =	0.81	<b>⊣"</b>
	<u> </u>	,5 -	0.01	
Outlet Protec	tion:			
	Flow/(Diameter^2.5)	Q/D^2.5 =	1.77	ft <sup>0.5</sup> /s
	Tailwater Surface Height	Y <sub>t</sub> =	0.80	⊣ <sub>ft</sub> ,,
	Tailwater/Diameter	Yt/D = -	0.40	<b>1</b>
	Expansion Factor	1/(2*tan(Θ)) =	5.96	1
	Flow Area at Max Channel Velocity	A <sub>t</sub> =	2.00	
	Width of Equivalent Conduit for Multiple Barrels	W <sub>eq</sub> =	-	ft
	Length of Riprap Protection	L <sub>p</sub> =	6	⊣'t
	Width of Riprap Protection at Downstream End	T =	4	ft
	Adjusted Diameter for Cunevarities Flour	r-	1.47	٦
	Adjusted Diameter for Supercritical Flow	Da = _	1.47	ft in
	Minimum Theoretical Riprap Size	d <sub>50</sub> min=	3	
	Nominal Riprap Size MHFD Riprap Type	d <sub>50</sub> nominal=	6 <b>VL</b>	in
	MHFD KINTAN IVNA	Type =	VI	1

# DETERMINATION OF CULVERT HEADWATER AND OUTLET PROTECTION

MHFD-Culvert, Version 4.00 (May 2020)
Project: RETREAT AT TIMBERRIDGE FILING NO. 3
1D: 18 in. RG 1 Pond Outfall to Channel





		upercritical Flow! Using Adjusted D	iameter to cald	culate protection type.
Design Infor	mation:			
	Design Discharge	Q =	7	cfs
	3			<u> </u>
Circular Culve	ert:			
	Barrel Diameter in Inches	D = [	18	inches
	Inlet Edge Type (Choose from pull-down list)	Groov	ed Edge Projectir	
OF	3 ,1 \		9	-5
Box Culvert:	<u>=</u>		OR	
DOX CUIVEIG	Barrel Height (Rise) in Feet	H (Rise) =	- Oit	ft
	Barrel Width (Span) in Feet	W (Span) =		ft
	Inlet Edge Type (Choose from pull-down list)	W (Spail) - [		
	Thet Lage Type (Choose from pail-down list)			
	Number of Barrels	# Barrels =	1	
	Inlet Elevation	# barreis =   Elev IN =	7231.2	
		-		ft
	Outlet Elevation <b>OR</b> Slope	Elev OUT =	7229	ft
	Culvert Length	L =	48.1	ft
	Manning's Roughness	n =	0.013	
	Bend Loss Coefficient	$k_b = 1$	0	
	Exit Loss Coefficient	$k_x = 1$	1	
	Tailwater Surface Elevation	$Y_{t, Elevation} = $		ft
	Max Allowable Channel Velocity	V =	5	ft/s
Calculated R	esults:			
	Culvert Cross Sectional Area Available	A =	1.77	ft²
	Culvert Normal Depth	$Y_n = $	0.72	ft
	Culvert Critical Depth	$Y_c = $	1.02	∏ft
	Froude Number	Fr =	1.95	Supercritical!
	Entrance Loss Coefficient	k <sub>e</sub> =	0.20	
	Friction Loss Coefficient	k <sub>f</sub> =	0.87	
	Sum of All Loss Coefficients	k <sub>s</sub> =	2.07	T <sub>ft</sub>
	Sum of All 2005 coefficients	NS — [	2.07	
Headwater:				
ricadwater.	Inlet Control Headwater	$HW_{\tau} = [$	1.52	∏ft
	Outlet Control Headwater	HW <sub>O</sub> =	N/A	⊢lt.
	Design Headwater Elevation	HW =	7232.72	⊢¦t
	Headwater/Diameter <u>OR</u> Headwater/Rise R	l de la companya de	1.01	<b>⊣</b> ''`
		- 1		Coloulations Beauticad
Outlet Protect	Outlet Control Headwater Approximat	tion method inaccurate for Low Flo	w - Backwater	Calculations Kequired
Outlet Protec		0/543.5	2.54	ft <sup>0.5</sup> /s
	Flow/(Diameter^2.5)	Q/D^2.5 =	2.54	
	Tailwater Surface Height	$Y_t =$	0.60	ft
	Tailwater/Diameter	Yt/D =	0.40	
	Expansion Factor	1/(2*tan(Θ)) =	5.01	
	Flow Area at Max Channel Velocity	$A_t =$	1.40	ft <sup>2</sup>
	Width of Equivalent Conduit for Multiple Barrels	$W_{eq} = $	-	ft
	Length of Riprap Protection	$L_p =  $	5	ft
	Width of Riprap Protection at Downstream I	End T = [	3	ft
	Adjusted Diameter for Supercritical Flow	Da =	1.11	∏ft
	Minimum Theoretical Riprap Size	d <sub>50</sub> min=	3	in in
	Nominal Riprap Size	$d_{50}$ nominal=	6	⊢lin
	Hommai Niprap Size	u <sub>50</sub> nonlinai – [	U	
	MHFD Riprap Type	Type =	٧L	

# Arroya Lane - North side of roadway (Sta. 0+25 to Sta. 3+50)

	<b>Erosion Control Blanket (ECB)</b>	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	,
Design Flow (cfs)	3.0	3.0	3.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.5%	3.5%	3.5%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.5	3.3	1.5
Velocity (ft./sec.)	1.5	0.3	1.5
Allowed Flow (cfs)	7.6	67.6	8.9

# Arroya Lane - South side of roadway (Sta. 0+25 to Sta. 3+50)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	1.0	1.0	1.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.5%	3.5%	3.5%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.5	3.3	1.5
Velocity (ft./sec.)	0.5	0.1	0.5
Allowed Flow (cfs)	7.6	67.6	8.9

# Arroya Lane - North side of roadway (Sta. 3+50 to Sta. 7+25)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	5.0	5.0	5.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	2.0%	2.0%	2.0%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	0.9	1.9	0.9
Velocity (ft./sec.)	2.6	0.6	2.6
Allowed Flow (cfs)	5.7	51.1	6.7

# Arroya Lane - South side of roadway (Sta. 3+50 to Sta. 7+00)

	<b>Erosion Control Blanket (ECB)</b>	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	2.0	2.0	2.0
Permissible Shear (lbs/ft.2)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	2.0%	2.0%	2.0%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	0.9	1.9	0.9
Velocity (ft./sec.)	1.0	0.2	1.0
Allowed Flow (cfs)	5.7	51.1	6.7

# Arroya Lane - North side of roadway (Sta. 13+75 to Sta. 17+50)

	<b>Erosion Control Blanket (ECB)</b>	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent) w/ rock check dams	(Permanent)	
Design Flow (cfs)	6.0	6.0	6.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	5.3%	5.3%	5.3%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	2.3	4.9	2.3
Velocity (ft./sec.)	3.1	0.7	3.1
Allowed Flow (cfs)	9.3	82.8	10.8

# Arroya Lane - South side of roadway (Sta. 14+75 to Sta. 17+50)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent) w/ rock check dams	(Permanent)	
Design Flow (cfs)	2.0	2.0	2.0
Permissible Shear (lbs/ft.2)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	5.3%	5.3%	5.3%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	2.3	4.9	2.3
Velocity (ft./sec.)	1.0	0.2	1.0
Allowed Flow (cfs)	9.3	82.8	10.8

# Arroya Lane - North side of roadway (Sta. 17+50 to Sta. 22+50)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	4.0	4.0	4.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.8%	3.8%	3.8%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.7	3.6	1.7
Velocity (ft./sec.)	2.0	0.4	2.0
Allowed Flow (cfs)	7.9	70.4	9.2

# Arroya Lane - South side of roadway (Sta. 17+50 to Sta. 22+50)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass line
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	1.0	1.0	1.0
Permissible Shear (lbs/ft. <sup>2</sup> )	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.8%	3.8%	3.8%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	9.00	1.96
Wetted Perimeter (ft.)	5.78	12.39	5.78
Hydraulic Radius	0.34	0.73	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	1.5	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.7	3.6	1.7
Velocity (ft./sec.)	0.5	0.1	0.5
Allowed Flow (cfs)	7.9	70.4	9.2

# Aspen Valley Rd. - West side of roadway (Sta. 25+67 to Sta. 29+00)

Frosion Control Blanket (FCB)	Turf Reinforcement Mat (TPM)	Revegetation - Grass lined
		(Native Seed Mix)
		(Native Seed Mix)
(Permanent)	(Permanent)	
8.0	8.0	8.0
2.0	8.0	0.1
8.0	16.0	3.0
1	1	1
2.5%	2.5%	2.5%
V-Ditch	V-Ditch	V-Ditch
1.96	9.00	1.96
5.78	12.39	5.78
0.34	0.73	0.34
0.035	0.030	0.030
0.7	1.5	0.7
1.1	2.3	1.1
4.1	0.9	4.1
6.4	57.1	7.5
	2.0 8.0 1 2.5%  V-Ditch  1.96 5.78 0.34  0.035  0.7	(North American Green - SC150)     (North American Green - P300)       (Permanent)     (Permanent)       8.0     8.0       2.0     8.0       8.0     16.0       1     1       2.5%     2.5%       V-Ditch     V-Ditch       1.96     9.00       5.78     12.39       0.34     0.73       0.035     0.030       0.7     1.5       1.1     2.3       4.1     0.9

# Aspen Valley Rd. - East side of roadway (Sta. 25+67 to Sta. 29+00)

Fresion Control Blanket (FCB)	Turf Reinforcement Mat (TRM) (North American Green - P300)	Revegetation - Grass lined (Native Seed Mix)
(North American Green - SC150)		
1.0	1.0	1.0
2.0	8.0	0.1
8.0	16.0	3.0
1	1	1
2.5%	2.5%	2.5%
V-Ditch	V-Ditch	V-Ditch
1.96	9.00	1.96
5.78	12.39	5.78
0.34	0.73	0.34
0.035	0.030	0.030
0.7	1.5	0.7
1.1	2.3	1.1
0.5	0.1	0.5
6.4	57.1	7.5
	1.0 2.0 8.0 1 2.5%  V-Ditch  1.96 5.78 0.34  0.035  0.7	(North American Green - SC150)     (North American Green - P300)       (Permanent)     (Permanent)       1.0     1.0       2.0     8.0       8.0     16.0       1     1       2.5%     2.5%       V-Ditch     V-Ditch       1.96     9.00       5.78     12.39       0.34     0.73       0.035     0.030       0.7     1.5       1.1     2.3       0.5     0.1

# Hawks Hill Ct. - Northeast side of roadway (Sta. 1+25 to Sta. 8+50)

	Erosion Control Blanket (ECB) (North American Green - SC150)	Turf Reinforcement Mat (TRM) (North American Green - P300)	Revegetation - Grass lined (Native Seed Mix)
Design Flow (cfs)	15.0	15.0	15.0
Permissible Shear (lbs/ft.2)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	1.2%	1.2%	1.2%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	9.00	9.00	9.00
Wetted Perimeter (ft.)	12.39	12.39	12.39
Hydraulic Radius	0.73	0.73	0.73
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	1.5	1.5	1.5
Calculations:			
Shear Stress (lbs/ft.²)	1.1	1.1	1.1
Velocity (ft./sec.)	1.7	1.7	1.7
Allowed Flow (cfs)	33.9	39.6	39.6

# Hawks Hill Ct. - Southwest side of roadway (Sta. 1+25 to Sta. 9+50)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM) (North American Green - P300)	Revegetation - Grass lined (Native Seed Mix)
	(North American Green - SC150)		
Given:	(Permanent)	(Permanent)	
	· ·		
Design Flow (cfs)	2.0	2.0	2.0
Permissible Shear (lbs/ft.2)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	1.2%	1.2%	1.2%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	1.96	1.96
Wetted Perimeter (ft.)	5.78	5.78	5.78
Hydraulic Radius	0.34	0.34	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	0.7	0.7
Calculations:			
Shear Stress (lbs/ft. <sup>2</sup> )	0.5	0.5	0.5
Velocity (ft./sec.)	1.0	1.0	1.0
velocity (it./3cc.)	1.0	1.0	1.0
Allowed Flow (cfs)	4.4	5.2	5.2
, ,			

### **ROADSIDE DITCH CALCUALTIONS**

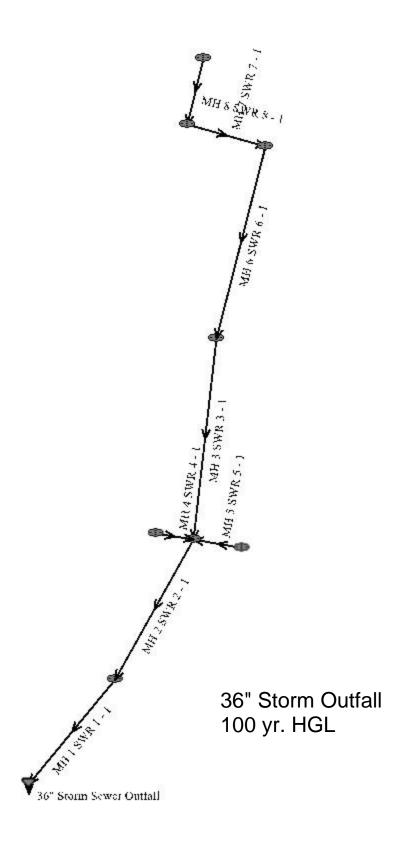
### Hawks Hill Ct. - North side of roadway (Sta. 8+50 to HP in Cul-de-sac)

	<b>Erosion Control Blanket (ECB)</b>	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	6.0	6.0	6.0
Permissible Shear (lbs/ft.²)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.5%	3.5%	3.5%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	1.96	1.96
Wetted Perimeter (ft.)	5.78	5.78	5.78
Hydraulic Radius	0.34	0.34	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	0.7	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.5	1.5	1.5
Velocity (ft./sec.)	3.1	3.1	3.1
Allowed Flow (cfs)	7.6	8.9	8.9

### **ROADSIDE DITCH CALCUALTIONS**

### Hawks Hill Ct. - South side of roadway (Sta. 10+10 to HP in Cul-de-sac)

	Erosion Control Blanket (ECB)	Turf Reinforcement Mat (TRM)	Revegetation - Grass lined
	(North American Green - SC150)	(North American Green - P300)	(Native Seed Mix)
Given:	(Permanent)	(Permanent)	
Design Flow (cfs)	2.0	2.0	2.0
Permissible Shear (lbs/ft.²)	2.0	8.0	0.1
Permissible Velocity (ft./sec.)	8.0	16.0	3.0
Safety Factor	1	1	1
Ditch Slope (Max.)	3.5%	3.5%	3.5%
Ditch Section (24 in. depth)	V-Ditch	V-Ditch	V-Ditch
Flow Area (ft. <sup>2</sup> )	1.96	1.96	1.96
Wetted Perimeter (ft.)	5.78	5.78	5.78
Hydraulic Radius	0.34	0.34	0.34
Mannings n	0.035	0.030	0.030
Depth of Flow (max.)	0.7	0.7	0.7
Calculations:			
Shear Stress (lbs/ft.²)	1.5	1.5	1.5
Velocity (ft./sec.)	1.0	1.0	1.0
Allowed Flow (cfs)	7.6	8.9	8.9



## 36" Storm Sewer Outfall - 100yr HGL

## **System Input Summary**

#### **Rainfall Parameters**

Rainfall Return Period: 100

Rainfall Calculation Method: Table

Time	Intensity
5	8.68
10	6.93
20	5.19
30	4.16
40	3.44
60	2.42
120	0.67

#### **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): 18.00 Maximum Depth to Rise Ratio: 0.90 Maximum Flow Velocity (fps): 18.0 Minimum Flow Velocity (fps): 2.0

### **Backwater Calculations:**

**Tailwater Elevation (ft):** 7206.90

## **Manhole Input Summary:**

		Giv	en Flow			Sub Basir	ı Informat	ion		
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)
36" Storm Sewer Outfall	7211.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 1 SWR 1 - 1	7223.66	55.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 2 SWR 2 - 1	7226.71	55.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 3 SWR 3 - 1	7234.90	30.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 6 SWR 6 - 1	7251.18	30.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 7 SWR 7 - 1	7251.18	30.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 8 SWR 8 - 1	7246.50	30.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 4 SWR 4 - 1	7227.02	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 5 SWR 5 - 1	7227.02	22.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

# **Manhole Output Summary:**

		Loca	al Contrib	ution			Total D	esign 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
36" Storm Sewer Outfall	0.00	0.00	0.00	0.00	0.00	58.61	0.94	0.54	55.00	
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	55.00	
MH 2 SWR 2 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	55.00	
MH 3 SWR 3 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	30.00	
MH 6 SWR 6 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	30.00	
MH 7 SWR 7 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	30.00	
MH 8 SWR 8 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	30.00	
MH 4 SWR 4 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.00	
MH 5 SWR 5 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	22.00	

# **Sewer Input Summary:**

Elevation					Loss C	Coefficio	ents	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	252.38	7205.10	4.0	7215.20	0.013	0.03	1.00	CIRCULAR	36.00 in	36.00 in	
MH 2 SWR 2 - 1	283.77	7215.70	1.9	7221.01	0.013	0.05	1.00	CIRCULAR	36.00 in	36.00 in	

MH 3 SWR 3 - 1	426.88	7221.46	1.7	7228.72	0.013	0.05	1.00	CIRCULAR	30.00 in	30.00 in
MH 6 SWR 6 - 1	352.96	7229.18	3.2	7240.47	0.013	0.05	1.00	CIRCULAR	30.00 in	30.00 in
MH 7 SWR 7 - 1	88.50	7240.95	1.3	7242.10	0.013	1.32	1.00	CIRCULAR	30.00 in	30.00 in
MH 8 SWR 8 - 1	40.03	7242.60	1.0	7243.00	0.013	1.32	1.00	CIRCULAR	30.00 in	30.00 in
MH 4 SWR 4 - 1	5.17	7222.51	5.8	7222.81	0.013	1.32	0.00	CIRCULAR	18.00 in	18.00 in
MH 5 SWR 5 - 1	25.17	7222.01	3.5	7222.88	0.013	1.32	0.00	CIRCULAR	24.00 in	24.00 in

## **Sewer Flow Summary:**

	Full Flov	w Capacity	Critic	cal Flow		Noi	rmal 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	133.76	18.92	28.88	9.05	16.08	18.00	3.13	Supercritical	55.00	0.00	
MH 2 SWR 2 - 1	91.48	12.94	28.88	9.05	20.12	13.53	2.04	Supercritical	55.00	0.00	
MH 3 SWR 3 - 1	53.62	10.92	22.40	7.63	16.04	11.23	1.91	Supercritical Jump	30.00	45.74	
MH 6 SWR 6 - 1	73.57	14.99	22.40	7.63	13.34	14.22	2.72	Supercritical	30.00	0.00	
MH 7 SWR 7 - 1	46.89	9.55	22.40	7.63	17.44	10.13	1.63	Supercritical	30.00	0.00	
MH 8 SWR 8 - 1	41.13	8.38	22.40	7.63	19.02	9.14	1.38	Supercritical	30.00	0.00	
MH 4 SWR 4 - 1	25.37	14.35	9.18	4.41	4.83	10.47	3.44	Pressurized	4.00	5.17	
MH 5 SWR 5 - 1	42.17	13.42	20.10	7.83	12.30	13.57	2.65	Pressurized	22.00	25.17	

- 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:**

			Exis	sting	Calcı	ılated		Used		
Element Name	Peak Flow (cfs)	Cross Section	Rise	Span	Rise	Span	Rise	Span	Area (ft^2)	Comment
MH 1 SWR 1 - 1	55.00	CIRCULAR	36.00 in	36.00 in	27.00 in	27.00 in	36.00 in	36.00 in	7.07	
MH 2 SWR 2 - 1	55.00	CIRCULAR	36.00 in	36.00 in	30.00 in	30.00 in	36.00 in	36.00 in	7.07	
MH 3 SWR 3 - 1	30.00	CIRCULAR	30.00 in	30.00 in	27.00 in	27.00 in	30.00 in	30.00 in	4.91	
MH 6 SWR 6 - 1	30.00	CIRCULAR	30.00 in	30.00 in	24.00 in	24.00 in	30.00 in	30.00 in	4.91	
MH 7 SWR 7 - 1	30.00	CIRCULAR	30.00 in	30.00 in	27.00 in	27.00 in	30.00 in	30.00 in	4.91	
MH 8 SWR 8 - 1	30.00	CIRCULAR	30.00 in	30.00 in	27.00 in	27.00 in	30.00 in	30.00 in	4.91	
MH 4 SWR 4 - 1	4.00	CIRCULAR	18.00 in	1.77						
MH 5 SWR 5 - 1	22.00	CIRCULAR	24.00 in	24.00 in	21.00 in	21.00 in	24.00 in	24.00 in	3.14	

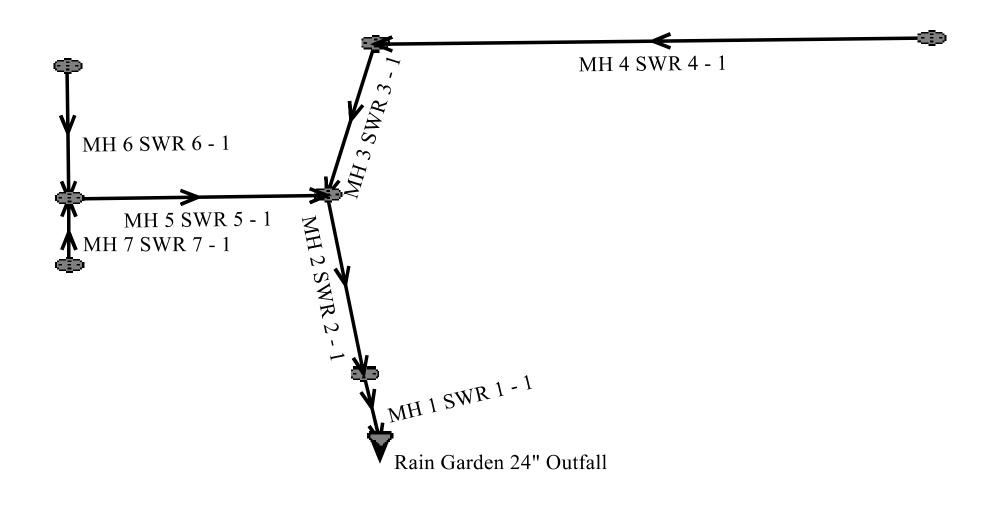
- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

## **Grade Line Summary:**

**Tailwater Elevation (ft):** 7206.90

	Invert 1	Elev.		am Manhole osses	HG	L	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	7205.10	7215.20	0.00	0.00	7206.90	7217.60	7211.47	7.40	7218.87	
MH 2 SWR 2 - 1	7215.70	7221.01	0.05	0.00	7217.65	7223.42	7220.22	4.47	7224.69	
MH 3 SWR 3 - 1	7221.46	7228.72	0.03	0.36	7224.50	7230.59	7225.08	6.41	7231.49	
MH 6 SWR 6 - 1	7229.18	7240.47	0.03	0.00	7230.62	7242.34	7233.43	9.81	7243.24	
MH 7 SWR 7 - 1	7240.95	7242.10	0.77	0.00	7243.42	7243.97	7244.01	0.86	7244.87	
MH 8 SWR 8 - 1	7242.60	7243.00	0.77	0.00	7245.05	7245.05	7245.64	0.17	7245.80	
MH 4 SWR 4 - 1	7222.51	7222.81	0.11	0.00	7224.71	7224.72	7224.79	0.01	7224.80	
MH 5 SWR 5 - 1	7222.01	7222.88	1.01	0.00	7224.93	7225.17	7225.69	0.24	7225.93	

- 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.



RAIN GARDEN 1 100YR HGL

## Rain Garden 1 Storm System 100-yr. HGL

## **System Input Summary**

### **Rainfall Parameters**

**Rainfall Return Period: 100** 

Rainfall Calculation Method: Table

Time	Intensity
5	8.68
10	6.93
20	5.19
30	4.16
40	3.44
60	2.42
120	0.67

#### **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): 18.00 Maximum Depth to Rise Ratio: 0.90 Maximum Flow Velocity (fps): 18.0 Minimum Flow Velocity (fps): 2.0

### **Backwater Calculations:**

**Tailwater Elevation (ft):** 7236.08

## **Manhole Input Summary:**

		Giv	en Flow			Sub Basii	n Informat	ion		
Element Name	Ground Elevation (ft)	Total Known Flow (cfs)	Local Contribution (cfs)	Drainage Area (Ac.)	Kunon	5yr Coefficient	Langth	Overland Slope (%)	Gutter Length (ft)	Gutter Velocity (fps)
Rain Garden 24" Outfall	7018.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 1 SWR 1 - 1	7242.00	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 2 SWR 2 - 1	7249.70	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 3 SWR 3 - 1	7250.10	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 4 SWR 4 - 1	7253.50	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 5 SWR 5 - 1	7248.50	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 6 SWR 6 - 1	7249.00	3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 7 SWR 7 - 1	7249.00	3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

# **Manhole Output Summary:**

		Local	Contril	bution			<b>Total De</b>	sign 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
Rain Garden 24" Outfall	0.00	0.00	0.00	0.00	0.00	43.33	0.23	0.13	10.00	Surface Water Present (Upstream)
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	10.00	Surface Water Present (Downstream)
MH 2 SWR 2 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	10.00	
MH 3 SWR 3 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00	
MH 4 SWR 4 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00	
MH 5 SWR 5 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00	
MH 6 SWR 6 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	
MH 7 SWR 7 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	

# **Sewer Input Summary:**

		Ele	Loss C	Coefficio	ents	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	25.39	7234.00	1.0	7234.25	0.013	0.03	1.00	CIRCULAR	24.00 in	24.00 in
MH 2 SWR 2 - 1	59.20	7234.25	8.0	7238.99	0.013	0.05	1.00	CIRCULAR	24.00 in	24.00 in
MH 3 SWR 3 - 1	35.61	7242.00	6.0	7244.14	0.013	0.05	1.00	CIRCULAR	18.00 in	18.00 in

MH 4 SWR 4 - 1 200.6	5 7244.64	1.5	7247.55	0.013	1.06	1.00	CIRCULAR	18.00 in	18.00 in
MH 5 SWR 5 - 1 115.5	2 7241.55	1.8	7243.63	0.013	1.19	0.00	CIRCULAR	24.00 in	24.00 in
MH 6 SWR 6 - 1 24.6	7 7244.13	1.5	7244.50	0.013	1.32	0.00	CIRCULAR	18.00 in	18.00 in
MH 7 SWR 7 - 1 8.67	7244.13	4.3	7244.50	0.013	1.32	0.00	CIRCULAR	18.00 in	18.00 in

## **Sewer Flow Summary:**

	Full Flo	w Capacity	Critic	cal Flow		Noi	rmal 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	22.68	7.22	13.58	5.46	11.15	6.99	1.46	Supercritical Jump	10.00	10.41	
MH 2 SWR 2 - 1	64.16	20.42	13.58	5.46	6.41	14.85	4.24	Supercritical	10.00	0.00	
MH 3 SWR 3 - 1	25.80	14.60	11.35	5.11	5.91	11.89	3.50	Supercritical	6.00	0.00	
MH 4 SWR 4 - 1	12.68	7.18	11.35	5.11	8.71	7.08	1.66	Supercritical	6.00	0.00	
MH 5 SWR 5 - 1	30.43	9.69	10.39	4.60	7.23	7.53	2.01	Supercritical	6.00	0.00	
MH 6 SWR 6 - 1	12.90	7.30	7.90	4.02	5.91	5.95	1.75	Supercritical	3.00	0.00	
MH 7 SWR 7 - 1	21.84	12.36	7.90	4.02	4.51	8.67	2.96	Supercritical	3.00	0.00	

- 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:**

			Exis	sting	Calcı	ılated		Used		
Element Name	Peak Flow (cfs)	Cross Section	Rise	Span	Rise	Span	Rise	Span	Area (ft^2)	Comment
MH 1 SWR 1 - 1	10.00	CIRCULAR	24.00 in	24.00 in	18.00 in	18.00 in	24.00 in	24.00 in	3.14	
MH 2 SWR 2 - 1	10.00	CIRCULAR	24.00 in	24.00 in	18.00 in	18.00 in	24.00 in	24.00 in	3.14	
MH 3 SWR 3 - 1	6.00	CIRCULAR	18.00 in	1.77						
MH 4 SWR 4 - 1	6.00	CIRCULAR	18.00 in	1.77						
MH 5 SWR 5 - 1	6.00	CIRCULAR	24.00 in	24.00 in	18.00 in	18.00 in	24.00 in	24.00 in	3.14	
MH 6 SWR 6 - 1	3.00	CIRCULAR	18.00 in	1.77						
MH 7 SWR 7 - 1	3.00	CIRCULAR	18.00 in	1.77						

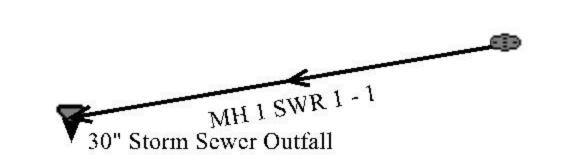
- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

## **Grade Line Summary:**

**Tailwater Elevation (ft):** 7236.08

	Invert Elev.			eam Manhole osses	HG	L	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	7234.00	7234.25	0.00	0.00	7236.08	7236.09	7236.24	0.02	7236.26	
MH 2 SWR 2 - 1	7234.25	7238.99	0.01	0.00	7236.10	7240.12	7238.21	2.37	7240.58	
MH 3 SWR 3 - 1	7242.00	7244.14	0.01	0.00	7242.50	7245.09	7244.69	0.80	7245.49	
MH 4 SWR 4 - 1	7244.64	7247.55	0.19	0.00	7245.37	7248.50	7246.14	2.76	7248.90	
MH 5 SWR 5 - 1	7241.55	7243.63	0.07	0.00	7242.15	7244.50	7243.03	1.79	7244.82	
MH 6 SWR 6 - 1	7244.13	7244.50	0.06	0.00	7244.62	7245.16	7245.17	0.24	7245.41	
MH 7 SWR 7 - 1	7244.13	7244.50	0.06	0.00	7244.56	7245.60	7245.67	0.00	7245.67	

- 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.



## Future 30" Storm Sewer Outfall - 100yr HGL

## **System Input Summary**

### **Rainfall Parameters**

**Rainfall Return Period: 100** 

Rainfall Calculation Method: Table

Time	Intensity
5	8.68
10	6.93
20	5.19
30	4.16
40	3.44
60	2.42
120	0.67

#### **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): 18.00 Maximum Depth to Rise Ratio: 0.90 Maximum Flow Velocity (fps): 18.0 Minimum Flow Velocity (fps): 2.0

### **Backwater Calculations:**

Tailwater Elevation (ft): 7239.00

## **Manhole Input Summary:**

		Giv	en Flow	Sub Basin Information							
Element Name	Ground Elevation (ft)	Total Known Flow (cfs)	Local Contribution (cfs)	Drainage Area (Ac.)	Runoff Coefficient	5yr	Overland Length (ft)	Overland Slope (%)		Gutter Velocity (fps)	
30" Storm Sewer Outfall	7211.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
MH 1 SWR 1 - 1	7253.00	45.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

# **Manhole Output Summary:**

		Local	Contril	oution			<b>Total De</b>	sign Flow		
Element Name	Overland Gutter Basin Time Time (min) (min)		Intensity (in/hr)	· I Antrin		Intensity (in/hr)	Manhole Tc (min)	Peak Flow (cfs)	Comment	
30" Storm Sewer Outfall	0.00	0.00	0.00	0.00	0.00	53.65	0.84	0.48	45.00	Surface Water Present (Upstream)
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	45.00	Surface Water Present (Downstream)

# **Sewer Input Summary:**

		Ele	evation		Loss C	Coefficio	ents	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	265.77	7235.00	2.0	7240.32	0.013	0.03	1.00	CIRCULAR	30.00 in	30.00 in	

# **Sewer Flow Summary:**

	Full Flo	w Capacity	Critic	cal Flow		No	rmal Flow				
Elemei Name	Flow (cfs)	Velocity (fps)	Depth (in)	Velocity (fps)	Depth (in)		Froude Number	Flow Condition	Flow (cfs)	Surcharged Length (ft)	Comment

MH 1 SWR 1 - 1 58.16	11.85	26.70 9.75	19.81	13.08	1.91	Supercritical Jump 45.00	186.28	
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- 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:**

		Exis	ting	Calcu	ılated	Used				
Element Name	Peak Flow (cfs)	Cross Section	Rise	Span	Rise	Span	Rise	Span	Area (ft^2)	Comment
MH 1 SWR 1 - 1	45.00	CIRCULAR	30.00 in	4.91						

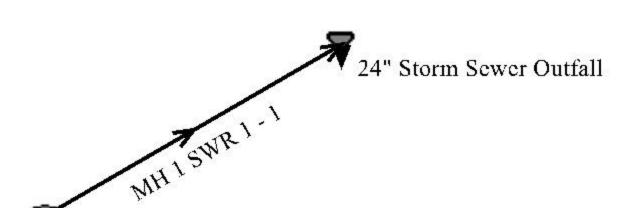
- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

## **Grade Line Summary:**

**Tailwater Elevation (ft):** 7239.00

	Invert 1	Elev.	Downstream Manhole Losses		HG	L	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	7235.00	7240.32	0.00	0.00	7239.00	7242.55	7240.30	3.72	7244.02	

- 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.



## 24" Storm Sewer Outfall - 100yr HGL

## **System Input Summary**

### **Rainfall Parameters**

**Rainfall Return Period: 100** 

Rainfall Calculation Method: Table

Time	Intensity
5	8.68
10	6.93
20	5.19
30	4.16
40	3.44
60	2.42
120	0.67

#### **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): 18.00 Maximum Depth to Rise Ratio: 0.90 Maximum Flow Velocity (fps): 18.0 Minimum Flow Velocity (fps): 2.0

### **Backwater Calculations:**

Tailwater Elevation (ft): 7239.00

## **Manhole Input Summary:**

		Giv	en Flow			Sub Basir	sin Information				
Element Name	Ground Elevation (ft)	Total Known Flow (cfs)	Local Contribution (cfs)	Drainage Area (Ac.)	Kunoii	5yr Coefficient	Overland Length (ft)	Overland Slope (%)		Gutter Velocity (fps)	
24" Storm Sewer Outfall	7211.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
MH 1 SWR 1 - 1	7251.00	11.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

# **Manhole Output Summary:**

		Local	Contril	oution			Total De	sign 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
24" Storm Sewer Outfall	0.00	0.00	0.00	0.00	0.00	7.56	1.45	0.84	11.00	Surface Water Present (Upstream)
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	11.00	Surface Water Present (Downstream)

# **Sewer Input Summary:**

	Elevation				Loss C	Coefficio	ents	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	175.97	7234.00	6.8	7246.00	0.013	0.03	1.00	CIRCULAR	24.00 in	24.00 in

# **Sewer Flow Summary:**

	Full Flow Capacity		Critical Flow			No	rmal Flow				
Elemei Name	Flow (cfs)	Velocity (fps)	Depth (in)	Velocity (fps)	Depth (in)		Froude Number	Flow Condition	Flow (cfs)	Surcharged Length (ft)	Comment

MH 1 SWR 1 - 1 59.23 18.86	4.27 5.65 7.01	14.42 3.92	Supercritical Jump 11.00	45.56	
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- 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:**

			Exis	sting	Calcı	ılated	Used			
Element Name	Peak Flow (cfs)	Cross Section	Rise	Span	Rise	Span	Rise	Span	Area (ft^2)	Comment
MH 1 SWR 1 - 1	11.00	CIRCULAR	24.00 in	24.00 in	18.00 in	18.00 in	24.00 in	24.00 in	3.14	

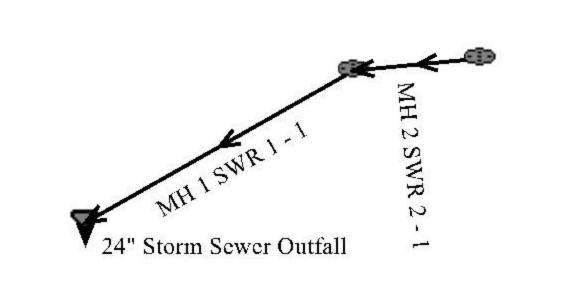
- Calculated diameter was determined by sewer hydraulic capacity rounded up to the nearest commercially available size.
- Sewer sizes should not decrease downstream.
- All hydraulics where calculated using the 'Used' parameters.

## **Grade Line Summary:**

**Tailwater Elevation (ft):** 7239.00

	Invert l	Elev.	Downstream Manhole Losses		HG	L	EGL			
Element Name	Downstream (ft)	Upstream (ft)	2001		Downstream (ft) (ft)		Downstream (ft)	Friction Loss (ft) Upstrea		
MH 1 SWR 1 - 1	7234.00	7246.00	0.00	0.00	7239.00	7247.19	7239.19	8.49	7247.68	

- 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.



## 24" Storm Sewer Outfall from TSB - 100yr HGL

## **System Input Summary**

### **Rainfall Parameters**

**Rainfall Return Period: 100** 

Rainfall Calculation Method: Table

Time	Intensity
5	8.68
10	6.93
20	5.19
30	4.16
40	3.44
60	2.42
120	0.67

#### **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): 18.00 Maximum Depth to Rise Ratio: 0.90 Maximum Flow Velocity (fps): 18.0 Minimum Flow Velocity (fps): 2.0

### **Backwater Calculations:**

**Tailwater Elevation (ft):** 7237.85

## **Manhole Input Summary:**

		Giv	en Flow	Sub Basin Information						
Element Name	Ground Elevation (ft)	Total Known Flow (cfs)	Local Contribution (cfs)	Drainage Area (Ac.)	Runoff Coefficient	5yr	Overland Length (ft)	Overland Slope (%)		Gutter Velocity (fps)
24" Storm Sewer Outfall	7211.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 1 SWR 1 - 1	7249.20	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MH 2 SWR 2 - 1	7250.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

# **Manhole Output Summary:**

		Local	Contril	oution			Total De	sign 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
24" Storm Sewer Outfall	0.00	0.00	0.00	0.00	0.00	76.66	0.46	0.26	35.00	Surface Water Present (Upstream)
MH 1 SWR 1 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	35.00	Surface Water Present (Downstream)
MH 2 SWR 2 - 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	35.00	

# **Sewer Input Summary:**

		Ele	evation		<b>Loss Coefficients</b>			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	175.81	7237.00	4.3	7244.59	0.013	0.03	1.00	CIRCULAR	24.00 in	24.00 in
MH 2 SWR 2 - 1	41.22	7244.59	1.0	7245.00	0.013	0.05	1.00	CIRCULAR	24.00 in	24.00 in

# **Sewer Flow Summary:**

Full Flow Capacity	<b>Critical Flow</b>	Normal Flow	

Element Name	Flow (cfs)	Velocity (fps)	Depth (in)	Velocity (fps)	Depth (in)		Froude Number	Flow Condition	Flow (cfs)	Surcharged Length (ft)	Comment
MH 1 SWR 1 - 1	47.13	15.00	23.04	11.29	15.40	16.43	2.75	Supercritical	35.00	0.00	
MH 2 SWR 2 - 1	22.63	7.20	24.00	11.14	24.00	11.14	0.00	Pressurized	35.00	41.22	

- 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	35.00	CIRCULAR	24.00 in	24.00 in	24.00 in	24.00 in	24.00 in	24.00 in	3.14	
MH 2 SWR 2 - 1	35.00	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):** 7237.85

	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	7237.00	7244.59	0.00	0.00	7238.28	7246.51	7242.48	6.01	7248.49
MH 2 SWR 2 - 1	7244.59	7245.00	0.10	0.00	7246.66	7247.64	7248.59	0.98	7249.57

- 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.

STORMWATER QUALITY CALCULATIONS



Design Procedure Form: Rain Garden (RG)									
		(Version 3.07, March 2018)	Sheet 1 of 2						
Designer:	Marc A. Whorton, P.E.								
Company: Date:	Classic Consulting September 20, 2023								
Project:	Retreat at TimberRidge Filing No. 3								
Location:	Arroya Lane (Rain Garden 1)								
		<del></del>							
1. Basin Stor	age Volume								
,	re Imperviousness of Tributary Area, I <sub>a</sub> if all paved and roofed areas upstream of rain garden)	l <sub>a</sub> = 22.4 %							
B) Tributa	ary Area's Imperviousness Ratio (i = I <sub>a</sub> /100)	i = 0.224							
	Quality Capture Volume (WQCV) for a 12-hour Drain Time CV= $0.8*(0.91*i^3-1.19*i^2+0.78*i)$	WQCV = 0.10 watershed inches							
D) Contril	outing Watershed Area (including rain garden area)	Area = 217,800 sq ft							
	Quality Capture Volume (WQCV) Design Volume (WQCV / 12) * Area	V <sub>WQCV</sub> = cu ft							
	atersheds Outside of the Denver Region, Depth of ge Runoff Producing Storm	d <sub>6</sub> = 0.42 in							
	atersheds Outside of the Denver Region, Quality Capture Volume (WQCV) Design Volume	V <sub>WQCV OTHER</sub> = 1,776 cu ft							
	nput of Water Quality Capture Volume (WQCV) Design Volume a different WQCV Design Volume is desired)	V <sub>WQCV USER</sub> = cu ft							
2. Basin Geo	ometry								
A) WQCV	Depth (12-inch maximum)	D <sub>WQCV</sub> = 12 in							
	arden Side Slopes (Z = 4 min., horiz. dist per unit vertical) " if rain garden has vertical walls)	Z = 4.00 ft / ft							
C) Mimim	um Flat Surface Area	A <sub>Min</sub> = 976 sq ft							
D) Actual	Flat Surface Area	A <sub>Actual</sub> = 1935 sq ft							
E) Area at	Design Depth (Top Surface Area)	$A_{Top} = 2718$ sq ft							
	arden Total Volume A <sub>Top</sub> + A <sub>Actual</sub> ) / 2) * Depth)	V <sub>T</sub> = 2,327 cu ft							
3. Growing N	/ledia	Choose One  18" Rain Garden Growing Media Other (Explain):							
4. Underdrai	n System	- G							
	derdrains provided?	Choose One  ● YES  ○ NO							
B) Underd	rain system orifice diameter for 12 hour drain time								
	i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y = 1.5 ft							
	ii) Volume to Drain in 12 Hours	Vol <sub>12</sub> = 1,776 cu ft							
	iii) Orifice Diameter, 3/8" Minimum	D <sub>O</sub> = 1 1/16 in							

UD-BMP\_v3.07 RG 1, RG 9/20/2023, 9:42 AM

	Design Procedu	re Form: Rain Garden (RG)
		Sheet 2 of 2
Designer:	Marc A. Whorton, P.E.	
Company:	Classic Consulting	
Date:	September 20, 2023	
Project:	Retreat at TimberRidge Filing No. 3	
Location:	Arroya Lane (Rain Garden 1)	
A) Is an	able Geomembrane Liner and Geotextile Separator Fabric impermeable liner provided due to proximity uctures or groundwater contamination?	Choose One ○ YES ■ NO
6. Inlet / Ou		Choose One Sheet Flow- No Energy Dissipation Required Concentrated Flow- Energy Dissipation Provided
7. Vegetatio	on	Choose One
8. Irrigation  A) Will th	ne rain garden be irrigated?	Choose One YES NO
Notes:		

UD-BMP\_v3.07 RG 1, RG 9/20/2023, 9:42 AM

## DETENTION BASIN STAGE-STORAGE TABLE BUILDER

MHFD-Detention, Version 4.06 (July 2022)

### Project: RETREAT AT TIMBERRIDGE FILING NO. 3

Optional User Overrides acre-feet acre-feet

inches

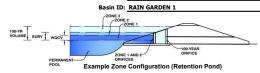
inches 2.25 inches 2.52 inches 3.85 inches

1.19 inches

1.50

1.75 inches

2.00



#### Watershed Information

RG	Selected BMP Type =
5.00 acres	Watershed Area =
1,000 ft	Watershed Length =
300 ft	Watershed Length to Centroid =
0.044 ft/ft	Watershed Slope =
22.40% percent	Watershed Imperviousness =
0.0% percent	Percentage Hydrologic Soil Group A =
100.0% percent	Percentage Hydrologic Soil Group B =
0.0% percent	Percentage Hydrologic Soil Groups C/D =
12.0 hours	Target WQCV Drain Time =
Denver - Capitol Building	Location for 1-hr Rainfall Denths =

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

the embedded Colorado Urban Hydrograph Procedure.							
Water Quality Capture Volume (WQCV) =	0.042	acre-feet					
Excess Urban Runoff Volume (EURV) =	0.112	acre-feet					
2-yr Runoff Volume (P1 = 1.19 in.) =	0.118	acre-feet					
5-yr Runoff Volume (P1 = 1.5 in.) =	0.209	acre-feet					
10-yr Runoff Volume (P1 = 1.75 in.) =	0.294	acre-feet					
25-yr Runoff Volume (P1 = 2 in.) =	0.430	acre-feet					
50-yr Runoff Volume (P1 = 2.25 in.) =	0.529	acre-feet					
100-yr Runoff Volume (P1 = 2.52 in.) =	0.664	acre-feet					
500-yr Runoff Volume (P1 = 3.85 in.) =	1.221	acre-feet					
Approximate 2-yr Detention Volume =	0.078	acre-feet					
Approximate 5-yr Detention Volume =	0.114	acre-feet					
Approximate 10-yr Detention Volume =	0.179	acre-feet					
Approximate 25-yr Detention Volume =	0.217	acre-feet					
Approximate 50-yr Detention Volume =	0.230	acre-feet					
Approximate 100-yr Detention Volume =	0.279	acre-feet					
		-					

### Define Zones and Basin Geometry

Zone 1 Volume (WQCV) =	0.042	acre-feet
Zone 2 Volume (100-year - Zone 1) =	0.238	acre-feet
Select Zone 3 Storage Volume (Optional) =		acre-feet
Total Detention Basin Volume =	0.279	acre-feet
Initial Surcharge Volume (ISV) =	N/A	ft <sup>3</sup>
Initial Surcharge Depth (ISD) =	N/A	ft
Total Available Detention Depth (H <sub>total</sub> ) =	user	ft
Depth of Trickle Channel ( $H_{TC}$ ) =	N/A	ft
Slope of Trickle Channel $(S_{TC}) =$	N/A	ft/ft
Slopes of Main Basin Sides (S <sub>main</sub> ) =	user	H:V
Basin Length-to-Width Ratio (R <sub>L/W</sub> ) =	user	

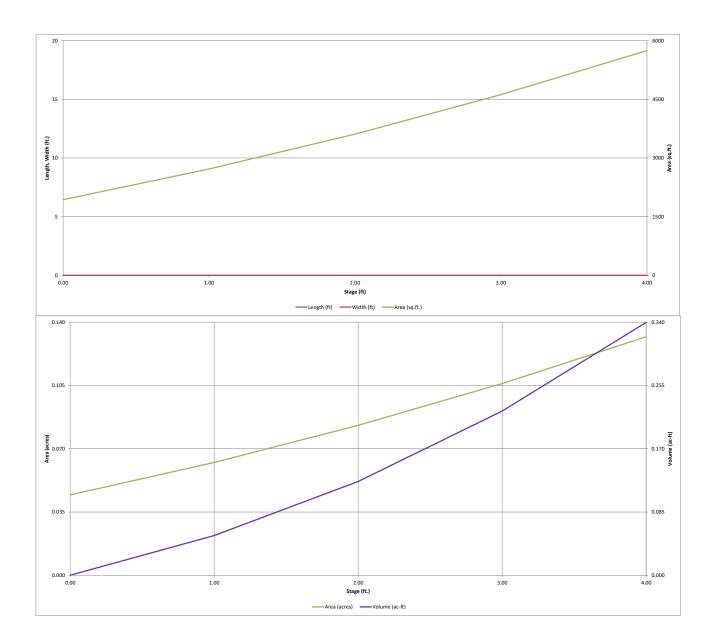
Initial Surcharge Area (A <sub>ISV</sub> ) =	user	ft 2
Surcharge Volume Length $(L_{ISV}) =$	user	ft
Surcharge Volume Width (W <sub>ISV</sub> ) =	user	ft
Depth of Basin Floor (H <sub>FLOOR</sub> ) =	user	ft
Length of Basin Floor $(L_{FLOOR})$ =	user	ft
Width of Basin Floor (WFLOOR) =	user	ft
Area of Basin Floor $(A_{FLOOR}) =$	user	ft <sup>2</sup>
Volume of Basin Floor (V <sub>FLOOR</sub> ) =	user	ft <sup>3</sup>
Depth of Main Basin (H <sub>MAIN</sub> ) =	user	ft
Length of Main Basin $(L_{MAIN}) =$	user	ft
Width of Main Basin (W <sub>MAIN</sub> ) =	user	ft
Area of Main Basin (A <sub>MAIN</sub> ) =	user	ft 2
Volume of Main Basin (V <sub>MAIN</sub> ) =	user	ft <sup>3</sup>
Calculated Total Basin Volume ( $V_{total}$ ) =	user	acre-f

Depth Increment =	1.00	ft						
Depair Incientent =	1.00	Optional				Optional		
Stage - Storage Description	Stage (ft)	Override Stage (ft)	Length (ft)	Width (ft)	Area (ft ²)	Override Area (ft <sup>2</sup> )	Area (acre)	Volume (ft <sup>3</sup> )
Media Surface		0.00				1,935	0.044	
7235		1.00				2,718	0.062	2,326
7236		2.00				3,613	0.083	5,492
7237		3.00				4,621	0.106	9,609
7238		4.00				5,745	0.132	14,792
				-				
				-				

Volume (ac-ft)

0.053 0.126 0.221 0.340

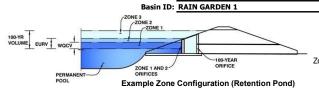
118530 MHFD-Detention\_v4-06 - RG 1, Basin 9/20/2023, 9:37 AM



118530 MHFD-Detention\_v4-06 - RG 1, Basin 9/20/2023, 9:37 AM

### DETENTION BASIN OUTLET STRUCTURE DESIGN

Project: RETREAT AT TIMBERRIDGE FILING NO. 3



	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
		, ,	
Zone 1 (WQCV)	0.81	0.042	Filtration Media
one 2 (100-year)	3.53	0.238	Weir&Pipe (Restrict)
Zone 3			
· ·	Total (all zones)	n 279	

<u>User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)</u>

Underdrain Orifice Invert Depth = 1.50 ft (distance below the filtration media surface) Underdrain Orifice Diameter = 1.08 inches

Calculated Parameters for Underdrain Underdrain Orifice Area 0.0 Underdrain Orifice Centroid = 0.05 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 BMP) Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft) N/A

Depth at top of Zone using Orifice Plate = N/A ft (relative to basin bottom at Stage = 0 ft) Orifice Plate: Orifice Vertical Spacing = N/A inches sq. inches Orifice Plate: Orifice Area per Row = N/A

Calculated Parameters for Plate WQ Orifice Area per Row = N/A ft2 Elliptical Half-Width = N/A feet Elliptical Slot Centroid = feet N/A ft<sup>2</sup> Elliptical Slot Area = N/A

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

	Row 1 (optional)	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)	N/A							
Orifice Area (sq. inches)	N/A							

	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)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Orifice Area (sq. inches)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

User Input: Vertical Orifice (Circular or Rectangular)

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

	Calculated Parame	Calculated Parameters for Vertical Orifice					
	Not Selected	Not Selected					
Vertical Orifice Area =			ft <sup>2</sup>				
Vertical Orifice Centroid =			feet				

Calculated Parameters for Outlet Pine w/ Flow Restriction Plate

Calculated Parameters for Overflow Weir

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

	Zone 2 Weir	Not Selected		Zone 2 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	1.00	1.95	ft (relative to basin bottom at Stage = 0 ft) Height of Grate Upper Edge, $H_t$ =	1.75		feet
Overflow Weir Front Edge Length =	3.00		feet Overflow Weir Slope Length =	3.09		feet
Overflow Weir Grate Slope =	4.00		H:V Grate Open Area / 100-yr Orifice Area =	9.67		
Horiz. Length of Weir Sides =	3.00		feet Overflow Grate Open Area w/o Debris =	7.34		ft <sup>2</sup>
Overflow Grate Type =	Close Mesh Grate		Overflow Grate Open Area w/ Debris =	3.67		ft <sup>2</sup>
Debris Clogging % =	50%		%			-

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

	Zone 2 Restrictor	Not Selected			Zone 2 Restrictor	Not Selected	l
Depth to Invert of Outlet Pipe =	2.00		ft (distance below basin bottom at Stage = 0 ft)	Outlet Orifice Area =	0.76		ft <sup>2</sup>
Outlet Pipe Diameter =	18.00		inches	Outlet Orifice Centroid =	0.39		feet
or Plate Height Above Pipe Invert =	8.00		inches Half-Central Angle of	Restrictor Plate on Pipe =	1.46	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

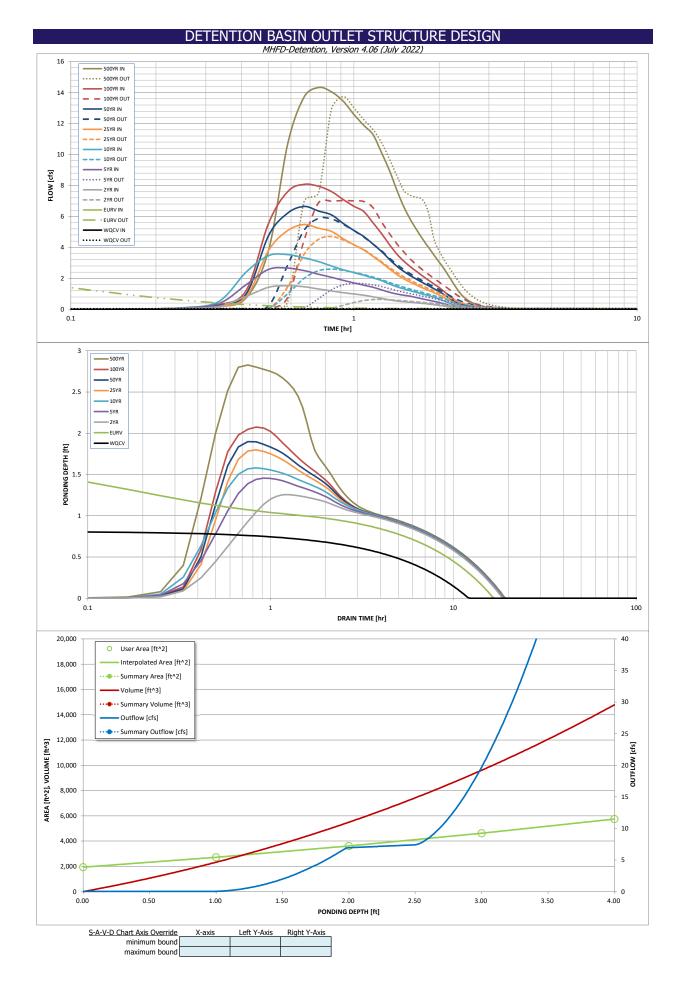
Spillway Invert Stage=	2.50	ft (rela	tive to basin bottom at Stage = 0 ft)
Spillway Crest Length =	10.00	feet	
Spillway End Slopes =	3.00	H:V	
Freeboard above Max Water Surface =	0.50	feet	1ft freeboard

required

Calculated Parameters for Spillway Spillway Design Flow Depth= 0.39 feet Stage at Top of Freeboard = 3.39 feet Basin Area at Top of Freeboard = 0.12 acres Basin Volume at Top of Freeboard = 0.26 acre-ft

Restrictor

Routed Hydrograph Results	The user can over	ride the default CUI	HP hydrographs and	d runoff volumes by	entering new value	es in the Inflow Hyd	drographs table (Co	olumns W through A	( <i>F</i> ).
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	1.19	1.50	1.75	2.00	2.25	2.52	3.85
CUHP Runoff Volume (acre-ft) =	0.042	0.112	0.118	0.209	0.294	0.430	0.529	0.664	1.221
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.118	0.209	0.294	0.430	0.529	0.664	1.221
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A	0.6	1.6	2.4	4.3	5.4	6.7	12.6
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.11	0.32	0.48	0.86	1.08	1.35	2.51
Peak Inflow Q (cfs) =	N/A	N/A	1.5	2.7	3.5	5.5	6.6	8.1	14.3
Peak Outflow Q (cfs) =	0.0	3.4	0.6	1.7	2.6	4.7	5.9	7.0	13.7
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.0	1.1	1.1	1.1	1.0	1.1
Structure Controlling Flow =	Filtration Media	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	0.68	0.08	0.2	0.3	0.6	0.8	0.9	1.0
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) =	12	16	18	17	16	15	14	13	9
Time to Drain 99% of Inflow Volume (hours) =	12	16	18	18	18	18	17	17	15
Maximum Ponding Depth (ft) =	0.82	1.83	1.26	1.46	1.58	1.80	1.90	2.08	2.83
Area at Maximum Ponding Depth (acres) =	0.06	0.08	0.07	0.07	0.07	0.08	0.08	0.08	0.10
Maximum Volume Stored (acre-ft) =	0.042	0.112	0.070	0.084	0.092	0.109	0.117	0.132	0.202



## 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.

1	SOURCE	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP	CUHP
T T										
Time Interval	TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]		25 Year [cfs]	50 Year [cfs]		500 Year [cfs]
5.00 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:05:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:10:00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.04
	0:15:00	0.00	0.00	0.05	0.09	0.11	0.07	0.09	0.09	0.17
	0:20:00 0:25:00	0.00	0.00	0.19	0.37	0.51	0.19	0.24	0.29	0.75
	0:30:00	0.00	0.00	0.83 1.43	1.59 2.58	2.41 3.47	0.81 3.83	1.00 4.78	1.21 5.58	3.85 10.67
	0:35:00	0.00	0.00	1.52	2.66	3.54	5.08	6.21	7.60	13.72
	0:40:00	0.00	0.00	1.46	2.50	3.33	5.47	6.64	8.08	14.33
	0:45:00	0.00	0.00	1.31	2.27	3.08	5.24	6.35	7.94	14.05
	0:50:00	0.00	0.00	1.19	2.07	2.80	5.03	6.10	7.61	13.44
	0:55:00	0.00	0.00	1.08	1.87	2.56	4.56	5.55	7.09	12.59
	1:00:00	0.00	0.00	0.99	1.71	2.37	4.16	5.09	6.66	11.88
	1:05:00	0.00	0.00	0.91	1.56	2.20	3.83	4.70	6.31	11.29
	1:10:00	0.00	0.00	0.81	1.42	2.03	3.42	4.22	5.60	10.14
	1:15:00	0.00	0.00	0.71	1.26	1.86	3.02	3.74	4.91	9.00
	1:20:00	0.00	0.00	0.61	1.10	1.64	2.62	3.24	4.20	7.72
	1:25:00	0.00	0.00	0.54	0.99	1.46	2.25	2.79	3.58	6.63
	1:30:00 1:35:00	0.00	0.00	0.49	0.91	1.32	1.97	2.45	3.12	5.80
}	1:40:00	0.00	0.00	0.45	0.84	1.20 1.09	1.75	2.17 1.94	2.75	5.13 4.54
ŀ	1:45:00	0.00	0.00	0.42	0.76 0.67	0.99	1.56 1.39	1.72	2.44 2.15	4.54
ŀ	1:50:00	0.00	0.00	0.34	0.59	0.99	1.23	1.72	1.89	3.52
	1:55:00	0.00	0.00	0.30	0.52	0.78	1.08	1.34	1.64	3.05
	2:00:00	0.00	0.00	0.26	0.44	0.66	0.93	1.16	1.40	2.61
	2:05:00	0.00	0.00	0.21	0.35	0.53	0.76	0.95	1.15	2.13
	2:10:00	0.00	0.00	0.16	0.27	0.41	0.59	0.74	0.90	1.66
	2:15:00	0.00	0.00	0.12	0.19	0.30	0.43	0.54	0.66	1.22
	2:20:00	0.00	0.00	0.09	0.14	0.23	0.29	0.37	0.45	0.88
	2:25:00	0.00	0.00	0.07	0.11	0.19	0.21	0.27	0.32	0.65
	2:30:00	0.00	0.00	0.05	0.09	0.15	0.16	0.20	0.24	0.48
	2:35:00	0.00	0.00	0.04	0.07	0.12	0.12	0.15	0.17	0.36
	2:40:00 2:45:00	0.00	0.00	0.03	0.06	0.10	0.09	0.11	0.12	0.26
	2:50:00	0.00	0.00	0.03	0.05 0.04	0.08	0.07 0.05	0.09 0.07	0.08	0.19
	2:55:00	0.00	0.00	0.02	0.03	0.05	0.03	0.07	0.04	0.09
	3:00:00	0.00	0.00	0.01	0.02	0.04	0.03	0.04	0.03	0.07
	3:05:00	0.00	0.00	0.01	0.02	0.03	0.02	0.03	0.03	0.06
	3:10:00	0.00	0.00	0.01	0.01	0.02	0.02	0.02	0.02	0.05
	3:15:00	0.00	0.00	0.01	0.01	0.02	0.01	0.02	0.02	0.04
	3:20:00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.03
	3:25:00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.02
	3:30:00	0.00	0.00	0.00	0.00	0.01	0.00	0.01	0.01	0.01
	3:35:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
	3:40:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:45:00 3:50:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
}	3:50:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ŀ	4:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ŀ	4:05:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:10:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:15:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:20:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
}	4:25:00 4:30:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:35:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:40:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:45:00 4:50:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ŀ	4:55:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:05:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:10:00 5:15:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ļ	5:20:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ļ	5:25:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:30:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:35:00 5:40:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:45:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
[	5:50:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:55:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
l	6:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

### DETENTION BASIN OUTLET STRUCTURE DESIGN

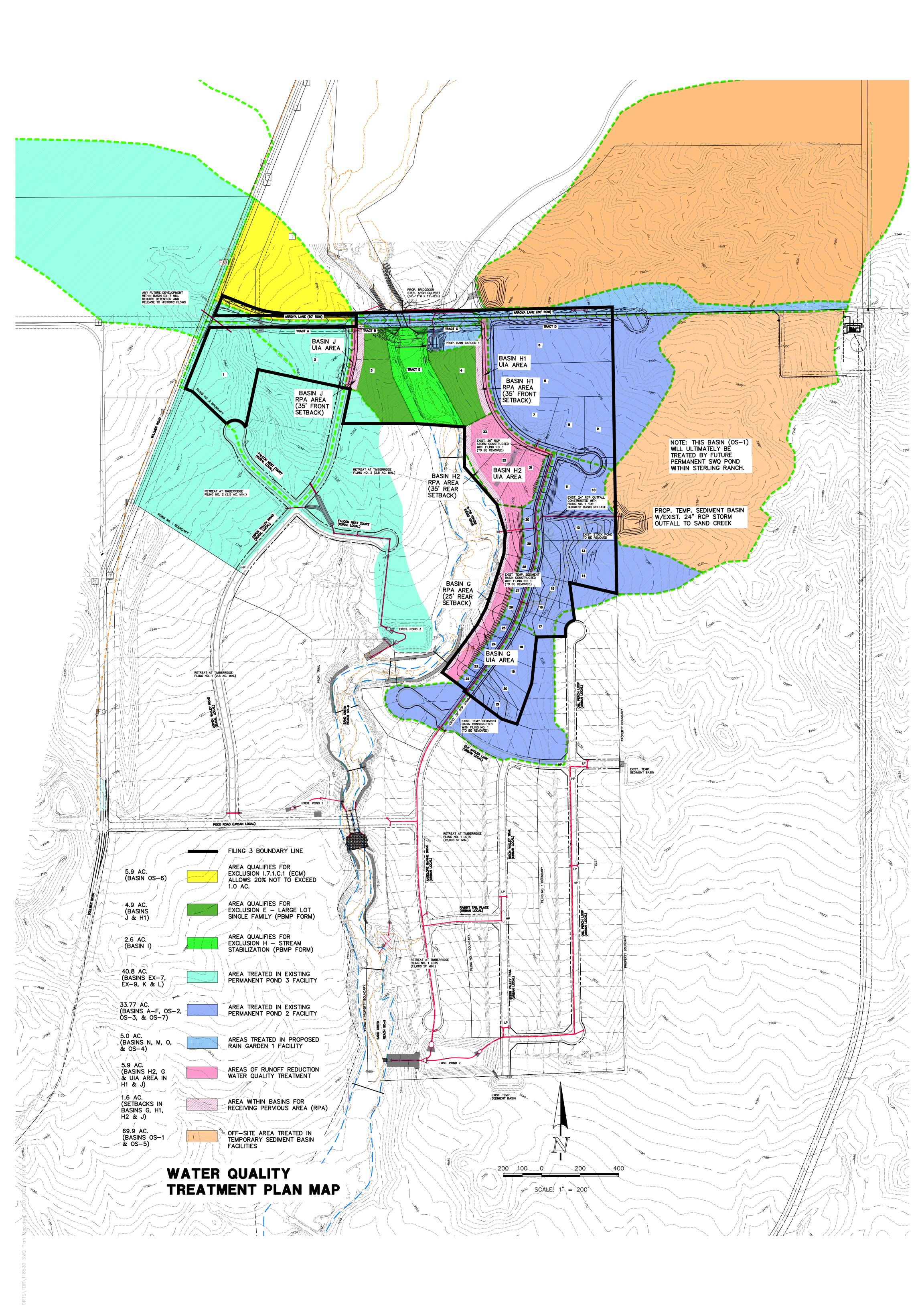
MHFD-Detention, Version 4.06 (July 2022)

Summary Stage-Area-Volume-Discharge Relationships

The user can create a summary S-A-V-D by entering the desired stage increments and the remainder of the table will populate automatically. The user should graphically compare the summary S-A-V-D table to the full S-A-V-D table in the chart to confirm it captures all key transition points.

Stage - Storage Description	Stage [ft]	Area [ft²]	Area [acres]	Volume [ft <sup>3</sup> ]	Volume [ac-ft]	Total Outflow [cfs]	
							For best results, include the
							stages of all grade slope
							changes (e.g. ISV and Floor
							from the S-A-V table on Sheet 'Basin'.
							- Silect Dasiii.
							Also include the inverts of a
							outlets (e.g. vertical orifice,
							overflow grate, and spillway where applicable).
							wriere applicable).
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			Desig	ın Procedu			luction					
Designer:	Marc A. Who	rton. P.E.		UD-BMP (Ve	rsion 3.07, Ma	rch 2018)						Sheet 1 of 1
Company:		Classic Consulting										
Date:	September 20, 2023											
Project: Retreat at TimberRidge Filing No. 3												
Location: BASINS NOT TRIBUTARY TO PERMANENT SWQ FACILITY												
SITE INFORMATION (User Input in Blue Cells)												
		ainfall Depth	0.53	inches								
Depth of Average Rui	noff Producing	g Storm, d <sub>6</sub> =	0.42	inches (for W	/atersheds O	utside of the I	Denver Regio	on, Figure 3-1	in USDCM V	ol. 3)		
Area Type		UIA:RPA	UIA:RPA	UIA:RPA								
Area ID	Basin G SC	Basin H1 SC	Basin H2 SC	Basin J SC								
Downstream Design Point ID Downstream BMP Type	None	None	None	None								
DCIA (ft²)												
UIA (ft²)		15,430	49,000	11,146								
RPA (ft²)	24,205	14,150	20,550	12,340								
SPA (ft²) HSG A (%)	0%	 0%	0%	 0%								
HSG B (%)	100%	100%	100%	100%								
HSG C/D (%)	0%	0%	0%	0%								
Average Slope of RPA (ft/ft)	0.025	0.080	0.140	0.030								
UIA:RPA Interface Width (ft)	900.00	360.00	550.00	300.00								
CALCULATED RUNOFF		B	D 1 1-10	l posteri				1	I			
Area ID UIA:RPA Area (ft²)	Basin G 79,345	Basin H1 29,580	Basin H2 69,550	Basin J 23,486								
L / W Ratio	0.10	0.23	0.23	0.26								
UIA / Area	0.6949	0.5216	0.7045	0.4746								
Runoff (in)	0.00	0.00	0.00	0.00								
Runoff (ft <sup>3</sup> )	0 1976	0 553	0 1756	0 399								
Runoff Reduction (ft <sup>3</sup> )	1970	333	1756	399				!				
CALCULATED WQCV RI	ESULTS											
Area ID		Basin H1	Basin H2	Basin J								
WQCV (ft <sup>3</sup> )	2244 2244	628 628	1994 1994	454 454								
WQCV Reduction (ft <sup>3</sup> ) WQCV Reduction (%)	100%	100%	100%	100%								
Untreated WQCV (ft <sup>3</sup> )		0	0	0								
CALCUL ATED DEGICAL	DOINT DECL	LTC (auma m		-111	ith the com	. D	Daniem De	int ID)				
CALCULATED DESIGN I Downstream Design Point ID	SC SC	LIS (sums re	esuits from a	all columns v	ith the sam	Downstrea	m Design Po	int iD)				
DCIA (ft <sup>2</sup> )	0											
UIA (ft²)	130,716											
RPA (ft²)												
SPA (ft²) Total Area (ft²)	0 201,961											
Total Impervious Area (ft²)	130,716											
WQCV (ft <sup>3</sup> )												
WQCV Reduction (ft <sup>3</sup> )												
WQCV Reduction (%)	100%											
Untreated WQCV (ft <sup>3</sup> )	0											
CALCULATED SITE RES	SULTS (sums	results from	n all columns	s in workshe	et)							
Total Area (ft <sup>2</sup> )												
Total Impervious Area (ft <sup>2</sup> )	130,716											
WQCV (ft <sup>3</sup> ) WQCV Reduction (ft <sup>3</sup> )												
WQCV Reduction (ft <sup>-</sup> ) WQCV Reduction (%)												
Untreated WQCV (ft <sup>3</sup> )												



# **Description**

A sediment basin is a temporary pond built on a construction site to capture eroded or disturbed soil transported in storm runoff prior to discharge from the site. Sediment basins are designed to capture site runoff and slowly release it to allow time for settling of sediment prior to discharge. Sediment basins are often constructed in locations that will later be modified to serve as post-construction stormwater basins.

# **Appropriate Uses**

Most large construction sites (typically greater than 2 acres) will require one or more sediment basins for effective



**Photograph SB-1.** Sediment basin at the toe of a slope. Photo courtesy of WWE.

management of construction site runoff. On linear construction projects, sediment basins may be impractical; instead, sediment traps or other combinations of BMPs may be more appropriate.

Sediment basins should not be used as stand-alone sediment controls. Erosion and other sediment controls should also be implemented upstream.

When feasible, the sediment basin should be installed in the same location where a permanent post-construction detention pond will be located.

# **Design and Installation**

The design procedure for a sediment basin includes these steps:

- Basin Storage Volume: Provide a storage volume of at least 3,600 cubic feet per acre of drainage area. To the extent practical, undisturbed and/or off-site areas should be diverted around sediment basins to prevent "clean" runoff from mixing with runoff from disturbed areas. For undisturbed areas (both on-site and off-site) that cannot be diverted around the sediment basin, provide a minimum of 500 ft³/acre of storage for undeveloped (but stable) off-site areas in addition to the 3,600 ft³/acre for disturbed areas. For stable, developed areas that cannot be diverted around the sediment basin, storage volume requirements are summarized in Table SB-1.
- **Basin Geometry:** Design basin with a minimum length-to-width ratio of 2:1 (L:W). If this cannot be achieved because of site space constraints, baffling may be required to extend the effective distance between the inflow point(s) and the outlet to minimize short-circuiting.

  Sediment Basins
- **Dam Embankment**: It is recommended that embankment slopes be 4:1 (H:V) or flatter and no steeper than 3:1 (H:V) in any location.

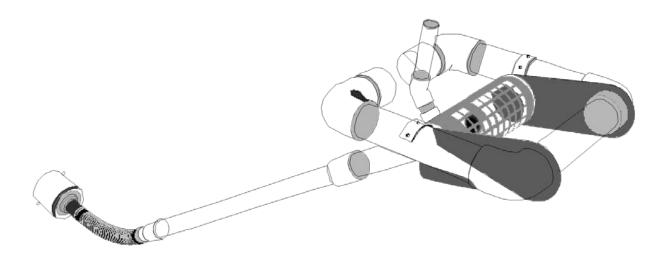
Sediment Basins	
Functions	
Erosion Control	No
Sediment Control	Yes
Site/Material Management	No

• **Inflow Structure**: For concentrated flow entering the basin, provide energy dissipation at the point of inflow.

Table SB-1. Additional Volume Requirements for Undisturbed and Developed Tributary Areas
Draining through Sediment Basins

Imperviousness (%)	Additional Storage Volume (ft <sup>3</sup> ) Per Acre of Tributary Area
Undeveloped	500
10	800
20	1230
30	1600
40	2030
50	2470
60	2980
70	3560
80	4360
90	5300
100	6460

- Outlet Works: The outlet pipe shall extend through the embankment at a minimum slope of 0.5 percent. Outlet works can be designed using one of the following approaches:
  - o **Riser Pipe (Simplified Detail):** Detail SB-1 provides a simplified design for basins treating no more than 15 acres.
  - Orifice Plate or Riser Pipe: Follow the design criteria for Full Spectrum Detention outlets in the EDB Fact Sheet provided in Chapter 4 of this manual for sizing of outlet perforations with an emptying time of approximately 72 hours. In lieu of the trash rack, pack uniformly sized 1½ to 2-inch gravel in front of the plate or surrounding the riser pipe. This gravel will need to be cleaned out frequently during the construction period as sediment accumulates within it. The gravel pack will need to be removed and disposed of following construction to reclaim the basin for use as a permanent detention facility. If the basin will be used as a permanent extended detention basin for the site, a trash rack will need to be installed once contributing drainage areas have been stabilized and the gravel pack and accumulated sediment have been removed.
  - o **Floating Skimmer**: If a floating skimmer is used, install it using manufacturer's recommendations. Illustration SB-1 provides an illustration of a Faircloth Skimmer Floating Outlet<sup>TM</sup>, one of the more commonly used floating skimmer outlets. A skimmer should be designed to release the design volume in no less than 48 hours. The use of a floating skimmer outlet can increase the sediment capture efficiency of a basin significantly. A floating outlet continually decants cleanest water off the surface of the pond and releases cleaner water than would discharge from a perforated riser pipe or plate.



**Illustration SB-1.** Outlet structure for a temporary sediment basin - Faircloth Skimmer Floating Outlet. Illustration courtesy of J. W. Faircloth & Sons, Inc., FairclothSkimmer.com.

- Outlet Protection and Spillway: Consider all flow paths for runoff leaving the basin, including protection at the typical point of discharge as well as overtopping.
  - Outlet Protection: Outlet protection should be provided where the velocity of flow will exceed the maximum permissible velocity of the material of the waterway into which discharge occurs. This may require the use of a riprap apron at the outlet location and/or other measures to keep the waterway from eroding.
  - Emergency Spillway: Provide a stabilized emergency overflow spillway for rainstorms that exceed the capacity of the sediment basin volume and its outlet. Protect basin embankments from erosion and overtopping. If the sediment basin will be converted to a permanent detention basin, design and construct the emergency spillway(s) as required for the permanent facility. If the sediment basin will not become a permanent detention basin, it may be possible to substitute a heavy polyvinyl membrane or properly bedded rock cover to line the spillway and downstream embankment, depending on the height, slope, and width of the embankments.

## **Maintenance and Removal**

Maintenance activities include the following:

- Dredge sediment from the basin, as needed to maintain BMP effectiveness, typically when the design storage volume is no more than one-third filled with sediment.
- Inspect the sediment basin embankments for stability and seepage.
- Inspect the inlet and outlet of the basin, repair damage, and remove debris. Remove, clean and replace the gravel around the outlet on a regular basis to remove the accumulated sediment within it and keep the outlet functioning.
- Be aware that removal of a sediment basin may require dewatering and associated permit requirements.
- Do not remove a sediment basin until the upstream area has been stabilized with vegetation.

Final disposition of the sediment basin depends on whether the basin will be converted to a permanent post-construction stormwater basin or whether the basin area will be returned to grade. For basins being converted to permanent detention basins, remove accumulated sediment and reconfigure the basin and outlet to meet the requirements of the final design for the detention facility. If the sediment basin is not to be used as a permanent detention facility, fill the excavated area with soil and stabilize with vegetation.

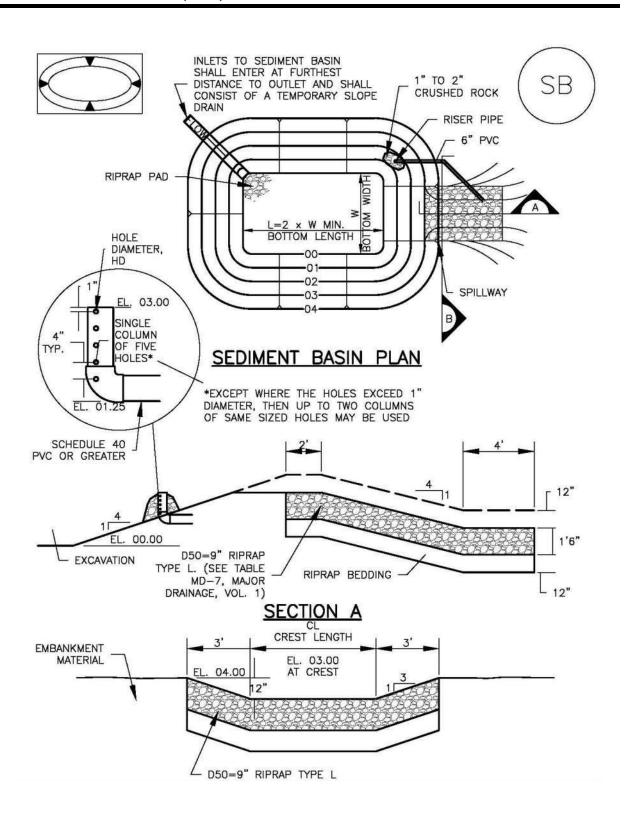


TABLE SB-1. SIZ	ZING INFORMATION FO	OR STANDARD SEDIMENT	BASIN
Upstream Drainage Area (rounded to nearest acre), (ac)	Basin Bottom Width (W), (ft)	Spillway Crest Length (CL), (ft)	Hole Diameter (HD), (in)
1 2 3 4 5 6 7 8 9 10 11 12 13 14	12 ½ 21 28 33 ½ 38 ½ 43 47 ¼ 51 55 58 ¼ 61 64 67 ½ 70 ½ 73 ¼	2 3 5 6 8 9 11 12 13 15 16 18 19 21 22	932 13/6 14 9/6 21/32 25/32 25/32 78 15/6 31/32 1 1 1/6 1 1/8

#### SEDIMENT BASIN INSTALLATION NOTES

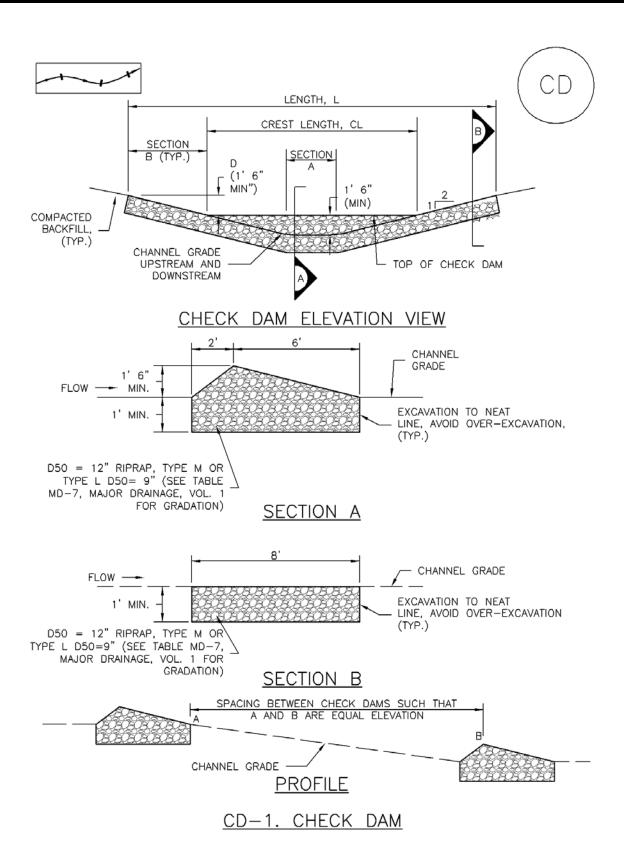
- 1. SEE PLAN VIEW FOR:
  - -LOCATION OF SEDIMENT BASIN.
  - -TYPE OF BASIN (STANDARD BASIN OR NONSTANDARD BASIN).
  - -FOR STANDARD BASIN, BOTTOM WIDTH W, CREST LENGTH CĹ, AND HOLE DIAMETER. HD.
  - -FOR NONSTANDARD BASIN, SEE CONSTRUCTION DRAWINGS FOR DESIGN OF BASIN INCLUDING RISER HEIGHT H, NUMBER OF COLUMNS N, HOLE DIAMETER HD AND PIPE DIAMETER D.
- 2. FOR STANDARD BASIN, BOTTOM DIMENSION MAY BE MODIFIED AS LONG AS BOTTOM AREA IS NOT REDUCED.
- 3. SEDIMENT BASINS SHALL BE INSTALLED PRIOR TO ANY OTHER LAND-DISTURBING ACTIVITY THAT RELIES ON ON BASINS AS A STORMWATER CONTROL.
- 4. EMBANKMENT MATERIAL SHALL CONSIST OF SOIL FREE OF DEBRIS, ORGANIC MATERIAL, AND ROCKS OR CONCRETE GREATER THAN 3 INCHES AND SHALL HAVE A MINIMUM OF 15 PERCENT BY WEIGHT PASSING THE NO. 200 SIEVE.
- 5. EMBANKMENT MATERIAL SHALL BE COMPACTED TO AT LEAST 95 PERCENT OF MAXIMUM DENSITY IN ACCORDANCE WITH ASTM D698.
- 6. PIPE SCH 40 OR GREATER SHALL BE USED.
- 7. THE DETAILS SHOWN ON THESE SHEETS PERTAIN TO STANDARD SEDIMENT BASIN(S) FOR DRAINAGE AREAS LESS THAN 15 ACRES. SEE CONSTRUCTION DRAWINGS FOR EMBANKMENT, STORAGE VOLUME, SPILLWAY, OUTLET, AND OUTLET PROTECTION DETAILS FOR ANY SEDIMENT BASIN(S) THAT HAVE BEEN INDIVIDUALLY DESIGNED FOR DRAINAGE AREAS LARGER THAN 15 ACRES.

### SEDIMENT BASIN MAINTENANCE NOTES

- 1. INSPECT BMPs EACH WORKDAY, AND MAINTAIN THEM IN EFFECTIVE OPERATING CONDITION. MAINTENANCE OF BMPs SHOULD BE PROACTIVE, NOT REACTIVE. INSPECT BMPs AS SOON AS POSSIBLE (AND ALWAYS WITHIN 24 HOURS) FOLLOWING A STORM THAT CAUSES SURFACE EROSION, AND PERFORM NECESSARY MAINTENANCE.
- 2. FREQUENT OBSERVATIONS AND MAINTENANCE ARE NECESSARY TO MAINTAIN BMPs IN EFFECTIVE OPERATING CONDITION. INSPECTIONS AND CORRECTIVE MEASURES SHOULD BE DOCUMENTED THOROUGHLY.
- 3. WHERE BMPs HAVE FAILED, REPAIR OR REPLACEMENT SHOULD BE INITIATED UPON DISCOVERY OF THE FAILURE.
- 4. SEDIMENT ACCUMULATED IN BASIN SHALL BE REMOVED AS NEEDED TO MAINTAIN BMP EFFECTIVENESS, TYPICALLY WHEN SEDIMENT DEPTH REACHES ONE FOOT (I.E., TWO FEET BELOW THE SPILLWAY CREST).
- 5. SEDIMENT BASINS ARE TO REMAIN IN PLACE UNTIL THE UPSTREAM DISTURBED AREA IS STABILIZED AND GRASS COVER IS ACCEPTED BY THE LOCAL JURISDICTION.
- 6. WHEN SEDIMENT BASINS ARE REMOVED, ALL DISTURBED AREAS SHALL BE COVERED WITH TOPSOIL, SEEDED AND MULCHED OR OTHERWISE STABILIZED AS APPROVED BY LOCAL JURISDICTION.

(DETAILS ADAPTED FROM DOUGLAS COUNTY, COLORADO)

NOTE: MANY JURISDICTIONS HAVE BMP DETAILS THAT VARY FROM UDFCD STANDARD DETAILS. CONSULT WITH LOCAL JURISDICTIONS AS TO WHICH DETAIL SHOULD BE USED WHEN DIFFERENCES ARE NOTED.



### CHECK DAM INSTALLATION NOTES

- 1. SEE PLAN VIEW FOR:
  - -LOCATION OF CHECK DAMS.
  - -CHECK DAM TYPE (CHECK DAM OR REINFORCED CHECK DAM).
  - -LENGTH (L), CREST LENGTH (CL), AND DEPTH (D).
- 2. CHECK DAMS INDICATED ON INITIAL SWMP SHALL BE INSTALLED AFTER CONSTRUCTION FENCE, BUT PRIOR TO ANY UPSTREAM LAND DISTURBING ACTIVITIES.
- 3. RIPRAP UTILIZED FOR CHECK DAMS SHOULD BE OF APPROPRIATE SIZE FOR THE APPLICATION. TYPICAL TYPES OF RIPRAP USED FOR CHECK DAMS ARE TYPE M (D50 12") OR TYPE L (D50 9").
- 4. RIPRAP PAD SHALL BE TRENCHED INTO THE GROUND A MINIMUM OF 1'.
- 5. THE ENDS OF THE CHECK DAM SHALL BE A MINIMUM OF 1' 6" HIGHER THAN THE CENTER OF THE CHECK DAM.

#### CHECK DAM MAINTENANCE NOTES

- 1. INSPECT BMPs EACH WORKDAY, AND MAINTAIN THEM IN EFFECTIVE OPERATING CONDITION. MAINTENANCE OF BMPs SHOULD BE PROACTIVE, NOT REACTIVE. INSPECT BMPs AS SOON AS POSSIBLE (AND ALWAYS WITHIN 24 HOURS) FOLLOWING A STORM THAT CAUSES SURFACE EROSION, AND PERFORM NECESSARY MAINTENANCE.
- 2. FREQUENT OBSERVATIONS AND MAINTENANCE ARE NECESSARY TO MAINTAIN BMPs IN EFFECTIVE OPERATING CONDITION. INSPECTIONS AND CORRECTIVE MEASURES SHOULD BE DOCUMENTED THOROUGHLY.
- 3. WHERE BMPs HAVE FAILED, REPAIR OR REPLACEMENT SHOULD BE INITIATED UPON DISCOVERY OF THE FAILURE.
- 4. SEDIMENT ACCUMULATED UPSTREAM OF THE CHECK DAMS SHALL BE REMOVED WHEN THE SEDIMENT DEPTH IS WITHIN  $\frac{1}{2}$  OF THE HEIGHT OF THE CREST.
- 5. CHECK DAMS ARE TO REMAIN IN PLACE UNTIL THE UPSTREAM DISTURBED AREA IS STABILIZED AND APPROVED BY THE LOCAL JURISDICTION.
- 6. WHEN CHECK DAMS ARE REMOVED, EXCAVATIONS SHALL BE FILLED WITH SUITABLE COMPACTED BACKFILL. DISTURBED AREA SHALL BE SEEDED AND MULCHED AND COVERED WITH GEOTEXTILE OR OTHERWISE STABILIZED IN A MANNER APPROVED BY THE LOCAL JURISDICTION.

(DETAILS ADAPTED FROM DOUGLAS COUNTY, COLORADO, NOT AVAILABLE IN AUTOCAD)

NOTE: MANY JURISDICTIONS HAVE BMP DETAILS THAT VARY FROM UDFCD STANDARD DETAILS. CONSULT WITH LOCAL JURISDICTIONS AS TO WHICH DETAIL SHOULD BE USED WHEN DIFFERENCES ARE NOTED.

# **Description**

A BMP that utilizes bioretention is an engineered, depressed landscape area designed to capture and filter or infiltrate the water quality capture volume (WQCV). BMPs that utilize bioretention are frequently referred to as rain gardens or porous landscape detention areas (PLDs). The term PLD is common in the UDFCD region as this manual first published the BMP by this name in 1999. In an effort to be consistent with terms most prevalent in the stormwater industry, this document generally refers to the treatment process as *bioretention* and to the BMP as a *rain garden*.

The design of a rain garden may provide detention for events exceeding that of the WQCV. There are generally two ways to achieve this. The design can provide the flood control volume above the WQCV or the design can provide and slowly release the flood control volume in an area downstream of one or more rain gardens. See the *Storage* chapter in Volume 2 of the USDCM for more information.

This infiltrating BMP requires consultation with a geotechnical engineer when proposed adjacent to a structure. A geotechnical engineer can assist with evaluating the suitability of soils, identifying potential impacts, and establishing minimum distances between the BMP and structures.

## **Terminology**

The term *bioretention* refers to the treatment process although it is also frequently used to describe a BMP that provides biological uptake and retention of the pollutants found in stormwater runoff. This BMP is sometimes referred to as a *porous landscape detention (PLD) area* or *rain garden*.



**Photograph B-1**. This recently constructed rain garden provides bioretention of pollutants, as well as an attractive amenity for a residential building. Treatment should improve as vegetation matures.

Bioretention (Rain Garden)						
Functions						
LID/Volume Red.	Yes					
WQCV Capture	Yes					
WQCV+Flood Control	Yes					
Fact Sheet Includes EURV Guidance No						
Typical Effectiveness for Targeted Pollutants <sup>3</sup>						
Sediment/Solids	Very Good <sup>1</sup>					
Nutrients	Moderate					
Total Metals	Good					
Bacteria Moderate						
<b>Other Considerations</b>						
Life-cycle Costs <sup>4</sup>	Moderate					

<sup>&</sup>lt;sup>1</sup> Not recommended for watersheds with high sediment yields (unless pretreatment is provided).

<sup>&</sup>lt;sup>3</sup> Based primarily on data from the International Stormwater BMP Database (www.bmpdatabase.org).

<sup>&</sup>lt;sup>4</sup> Based primarily on BMP-REALCOST available at <u>www.udfcd.org</u>. Analysis based on a single installation (not based on the maximum recommended watershed tributary to each BMP).

## **Site Selection**

This BMP allows WQCV treatment within one or more areas designated for landscape (see design step 7 for suggusted vegetation). In this way, it is an excellent alternative to extended detention basins for small sites. A typical rain garden serves a tributary area of one impervious acre or less, although they can be designed for larger tributary areas. Multiple installations can be used within larger sites. Rain gardens should not be used when a baseflow is anticipated. They are typically small and installed in locations such as:

- Parking lot islands
- Street medians
- Landscape areas between the road and a detached walk
- Planter boxes that collect roof drains

Bioretention requires a stable watershed. Retrofit applications are typically successful for this reason. When the watershed includes phased construction, sparsely vegetated areas, or steep slopes in sandy soils, consider another BMP or provide pretreatment before runoff from these areas reaches the rain garden.

The surface of the rain garden should be flat. For this reason, rain gardens can be more difficult to incorporate into steeply sloping terrain; however, terraced applications of these facilities have been successful in other parts of the country.

When bioretention (and other BMPs used for infiltration) are

located adjacent to buildings or pavement areas, protective measures should be implemented to avoid adverse impacts to these structures. Oversaturated subgrade soil underlying a structure can cause the structure to settle or result in moisture-related problems. Wetting of expansive soils or bedrock can cause swelling, resulting in structural movements. A geotechnical engineer should evaluate the potential impact of the BMP on adjacent structures based on an evaluation of the subgrade soil, groundwater, and bedrock conditions at the site. Additional minimum requirements include:

- In locations where subgrade soils do not allow infiltration and/or where infiltration could adversely impact adjacent structures, include a drainage layer (with underdrain) under the growing medium.
- In locations where potentially expansive soils or bedrock exist, placement of a rain garden adjacent to structures and pavement should only be considered if the BMP includes a drainage layer (with underdrain) and an impermeable geomembrane liner designed to restrict seepage.

## **Benefits**

- Bioretention uses multiple treatment processes to remove pollutants, including sedimentation, filtering, adsorption, evapotranspiration, and biological uptake of constituents.
- Stormwater treatment occurs within attractive landscaped areas.
- There is a potential reduction of irrigation requirements by taking advantage of site runoff.

## Limitations

- Additional design and construction steps are required for placement of any ponding or infiltration area near or upgradient from a building foundation and/or when expansive (low to high swell) soils exist. This is discussed in the design procedure section.
- In developing or otherwise erosive watersheds, high sediment loads can clog the facility.

# **Designing for Maintenance**

Recommended maintenance practices for all BMPs are in Chapter 6 of this manual. During design, consider the following to ensure ease of maintenance over the long-term:

- Do not put a filter sock on the underdrain. This is not necessary and can cause the underdrain to clog.
- The best surface cover for a rain garden is full vegetation. Use rock mulch sparingly within the rain garden because rock mulch limits infiltration and is more difficult to maintain. Wood mulch handles sediment build-up better than rock mulch; however, wood mulch floats and may clog the overflow depending on the configuration of the outlet or settle unevenly. Some municipalities may not allow wood mulch for this reason.

### Is Pretreatment Needed?

Designing the inflow gutter to the rain garden at a minimal slope of 0.5% can facilitate sediment and debris deposition prior to flows entering the BMP. Be aware, this will reduce maintenance of the BMP, but may require more frequent sweeping of the gutter to ensure that the sediment does not impede flow into the rain garden.

- Consider all potential maintenance requirements such as mowing (if applicable) and replacement of the growing medium. Consider the method and equipment for each task required. For example, in a large rain garden where the use of hand tools is not feasible, does the shape and configuration of the rain garden allow for removal of the growing medium using a backhoe?
- Provide pre-treatment when it will reduce the extent and frequency of maintenance necessary to maintain function over the life of the BMP. For example, if the tributary is larger than one acre, prone to debris or the use of sand for ice control, consider a small forebay.
- Make the rain garden as shallow as possible. Increasing the depth unnecessarily can create erosive side slopes and complicate maintenance. Shallow rain gardens are also more attractive.
- Design and adjust the irrigation system (temporary or permanent) to provide appropriate water for the establishment and maintenance of selected vegetation.

# **Design Procedure and Criteria**

- 1. Subsurface Exploration and Determination of a No-Infiltration, Partial Infiltration, or Full Infiltration Section: Infiltration BMPs can have three basic types of sections. The appropriate section will depend on land use and activities, proximity to adjacent structures and soil characteristics. Sections of each installation type are shown in Figure B-1.
  - **No-Infiltration Section**: This section includes an underdrain and an impermeable liner that prevents infiltration of stormwater into the subgrade soils. Consider using this section when any of the following conditions exist:
    - The site is a stormwater hotspot and infiltration could result in contamination of groundwater.
    - The site is located over contaminated soils and infiltration could mobilize these contaminants.
    - o The facility is located over potentially expansive soils or bedrock that could swell due to infiltration and potentially damage adjacent structures (e.g., building foundation or pavement).
  - Partial Infiltration Section: This section does not include an impermeable liner, and allows some infiltration. Stormwater that does not infiltrate is collected and removed by an underdrain

system.

• Full Infiltration Section: This section is designed to infiltrate the water stored in the basin into the subgrade below. UDFCD recommends a minimum infiltration rate of 2 times the rate needed to drain the WQCV over 12 hours. A conservative design could utilize the partial infiltration section with the addition of a valve at the underdrain outlet. In the event that infiltration does not remain adequate following construction, the valve could be opened and allow this section to operate as a partial infiltration section.

A geotechnical engineer should scope and perform a subsurface study. Typical geotechnical investigation needed to select and design the section includes:

- Prior to exploration review geologic and geotechnical information to assess near-surface soil, bedrock and groundwater conditions that may be encountered and anticipated ranges of infiltration rate for those materials. For example, if the facility is located adjacent to a structure and the site is located in a general area of known shallow, potentially expansive bedrock, a no-infiltration section will likely be required. It is also possible that this BMP may be infeasible, even with a liner, if there is a significant potential for damage to the adjacent structures (e.g., areas of dipping bedrock).
- Drill exploratory borings or exploratory pits to characterize subsurface conditions beneath the subgrade and develop requirements for subgrade preparation. Drill at least one boring or pit for every 40,000 ft², and at least two borings or pits for sites between 10,000 ft² and 40,000 ft². The boring or pit should extend at least 5 feet below the bottom of the base, and at least 20 feet in areas where there is a potential of encountering potentially expansive soils or bedrock. More borings or pits at various depths may be required by the geotechnical engineer in areas where soil types may change, in low-lying areas where subsurface drainage may collect, or where the water table is likely within 8 feet below the planned bottom of the base or top of subgrade. Installation of temporary monitoring wells in selected borings or pits for monitoring groundwater levels over time should be considered where shallow groundwater is encountered.
- Perform laboratory tests on samples obtained from the borings or pits to initially characterize the subgrade, evaluate the possible section type, and to assess subgrade conditions for supporting traffic loads. Consider the following tests: moisture content (ASTM D 2216); dry density (ASTM D 2936); Atterberg limits (ASTM D 4318); gradation (ASTM D 6913); swell-consolidation (ASTM D 4546); subgrade support testing (R-value, CBR or unconfined compressive strength); and hydraulic conductivity. A geotechnical engineer should determine the appropriate test method based on the soil type.
- For sites where a full infiltration section may be feasible, perform on-site infiltration tests using a double-ring infiltrometer (ASTM D 3385). Perform at least one test for every 160,000 ft² and at least two tests for sites between 40,000 ft² and 160,000 ft². The tests should be located near completed borings or pits so the test results and subsurface conditions encountered in the borings can be compared, and at least one test should be located near the boring or pit showing the most unfavorable infiltration condition. The test should be performed at the planned top of subgrade underlying the growing media.
- Be aware that actual infiltration rates are highly variable dependent on soil type, density and moisture content and degree of compaction as well as other environmental and construction influences. Actual rates can differ an order of magnitude or more from those indicated by infiltration or permeability testing. Select the type of section based on careful assessment of the subsurface exploration and testing data.

The following steps outline the design procedure and criteria, with Figure B-1 providing a corresponding cross-section.

2. **Basin Storage Volume**: Provide a storage volume based on a 12-hour drain time.

Find the required WQCV (watershed inches of runoff). Using the imperviousness of the tributary area (or effective imperviousness where LID elements are used upstream), use Figure 3-2 located in Chapter 3 of this manual to determine the WQCV based on a 12-hour drain time.

Calculate the design volume as follows:

$$V = \left[\frac{\text{WQCV}}{12}\right] A$$
 Equation B-1

Where:

 $V = \text{design volume (ft}^3)$ 

A = area of watershed tributary to the rain garden (ft<sup>2</sup>)

3. **Basin Geometry:** UDFCD recommends a maximum WQCV ponding depth of 12 inches to maintain vegetation properly. Provide an inlet or other means of overflow at this elevation. Depending on the type of vegetation planted, a greater depth may be utilized to detain larger (more infrequent) events. The bottom surface of the rain garden, also referred to here as the filter area, should be flat. Sediment will reside on the filter area of the rain garden; therefore, if the filter area is too small, it may clog prematurely. If the filter area is not flat, the lowest area of the filter is more likely to clog as it will have a higher sediment loading. Increasing the filter area will reduce clogging and decrease the frequency of maintenance. Equation B-2 provides a minimum filter area allowing for some of the volume to be stored beyond the area of the filter (i.e., above the sideslopes of the rain garden).

Note that the total surcharge volume provided by the design must also equal or exceed the design volume. Where needed to meet the the required volume, also consider the porosity of the media at 14 percent. Use vertical walls or slope the sides of the basin to achieve the required volume. Sideslopes should be no steeper than 4:1 (horizontal:vertical).

$$A_{E} = 0.02AI$$

Equation B-2

Where:

 $A_F$ = minimum (flat) filter area (ft<sup>2</sup>)

A = area tributary to the rain garden (ft<sup>2</sup>)

I = imperviousness of area tributary to the rain garden (percent expressed as a decimal)

4. **Growing Medium:** Provide a minimum of 18 inches of growing medium to enable establishment of the roots of the vegetation (see Figure B-1). A previous version of this manual specified a mixture consisting of 85% coarse sand and a 15% compost/shredded paper mixture (by volume). Based on field monitoring of this medium, compost was removed to reduce export of nutrients and fines and silts were added to both benefit the vegetation and increase capture of metals in stormwater.

Table B-1 specifies the growing media as well as other materials discussed in this Fact Sheet. Growing media is engineered media that requires a high level of quality control and must almost always be imported. Obtaining a particle size distribution and nutrient analysis is the only way to ensure that the media is acceptable. UDFCD has identified placement of media not meeting the specification as the most frequent cause of failure. Sample the media after delivery and prior to placement or obtain a sample from the supplier in advance of delivery and placement and have this analyzed prior to delivery.

### **Other Rain Garden Growing Medium Amendments**

The specified growing medium was designed for filtration ability, clogging characteristics, and vegetative health. It is important to preserve the function provided by the rain garden growing medium when considering additional materials for incorporation into the growing medium or into the standard section shown in Figure B-1. When desired, amendments may be included to improve water quality or to benefit vegetative health as long as they do not add nutrients, pollutants, or modify the infiltration rate. For example, a number of products, including steel wool, capture and retain dissolved phosphorus (Erickson 2009). When phosphorus is a target pollutant, proprietary materials with similar characteristics may be considered. Do not include amendments such as top soil, sandy loam, and compost.

Table B-1. Material specification for bioretention/rain garden facilities

hdakanial		0.000					Notes
Material Bioretention Growing Media (soil + organics)	Bioretention soil Bioretention organics	Specification Particle size distribution: Barticle size distribution: 3-17% sold (0.05 - 2.0 mm diameter) 3-17% clay (<0.002 diameter) 3-17% clay (<0.002 diameter) Chemical attribute and nutrient analysis: pH 6.8 - 7.5 organic matter < 1.5% nitrogen < 15 ppm phosphorus < 15 ppm salinity < 6 mmhosfcm 3 to 5% shredded mulch (by weight of growing media)	wing media)		Submittals Particle size distribution and nutrient analysis required.	Diusa.	Notes Percentages are in weight.  bioretention soil required. Aged 6 months (minimum).
Landscape mulch		Shredded hardwood					Ageu o mortans (minimum). No weed fabric allowed.
			Mass Percen	Mass Percent Passing Square Mesh Siev			
		Sieve Size	Class B (	Class C			
	CDOT filter	19.0 mm (0.75")		100			
Underdrain	material (Class B	4.75 mm (No.4)	20-60	60-100	Particle size		
aggregate	or C as specified		10-30	3	distribution		
				10-30	required.		
		150 (No. 100)		0.10			
		75 um (No. 200)	63	0-3			•
		(000 000)				Pipe must conform to	
		Pine diameter and tune	Maximum	Minimum open area (per		requirements of ASTM designation F949. There shall be	
Underdrain Pipe			(inches)	Foot)	Required	no evidence of splitting, cracking, or breaking when the pipe is tested per ASTM test	Contech A-2000 slotted pipe (or equal)
		4-inch slotted PVC	0.032	1.90 in.²		method D2412 in accordance	
		6-inch slotted PVC	0.032	1.98 in.²		with F343 section 7.5 and ASTM F794 section 8.5.	
			Thickness 0.76 mm	Test method			
		Thickness, % Tolerance	+4-5	ASTMD 1593			
		Tensile strength, kN/m (Ibiin)	12.25 (70)	ASTM D8 82, method B		Thermal welding required for	
Impermeable liner			5.25 (30)	ASTM D8 82, method B	Reauired	fully lined facilities (not a	
		Ultimate elongation, %	320	ASTM D8 82, method A		cutain]. Leak testing in the field	
		Tear resistance, N(Ibs)	38 (8.5)	ASTM D 1004		required.	
		Low temperature impact, * C (* F)	-29 (-20)	ASTMD 1790	_		
		Volatile loss, % maximum Disholog no nor 9 m2 (no nor 10 ud 2)	1(2027)	ASTM D8 82, method A			
,		Bonded seam strength, % of tensile	(Yell)	¥ 2 × 2			•

5. **Underdrain System**: When using an underdrain system, provide a control orifice sized to drain the design volume in 12 hours or more (see Equation B-3). Use a minimum orifice size of 3/8 inch to avoid clogging. This will provide detention and slow release of the WQCV, providing water quality benefits and reducing impacts to downstream channels. Space underdrain pipes a maximum of 20 feet on center. Provide cleanouts to enable maintenance of the underdrain. Cleanouts can also be used to conduct an inspection (by camera) of the underdrain system to ensure that the pipe was not crushed or disconnected during construction.

Calculate the diameter of the orifice for a 12-hour drain time using Equation B-3 (Use a minimum orifice size of 3/8 inch to avoid clogging.):

$$D_{12 \text{ hour drain time}} = \sqrt{\frac{V}{1414 \, y^{0.41}}}$$
 Equation B-3

Where:

D = orifice diameter (in)

y = distance from the lowest elevation of the storage volume (i.e., surface of the filter) to the center of the orifice (ft)

V = volume (WQCV or the portion of the WQCV in the rain garden) to drain in 12 hours (ft<sup>3</sup>)

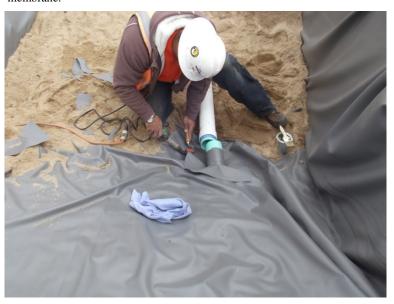
In previous versions of this manual, UDFCD recommended that the underdrain be placed in an aggregate layer and that a geotextile (separator fabric) be placed between this aggregate and the growing medium. This version of the manual replaces that section with materials that, when used together, eliminate the need for a separator fabric.

The underdrain system should be placed within an 6-inch-thick section of CDOT Class B or Class C filter material meeting the gradation in Table B-1. Use slotted pipe that meets the slot dimensions provided in Table B-3.

6. Impermeable Geomembrane **Liner and Geotextile Separator Fabric:** For noinfiltration sections, install a 30 mil (minimum) PVC geomembrane liner, per Table B-1, on the bottom and sides of the basin, extending up at least to the top of the underdrain layer. Provide at least 9 inches (12 inches if possible) of cover over the membrane where it is attached to the wall to protect the membrane from UV deterioration. The geomembrane should be fieldseamed using a dual track welder, which allows for nondestructive testing of almost all field seams. A small amount of single track is allowed in limited areas to seam around pipe perforations, to patch seams removed for destructive seam testing, and for limited repairs. The liner should be installed with slack to prevent tearing due to backfill, compaction, and settling. Place CDOT Class B geotextile separator fabric above the geomembrane to protect it from being punctured during the placement of the filter material above the liner. If the subgrade contains angular rocks or other material that could puncture the geomembrane, smooth-roll the surface to create a suitable surface. If smooth-rolling the surface does not provide a



**Photograph B-2.** The impermeable membrane in this photo has ripped from the bolts due to placement of the media without enough slack in the membrane.



**Photograph B-3**. Ensure a water-tight connection where the underdrain penetrated the liner. The heat-welded "boot" shown here is an alternative to the clamped detail shown in Figure B-2.

suitable surface, also place the separator fabric between the geomembrane and the underlying subgrade. This should only be done when necessary because fabric placed under the geomembrane can increase seepage losses through pinholes or other geomembrane defects. Connect the geomembrane to perimeter concrete walls around the basin perimeter, creating a watertight seal between the geomembrane and the walls using a continuous batten bar and anchor connection (see Figure B-3). Where the need for the impermeable membrane is not as critical, the membrane can be attached with a nitrile-based vinyl adhesive. Use watertight PVC boots for underdrain pipe penetrations through the liner (see Figure B-2) or the technique shown in photo B-3.

December	Class	В	Took Modes d		
Property	Elongation < 50% <sup>2</sup>	Elongation > 50% <sup>2</sup>	Test Method		
Grab Strength, N (lbs.)	800 (180)	510 (115)	ASTM D 4632		
Puncture Resistance, N (lbs.)	310 (70)	180 (40)	ASTM D 4833		
Trapezoidal Tear Strength, N (lbs.)	310 (70)	180 (40)	ASTM D 4533		
Apparent Opening Size, mm (US Sieve Size)	AOS < 0.3mm (US S	AOS < 0.3mm (US Sieve Size No. 50)			
Permittivity, sec <sup>-1</sup>		0.02 default value, must also be greater than that of soil			
Permeability, cm/sec	k fabric > k soil for all classes		ASTM D 4491		
Ultraviolet Degradation at 500 hours	50% strength retained	ed for all classes	ASTM D 4355		

Table B-2. Physical requirements for separator fabric<sup>1</sup>

7. **Inlet and Outlet Control:** In order to provide the proper drain time, the bioretention area can be restricted at the underdrain outlet with an orifice plate or can be designed without an underdrain

(provided the subgrade meets the requirements above). Equation B-3 is a simplified equation for sizing an orifice plate for a 12-hour drain time. UD-BMP or UD-Detention, available at <a href="www.udfcd.org">www.udfcd.org</a>, also perform this calculation.

How flow enters and exits the BMP is a function of the overall drainage concept for the site. Curb cuts can be designed to both allow stormwater into the rain garden as well as to provide release of stormwater in excess of the WQCV. Roadside rain gardens located on a steep site might pool and overflow into downstream cells with a single curb cut, level spreader, or outlet structure located at the most downstream cell. When selecting the



**Photograph B-4.** The curb cut shown allows flows to enter this rain garden while excess flows bypass the facility.

<sup>&</sup>lt;sup>1</sup> Strength values are in the weaker principle direction

<sup>&</sup>lt;sup>2</sup> As measured in accordance with ASTM D 4632

type and location of the outlet structure, ensure runoff will not short-circuit the rain garden. This is a frequent problem when using a curb inlet located outside the rain garden for overflow.

For rain gardens with concentrated points of inflow, provide a forebay and energy dissipation. A depressed concrete slab works best for a forebay. It helps maintain a vertical drop at the inlet and allows for easily removal of sediment using a square shovel. Where rock is used for energy dissipation, provide separator fabric between the rock and growing medium to minimize subsidence.

8. **Vegetation:** UDFCD recommends that the filter area be vegetated with drought tolerant species that thrive in sandy soils. Table B-3 provides a suggested seed mix for sites that will not need to be irrigated after the grass has been established.

Mix seed well and broadcast, followed by hand raking to cover seed and then mulched. Hydromulching can be effective for large areas. Do not place seed when standing water or snow is present or if the ground is frozen. Weed control is critical in the first two to three years, especially when starting with seed.

When using sod, specify sand–grown sod. Do not use conventional sod. Conventional sod is grown in clay soil that will seal the filter area, greatly reducing overall function of the BMP.

When using an impermeable liner, select plants with diffuse (or fibrous) root systems, not taproots. Taproots can damage the liner and/or underdrain pipe. Avoid trees and large shrubs that may interfere with restorative maintenance. Plant these outside of the area of growing medium. Use a cutoff wall to ensure that roots do not grow into the underdrain or place trees and shrubs a conservative distance from the underdrain.

9. **Irrigation:** Provide spray irrigation at or above the WQCV elevation or place temporary irrigation on top of the rain garden surface. Do not place sprinkler heads on the flat surface. Remove temporary irrigation when vegetation is established. If left in place this will become buried over time and will be damaged during maintenance operations.

Adjust irrigation schedules during the growing season to provide the minimum water necessary to maintain plant health and to maintain the available pore space for infiltration.

### **Designing for Flood Protection**

Provide the WQCV in rain gardens that direct excess flow into to a landscaped basin designed for flood control or design a single basin to provide water quality and flood control. See the *Storage* chapter in Volume 2 of the USDCM for more information. UD-Detention, available at www.udfcd.org, will facilitate design either alternative.

Table B-3. Native seed mix for rain gardens

Common Name	Scientific Name	Variety	PLS <sup>2</sup> lbs per Acre	Ounces per Acre
Sand bluestem	Andropogon hallii	Garden	3.5	
Sideoats grama	Bouteloua curtipendula	Butte	3	
Prairie sandreed	Calamovilfa longifolia	Goshen	3	
Indian ricegrass	Oryzopsis hymenoides	Paloma	3	
Switchgrass	Panicum virgatum	Blackwell	4	
Western wheatgrass	Pascopyrum smithii	Ariba	3	
Little bluestem	Schizachyrium scoparium	Patura	3	
Alkali sacaton	Sporobolus airoides		3	
Sand dropseed	Sporobolus cryptandrus		3	
Pasture sage <sup>1</sup>	Artemisia frigida			2
Blue aster <sup>1</sup>	Aster laevis			4
Blanket flower <sup>1</sup>	Gaillardia aristata			8
Prairie coneflower <sup>1</sup>	Ratibida columnifera			4
Purple prairieclover <sup>1</sup>	Dalea (Petalostemum) purpurea			4
Sub-Totals:			27.5	22
Total lbs per acre:			28.9	

Wildflower seed (optional) for a more diverse and natural look.

PLS = Pure Live Seed.

# **Aesthetic Design**

In addition to effective stormwater quality treatment, rain gardens can be attractively incorporated into a site within one or several landscape areas. Aesthetically designed rain gardens will typically either reflect the character of their surroundings or become distinct features within their surroundings. Guidelines for each approach are provided below.

### **Reflecting the Surrounding**

- Determine design characteristics of the surrounding. This becomes the context for the drainage improvement. Use these characteristics in the structure.
- Create a shape or shapes that "fix" the forms surrounding the improvement. Make the improvement part of the existing surrounding.
- The use of material is essential in making any new improvement an integral part of the whole. Select materials that are as similar as possible to the surrounding architectural/engineering materials. Select materials from the same source if possible. Apply materials in the same quantity, manner, and method as original material.
- Size is an important feature in seamlessly blending the addition into its context. If possible, the overall size of the improvement should look very similar to the overall sizes of other similar objects in the improvement area.

## **Reflective Design**

A reflective design borrows the characteristics, shapes, colors, materials, sizes and textures of the built surroundings. The result is a design that fits seamlessly and unobtrusively in its environment.

• The use of the word texture in terms of the structure applies predominantly to the selection of plant material. The materials used should as closely as possible, blend with the size and texture of other plant material used in the surrounding. The plants may or may not be the same, but should create a similar feel, either individually or as a mass.

### **Creating a Distinct Feature**

Designing the rain garden as a distinct feature is limited only by budget, functionality, and client preference. There is far more latitude in designing a rain garden that serves as a distinct feature. If this is the intent, the main consideration beyond functionality is that the improvement create an attractive addition to its surroundings. The use of form, materials, color, and so forth focuses on the improvement itself and does not necessarily reflect the surroundings, depending on the choice of the client or designer.

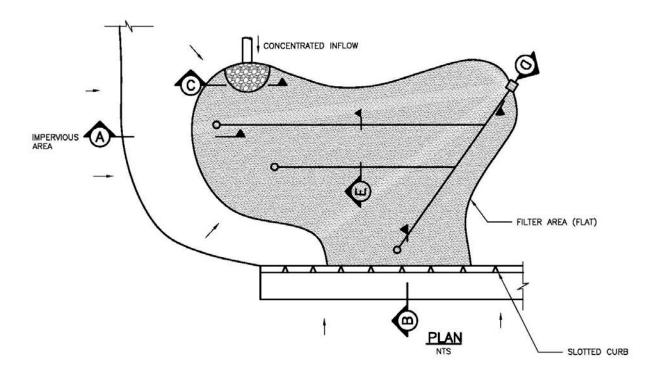
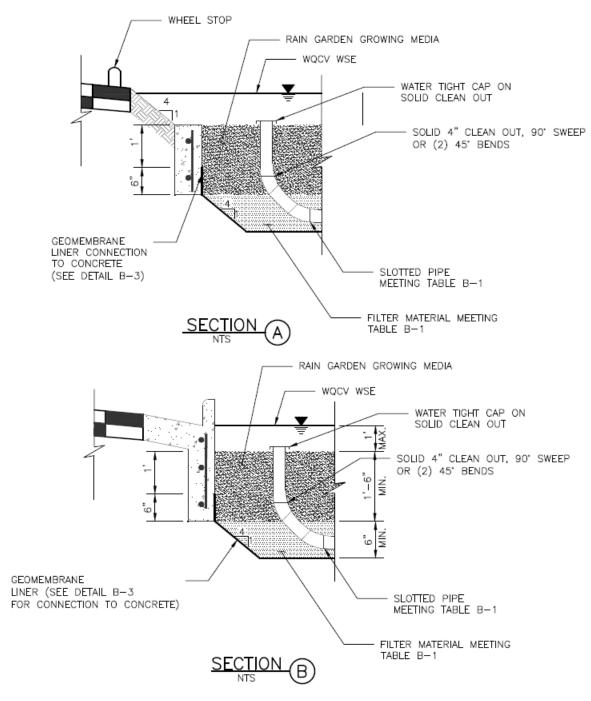
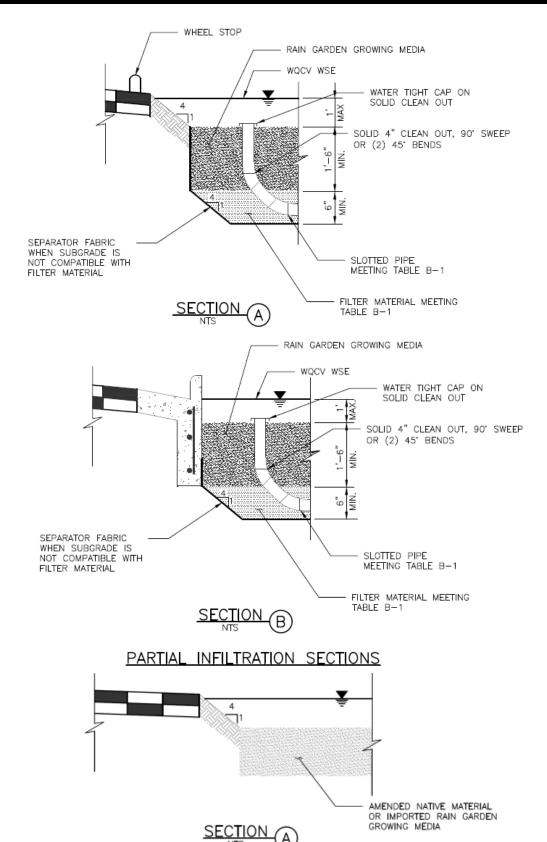


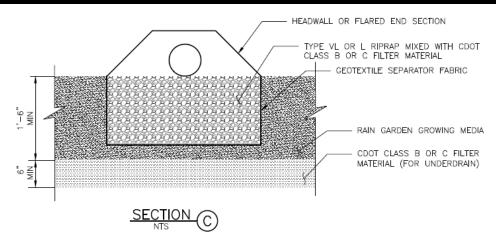
Figure B-1 – Typical rain garden plan and sections

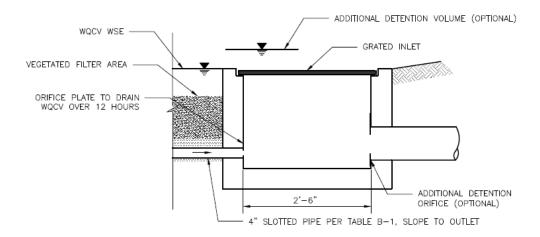


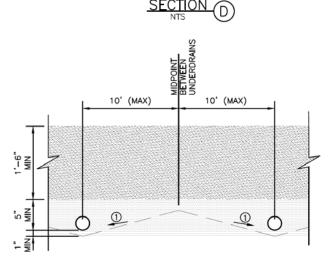
NO-INFILTRATION SECTIONS



## FULL INFILTRATION SECTION







① SLOPE (STRAIGHT GRADE) SUBGRADE (2-10%) TO UNDERDRAIN TO REDUCE SATURATED SOIL CONDITIONS BETWEEN STORM EVENTS (OPTIONAL)



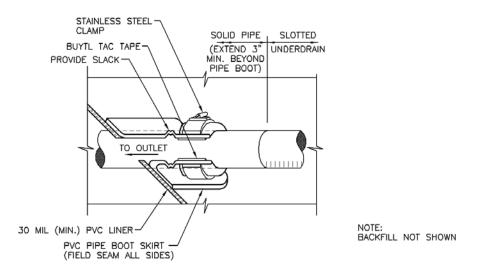


Figure B-2. Geomembrane Liner/Underdrain Penetration Detail

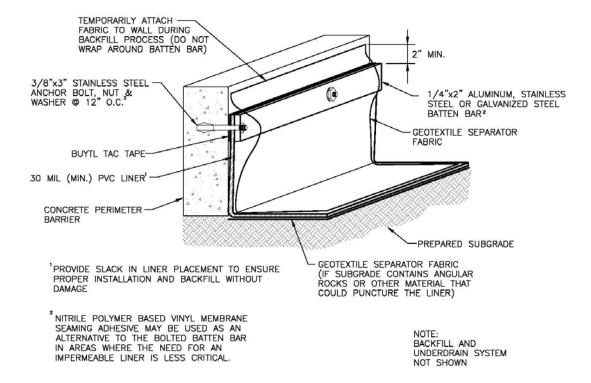


Figure B-3. Geomembrane Liner/Concrete Connection Detail

Bioretention T-3

#### **Construction Considerations**

Proper construction of rain gardens involves careful attention to material specifications, final grades, and construction details. For a successful project, implement the following practices:

- Protect area from excessive sediment loading during construction. This is the most common cause of clogging of rain gardens. The portion of the site draining to the rain garden must be stabilized before allowing flow into the rain garden. This includes completion of paving operations.
- Avoid over compaction of the area to preserve infiltration rates (for partial and full infiltration sections).
- Provide construction observation to ensure compliance with design specifications. Improper installation, particularly related to facility dimensions and elevations and underdrain elevations, is a common problem with rain gardens.
- When using an impermeable liner, ensure enough slack in the liner to allow for backfill, compaction, and settling without tearing the liner.
- Provide necessary quality assurance and quality control (QA/QC) when constructing an impermeable geomembrane liner system, including but not limited to fabrication testing, destructive and non-destructive testing of field seams, observation of geomembrane material for tears or other defects, and air lace testing for leaks in all field seams and penetrations. QA/QC should be overseen by a professional engineer. Consider requiring field reports or other documentation from the engineer.
- Provide adequate construction staking to
  ensure that the site properly drains into the
  facility, particularly with respect to surface drainage away from adjacent buildings. Photo B-3 and
  Photo B-4 illustrate a construction error for an otherwise correctly designed series of rain gardens.



**Photograph B-3.** Inadequate construction staking may have contributed to flows bypassing this rain garden.



**Photograph B-4.** Runoff passed the upradient rain garden, shown in Photo B-3, and flooded this downstream rain garden.

T-3 Bioretention

#### References

Erickson, Andy. 2009. Field Applications of Enhanced Sand Filtration. University of Minnesota *Stormwater Management Practice Assessment Project Update*. <a href="http://wrc.umn.edu">http://wrc.umn.edu</a>.

Hunt, William F., Davis, Allen P., Traver, Robert. G. 2012. "Meeting Hydrologic and Water Quality Goals through Targeted Bioretention Design" *Journal of Environmental Engineering*. (2012) 138:698-707. Print.

#### **HEC-RAS CALCULATIONS**

Unresolved:

Scour analysis needs to be provided for structure at Arroya Lane since Culvert report did not address this





SAND CREEK REACH 9 - LOOKING UPSTREAM

(EX. 60" CMP AT ARROYA LN.)



SAND CREEK REACH 9 - LOOKING UPSTREAM

(AT ARROYA LN.)



SAND CREEK REACH 9 – JUST SOUTH OF ARROYA LN.

(EX. 60" CMP)



SAND CREEK REACH 9 - LOOKING DOWNSTREAM (JUST SOUTH OF ARROYA LN.)



SAND CREEK REACH 9 - LOOKING UPSTREAM

(NEAR STA. 4+00.58)



SAND CREEK REACH 9 - LOOKING DOWNSTREAM (NEAR STA: 4+00.58)



SAND CREEK REACH 9 - LOOKING UPSTREAM



SAND CREEK REACH 9 - LOOKING DOWNSTREAM (NEAR STA: 3+00.14)



SAND CREEK REACH 9 - LOOKING UPSTREAM (NEAR STA: 2+00.22)



SAND CREEK REACH 9 - LOOKING DOWNSTREAM (NEAR STA: 2+00.22)

TABLE 10-1

## COMPOSITE ROUGHNESS COEFFICIENTS FOR UNLINED OPEN CHANNELS (Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m$$
 (10-2)

	Channel Conditions	<u>Value</u>
Material Type  n o	Earth Fine Gravel Coarse Gravel	0.020 0.024 0.028
Degree of Irregularity  n 1	Smooth Minor Moderate Severe	0.000 0.005 0.010 0.020
Variation of Channel Cross Section <sup>n</sup> 2	Gradual Alternating Occasionally Alternating Frequently	0.000 0.005 0.010 - 0.015
Relative Effect of Obstructions n <sub>3</sub>	Negligible Minor Appreciable Severe	0.000 0.010 - 0.015 0.020 - 0.030 0.040 - 0.060
Vegetation <sup>n</sup> 4	Low Medium High Very High	0.005 - 0.010 0.010 - 0.025 0.025 - 0.050 0.050 - 0.100
Degree of Meandering '	Minor Appreciable Severe	1.000 - 1.200 1.200 - 1.500 1.500

#### TRBLE 10-2 (Continued)

#### TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Type	of Channel and Description	Minimum	Normal	Maximum								
NATURAL STREAMS												
	streams (top width at flood 100 ft)											
а.	Streams on plain											
a.	1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033								
	2. Same as above, but more stones and weeds	0.030	0.035	0.040								
	3. Clean, winding, some pools and shoals	0.033	0.040	0.045								
	4. Same as above, but some weeds and stones	0.035	0.045	0.050								
	5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055								
	6. Same as 4, but more stones	0.045	0.050	0.060								
	7. Sluggish reaches, weedy, deep pools	0.050	0.070									
	8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150								
LINED	OR BUILT-UP CHANNELS											
a.	Corrugated Metal	0.021	0.025	0.030								
b.	Concrete											
	1. Trowel finish	0.011	0.013	0.015								
	2. Float finish	0.013	0.015	0.016								
	3. Finished, with gravel on bottom		0 017	0.020								
	4. Unfinished	0.014	0.017	0.020								
	5. Gunite, good section	0.016										
	6. Gunite, wavy section	0.018	0.022	0.025								
	7. On good excavated rock	0.017	0.020									
	8. On irregular excavated rock	0.022	0.027									

**Table 3.** Adjustment values for factors that affect roughness of flood plains [Modified from Aldridge and Garrett, 1973, table 2]

Flood-plain co	onditions	n value adjustment	Example
10752	Smooth	0.000	Compares to the smoothest, flattest flood plain attainable in a given bed material.
Degree of	Minor	0.001-0.005	Is a flood plain slightly irregular in shape. A few rises and dips or sloughs may be visible on the flood plain.
irregularity $(n_1)$	Moderate Severe	0.006-0.010 0.011-0.020	Has more rises and dips. Sloughs and hummocks may occur.  Flood plain very irregular in shape. Many rises and dips or sloughs are visible.  Irregular ground surfaces in pastureland and furrows perpendicular to the flow are also included.
Variation of flood-plain cross section (n <sub>2</sub> )	,	0.0	Not applicable.
Effect of obstructions	Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, or isolated boulders, occupy less than 5 percent of the cross-sectional area.
$(n_3)$	Minor Appreciable	0.005-0.019 0.020-0.030	Obstructions occupy less than 15 percent of the cross-sectional area.  Obstructions occupy from 15 to 50 percent of the cross-sectional area.
	Small	0.001-0.010	Dense growth of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation, or supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.011-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation, or moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season.
Amount of vegetation $(n_4)$	Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation, or 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft, or mature row crops such as small vegetables, or mature field crops where depth of flow is at least twice the height of the vegetation.
	Very large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation, or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is below branches, or mature field crops where depth of flow is less than the height of the vegetation.
	Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar (all vegetation in full foliage), or heavy stand of timber, few down trees, depth of flow reaching branches.
Degree of meander (m)	V	1.0	Not applicable.

Chow (1959) presents a table showing minimum, normal, and maximum values of n for flood plains covered by pasture and crops. These values are helpful for comparing the roughness values of flood plains having similar vegetation.

#### Vegetation-Density Method

For a wooded flood plain, the vegetation-density method can be used as an alternative to the previous method for determining *n* values for flood plains. In a wooded flood plain, where the tree diameters can be measured, the vegetation density of the flood plain can be determined.

Determining the vegetation density is an effective way of relating plant height and density characteristics, as a function of depth of flow, to the flow resistance of vegetation. Application of the flow-resistance model presented below requires an estimate of the vegetation density as a function of depth of flow. The procedure requires a direct or indirect determination of vegetation density at a given depth. If the change in n value through a range in depth is required, then an estimation of vegetation density through that range is necessary.

#### **Techniques for Determining Vegetation Density**

Petryk and Bosmajian (1975) developed a method of analysis of the vegetation density to determine the rough**Classification of Vegetal Covers** 

Retardance Class	Cover	Condition			
Α	Weeping lovegrass	Excellent stand, tall, average 30 in.			
A	Yellow bluestem Ischaemum	Excellent stand, tall, average 36 in.			
	Bermuda grass	Good stand, tall, average 12 in.			
	Native grass mixture (little bluestem, bluestem, blue gamma, and other long and short Midwest grasses	Good stand, unmowed			
	Weeping lovegrass	Good stand, tall, average 24 in.			
3	Lespedeza serica	Good stand, not woody, tall, average 19 in.			
	Alfalfa	Good stand uncut, average 11 in.			
	Weeping lovegrass	Good stand, unmowed, average 13 in.			
	Kudzu	Dense growth, uncut			
	Blue gamma	Good stand, uncut, average 13 in.			
	Crabgrass	Fair stand, uncut, avg. 10 in.			
	Bermuda grass	Good stand, mowed, average 6 in.			
	Common lespedeza	Good stand, uncut, average 11 in.			
	Grass-legume mixture - summer (orchard grass, redtop Italian ryegrass, and common lespedeza)	Good stand, uncut, average 6 to 8 in.			
	Centipedegrass	Very dense cover, average 6 in.			
	Kentucky Bluegrass	Good stand, headed, 6 to 12 in.			
	Bermuda grass	Good stand, cut to 2.5 in. height			
	Common lespedeza	Excellent stand, uncut, average 4.5 in.			
	Buffalo Grass	Good stand, uncut, 3 t 6 in.			
1	Grass-legume mixture - fall (orchard grass, redtop Italian ryegrass, and common lespedeza)	Good stand, uncut, 3 to 5 in.			
	Lespedeza serica	After cutting to 2 in. height, good stand before cutting			
	Bermuda grass	Good stand, cut to average 1.5 in. heigh			
	Bermuda grass	Burned stubble			
lote: Covers cla niform.	ssified have been tested in experimental char	nnels. Covers were green and generally			
ource: HEC-15					



#### Coefficients for Roughness of Grass-Lined Channels

SCS Retardance Class	$C_{n}$
A	0.605
В	0.418
С	0.220
D	0.147
E	0.093

#### **Composite Roughness**

Culverts using different materials for portions of the perimeter such as embedded culverts or culverts with an invert liner should use a composite Manning's n value. A weighted n value based on the materials can be derived using the following equation:

$$n_c = \left[rac{\Sigma(p_i n_i^{1.5})}{p}
ight]^{0.67}$$

Where:

 $n_c$  = Composite/weighted Manning's n.

 $p_i$  = Wetted perimeter for the material, ft.

 $n_i$  = Manning's n value for the material.

p = Total wetted perimeter, ft.

#### 750.1.4.1.2 Hydraulic Radius

The hydraulic radius is a characteristic depth of flow and is defined as the cross-sectional area of flow divided by the wetted perimeter of the channel. The hydraulic radius is computed as follows:

$$R = \frac{A}{P}$$

where:

R = hydraulic radius, ft

 $A = cross-sectional area of flow, ft^2$ 

P = wetted perimeter of the channel cross section, ft

#### 750.1.4.1.3 Slope

Table 8–8 Characteristics of selected grass species for use in channels and waterways

0	Height at maturity					
Grass species	(ft)	(m)				
Cool-season grasses						
Creeping foxtail	3–4	0.9-1.2				
Crested wheatgrass	2–3	0.6-0.9				
Green needlegrass	3-4	0.9-1.2				
Russian wild rye	3–4	0.9-1.2				
Smooth bromegrass	3–4	0.9-1.2				
Tall fescue	3-4	0.9-1.2				
Tall wheatgrass		1.2-1.5				
Western wheatgrass	2–3	0.6-0.9				
Warm-season grasses	•					
Bermudagrass	3/42	0.2-0.6				
Big bluestem	4-6	1.2-1.8				
Blue grama	1–2	0.3-0.6				
Buffalograss	1/3-1	0.1-0.3				
Green spangletop	3–4	0.9-1.2				
Indiangrass	5–6	1.5-1.8				
Klein grass	3-4	0.9-1.2				
Little bluestem	3-4	0.9-1.2				
Plains bristlegrass	1–2	0.3-0.6				
Sand bluestem	56	1.5-1.8				
Sideoats grama	2–3	0.6-0.9				
Switchgrass	4–5	1.2-1.5				
Vine mesquitegrass	1–2	0.3-0.6				
Weeping lovegrass	3–4	0.9-1.2				
Old World bluestems						
Caucasian bluestem	4–5	1.2-1.5				
Ganada yellow bluestem	3-4	0.9 - 1.2				



Table 8–9 Retardance curve index by SCS retardance class

SCS retardance class	Retardance curve index
A	10.0
В	7.64
C	5.60
D	4.44
E	2.88

this table were obtained from a review of the available qualitative descriptions and stem counts reported by researchers studying channel resistance and stability.

Since cover conditions vary from year to year and season to season, it is recommended that an upper and lower bound be determined for  $C_\Gamma$ . The lower bound should be used in stability computations, and the upper bound should be used to determine channel capacity. Some practitioners find that the use of SCS retardance class (table 8–9) is a preferable approach.

The vegetal cover index, C<sub>F</sub>, depends primarily on the density and uniformity of density in the immediate vicinity of the soil boundary. Because this parameter is associated with the prevention of local erosion damage which may lead to channel unraveling, the cover factor should represent the weakest area in a reach, rather than the average for the cover species. Recommended values for the cover factor are presented in table 8–10. Values in this table do not account for such considerations as maintenance practices or uniformity of soil fertility or moisture. Therefore, appropriate engineering judgment should be used in its application.

**Table 8–10** Properties of grass channel linings values (apply to good uniform stands of each cover)

Cover factor (C <sub>F</sub> )	Covers tested	Reference stem density (stems/ft²)	Reference stem density (stems/m²)		
0.90	Bermudagrass	500	5,380		
	Centipede grass	500	5,380		
0.87	Buffalograss	400	4,300		
	Kentucky bluegrass	350	3,770		
	Blue grama	350	3,770		
0.75	Grass mixture	200	2,150		
0.50	Weeping lovegrass Yellow bluestem	$\frac{350}{250}$	3,770 2,690		
0.50	Alfalfa	500	5,380		
	Lespedeza sericea	300	3,280		
0.50	Common lespedeza	150	1,610		
	Sudangrass	50	538		

Multiply the stem densities given by 1/3, 2/3, 1, 4/3, and 5/3 for poor, fair, good, very good, and excellent covers, respectively. Reduce the  $\rm C_p$  by 20% for fair stands and 50% for poor stands.

Two soil parameters are required for application of effective stress concepts to the stability design of lined or unlined channels having an erodible soil boundary: soil grain roughness,  $n_{\rm s}$ , and allowable effective stress,  $\tau_{\rm a}$ . When the effective stress approach is used, the soil parameters are the same for both lined and unlined channels with negligible bed-material sediment transport.

Soil grain roughness is defined as the roughness associated with particles or aggregates of a size that can be independently moved by the flow at incipient channel failure. For noncohesive soils, the soil grain roughness and effective shear stress are both a function of the  $D_{75}$  grain size. When  $D_{75}$  is greater than 1.3 millimeter, the soil is considered coarse grained. When  $D_{75}$  is less than 1.3 millimeter, the soil is considered fine grained. Fine-grained roughness is considered to have a constant value of 0.0156. Fine-grained effective shear stress is taken to have a constant value of 0.02 pound per square foot. Coarse-grained shear stress and roughness are given in figures 8–21 and 8–22.

A soil grain roughness of 0.0156 is assigned to all cohesive soils. The allowable effective stresses are a function of the unified soil classification system soil type, the plasticity index, and the void ratio. The basic allowable shear stress,  $\tau_{ab}$ , is determined from the plasticity index and soil classification, and then adjusted by the void ratio correction factor,  $C_{\rm e}$ , using the following equation:

$$\tau_a = \tau_{ab} C_e^2$$
 (eq. 8–29)

The basic allowable effective stress can be determined from figure 8–23 and the void ratio correction factor from figure 8–24. These two figures were developed directly from the allowable velocity curves in AH 667. Stress partitioning (slope partitioning) is essential to application of figures 8–21 to 8–24, with or without vegetation (Temple et al. 1987).

#### (e) General design procedure

Use the basic shear stress equation to determine effective shear stress on the soil beneath the vegetation. Use any consistent units of measurement.

$$\tau_e = \gamma dS (1 - C_F) \left(\frac{n_s}{n}\right)^2$$
 (eq. 8–30)

#### where:

τ<sub>e</sub> = effective shear stress exerted on the soil beneath vegetation (lb/ft² or N/m²)

 $\gamma$  = specific weight of water (lb/ft<sup>3</sup> or N/m<sup>3</sup>)

S = energy slope, dimensionless

C<sub>F</sub> = vegetation cover factor (0 for unlined channel), dimensionless

 $n_s$  = grain roughness of underlying soil, typically taken as dimensionless

n = roughness coefficient of vegetation, typically taken as dimensionless

The flow depth is used instead of the hydraulic radius because this will result in the maximum local shear stress, rather than the average shear stress. The cover factor is a function of the grass and stem density. Roughness coefficients are standard Manning's roughness values;  $n_{\rm s}$  can be determined from figure 8–22, n can be determined from the old SCS curves (fig. 8–20) or from the following equation.

$$n_{_{\rm R}} = \exp \left\{ C_{_{\rm I}} \left[ \, 0.0133 \left( \ln R_{_{\rm V}} \, \right)^2 - 0.0954 \ln R_{_{\rm V}} + 0.297 \, \right] - 4.16 \right\} \tag{eq. 8-31}$$

where:

 $R_{\nu} = (VR/\nu) \times 10^{5}$  (this dimensionless term reduces to VR for practical application in English units)

V = channel velocity (ft/s or m/s)

R = hydraulic radius (ft or m)

Limited to  $0.0025C_1^{2.5} < R_y < 36$ 

A reference value of Manning's resistance coefficient,  $n_R$  is applicable to vegetation established on relatively smoothly graded fine-grained soil.

If vegetated channel liner mats are used, manufacturer-supplied roughness coefficients for particular mats may be used in the equation.

Maximum allowable shear stress,  $\tau_{va}$ , in pound per square foot is determined as a function of the retardance curve index,  $C_{\Gamma}$ . Very little information is available for vegetal performance under very high stresses and this relation is believed to be conservative.

$$\tau_{\rm va} = 0.75C_{\rm I}$$
 (eq. 8–32)

#### 8.1 Riprap Sizing

Procedures for sizing rock to be used in soil riprap, void-filled riprap, and riprap over bedding are the same.

#### 8.1.1 Mild Slope Conditions

When subcritical flow conditions occur and/or slopes are mild (less than 2 percent), UDFCD recommends the following equation (Hughes, et al, 1983):

$$d_{50} \ge \left\lceil \frac{VS^{0.17}}{4.5(G_s - 1)^{0.66}} \right\rceil^2$$
 Equation 8-11

Where:

V = mean channel velocity (ft/sec)

S = longitudinal channel slope (ft/ft)

 $d_{50}$  = mean rock size (ft)

Gs = specific gravity of stone (minimum = 2.50, typically 2.5 to 2.7), Note: In this equation (Gs -1) considers the buoyancy of the water, in that the specific gravity of water is subtracted from the specific gravity of the rock.

Note that Equation 8-11 is applicable for sizing riprap for channel lining with a longitudinal slope of no more than 2%. This equation is not intended for use in sizing riprap for steep slopes (typically in excess of 2 percent), rundowns, or protection downstream of culverts. Information on rundowns is provided in Section 7.0 of the *Hydraulic Structures* chapter of the USDCM, and protection downstream of culverts is discussed in the *Culverts and Bridges* chapter. For channel slopes greater than 2% use one of the methods presented in 8.1.2.

Rock size does not need to be increased for steeper channel side slopes, provided the side slopes are no steeper than 2.5H:1V (UDFCD 1982). Channel side slopes steeper than 2.5H:1V are not recommended because of stability, safety, and maintenance considerations. See Figure 8-34 for riprap placement specifications. At the upstream and downstream termination of a riprap lining, the thickness should be increased 50% for at least 3 feet to prevent undercutting.

#### 8.1.2 Steep Slope Conditions

Steep slope rock sizing equations are used for applications where the slope is greater than 2 percent and/or flows are in the supercritical flow regime. The following rock sizing equations may be referred to for riprap design analysis on steep slopes:

- CSU Equation, Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase II
  (prepared by S.R. Abt, et al, Colorado State University, 1988). This method was developed for steep
  slopes from 2 to 20 percent.
- USDA- Agricultural Research Service Equations, Design of Rock Chutes (by K.M. Robinson, et al, USDA- ARS, 1998 Transactions of ASAE) and An Excel Program to Design Rock Chutes for Grade



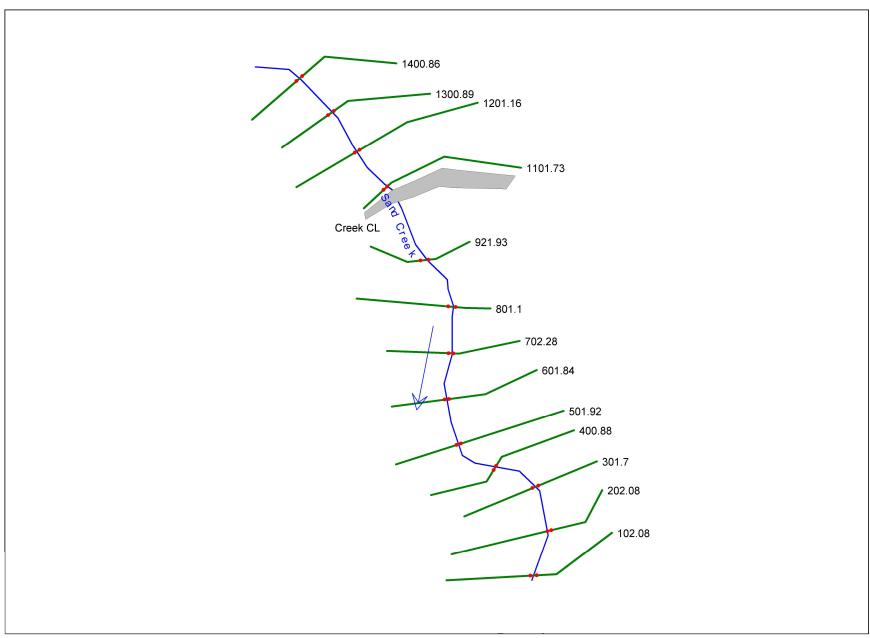
619 N. Cascade Avenue, Suite 200 Colorado Springs, CO 80903

D.	Timber Ridge 3	
Project:		<u>NOTES</u>
Date:	12 16/22	☐ Telephone Record
Contact:		☐ Note to the File
Phone:		☐ Job Information
	MA.1.	— □ Meeting Minutes
By:	L. L.O.	

Rip-rap sizing for channel stabilization:
- Mild slope andiform
- uce equation 8-11 for MHFD
$d_{20} \ge \left(\frac{4.2(6^{2}-1)^{0.00}}{\Lambda^{20.13}}\right)^{2}$
mean value ity = V = 4.2 ft/r. (Obtained from HEC-RAS Model)
longifudinal rlope = S = 1.58%.
Gr specific gravity of Abre = Gr = 2.5
14.2(0.0158)0.17,2
$d_{r} \ge \frac{(4.5(0.0158)^{0.17})^2}{4.5(2.5-1)^{0.66}} \ge 0.12 \text{ ft.}$
dro min. = 60
Juce dro = 9" type L

Ph: 719.785.0790 Fax: 719.785.0799

Innovative Design....Classic Results



#### **HEC-RAS Model (Ver. 6.3.1) Input Data**

#### Input based on the following:

Table 10-1 & 10-2 from DCM

Table 3 from USGS Guide for selecting Manning's Roughness Coefficients

Site visits and photographic documentation of the channel

Classification of Vegetal covers from HEC-15

SCS Retardance Class form HEC-15

Flows based on current approved Sand Creek DBPS, FEMA and SR MDDP

Manning's n Values: 0.12 overbank

0.07 channel

Standard Channel coefficients: Contraction 0.1

Expansion 0.3

Flows modeled: FEMA 100-yr 2600 cfs

DBPS 100-yr 2170 cfs

SR MDDP 100-yr 1468 cfs

DBPS 10-yr 630 cfs

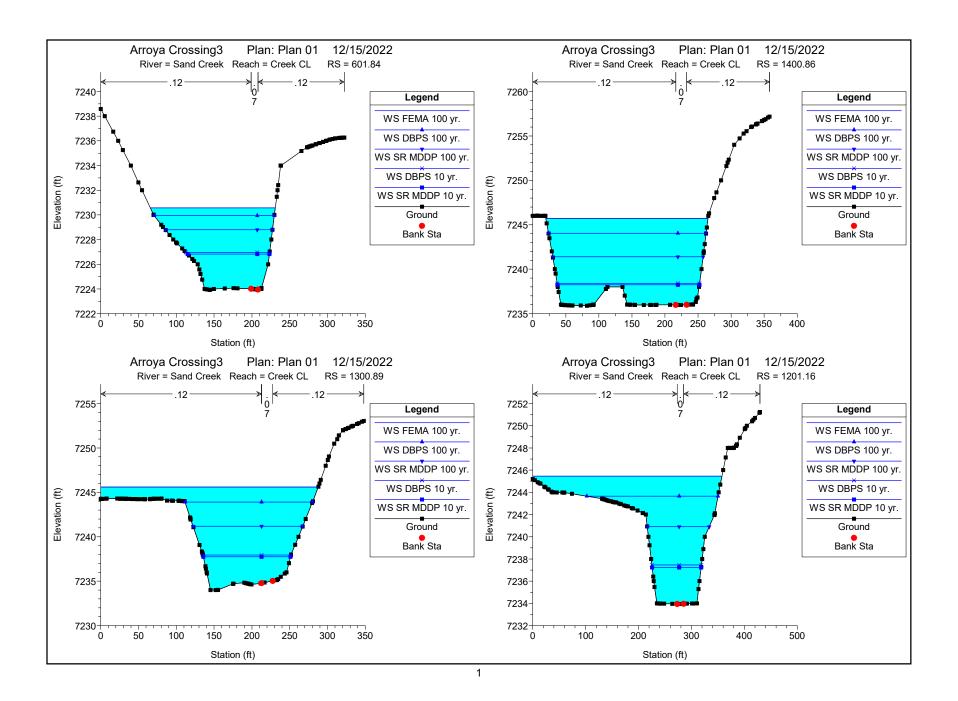
SR MDDP 10-yr 581 cfs

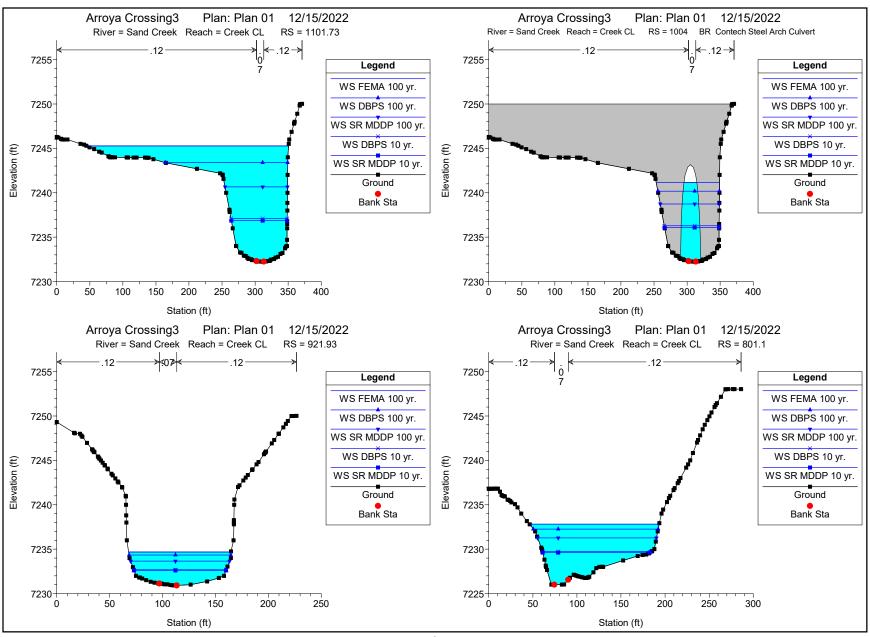
HEC-RAS Plan: Plan 01 River: Sand Creek Reach: Creek CL

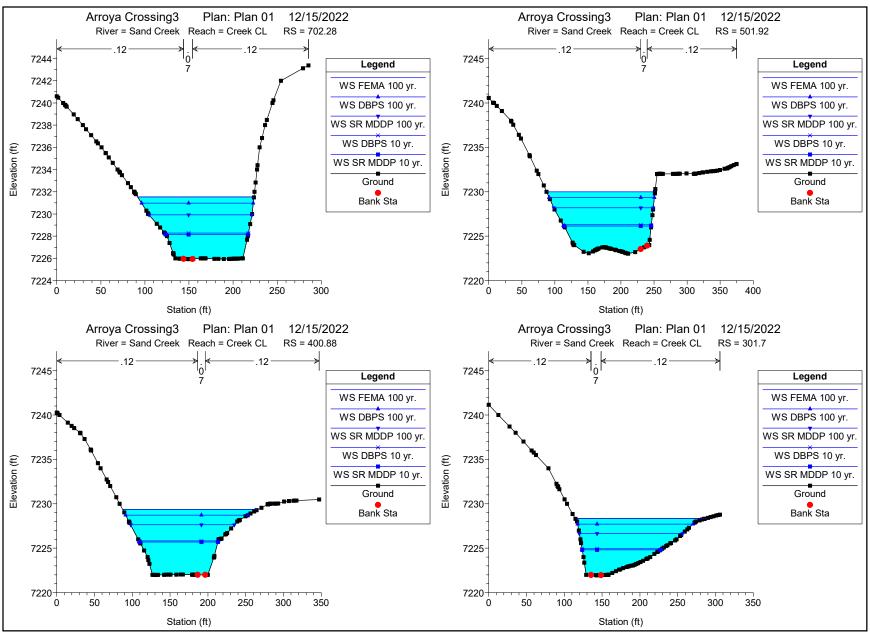
HEC-RAS Plan: Plan 01 River: Sand Creek Reach: Creek CL															
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Max Chl Dpth	Hydr Radius	E.G. Elev	E.G. Slope	Vel Total	Shear Total	Flow Area	Top Width	Froude # XS
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(lb/sq ft)	(sq ft)	(ft)	
Creek CL	1400.86	FEMA 100 yr.	2600	7235.97	7245.72		9.83	8.49	7245.75	0.000500	1.23	0.26	2120.99	245.14	0.08
Creek CL	1400.86	DBPS 100 yr.	2170	7235.97	7244.03		8.14	7.07	7244.06	0.000682	1.27	0.30	1712.36	238.34	0.09
Creek CL	1400.86	SR MDDP 100 yr.	1468	7235.97	7241.39		5.50	4.76	7241.42	0.001287	1.34	0.38	1096.16	227.49	0.12
Creek CL	1400.86	DBPS 10 yr.	630	7235.97	7238.41		2.52	2.01	7238.45	0.004620	1.44	0.58	436.23	215.24	0.20
Creek CL	1400.86	SR MDDP 10 yr.	581	7235.97	7238.24		2.35	1.86	7238.28	0.005171	1.45	0.60	400.76	214.58	0.21
Creek CL	1300.89	FEMA 100 yr.	2600	7234.78	7245.60		11.60	5.65	7245.67	0.001262	1.58	0.44	1647.31	287.76	0.16
Creek CL	1300.89	DBPS 100 yr.	2170	7234.78	7243.90		9.90	7.00	7243.97	0.001289	1.81	0.56	1198.34	168.69	0.14
Creek CL	1300.89	SR MDDP 100 yr.	1468	7234.78	7241.18		7.18	5.25	7241.26	0.002085	1.90	0.68	771.69	145.25	0.17
Creek CL	1300.89	DBPS 10 yr.	630	7234.78	7237.95		3.95	2.97	7238.01	0.004065	1.81	0.75	348.91	116.50	0.20
Creek CL	1300.89	SR MDDP 10 yr.	581	7234.78	7237.75		3.75	2.80	7237.81	0.004279	1.78	0.75	325.98	115.40	0.20
Creek CL	1201.16	FEMA 100 yr.	2600	7233.95	7245.45		11.50	4.62	7245.54	0.001450	1.56	0.42	1667.26	357.53	0.19
Creek CL	1201.16	DBPS 100 yr.	2170	7233.95	7243.65		9.70	4.31	7243.78	0.002552	2.01	0.69	1080.54	247.89	0.25
Creek CL	1201.16	SR MDDP 100 yr.	1468	7233.95	7240.90		6.95	5.44	7241.01	0.002776	2.25	0.94	651.10	117.13	0.20
Creek CL	1201.16	DBPS 10 yr.	630	7233.95	7237.46		3.51	3.11	7237.55	0.005286	2.13	1.03	295.40	93.73	0.24
Creek CL	1201.16	SR MDDP 10 yr.	581	7233.95	7237.23		3.28	2.92	7237.32	0.005655	2.12	1.03	274.30	92.59	0.24
Creek CL	1101.73	FEMA 100 yr.	2600	7232.23	7245.27	7236.42	13.04	4.65	7245.38	0.001550	1.77	0.45	1471.35	305.27	0.22
Creek CL	1101.73	DBPS 100 yr.	2170	7232.23	7243.39	7236.02	11.16	5.05	7243.55	0.002228	2.20	0.70	986.02	185.22	0.24
Creek CL	1101.73	SR MDDP 100 yr.	1468	7232.23	7240.67	7235.31	8.44	6.54	7240.77	0.002056	2.20	0.84	666.72	94.45	0.17
Creek CL	1101.73	DBPS 10 yr.	630	7232.23	7237.10	7234.26	4.87	3.91	7237.17	0.002744	1.82	0.67	346.54	85.06	0.19
Creek CL	1101.73	SR MDDP 10 yr.	581	7232.23	7236.86	7234.19		3.72	7236.93	0.002817	1.78	0.65	325.98	84.42	0.19
		,									-			-	
Creek CL	1004		Bridge												
			J												
Creek CL	921.93	FEMA 100 yr.	2600	7230.92	7234.69	7234.69	3.77	3.10	7236.24	0.077887	8.55	15.06	303.96	96.74	0.99
Creek CL	921.93	DBPS 100 yr.	2170	7230.92	7234.34	7234.34	3.42	2.79	7235.72	0.077980	8.01	13.57	271.03	96.20	0.99
Creek CL	921.93	SR MDDP 100 yr.	1468	7230.92	7233.66	7233.66	2.74	2.19	7234.77	0.083660	7.12	11.44	206.14	93.56	1.00
Creek CL	921.93	DBPS 10 yr.	630	7230.92	7232.68	7232.68	1.76	1.34	7233.33	0.087755	5.37	7.33	117.40	87.46	
Creek CL	921.93	SR MDDP 10 yr.	581	7230.92	7232.61	7232.61	1.69	1.27	7233.23	0.089498	5.25	7.09	110.71	86.98	
0.1102					1 = 2 = 10 1					0.000.00	0.20				
Creek CL	801.1	FEMA 100 yr.	2600	7226.00	7232.81		6.81	4.40	7233.21	0.010288	3.97	2.83	655.02	146.93	0.42
Creek CL	801.1	DBPS 100 yr.	2170	7226.00	7232.27		6.27	4.02	7232.63	0.010344	3.76	2.60	576.68	141.67	0.42
Creek CL	801.1	SR MDDP 100 yr.	1468	7226.00	7231.27		5.27	3.22	7231.57	0.010538	3.35	2.12	438.72	134.87	0.43
Creek CL	801.1	DBPS 10 yr.	630	7226.00	7229.72		3.72	1.91	7229.95	0.010921	2.65	1.30	237.33	123.27	0.49
Creek CL	801.1	SR MDDP 10 yr.	581	7226.00	7229.61		3.61	1.83	7229.83	0.010877	2.60	1.24	223.41	121.51	0.49
		,											-		
Creek CL	702.28	FEMA 100 yr.	2600	7225.93	7231.54		5.60	4.27	7231.96	0.015564	4.55	4.15	571.27	132.13	0.45
Creek CL	702.28	DBPS 100 yr.	2170	7225.93	7230.97		5.04	3.89	7231.36	0.016116	4.35	3.91	498.38	126.81	0.45
Creek CL	702.28	SR MDDP 100 yr.	1468	7225.93	7229.93		4.00	3.15	7230.25	0.017513	3.96	3.45	371.10	116.69	0.45
Creek CL	702.28	DBPS 10 yr.	630	7225.93	7228.28		2.34	2.05	7228.48	0.021117	3.21	2.71	196.19	94.86	0.44
Creek CL	702.28	SR MDDP 10 yr.	581	7225.93	7228.15		2.22	1.97	7228.34	0.021572	3.15	2.65	184.67	93.34	0.44
,			551		1220.10				. 220.04	5.52.0.2	30	2.30	.001	55.54	
Creek CL	601.84	FEMA 100 yr.	2600	7223.93	7230.57		6.65	4.62	7230.80	0.008293	3.37	2.39	772.26	165.36	0.32
Creek CL	601.84	DBPS 100 yr.	2170	7223.93	7229.94		6.02	4.18	7230.16	0.008719	3.24	2.28	670.44		0.32
Creek CL	601.84	SR MDDP 100 yr.	1468	7223.93	7228.80		4.88	3.49	7228.98	0.009075	2.94	1.98	499.28	141.84	0.32
Creek CL	601.84	DBPS 10 yr.	630	7223.93	7226.96		3.04	2.38	7227.07	0.009772	2.37	1.45	265.56	110.79	0.31
Creek CL	601.84	SR MDDP 10 yr.	581	7223.93	7226.82		2.90	2.30	7226.93	0.009731	2.32	1.40	250.58	108.34	0.30
2.55,, 52			301	. 220.00	. 220.02		2.50	2.00	. 223.00	3.333.01	2.02	40	200.00	.55.04	
Creek CL	501.92	FEMA 100 yr.	2600	7223.54	7229.99		6.97	5.40	7230.14	0.005104	2.89	1.72	898.54	164.00	0.24
STOCK OL	1001.02	1. 2.100 yr.	2000	1 220.04	1220.00		0.81	5.40	, 200.14	0.000104	2.03	1.72	030.34	104.00	0.24

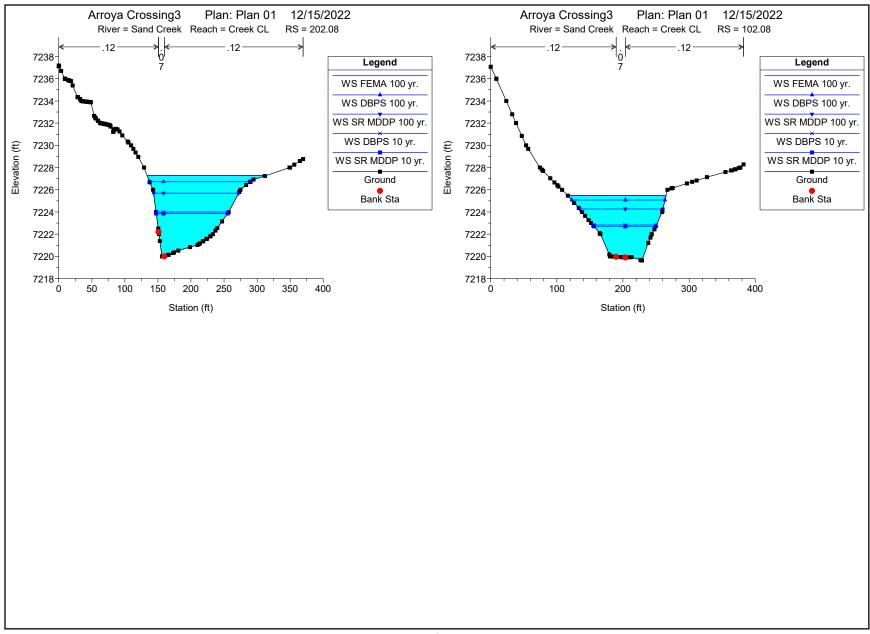
HEC-RAS Plan: Plan 01 River: Sand Creek Reach: Creek CL (Continued)

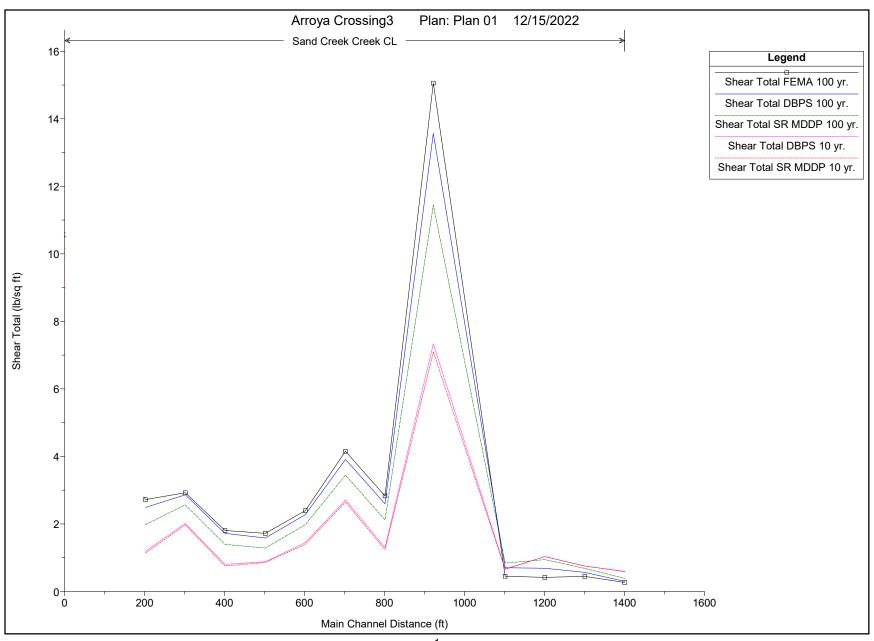
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	Max Chl Dpth	Hydr Radius	E.G. Elev	E.G. Slope	Vel Total	Shear Total	Flow Area	Top Width	Froude # XS
rtodori	Tuvor ou	Tromo	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(lb/sq ft)	(sq ft)	(ft)	1 Todde II Ac
Creek CL	501.92	DBPS 100 yr.	2170	7223.54	7229.35	( )	6.32	4.93	7229.48	0.005147	2.73	1.58	794.54	159.21	0.23
Creek CL	501.92	SR MDDP 100 yr.	1468	7223.54	7228.20		5.18	4.05	7228.30	0.005083	2.38	1.29	616.68	150.50	0.22
Creek CL	501.92	DBPS 10 yr.	630	7223.54	7226.29		3.27	2.58	7226.35	0.005489	1.82	0.88	345.84	133.28	0.21
Creek CL	501.92	SR MDDP 10 yr.	581	7223.54	7226.14		3.12	2.46	7226.20	0.005589	1.78	0.86	326.35	131.90	0.21
Creek CL	400.88	FEMA 100 yr.	2600	7222.00	7229.36		7.37	4.58	7229.59	0.006323	3.15	1.81	826.28	179.06	0.31
Creek CL	400.88	DBPS 100 yr.	2170	7222.00	7228.72		6.73	4.40	7228.92	0.006266	3.02	1.72	717.58	161.79	0.30
Creek CL	400.88	SR MDDP 100 yr.	1468	7222.00	7227.63		5.64	4.02	7227.78	0.005561	2.64	1.39	555.95	137.12	0.27
Creek CL	400.88	DBPS 10 yr.	630	7222.00	7225.82		3.83	3.18	7225.89	0.004024	1.87	0.80	336.50	104.72	0.21
Creek CL	400.88	SR MDDP 10 yr.	581	7222.00	7225.68		3.69	3.08	7225.75	0.003904	1.81	0.75	321.81	103.44	0.20
Creek CL	301.7	FEMA 100 yr.	2600	7221.95	7228.35		6.40	3.88	7228.73	0.012090	3.85	2.93	675.90	172.75	0.44
Creek CL	301.7	DBPS 100 yr.	2170	7221.95	7227.72		5.77	3.72	7228.07	0.012322	3.78	2.86	574.64	152.79	0.43
Creek CL	301.7	SR MDDP 100 yr.	1468	7221.95	7226.64		4.69	3.04	7226.95	0.013532	3.50	2.56	419.10	136.83	0.45
Creek CL	301.7	DBPS 10 yr.	630	7221.95	7224.94		2.99	1.97	7225.17	0.016340	2.99	2.01	210.87	106.06	0.48
Creek CL	301.7	SR MDDP 10 yr.	581	7221.95	7224.81		2.86	1.89	7225.04	0.016744	2.95	1.97	197.05	103.56	0.49
Creek CL	202.08	FEMA 100 yr.	2600	7219.99	7227.28		7.29	3.97	7227.57	0.010968	3.59	2.72	724.26	180.67	0.38
Creek CL	202.08	DBPS 100 yr.	2170	7219.99	7226.72		6.73	4.09	7226.96	0.009730	3.44	2.49	631.06	152.61	0.34
Creek CL	202.08	SR MDDP 100 yr.	1468	7219.99	7225.70		5.71	3.77	7225.87	0.008428	3.00	1.98	489.56	128.58	0.31
Creek CL	202.08	DBPS 10 yr.	630	7219.99	7224.00		4.01	2.59	7224.09	0.007364	2.19	1.19	287.09	109.93	0.27
Creek CL	202.08	SR MDDP 10 yr.	581	7219.99	7223.86		3.87	2.50	7223.95	0.007292	2.13	1.14	272.30	108.00	0.27
Creek CL	102.08	FEMA 100 yr.	2600	7219.90	7225.48	7224.11	5.85	3.56	7226.08	0.020806	4.89	4.62	532.12	148.81	0.58
Creek CL	102.08	DBPS 100 yr.	2170	7219.90	7225.07	7223.65	5.44	3.33	7225.60	0.019986	4.60	4.15	471.98	141.04	0.56
Creek CL	102.08	SR MDDP 100 yr.	1468	7219.90	7224.26	7222.87	4.63	2.89	7224.67	0.018369	4.03	3.31	364.38	125.65	0.53
Creek CL	102.08	DBPS 10 yr.	630	7219.90	7222.83		3.20	2.12	7223.05	0.015837	3.08	2.10	204.78	95.97	0.46
Creek CL	102.08	SR MDDP 10 yr.	581	7219.90	7222.72		3.09	2.07	7222.93	0.015543	2.99	2.01	194.19	93.51	0.45

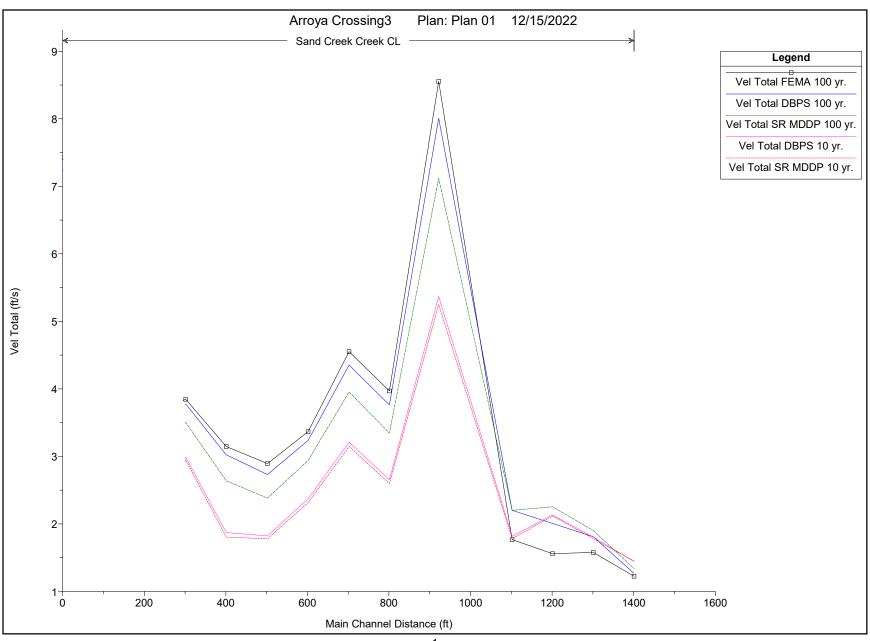












# ARROYA LANE CULVERT CALCULATIONS CULVERT DESIGN DOCUMENTS (CONTECH/KBJW PLANS)



				I 15 X 5½	-RADIUS ARCH	. BRIDGECOR 2	TABLE 74				
				LOADING)	GUIDE (HL-93	GHT OF COVER	LRFD HEI				
	Dimensions to Inside Corrugation  Gage Thickness (Inches) - Height of Cover Shown in Feet  Maximum Height of Cover (Minimum Height of Cover)										
Precon	3/8 (0.380)	5/16 (0.318)	1 (0.280)	3 (0.249)	5 (0.218)	7 (0.188)	8 (0.170)	Total S	Approx. Area (Sq. Ft.)	Rise (FtIn.)	Maximum Span (FtIn.)
2	48 (1.5)	43 (1.5)	38 (1.5)	33 (1.5)	28 (1.5)	24 (2.0)	20 (2.0)	21	124	8-4	18-5
2	44	39	34 (1.5)	29	25	21	17	25	173	10-0	22-0
2	(1.5)	(1.5)	27	(1.5)	(1.5)	(2.0)	(2.0)	25	172	9-3	23-5
2	(1.5)	(1.5)	(1.5)	(1.5)	(1.5)	(2.0)	(2.0)	29	228	11-7	25-5
2	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	29	233	10-10	26-11
2	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.5)	28	212	9-10	27-2
2	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.5) 15	(2.5)	33	306	13-2	28-11
2	(2.0)	(2.0)	(2.0)	(2.0) 18	(2.0) 15	(2.0)	(2.0)	34	320	12-8	31-8
2	(2.0)	(2.0)	(2.0) 18	(2.0) 16	(2.0)	(2.0)	(2.5)	33	295	11-8	31-11
2	(2.0)	(2.0) 16	(2.0) 14	(2.0) 12	(2.0) 11	(2.5)	(2.5)	32	273	10-8	32-2
2	(2.0)	(2.0)	(2.0)	(2.0) 17	(2.0)	(2.0)	(2.0)	37	385	14-9	32-2
2	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	35	318	11-4	35-10
+	(2.0)	(2.0)	(2.0)	(2.0) 15	(2.0)	(2.0)	(2.0)				
2	(2.0)	(2.0)	(2.0)	(2.0) 15	(2.0)	(2.0)	(2.0)	41	473	16-5	35-11
2	(2.0)	(2.0) 18	(2.0) 16	(2.0) 14	(2.0) 13	(2.0) 11	(2.0)	39	420	14-5	36-5
2	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)	(2.5)	39	412	13-8	37-10
3	16 (2.0)	14 (2.5)	12 (2.0)	11 (2.0)	10 (2.0)	(2.0)	8 (2.0)	38	382	12-9	38-1
3	20 (2.0)	18 (2.0)	16 (2.0)	14 (2.0)	12 (2.0)	10 (2.0)		45	569	18-0	39-5
3	(2.0)	18 (2.0)	16 (2.0)	14 (2.0)	12 (2.0)	10 (2.0)		43	512	16-1	39-11
3	10 (2.0)	9 (2.0)	8 (2.0)	7 (2.0)	6 (2.0)			39	387	12-2	40-10
3	19 (2.0)	17 (2.0)	15 (2.0)	13 (2.0)	11 (2.0)	9 (2.0)		44	525	15-6	42-7
3	18 (2.0)	17 (2.0)	15 (2.0)	14 (2.0)	(2.0)			49	675	19-7	42-11
3	(2.0)	(2.0)	(2.0)	(2.0)	(2.0)			42	458	13-7	43-1
3	13 (2.0)	12 (2.0)	(2.0)	10 (2.0)	(2.0)			44	513	14-9	44-1
3	18 (2.0)	16 (2.0)	14 (2.0)	13 (2.0)	(2.0)			48	637	17-10	44-7
3	16 (2.0)	14 (2.0)	13 (2.0)	12 (2.0)	10 (2.0)			53	790	21-2	46-5
4	16 (2.0)	14 (2.0)	12 (2.0)	11 (2.0)	10 (2.0)			49	652	17-4	47-4
4	9 (2.0)	8 (2.0)	7 (2.0)	6 (2.0)	5 (2.0)			46	539	14-5	48-0
4	16 (2.0)	14 (2.0)	13 (2.0)	12 (2.0)	10 (2.0)			52	749	19-6	48-1
4	16 (2.0)	14 (2.0)	12 (2.0)	11 (2.0)	(2.0)			50	676	17-7	48-7
4	15 (2.0)	13 (2.0)	11 (2.0)	10 (2.0)	(2.0)			57	914	22-10	49-11
4	14 (2.0)	12	10	8				50	662	16-10	50-0
4	10	(2.5)	(2.5)	(2.5) 7 (2.5)				49	623	15-10	50-3
4	(2.0) 15 (2.0)	(2.5) 13 (2.5)	(2.5) 12 (2.5)	(2.5) 11 (2.5)				54	804	19-11	50-7

- Notes:

  1. Not for a specific structural design. Use for budget estimating only. Finite Element Analysis (CANDE) is required for final design and quotation.

  2. The above table is based upon the general requirements of the AASHTO LRFD Design Specification, Section 12, and:

  a. These tables are based upon a trench condition installation. A finite element analysis is required to determine this width in an embankment installation.

  b. Backfill material per AASHTO M 145. For estimating, assume any A-1 material.

  With specific site information, alternate backfill materials may be allowed. Refer to Table 69 (pg. 93).

  c. Select backfill 120 pcf in density and compacted to a minimum 90% density per AASHTO T-180.

  d. The minimum cover is the vertical distance from the mid-depth (neutral axis) of the corrugations to the top of rigid or bottom of flexible pavement per AASHTO 12.8.9.4

  e. A minimum select backfill width of 8 feet was used when analyzing the table above. Other widths may be acceptable based on an alternate finite element analysis. This width was measured from outside the maximum span on each side of the structure. This width only applies when the material adjacent to the select zone is determined to be competent, well consolidated material.

  3. Select backfill width may increase for situations where lower strength fill exists in either the select fill zone or the adiacent embankment zone
- 3. Select backfill width may increase for situations where lower strength fill exists in either the select fill zone or the adjacent embankment zone.

  4. This estimate is for single barrel structures. For multiple barrels, more investigation is required.

## **HY-8 Culvert Analysis Report**

#### **Crossing Discharge Data**

Discharge Selection Method: User Defined

Table 1 - Summary of Culvert Flows at Crossing: Arroya Lane

Headwater Elevation (ft)	Discharge Names	Total Discharge (cfs)	BridgeCor Steel Culvert	Roadway Discharge (cfs)	Iterations
			Discharge (cfs)		
7238.58	SR MDDP	1482.00	1482.00	0.00	1
7240.75	SC DBPS	2170.00	2170.00	0.00	1
7242.03	FEMA	2600.00	2600.00	0.00	1
7248.60	Overtopping	4781.84	4781.84	0.00	Overtopping

#### **Culvert Data: BridgeCor Steel Culvert**

#### **Culvert Data Summary - BridgeCor Steel Culvert**

Barrel Shape: User Defined

Barrel Span: 32.00 ft

Barrel Rise: 11.70 ft

Barrel Material: Corrugated Metal Riveted or Welded

Embedment: 0.00 in

Barrel Manning's n: 0.0240 (top and sides)

Manning's n: 0.0350 (bottom)

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall (Ke=0.5)

Inlet Depression: None

#### **Site Data - BridgeCor Steel Culvert**

Site Data Option: Culvert Invert Data

Inlet Station: 100.00 ft

Inlet Elevation: 7231.50 ft

Outlet Station: 190.00 ft

Outlet Elevation: 7230.60 ft

Number of Barrels: 1

#### **Roadway Data for Crossing: Arroya Lane**

Roadway Profile Shape: Constant Roadway Elevation

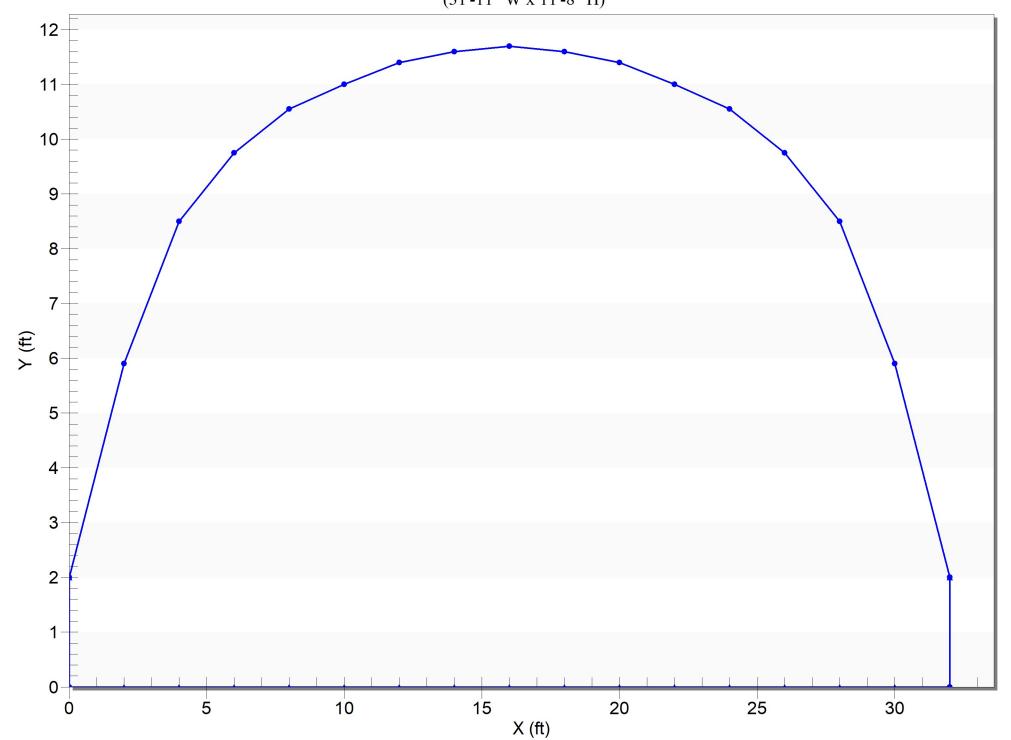
Crest Length: 100.00 ft

Crest Elevation: 7248.60 ft

Roadway Surface: Paved

Roadway Top Width: 80.00 ft

## Bridgecor Steel Culvert - Arroya Lane (31'-11" W x 11'-8" H)



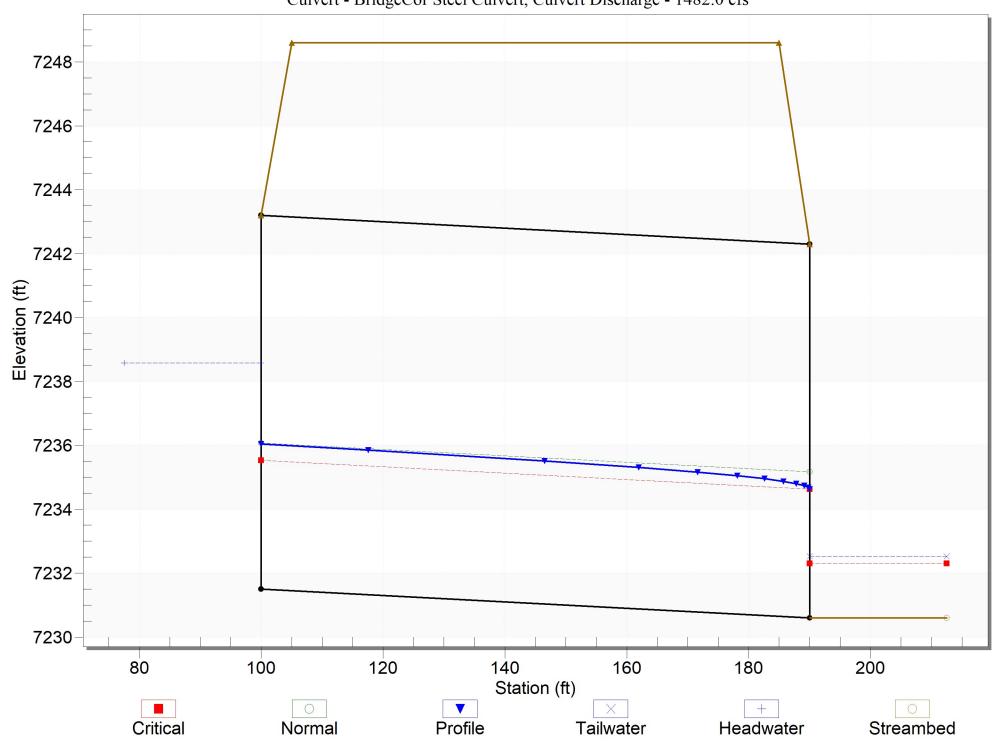
## Culvert Crossing: Arroya Lane

#### **Culvert Summary Table - BridgeCor Steel Culvert**

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
SR MDDP	1482.00	1482.00	7238.58	6.69	7.08	2- M2c	4.57	4.03	4.03	1.92	11.68	6.18
SC DBPS	2170.00	2170.00	7240.75	8.75	9.25	2- M2c	5.97	5.21	5.21	2.40	13.45	7.12
FEMA	2600.00	2600.00	7242.03	9.87	10.53	2- M2c	6.84	5.88	5.88	2.66	14.40	7.61

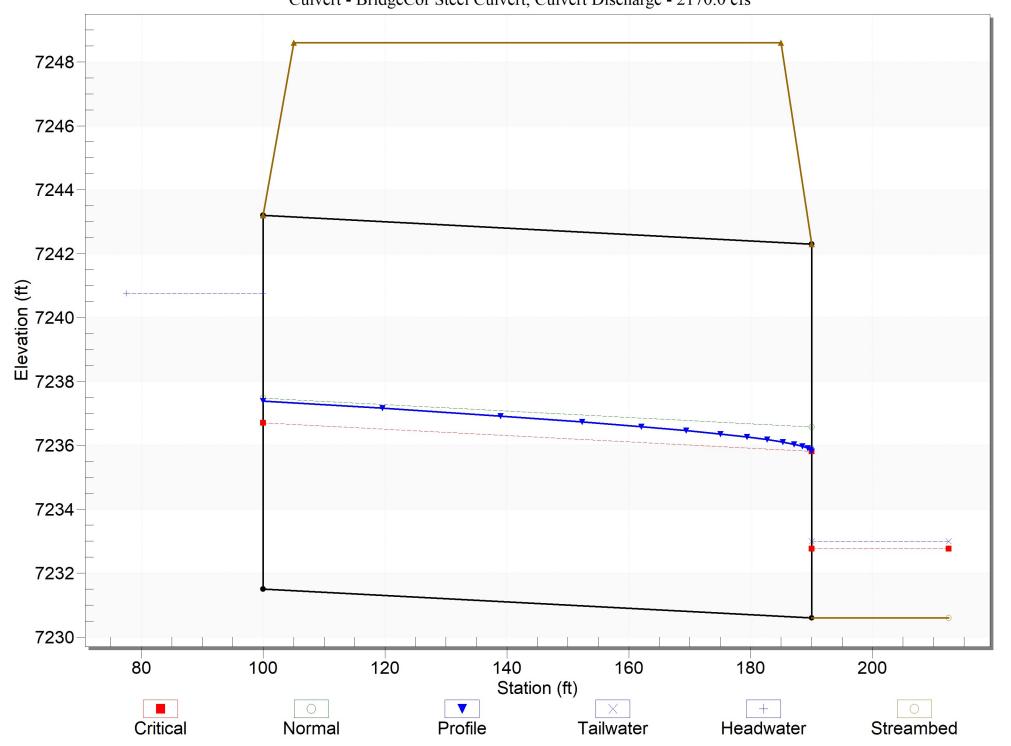
## Crossing - Arroya Lane, Design Discharge - 2170.0 cfs

Culvert - BridgeCor Steel Culvert, Culvert Discharge - 1482.0 cfs



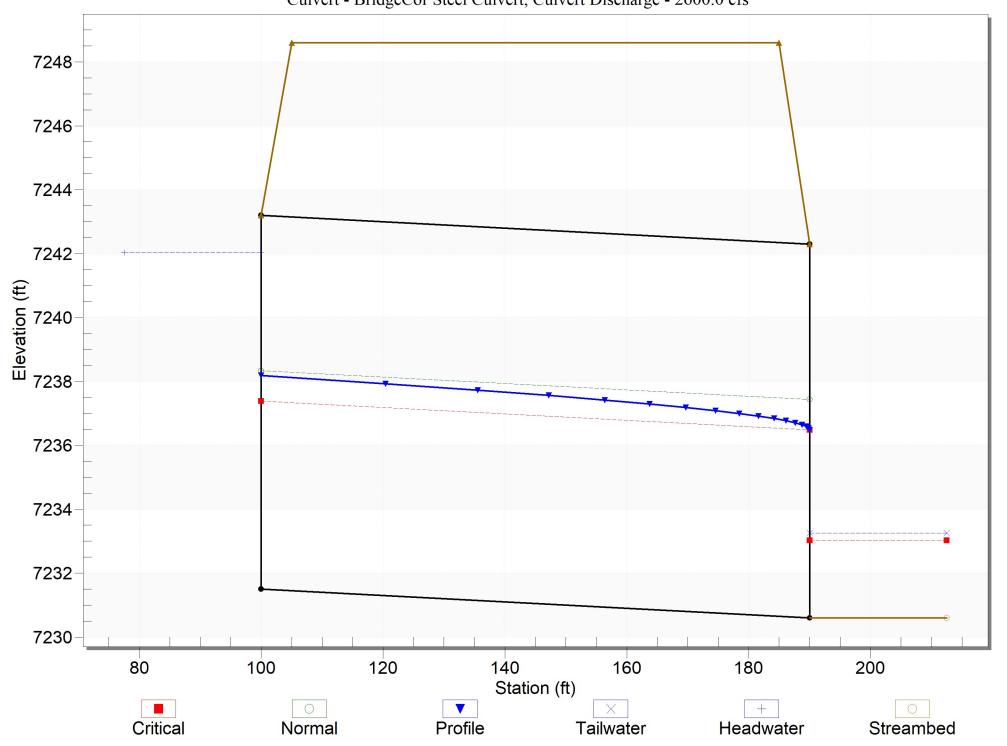
## Crossing - Arroya Lane, Design Discharge - 2170.0 cfs

Culvert - BridgeCor Steel Culvert, Culvert Discharge - 2170.0 cfs



## Crossing - Arroya Lane, Design Discharge - 2170.0 cfs

Culvert - BridgeCor Steel Culvert, Culvert Discharge - 2600.0 cfs

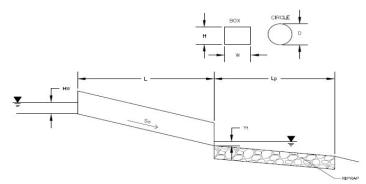


## DETERMINATION OF CULVERT HEADWATER AND OUTLET

MHFD-Culvert, Version 4.00 (May 2020)

Project: RETREAT AT TIMBERRIDGE FILING NO. 3

ID: BRIDGECOR STEEL BOX (31'-11" span X 11'-8" rise) 295 sq-ft. area



Soil Type:	
Choose One:  Sandy	
O Non-Sandy	

		- KIP KAP	<u> </u>	
Design Info	ormation:	_		_
	Design Discharge	Q =	2170	cfs
Circular Culv	ert:	_		_
	Barrel Diameter in Inches	D = [		inches
	Inlet Edge Type (Choose from pull-down list)			
_	<u>R:</u>			
Box Culvert:		_	OR	
	Barrel Height (Rise) in Feet	H (Rise) =	9.24	ft
	Barrel Width (Span) in Feet	W (Span) =	31.92	ft
	Inlet Edge Type (Choose from pull-down list)	1:1 Bevel w/ 45 deg	g. Flared Wingwa	ıll
	Number of Barrels	# Barrels =	1	
	Inlet Elevation	Elev IN =	7231.5	<b>T</b> ft
	Outlet Elevation OR Slope	Elev OUT =	7230.7	Tft .
	Culvert Length	L =	83	∏ft
	Manning's Roughness	n =	0.035	
	Bend Loss Coefficient	k <sub>b</sub> =	0	
	Exit Loss Coefficient	k <sub>x</sub> =	1	
	Tailwater Surface Elevation	Y <sub>t, Elevation</sub> =		<b>T</b> ft
	Max Allowable Channel Velocity	V =	5	ft/s
	,			<b>_</b> ·
Calculated I	Results:			
	Culvert Cross Sectional Area Available	A =	294.94	Tft²
	Culvert Normal Depth	Y <sub>n</sub> =	6.06	Πft
	Culvert Critical Depth	Y <sub>c</sub> =	5.23	Πft
	Froude Number	Fr =	0.80	
	Entrance Loss Coefficient	k <sub>e</sub> =	0.50	
	Friction Loss Coefficient	$k_f =$	0.54	
	Sum of All Loss Coefficients	$k_s =$	2.04	ft
Headwater:				
	Inlet Control Headwater	$HW_{T} = \Gamma$	8.16	Πft
	Outlet Control Headwater	HW <sub>O</sub> =	8.15	Π <sub>ft</sub>
	Design Headwater Elevation	HW =	7239.66	ft
	Headwater/Diameter OR Headwater/Rise Ratio	HW/H=	0.88	
Outlet Prote	otion.			
Jouliet Fible	Flow/(Span * Rise^1.5)	Q/WH^1.5 =	2.42	ft <sup>0.5</sup> /s
	Tailwater Surface Height	V <sub>t</sub> =	3.70	ft /3
	Tailwater/Rise	Yt/H =	0.40	Ⅎ``
	Expansion Factor	$1/(2*tan(\Theta)) =$	5.04	┥
	Flow Area at Max Channel Velocity	$A_t = \begin{bmatrix} 1/(2 & tan(O)) - 1 \\ A_t = \begin{bmatrix} 1/(2 & tan(O)) \end{bmatrix} \end{bmatrix}$	434.00	ft²
	Width of Equivalent Conduit for Multiple Barrels	W <sub>eq</sub> =	-	⊣'t
	Length of Riprap Protection	L <sub>p</sub> =	93	⊣"t
	Width of Riprap Protection at Downstream End	T =	51	ft
	Adjusted Rise for Supercritical Flow	- На = Г	_	
	Minimum Theoretical Riprap Size	d <sub>50</sub> min=	9	in
	Nominal Riprap Size	$d_{50}$ nominal=	12	
	MHFD Riprap Type	Type =	M	<b>⊣</b> "'

Type M Rip-rap used d50=12", Depth = 24"

#### GENERAL NOTES:

- 1. THIS SPECIFICATION COVERS THE MANUFACTURE AND INSTALLATION OF THE GALVANIZED STEEL DEEP CORRUGATED ARCH STRUCTURE (BRIDGECOR®) AS DETAILED IN THE PLANS. ANY INSTALLATION INFORMATION PROVIDED HEREIN SHALL BE REVIEWED AND APPROVED BY THE ENGINEER. CHANGES AND DISCREPANCIES MUST BE FORWARDED TO THE MANUFACTURER TO BE INCORPORATED IN A REVISED DRAWING SET.
- 2. THE ENGINEER SHALL VERIFY THAT THE PROPOSED STRUCTURE IS APPROPRIATE FOR THE SITE CONDITIONS AND THE DESIGN PARAMETERS ARE CONSISTENT WITH THE PROJECT REQUIREMENTS. ALL ASPECTS OF THE STRUCTURE DESIGN AND SITE LAYOUT NOT EXPLICITLY INCLUDED IN THESE DRAWINGS SHALL BE PROVIDED OR COORDINATED BY THE ENGINEER. THIS MAY INCLUDE BUT IS NOT LIMITED TO: FOUNDATIONS, BACKFILL, END TREATMENTS, HYDRAULIC ANALYSIS AND SCOUR ANALYSIS AS REQUIRED.
- CONTECH PRODUCT DRAWINGS MAY NOT BE USED, REPRODUCED, COPIED, OR ISSUED TO A THIRD PARTY WITHOUT THE PRIOR WRITTEN PERMISSION OF CONTECH ENGINEERED SOI LITIONS
- 4. ALL DIMENSIONS ARE TO THE INSIDE CREST OF THE CORRUGATION PROFILE UNLESS NOTED OTHERWISE AND ARE SUBJECT TO MANUFACTURING TOLERANCES.
- 5. THE STRUCTURE GEOMETRY, DESIGN, AND MATERIALS AS SHOWN IN THESE DRAWINGS ONLY APPLY FOR BRIDGECOR®, DESIGNED AND FABRICATED BY CONTECH ENGINEERED SOLUTIONS (CONTECH). ALTERNATIVE SYSTEMS SHALL BE FULLY DESIGNED AND APPROVED PRIOR TO BIDDING, WITH SEALED DRAWINGS AND CALCULATIONS PROVIDED TO DEMONSTRATE COMPLIANCE WITH THE SAME GEOMETRY, DESIGN, AND MATERIAL REQUIREMENTS AS SHOWN HEREIN.
- 6. CIRCUMFERENTIAL PLATE LENGTHS ARE IN TERMS OF S = 16 INCHES.
- 7. FOR WATER CONVEYANCE APPLICATIONS THE STRUCTURE MUST MEET HYDRAULIC REQUIREMENTS OF THE SITE AS DETERMINED BY THE ENGINEER. SHEET PILING, INLET AND OUTLET APRONS, CUTOFF WALLS, RIP RAP, AND/OR OTHER MEASURES SHALL BE INSTALLED AS NECESSARY TO PREVENT LOSS OF ENGINEERED BACKFILL AND/OR FOUNDATION SOILS DUE TO SCOUR. THE EXTENT, SIZE, AND LOCATION OF SCOUR PROTECTION SHALL BE DETERMINED BY THE ENGINEER.
- 8. PERIMETER DRAINAGE, SURFACE DRAINAGE, AND GRADING AROUND THE STRUCTURE SHALL BE DESIGNED, SPECIFIED, SUPPLIED, AND INSTALLED BY OTHERS.

#### DESIGN PARAMETERS

 DESIGN BY CONTECH ENGINEERED SOLUTIONS IS BASED ON THE FOLLOWING DESIGN CRITERIA:

VEHICLE LIVE LOAD: HL-93

MINIMUM COVER: 4.0'

MAXIMUM COVER: 6.6'

DESIGN UNIT WEIGHT OF ENGINEERED BACKFILL = 120 LBS/FT3.

- 2. ENGINEERED BACKFILL MATERIAL SHALL COMPLY WITH THE ENGINEERED BACKFILL MATERIAL REQUIREMENTS SHOWN IN THESE DRAWINGS.
- DESIGN COVER AND LATERAL EXTENT OF ENGINEERED BACKFILL ZONE SHALL BE AS SHOWN IN THESE DRAWINGS AND REQUIRED BY THE DESIGN STANDARDS REFERENCED IN THESE NOTES.
- 4. REFERENCE AASHTO LRFD SECTION 12.6.1 FOR SEISMIC DESIGN CONSIDERATIONS
- 5. STRUCTURE DESIGN BASED ON SITE SOIL INFORMATION PROVIDED IN CONTRACT DOCUMENTS. IF UNEXPECTED SITE SOIL CONDITIONS ARE ENCOUNTERED, CONTECH MUST BE NOTIFIED TO DETERMINE IF DESIGN CHANGES ARE NEEDED.
- 6. TEMPORARY CONSTRUCTION VEHICLE LOADING HEAVIER THAN THE DESIGN VEHICLE LIVE LOAD SHALL NOT BE PERMITTED TO CROSS OVER THE STRUCTURE WITHOUT THE APPROVAL OF CONTECH. IT IS THE CONTRACTOR'S RESPONSIBILITY TO NOTIFY CONTECH OF THE SIZE, TYPE AND WEIGHT OF ANY CONSTRUCTION VEHICLES INTENDED TO CROSS OVER THE STRUCTURE.

# BRIDGECOR TWO RADIUS ARCH ARROYA CROSSING TIMBER RIDGE COLORADO SPRINGS, CO

#### APPROVALS:

#### DESIGN ENGINEER'S STATEMENT:

THESE DETAILED PLANS AND SPECIFICATIONS WERE PREPARED UNDER MY DIRECTION AND SUPERVISION. SAID PLANS AND SPECIFICATIONS HAVE BEEN PREPARED ACCORDING TO THE CRITERIA ESTABLISHED BY THE COUNTY FOR DETAILED ROADWAY, DRAINAGE, GRADING AND EROSION CONTROL PLANS AND SPECIFICATIONS, AND SAID PLANS AND SPECIFICATIONS ARE IN CONFORMITY WITH APPLICABLE MASTER DRAINAGE PLANS AND MASTER TRANSPORTATION PLANS. SAID PLANS AND SPECIFICATIONS MEET THE PURPOSES FOR WHICH THE PARTICULAR ROADWAY AND DRAINAGE FACILITIES ARE DESIGNED AND ARE CORRECT TO THE BEST OF MY KNOWLEDGE AND BELIEF. I ACCEPT RESPONSIBILITY FOR ANY LIABILITY DIRECTLY CAUSED BY THE NEGLIGENT ACTS, ERRORS, OR OMISSIONS ON MY PART IN PREPARATION OF THESE DETAILED PLANS AND SPECIFICATIONS.

DARRELL J. SANDERS, COLORADO P.E. #55263

9/19/23

#### OWNER/DEVELOPER'S STATEMENT:

I, THE OWNER/DEVELOPER HAVE READ AND WILL COMPLY WITH THE REQUIREMENTS OF THE GRADING AND EROSION CONTROL PLAN AND ALL OF THE REQUIREMENTS SPECIFIED IN THESE DETAILED PLANS AND SPECIFICATIONS.

OREN & MORELAND

9/21/2023 DATE

#### L PASO COUNTY:

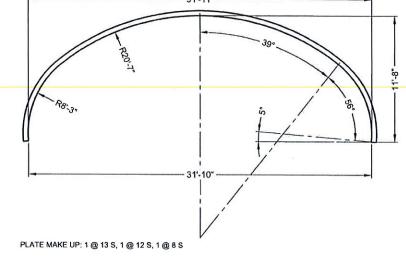
COUNTY PLAN REVIEW IS PROVIDED ONLY FOR GENERAL CONFORMANCE WITH COUNTY DESIGN CRITERIA. THE COUNTY IS NOT RESPONSIBLE FOR THE ACCURACY AND ADEQUACY OF THE DESIGN, DIMENSIONS, AND/OR ELEVATIONS WHICH SHALL BE CONFIRMED AT THE JOB SITE. THE COUNTY THROUGH THE APPROVAL OF THIS DOCUMENT ASSUMES NO RESPONSIBILITY FOR COMPLETENESS AND/OR ACCURACY OF THIS DOCUMENT.

FILED IN ACCORDANCE WITH THE REQUIREMENTS OF THE EL PASO COUNTY LAND DEVELOPMENT CODE, DRAINAGE CRITERIA MANUAL, VOLUMES 1 AND 2 AND ENGINEERING CRITERIA MANUAL AS AMENDED.

IN ACCORDANCE WITH WITH ECM SECTION 1.12, THESE CONSTRUCTION DOCUMENTS WILL BE VALID FOR CONSTRUCTION FOR A PERIOD OF 2 YEARS FROM THE DATE SIGNED BY THE EL PASO COUNTY ENGINEER. IF CONSTRUCTION HAS NOT STARTED WITHIN THOSE 2 YEARS, THE PLANS WILL NEED TO BE RESUBMITTED FOR APPROVAL, INCLUDING PAYMENT OF REVIEW FEES AT THE PLANNING AND COMMUNITY DEVELOPMENT DIRECTOR'S DISCRETION.

JOSHUA PALMER, P.E.
COUNTY ENGINEER / ECM ADMINISTRATOR

DATE



#### NOTES:

- MEASUREMENTS ARE TO THE INSIDE CRESTS OF THE CORRUGATIONS.
   DIMENSIONS ARE SUBJECT TO MANUFACTURING TOLERANCES.
   CIPCLIMEEDING OF A CHARGING.
  - BRIDGECOR TWO RADIUS ARCH INSIDE END AREA = 295.4 ft<sup>2</sup> TOTAL S = 33

STRUCTURE CROSS-SECTION

FOR APPROVAL

The despir and information shales on this orienting is provided as a service in the project caves, respirate and contractor by orientation and contractor between constant of contract, in contractor is comply to done at the user's own risk and Contract expressly disclaims any fability or responsibility for contractor between the supplied information upon which be duranced as based and cautal efficiency orientation and contractor between the supplied information upon which be duranced as base wink programme. Here discrepancies must be reported as a sew wink programme. Here discrepancies must be reported to Contractor between the supplied contractor and the proposal contractor and the programme. Here discrepancies must be reported to Contractor accepts no labelity for designs based on missing, recomplete or waters and which for designs based on missing, recomplete or waters and the programme contractor to be a supplied to the programme contractor to the programme contractor to the programme. Best discrepancies must be reported to Contractor accepts no labelity for designs based on missing, recomplete or waters and the programme. A programme contractor accepts no labelity or designs based on missing, recomplete or waters and the programme contractor and the programme con

CINTECH® ENGINEERED SOLUTIONS LLC

www.ContechES.com 9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069 800-338-1122 513-645-7000 513-645-7993 FAX



CONTECH CONTRACT DRAWING BRIDGECOR TWO RADIUS ARCH 31'-11" MAX. SPAN X 11'-8" RISE ARROYA CROSSING TIMBER RIDGE COLORADO SPRINGS, CO

ORADO LICENO

PROJECT No.: 723523						
DESIGNED:		DRA		СС		
CHECKED: EDE		APP	ROVE	D. XX		
SHEET NO.:	1		)F	7		

- 1.1 ALL STANDARDS REFER TO THE CURRENT ASTM/AASHTO EDITION UNLESS
- 1.2 AASHTO M111 STANDARD SPECIFICATION FOR ZINC (HOT-DIP GALVANIZED) COATINGS ON IRON AND STEEL PRODUCTS
- 1,3 AASHTO M232 STANDARD SPECIFICATION FOR ZINC COATING (HOT-DIP) ON IRON
- 1.4 AASHTO M 167 STANDARD SPECIFICATION FOR CORRUGATED STEEL STRUCTURAL PLATE, ZINC COATED, FOR FIELD-BOLTED PIPE, PIPE-ARCHES AND ARCHES
- 1,5 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS SECTION 12
- 1.6 AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS SECTION 26
- 1.7 ASTM A761 STANDARD SPECIFICATION FOR CORRUGATED STEEL STRUCTURAL PLATE, ZINC COATED, FOR FIELD-BOLTED PIPE, PIPE-ARCHES AND ARCHES
- 1,8 ASTM A796 STANDARD PRACTICE FOR STRUCTURAL DESIGN OF CORRUGATED STEEL PIPE, PIPE-ARCHES, AND ARCHES FOR STORM AND SANITARY SEWERS AND
- 1,9 ASTM A807 STANDARD PRACTICE FOR INSTALLING CORRUGATED STEEL STRUCTURAL PLATE PIPE FOR SEWERS AND OTHER APPLICATIONS
- 1.10 ASTM A449 STANDARD SPECIFICATION FOR HEX CAP SCREWS, BOLTS AND STUDS, HEAT TREATED, 120/105/90 KSI MINIMUM TENSILE STRENGTH, GENERAL USE
- 1,11 ASTM A123 STANDARD SPECIFICATION FOR ZINC (HOT DIP GALVANIZED) COATINGS ON IRON AND STEEL PRODUCTS
- 1,12 ASTM A153 STANDARD SPECIFICATION FOR ZINC COATING (HOT-DIP) ON IRON AND

#### 2. DEFINITIONS

- 2,1 ENGINEER IN THESE NOTES THE WORD "ENGINEER" SHALL MEAN THE ENGINEER OF RECORD OR OWNER'S DESIGNATED ENGINEERING REPRESENTATIVE
- 2.2 MANUFACTURER IN THESE NOTES THE WORD "MANUFACTURER" SHALL MEAN THE MANUFACTURER OF THE DEEP CORRUGATED ARCH PLATES, CONTECH ENGINEERED SOLUTIONS @ PHONE 800/338-1122.
- 2,3 CONTRACTOR IN THESE NOTES THE WORD "CONTRACTOR" SHALL MEAN THE FIRM OR CORPORATION UNDERTAKING THE EXECUTION OF ANY INSTALLATION WORK UNDER THE TERMS OF THESE SPECIFICATIONS:

#### 3. BRIDGECOR® STRUCTURE ASSEMBLY

- 3.1 ASSEMBLY OF THE BRIDGECOR® STRUCTURE SHALL BE IN ACCORDANCE WITH SECTION 26 OF AASHTO LRFD CONSTRUCTION SPECIFICATIONS (LATEST EDITION, WITH INTERIMS) AND ANY SUPPLEMENTAL RECOMMENDATIONS PROVIDED BY THE
- 3,2 ALL PLATES SHALL BE UNLOADED AND HANDLED WITH REASONABLE CARE, PLATES SHALL NOT BE ROLLED OR DRAGGED OVER GRAVEL ROCK AND SHALL BE PREVENTED FROM STRIKING ROCK OR OTHER HARD OBJECTS DURING PLACEMENT IN TRENCH OR ON BEDDING.
  - GALVANIZED COATINGS THAT HAVE BEEN COMPROMISED SHALL BE RESTORED WITH A MINIMUM OF TWO COATS OF COLD GALVANIZING COMPOUND (SPRAY OR PAINT) IN ACCORDANCE WITH ASTM A761, SECTION 7 OR OTHER MEASURES AS
- 3.3 FOR STRUCTURES SET ON FOOTINGS, PLATE ASSEMBLY CAN BEGIN AFTER PLACEMENT OF STRUCTURE FOOTINGS HAS BEEN APPROVED BY THE ENGINEER. VERIFICATION OF PROPER SPACING, ALIGNMENT, AND ORIENTATION OF THE FOUNDATIONS IS STRONGLY RECOMMENDED PRIOR TO BEGINNING PLATE ASSEMBLY, ANY MODIFICATIONS TO THE FOUNDATIONS SHALL BE MADE PRIOR TO
  - BEDDING PREPARATION FOR ROUND AND ELLIPTICAL SHAPES SHOULD BE APPROVED BY THE ENGINEER PRIOR TO ASSEMBLY. THE BED SHOULD BE CONSTRUCTED TO UNIFORM LINE AND GRADE: IT SHOULD BE FREE OF ROCK FORMATIONS, PROTRUDING STONES, FROZEN LUMPS, ROOTS AND OTHER FOREIGN 5,7 HUMIC (DECAYING) ORGANIC MATTER SHALL NOT EXCEED 1,0% (DRY WEIGHT
- 3.4 THE SPAN AND RISE OF THE STRUCTURE SHOULD BE CHECKED FREQUENTLY DURING THE EARLY STAGES OF ASSEMBLY TO VERIFY THAT ASSEMBL' TOLERANCES ARE BEING ACHIEVED AND TO ALLOW FOR ADJUSTMENTS TO PROCEDURES, IF NECESSARY, BEFORE ASSEMBLY IS COMPLETE.
- 3.5 CUT PLATES FOR SKEWED OR BEVELED ENDS SHOULD BE ATTACHED AFTER THE MAIN BARREL OF A STRUCTURE HAS BEEN ASSEMBLED. WHEN CAST-IN-PLACE CONCRETE COLLARS OR HEADWALLS ARE TO BE CONSTRUCTED, THE CONTRACTOR MUST ALLOW FOR FORMWORK TO BE ERECTED ON THE END OF THE 5.9 IF THE ELECTROCHEMICAL PROPERTIES OF THE BACKFILL OR WATER FALL STRUCTURE. FORMING AGAINST A SKEWED CORRUGATION PROFILE IS COMPLEX AND SHOULD BE ACCOUNTED FOR BY THE FORMING CONTRACTOR WHEN PLANNING PROCEDURES AND METHODS FOR FORMWORK CONSTRUCTION, IN SOME CASES, FIELD TRIMMING OF THE STRUCTURE MAY BE NECESSARY. SPECIAL

- BRACING AND/OR SCAFFOLDING (DESIGN BY OTHERS) IS REQUIRED TO SUPPORT SKEWED ENDS UNTIL ADEQUATE COMPRESSIVE STRENGTH IS ACHIEVED AS REQUIRED BY THE COLLAR/HEADWALL DESIGNER
- 3.6 NUTS SHALL BE PLACED WITH THE ROUNDED FACE IN CONTACT WITH THE PLATES UNLESS NOTED OTHERWISE, NUTS CAN BE ON EITHER THE INSIDE OR OUTSIDE OF THE STRUCTURE TO FACILITATE ASSEMBLY,
- 3,7 RECOMMENDED TORQUE RANGE ON THE BOLTS IS 150 TO 300 FT-LBS FOR 3/4" DIAMETER BOLTS AND 200-350 FT-LBS FOR 7/8" DIAMETER BOLTS
- 3,8 INSIDE SPAN AND RISE OF THE ASSEMBLED STRUCTURE AFTER BOLTING SHALL BE WITHIN 2% (OR 5 INCHES, WHICHEVER IS LESS) OF THE PLAN DIMENSIONS. THE TOLERANCE ON STRUCTURE LENGTH IS ± 1/2" PER 10 FT OF STRUCTURE LENGTH

#### ENGINEERED BACKFILL ENVELOPE

- 4.1 ENGINEERED BACKFILL MATERIAL SHALL BE PLACED WITHIN THE ENGINEERED BACKFILL ENVELOPE TO THE MINIMUM WIDTH AND WITHIN THE COVER LIMITS SHOWN ON THESE DRAWINGS, THE ENGINEERED BACKFILL ENVELOPE SHALL NOT BE ALTERED WITHOUT WRITTEN APPROVAL FROM CONTECH
- 4,2 IN-SITU SOILS BELOW AND ADJACENT TO THE ENGINEERED BACKFILL ENVELOPE SHALL PROVIDE A FIRM SURFACE AGAINST WHICH TO COMPACT THE ENGINEERED BACKFILL MATERIAL. THE GEOTECHNICAL ENGINEER SHALL EVALUATE THE IN-SITU SOILS AND DETERMINE THE TYPE AND DEGREE OF ANY SOIL IMPROVEMENTS REQUIRED. THESE MAY INCLUDE BENCHING OR SLOPING OF THE SIDE SOILS. BACKFILL PLACEMENT MAY NOT BEGIN UNTIL THE GEOTECHNICAL ENGINEER HAS APPROVED THE IN-SITU SOILS AND ANY REQUIRED IMPROVEMENTS.
- 4.3. A DRAINED CONDITION WAS ASSUMED FOR THE ENGINEERED BACKFILL ENVELOPE. DESIGN AND SUPPLY OF A SUBSURFACE DRAINAGE SYSTEM (IF NEEDED) SHALL BE 4.4. IF REQUIRED BY THE GEOTECHNICAL ENGINEER, A GEOTEXTILE OR GRADED SOIL
- FILTER MAY BE USED BETWEEN THE ENGINEERED BACKFILL AND IN-SITU SOIL TO PREVENT MIGRATION OF FINES AND POSSIBLE INTERNAL EROSION OF THE SOIL.

#### ENGINEERED BACKFILL MATERIAL REQUIREMENTS

- 5.1 THE DESIGN OF BRIDGECOR® STRUCTURES RELIES ON COMPACTED SOIL PROPERTIES PROVIDED BY THE ENGINEERED BACKFILL MATERIALS. THE CORRECT SELECTION AND PLACEMENT OF COMPACTED ENGINEERED BACKELL MATERIAL IS CRITICAL TO THE PERFORMANCE OF BRIDGECOR® STRUCTURES.
- 5.2 THE ENGINEERED BACKFILL MATERIAL SHALL NOT BE ADVERSELY AFFECTED BY WETTING, DRYING, SATURATION, FREEZE/THAW, VIBRATIONS, OR FLOWING WATER,
- 5.3 BACKFILL MATERIALS SHALL CONFORM TO PROPERTIES REFERENCED IN THE PROJECT SPECIFICATIONS OR THE PROPERTIES DESCRIBED HEREIN, WHICHEVER IS MORE STRINGENT.
- BACKFILL MATERIAL SHOULD CONSIST OF A WELL-GRADED, ANGULAR GRANULAR SOIL WITH A MAXIMUM PARTICLE SIZE OF 3 INCHES, OPEN GRADED OR GAP GRADED MATERIALS ARE NOT PREFERRED, THE BACKFILL MATERIAL SHOULD BE REE OF FROZEN LUMPS, FOREIGN MATERIAL OR ORGANIC DECOMPOSABLE MATERIALS, FINE BEACH SANDS, WINDBLOWN SANDS, AND STREAM DEPOSITED SANDS ALL OF WHICH EXHIBIT FINE, ROUNDED PARTICLES AND TYPICALLY ARE CLASSIFIED BY AASHTO M 145 AS A-3 MATERIALS ARE NOT ALLOWED. WHEN USING A-2 MATERIALS, MOISTURE CONTENT MUST BE BETWEEN -3% AND +2% OPTIMUM AS DEFINED BY AASHTO T 180
- GRAIN SIZE DISTRIBUTION (GRADATION) OF THE ENGINEERED BACKFILL MATERIAL SHALL SATISFY: Cu GREATER THAN OR EQUAL TO 3 AND Cc BETWEEN 0.7 AND 3, WHERE Cu = COEFFICIENT OF UNIFORMITY = D60/D10 AND Cc = COEFFICIENT OF CURVATURE = (D30)2/(D60XD10), DXX IS THE PARTICLE SIZE CORRESPONDING TO XX% FINER ON THE CUMULATIVE PARTICLE SIZE DISTRIBUTION CURVE (ASTM
- REQUIREMENTS ARE WAIVED FOR BACKFILL MATERIAL CONTAINING MORE THAN 60% GRAVEL (> #4 SIEVE).

Cu REQUIREMENTS ARE WAIVED FOR CRUSHER RUN SCREENED AGGREGATES. Co

- 5.6 ABRASION LOSS SHALL NOT EXCEED 45% AS DETERMINED BY THE LOS ANGELES ABRASION TEST (ASTM C131).
- 5.8 ELECTROCHEMICAL REQUIREMENTS FOR SOIL AND WATER IN CONTACT WITH BOTH THE INSIDE AND OUTSIDE OF THE BRIDGECOR® STRUCTURE ARE AS FOLLOWS, PER THE NCSPA DESIGN MANUAL:
- RESISTIVITY = 2,000 to 10,000 OHM-CM
- OUTSIDE OF THE RECOMMENDED RANGE, A SECONDARY PROTECTION SYSTEM MAY BE NEEDED TO ACHIEVE THE DESIGN SERVICE LIFE. SECONDARY PROTECTION SYSTEMS (IF REQUIRED) SHALL BE DESIGNED AND PROVIDED BY OTHERS.
- 5.10 THE SELECTION AND EVALUATION OF PROPOSED ENGINEERED BACKFILL MATERIAL

IS THE RESPONSIBILITY OF THE CONTRACTOR, THE CONTRACTOR SHALL PROVIDE CONTECH WITH DOCUMENTATION FROM A QUALIFIED GEOTECHNICAL ENGINEER. THAT THE PROPOSED ENGINEERED BACKFILL MATERIAL MEETS OR EXCEEDS THE

#### ENGINEERED BACKFILL PLACEMENT PROCEDURE

- 6.1 A CONTECH REPRESENTATIVE WILL CONDUCT A PRECONSTRUCTION CONFERENCE 7.3 PRIOR TO COMMENCEMENT OF WORK TO REVIEW AND DISCUSS THE RECOMMENDED PROCEDURES FOR BACKFILLING, AND STRUCTURE SHAPE MEASUREMENTS: ANY CONTRACTOR WITH RESPONSIBILITY FOR BACKFILLING OR CONSTRUCTION OF END TREATMENTS MUST BE REPRESENTED AT THIS MEETING. IT IS RECOMMENDED THAT THE ENGINEER AND ANY THIRD PARTY INVOLVED IN COMPACTION TESTING OR OTHER QUALITY CONTROL MEASURES ALSO ATTEND.
- 6.2 ANY IMPROVEMENT OF THE SUBGRADE AND EMBANKMENT SOILS REQUIRED BY THE GEOTECHNICAL ENGINEER SHALL BE COMPLETED AND APPROVED PRIOR TO BEGINNING PLACEMENT OF ENGINEERED BACKFILL MATERIAL
- 6,3 THE ENGINEERED BACKFILL MATERIAL SHALL BE PLACED UNIFORMLY ON BOTH SIDES OF THE STRUCTURE IN LAYERS OF 8 INCHES OR LESS (BEFORE
- 6.4 BACKFILL SHALL BE COMPACTED TO THE MINIMUM DENSITY INDICATED IN THESE DRAWINGS, DEPENDING ON THE COMPACTION EQUIPMENT AND ENGINEERED BACKFILL MATERIAL USED, IT MAY BE NECESSARY TO DECREASE THE LIFT THICKNESS AND/OR MOISTURE CONDITION THE LOOSE SOIL TO ACHIEVE THE SPECIFIED MINIMUM LEVEL OF COMPACTION.
- 6.5 IF THE ENGINEERED BACKFILL MATERIAL DOES NOT PRODUCE A PROCTOR CURVE AND/OR IS NOT CONDUCIVE TO TRADITIONAL FIELD-TESTING METHODS QUALITATIVE METHODS OF EVALUATING COMPACTION MAY BE USED. SUCH METHODS SHALL BE EVALUATED AND APPROVED BY THE GEOTECHNICAL ENGINEER AND A COPY OF THE METHOD BE PROVIDED TO THE DESIGNER,
- 6,6 THE DIFFERENCE IN BACKFILL LEVELS ON THE TWO SIDES OF THE STRUCTURE AT ANY TRANSVERSE SECTION SHALL NOT EXCEED 24 INCHES WITHOUT PRIOR APPROVAL FROM CONTECH.
- 6.7 CONSTRUCTION EQUIPMENT USED WITHIN 5 FEET LATERALLY OF THE WIDEST PART OF THE STRUCTURE, UP TO THE MINIMUM DESIGN COVER HEIGHT ABOVE THE STRUCTURE, SHALL HAVE A STATIC MASS OF 10 TONS OR LESS. IT MAY BE POSSIBLE TO USE HEAVIER EQUIPMENT IF IT CAN BE DEMONSTRATED THAT THE STRUCTURE SHAPE IS NOT ADVERSELY AFFECTED. ENGINEERED BACKFILL MATERIAL PLACED WITHIN 1 FOOT LATERALLY OF THE WIDEST PART OF THE STRUCTURE SHALL BE COMPACTED USING HAND OPERATED EQUIPMENT UNTIL THE MINIMUM COVER HEIGHT IS REACHED. OVER-COMPACTION OF ENGINEERED BACKFILL IN THIS ZONE SHOULD BE AVOIDED. AS THIS CAN CONTRIBUTE TO EXCESSIVE DEFLECTION OF SOME STRUCTURES. AREAS CLOSEST TO THE STRUCTURE SHALL BE COMPACTED RUNNING PARALLEL TO THE LENGTH OF THE STRUCTURE
- 6.8 ONCE THE BACKFILL ELEVATION REACHES APPROXIMATELY % OF THE STRUCTURE RISE (DEPENDING ON THE SHAPE OF THE STRUCTURE AND RELATIVE MOVEMENT DURING THE BACKFILL PROCESS), PLACE AND COMPACT ENGINEERED BACKFILL MATERIAL IN RADIAL LIFTS OVER THE TOP OF THE STRUCTURE USING EQUIPMENT AS DESCRIBED ABOVE. THE FIRST RADIAL LIFT SHOULD BE THICKER AND PROVIDE A MINIMUM 12 INCHES OF COVER BETWEEN THE STRUCTURE AND COMPACTION EQUIPMENT. EQUIPMENT SHOULD RUN PERPENDICULAR TO THE LONGITUDINAL AXIS OF THE STRUCTURE. NO EQUIPMENT SHALL BE ALLOWED OVER THE STRUCTURE THAT WOULD EXCEED THE DESIGN LOAD AT THE MINIMUM DESIGN HEIGHT OF COVER. NO CONSTRUCTION EQUIPMENT SHALL BE ALLOWED TO PARK ON TOP OF A PARTIALLY BACKFILLED STRUCTURE.
- 6,9 AT NO TIME SHALL THE ENGINEERED BACKFILL MATERIAL BE DUMPED OR PUSHED AGAINST THE STRUCTURE WALL(S) SO AS TO CHANGE THE SHAPE OR ALIGNMENT OF THE STRUCTURE, MATERIAL SHALL NOT BE DUMPED ON TOP OF THE STRUCTURE AT ANY TIME, TRUCKS MAY UNLOAD IN ROUGH LAYERS NO CLOSER THAN 5 FEET FROM THE WIDEST PART OF THE STRUCTURE:
- 6,10 THE STRUCTURE SHALL BE CHECKED PERIODICALLY DURING BACKFILLING TO ENSURE THE SHAPE OF THE STRUCTURE MEETS DESIGN REQUIREMENTS AND IS CONSISTENT WITH THE ASSEMBLY TOLERANCES AS STATED IN THESE NOTES, IF DEFLECTION OF THE STRUCTURE IS GREATER THAN EXPECTED, BACKFILLING SHALL BE HALTED AND BACKFILL PLACEMENT AND COMPACTION PROCEDURES MODIFIED TO CORRECT THE STRUCTURE SHAPE, IT MAY BE NECESSARY TO REMOVE SOME OF THE BACKELL TO CORRECT EXCESSIVE DEFLECTION. AFTER COMPLETION OF BACKFILL PLACEMENT. THE FINAL SHAPE OF THE STRUCTURE SHALL BE WITHIN 2% (OR 5 INCHES, WHICHEVER IS LESS) OF THE PLAN

#### CONSTRUCTION OBSERVATIONS AND TESTING

7.1. OBSERVATION AND TESTING SHALL BE PERFORMED DURING CONSTRUCTION TO VERIFY COMPLIANCE WITH THESE DRAWINGS, APPLICABLE PROJECT DOCUMENTS, AND STANDARDS REFERENCED IN THESE NOTES.

- 7,2, THE CONTRACTOR IS RESPONSIBLE FOR QUALITY CONTROL PROCEDURES, VERIFICATION MEASUREMENTS, ADEQUATE SUPERVISION, PROGRESS TESTING EVALUATION OF PROPOSED ENGINEERED BACKFILL MATERIALS, AND/OR OTHER MEASURES AS NEEDED TO ENSURE THAT THE COMPLETED PROJECT COMPLIES WITH THESE DRAWINGS AND NOTES:
- THE PROJECT OWNER (OR THEIR DESIGNATED REPRESENTATIVE) IS RESPONSIBLE FOR PROJECT OVERSIGHT AND FINAL ACCEPTANCE OF THE CONSTRUCTED STRUCTURE, THE OWNER MAY ACCEPT THE CONTRACTOR'S QUALITY CONTROL PROGRAM OR ADOPT AN INDEPENDENT QUALITY ASSURANCE PROGRAM TO VERIFY COMPLIANCE.
- 7.4. OBSERVATIONS AND TESTING PRIOR TO STRUCTURE ASSEMBLY & BACKFILL SHALL INCLUDE BUT NOT BE LIMITED TO:
  - EVALUATION OF FOUNDATION SOILS BELOW FOOTINGS AND THE ENGINEERED BACKFILL ENVELOPE
- VERIFICATION OF PROPER ALIGNMENT, DIMENSIONS, AND PLACEMENT OF **FOUNDATIONS**
- VERIFICATION OF PROPER SHAPING, PLACEMENT, AND PREPARATION OF BEDDING SOILS (STRUCTURES WITH INVERTS)
- VÉRIFICATION OF PROPÈR PREPARATION OF EMBANKMENT SOILS ADJACENT TO THE ENGINEERED BACKFILL ENVELOPE
- EVALUATION AND APPROVAL OF ENGINEERED BACKFILL MATERIALS
- VERIFICATION OF PROPER PLACEMENT OF GEOTEXTILES (WHEN REQUIRED)
- 7.5 OBSERVATIONS AND TESTING DURING ASSEMBLY & BACKFILLING SHALL INCLUDE BUT NOT BE LIMITED TO:
- STRUCTURE ALIGNMENT
- PLATE TIGHTNESS OBSERVATIONS
- INITIAL STRUCTURE SHAPE MEASUREMENTS (PRIOR TO STARTING BACKFILLING)
- PERIODIC STRUCTURE SHAPE MEASUREMENTS (DURING BACKFILLING)
- ENGINEERED BACKFILL MATERIAL SAMPLING AND TESTING
- OBSERVATIONS OF PROPER FILL PLACEMENT AND COMPACTION PROCEDURES.
- 7.6 CONTECH MAY REQUIRE ADDITIONAL OBSERVATIONS AND/OR TESTING WHICH MAY INCLUDE, BUT NOT BE LIMITED TO, FULL TIME SHAPE MONITORING, ADDITIONAL SOIL TESTING, AND SITE EVALUATIONS DEPENDING ON THE STRUCTURE GEOMETRY, DESIGN, AND/OR OTHER PROJECT SPECIFIC FACTORS



**FOR APPROVAL** 

MARK DATE REVISION DESCRIPTION

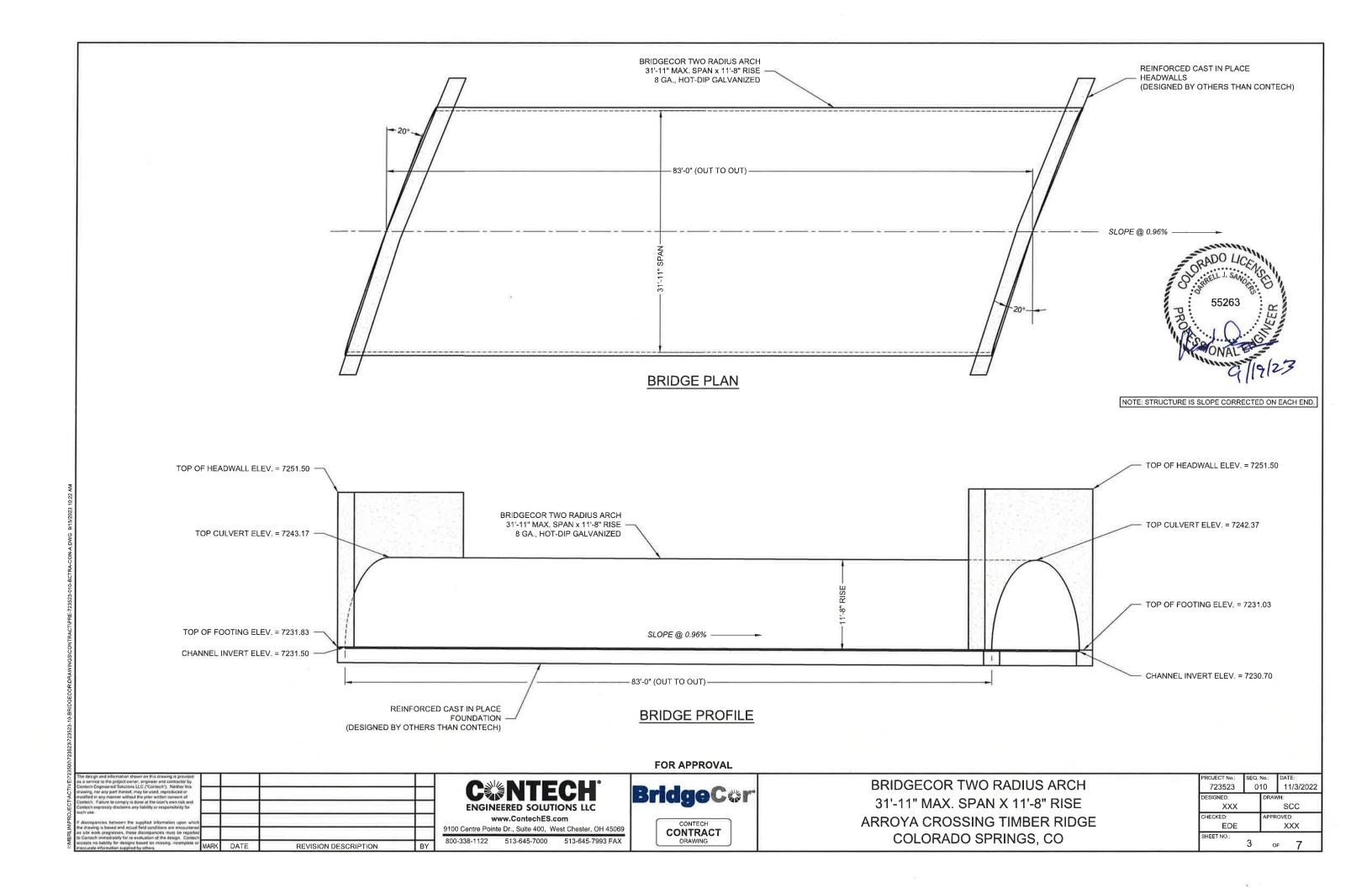
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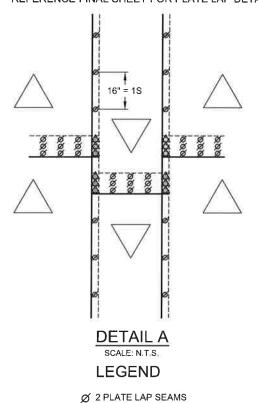
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- ALL COMPONENTS MUST BE ASSEMBLED AS SHOWN. PAY CLOSE ATTENTION TO MARK NUMBERS ON THE COMPONENTS.
- 2. MANUFACTURING FACILITY: MANSFIELD, TX.
- 3. REFERENCE FINAL SHEET FOR PLATE LAP DETAIL.



⚠ 3-PLATE SEAM

ANCHOR BOLTS PLACED MOVEMENT CONTROL HOOKS 9" FROM PLATE EDGE @ 16" O.C. IN NEAREST CREST OR VALLEY AS SHOWN (QTY, OF 21) FOR SHAPE MONITORING
PLACED AT EVERY FOURTH SEAM QTY OF (64x) (32x EACH SIDE) MARK ENDS RED-HAUNCH CROWN FLOW CENTERLINE CROWN HAUNCH MARK ENDS RED-

			BRIDGECOR	PLATE DETAILS (3	0" LONG PLATES)	
PLATE WIDTH "S"	GAGE	PLATE QTY.	MK#	OVERALL WIDTH (INCHES)	NUMBER OF CIRCUMFERENTIAL BOLT HOLES AT 16" CENTERS	PLATE WEIGHT WITHOUT FASTENERS
8	8	34	3;6;11;14	137	9	284
10	8	1	5	169	11	350
12	8	35	2;8;9;12	201	13	417
13	8	32	1;4;7;13	217	14	450
14	8	1	10	233	15	483

PLATE LAYOUT DRAWING
BRIDGECOR TWO RADIUS ARCH (OUTSIDE VIEW)
31'-11" SPAN x 11'-8" RISE



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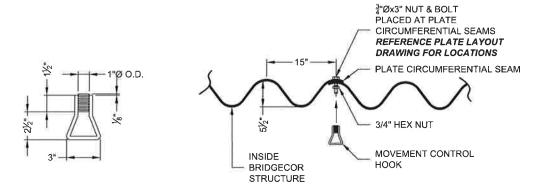


CONTECH CONTRACT DRAWING

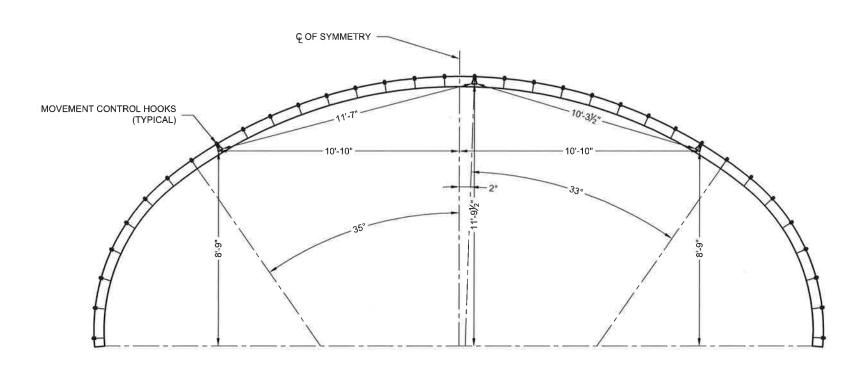
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	723523	0.	10	11/3/2022
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	XXX			SCC
1	CHECKED:		APPR	OVED:
	EDE			XXX
	SHEET NO :			
		4	Of	7

#### NOTES:

 REFERENCE PLATE LAYOUT DIAGRAM FOR MOVEMENT CONTROL HOOK LOCATIONS.



## MOVEMENT CONTROL HOOK DETAIL



MOVEMENT CONTROL HOOK PLACEMENT DETAIL



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SELECT GRANULAR STRUCTURAL BACKFILL LIMITS.



INITIAL LIFTS OVER THE CROWN OF STRUCTURE AS INDICATED BY SHADED AREA TO BE COMPACTED TO REQUIRED DENSITY WITH HAND OPERATED EQUIPMENT OR WITH LIGHTWEIGHT TRACTOR (D-4 OR LIGHTER) EQUIPMENT.

#### NOTES:

- MINIMUM SELECT GRANULAR STRUCTURAL BACKFILL WIDTH IS BASED ON AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS SECTION 12 AND/OR THE RESULTS OF THE PROJECT SPECIFIC FINITE ELEMENT ANALYSIS.
- 2. ALL SELECT GRANULAR STRUCTURAL BACKFILL TO BE PLACED IN A BALANCED FASHION IN THIN LIFTS (8" LOOSE TYPICALLY) AND COMPACTED TO 90 PERCENT DENSITY PER AASHTO T-180,
- MONITORING OF THE TWO RADIUS ARCH STRUCTURE IS REQUIRED DURING THE BACKFILLING PROCESS. THE METHOD, FREQUENCY AND DURATION SHALL BE DETERMINED BASED ON THE SIZE AND SHAPE OF THE STRUCTURE.
- PREVENT DISTORTION OF SHAPE AS NECESSARY BY VARYING COMPACTION
  METHODS AND EQUIPMENT.
- PLACE SELECT GRANULAR STRUCTURAL BACKFILL IN RADIAL LIFTS AT APPROXIMATELY 75% OF THE RISE OF THE TWO RADIUS ARCH STRUCTURE.
- BECAUSE OF THE FLEXING AND VIBRATION OF THE CROWN PLATES, THE FULL COMPACTION DENSITY LEVELS OFTEN CAN NOT BE ACHIEVED IN THE FIRST SEVERAL INCHES OF FILL OVER THE CROWN.

BASED ON A PROJECT SPECIFIC ANALYSIS GAGE TO BE  $\frac{8}{8}$ , BACKFILL MATERIAL TO BE  $\frac{A-1}{8}$ , COMPACTED TO  $\frac{90}{8}$ % PER AASHTO T-180.

IF OTHER BACKFILL MATERIALS ARE TO BE EVALUATED, CONTACT YOUR LOCAL CONTECH REPRESENTATIVE.

#### ADDITIONAL SELECT GRANULAR STRUCTURAL BACKFILL NOTES:

so provide a cross

SATISFACTORY BACKFILL MATERIAL, PROPER PLACEMENT, AND COMPACTION ARE KEY FACTORS IN OBTAINING MAXIMUM STRENGTH AND STABILITY.

THE BACKFILL MATERIAL SHOULD BE FREE OF ROCKS, FROZEN LUMPS, AND FOREIGN MATERIAL THAT COULD CAUSE HARD SPOTS OR DECOMPOSE TO CREATE VOIDS. BACKFILL MATERIAL SHOULD BE WELL GRADED GRANULAR MATERIAL THAT MEETS THE REQUIREMENTS OF THE PROJECT SPECIFIC ANALYSIS NOTED BELOW. REFERENCE THE STRUCTURAL PLATE BACKFILL GROUP CLASSIFICATION TABLE ON THIS SHEET. RECYCLED CONCRETE/SLAG ARE NOT RECOMMENDED FOR STRUCTURAL BACKFILL MATERIAL, BACKFILL MUST BE PLACED SYMMETRICALLY ON EACH SIDE OF THE STRUCTURE IN 8" LOOSE LIFTS. EACH LIFT IS TO BE COMPACTED TO THE SPECIFIED DENSITY.

A HIGH PERCENTAGE OF SILT OR FINE SAND IN THE NATIVE SOILS SUGGESTS THE NEED FOR A WELL GRADED GRANULAR BACKFILL MATERIAL TO PREVENT SOIL MIGRATION. IF THE PROPOSED BACKFILL IS NOT A WELL GRADED GRANULAR MATERIAL, A NON-WOVEN GEOTEXTILE FILTER FABRIC SHALL BE PLACED BETWEEN THE SELECT BACKFILL AND THE IN SITU MATERIAL.

DURING BACKFILL, ONLY LIGHTWEIGHT TRACKED VEHICLES (D-4 OR LIGHTER) SHOULD BE NEAR THE STRUCTURE AS FILL PROGRESSES ABOVE THE CROWN AND TO THE FINISHED GRADE. THE ENGINEER AND CONTRACTOR ARE CAUTIONED THAT THE MINIMUM COVER MAY NEED TO BE INCREASED TO HANDLE TEMPORARY CONSTRUCTION VEHICLE LOADS (HEAVIER THAN D-4).

Usual Materials	Stone Fragment, Gravel and Sand				
Plaslicity Index	6 max.	6 max.			
Liquid Limits	<del>200</del>				
Alterberg Limits for Fraction	on Passing No. 40 (0	).425 mm)			
No. 200 (0.075 mm)	15 max.	25 max.			
No. 40 (0.425 mm)	30 max.	50 max.			
No. 10 (2,000 mm)	50 max.				
Sieve Analysis Percent Passing					
GROUP CLASSIFICATION	A-1-a	A-1-b			
STRUCTURAL PLATE BACKF REFERENCE AASHTO M-145	LL GROUP CLAS	SIFICATION			

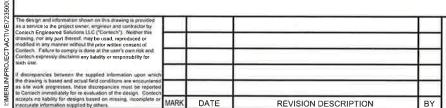
\*Modified from AASHTO M-145

Fine beach sands, windblown sands, stream deposited sands, etc., exhibiting fine, rounded particles and typically classified by AASHTO M-145 as A-3 Materials should not be used.

If 95% Compaction per AASHTO T-180 is specified, the backfill material shall be a A-1-a.

Reference the most current version of ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), for comparable soil groups,

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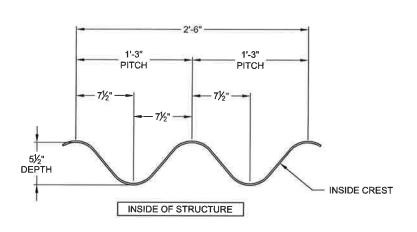


CONTRACT

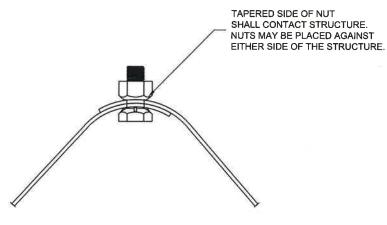
BRIDGECOR TWO RADIUS ARCH 31'-11" MAX. SPAN X 11'-8" RISE ARROYA CROSSING TIMBER RIDGE COLORADO SPRINGS. CO

	Service.	NO.	DATE:
723523	0	10	11/3/2022
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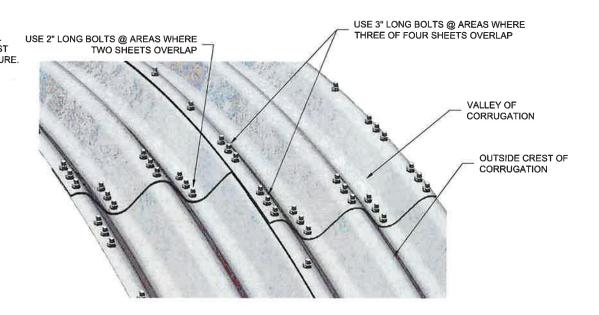
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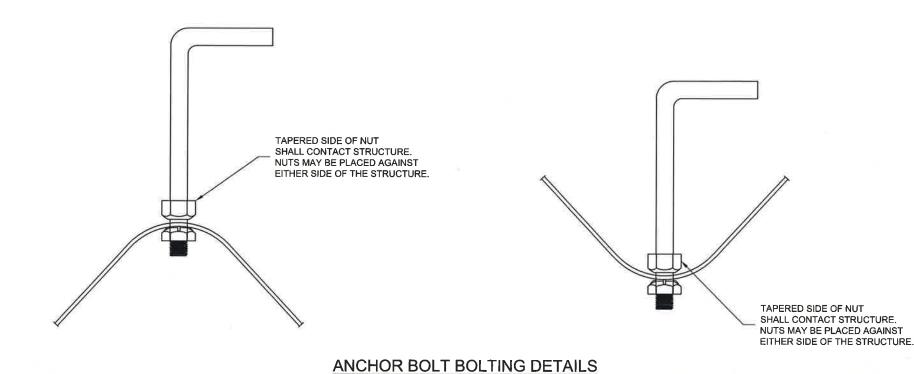
BRIDGECOR CORRUGATION PROFILE

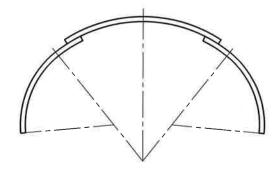


TYPICAL BOLTING DETAIL



## PLATE LAP DETAIL





## PLATE LAPPING DETAIL (LOOKING DOWNSTREAM)



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SJECT/ACTIVE/72	The design and information shows on this drawing is provided as a service by the project owner, onlinear and contradict by Contact Engineered Scholichers LLC ("Contact). Notifier this drawing, nor any part thireod, may be used, reproduced or modified in any manner without the prior written content of Contacts. Fallow to comply is done at the user's own risk and Contacts or specially contact and the contact in the comply is done at the user's own risk and Contacts or specially disclaims any liability or responsibility for such uses.			1	
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PROJECT No.: 723523	SEQ.	No_ 10	DAT	E: /3/2022
DESIGNED: XXX		DRAW		СС
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# CONTECH ENGINEERED SOLUTIONS, LLC Design of Spread Footing Foundations, Concrete Headwalls and Wingwalls for a BridgeCor Arch (723523); Arroya Lane, El Paso County, Colorado

## INDEX

- TITLE SHEET / INDEX
- PLAN, PROFILE, & FOOTING SECTIONS
- **END ELEVATION VIEWS**
- **UPSTREAM HEADWALL DETAILS**
- DOWNSTREAM HEADWALL DETAILS
- TYPICAL WINGWALL DETAILS
- 7. SPECIFICATIONS

#### APPROVALS:

DESIGN ENGINEER'S STATEMENT:
THESE DETAILED PLANS AND SPECIFICATIONS WERE PREPARED UNDER MY DIRECTION AND SUPERMISON, SAID PLANS AND SPECOFICATIONS HAVE BEEN PREPARED ACCORDING TO THE CRITERIA ESTABLISHED BY THE COUNTY FOR DETAILED ROADWAY, DRANAGE, GRADING AND EROSION CONTROL PLANS AND SPECOFICATIONS, AND SAID PLANS AND SPECOFICATIONS ARE IN CONFORMITY WITH APPLICABLE MASTER DRAINED PLANS AND MASTER TRANSPECTRATION PLANS, SAID PLANS AND SPECIFICATIONS MEET THE PURPOSES FOR WHICH THE PARTICULAR ROADWAY AND DRAINAGE FACILITIES ARE DESIGNED AND ARE CORRECT TO THE BEST OF MY KNOWLEDGE AND SELET, IACCEPT RESPONSELITY FOR ANY LIABILITY DIRECTLY CAUSED BY THE NEGLIGENT ACTS, ERRORS, OR CMISSIONS ON MY PART IN PREPARATION OF THESE DETAILED PLANS AND SPECIFICATIONS.

Nitall Halit 9/15/23 MITCHELL T. HARDERT, COLCRADO P.E. #45539

OWNER/DEVELOPER'S STATEMENT:

1. THE COMMER/DEVELOPER HAVE READ AND WILL COMPLY WITH THE REQUIREMENTS OF THE GRADING AND ERGS ON CONTROL PLAN AND ALL OF THE REQUIREMENTS SYDICHED IN THESE DETAILED PLANS AND SPECIFICATIONS.

9/22/2023

COUNTY PLAN REVEW IS PROVIDED ONLY FOR GENERAL CONFORMANCE WITH COUNTY DESIGN ORITERIA. THE COUNTY IS NOT RESPONSIBLE FOR THE ACCURACY AND ADEQUACY OF THE DESIGN, DIMENSIONS, AND/OR ELEVATIONS WHICH SHALL BE CONFIRMED AT THE UCB SITE. THE COUNTY THROUGH THE APPROVAL OF THIS DOCUMENT ASSUMES NO RESPONSIBILITY FOR COMPLETENESS

FILED IN ACCORDANCE WITH THE REQUIREMENTS OF THE EL PASO COUNTY LAND DEVELOPMENT CODE, DRAINAGE CRITERIA MANUAL, VOLUMES 1 AND 2 AND ENGINEERING CRITERIA MANUAL AS AMENDED.

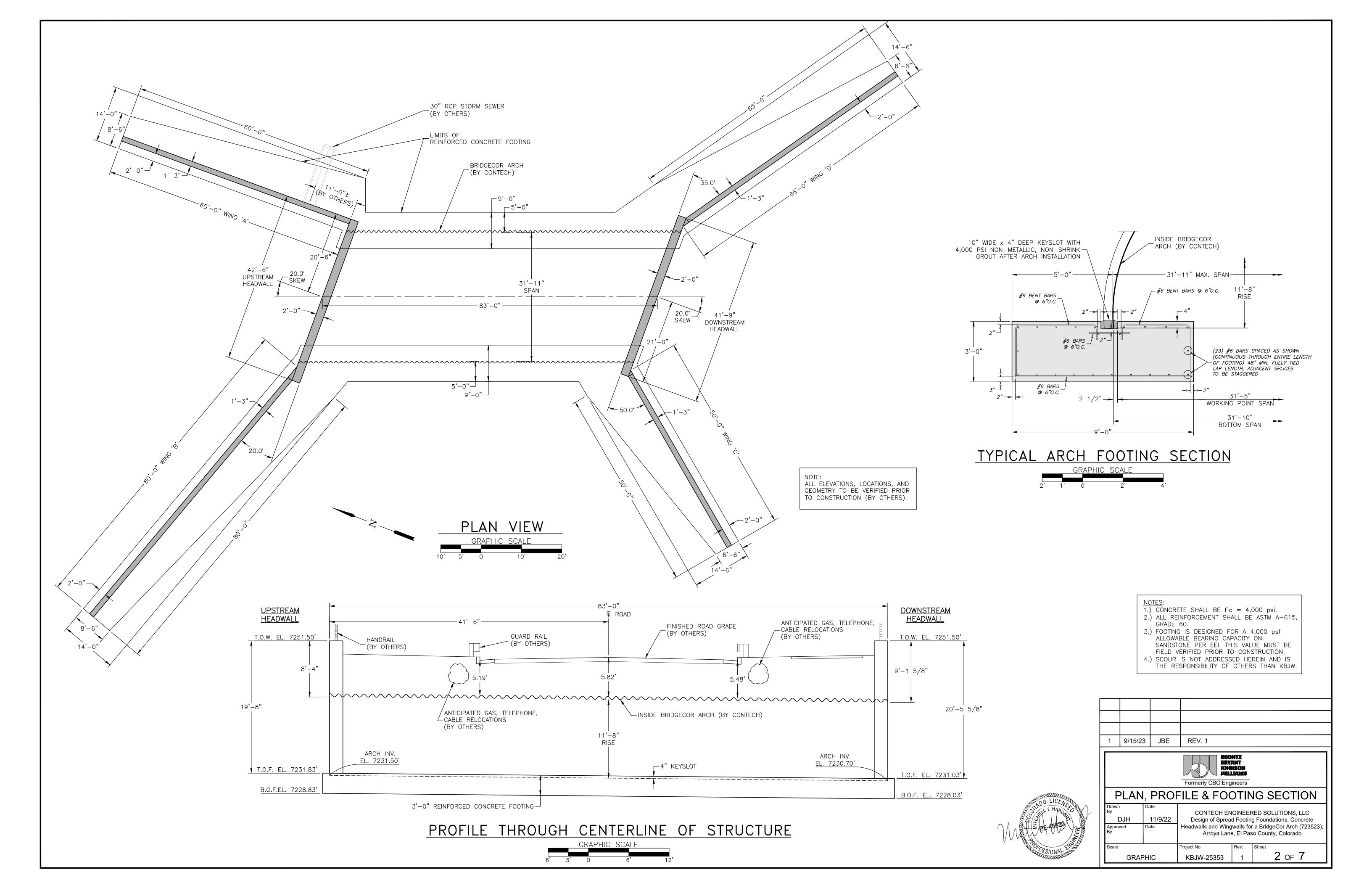
IN ACCORDANCE WITH WITH ECV SECTION 1.12, THESE CONSTRUCTION DOCUMENTS WILL BE VALID FOR CONSTRUCTION FOR A PERIOD OF 2 YEARS FROM THE DATE SCRIDD BY THE EL PASO COUNTY ENGREER. IF CONSTRUCTION HAS NOT STARTED WITHIN THOSE 2 YEARS, THE PLANS WILL RED TO BE RESUBNITTED FOR APPROVAL, INCLUDING PAYMENT OF REVIEW FEES AT THE PLANNING AND COMMUNITY DEVELOPMENT DIRECTOR'S DISCRETION.

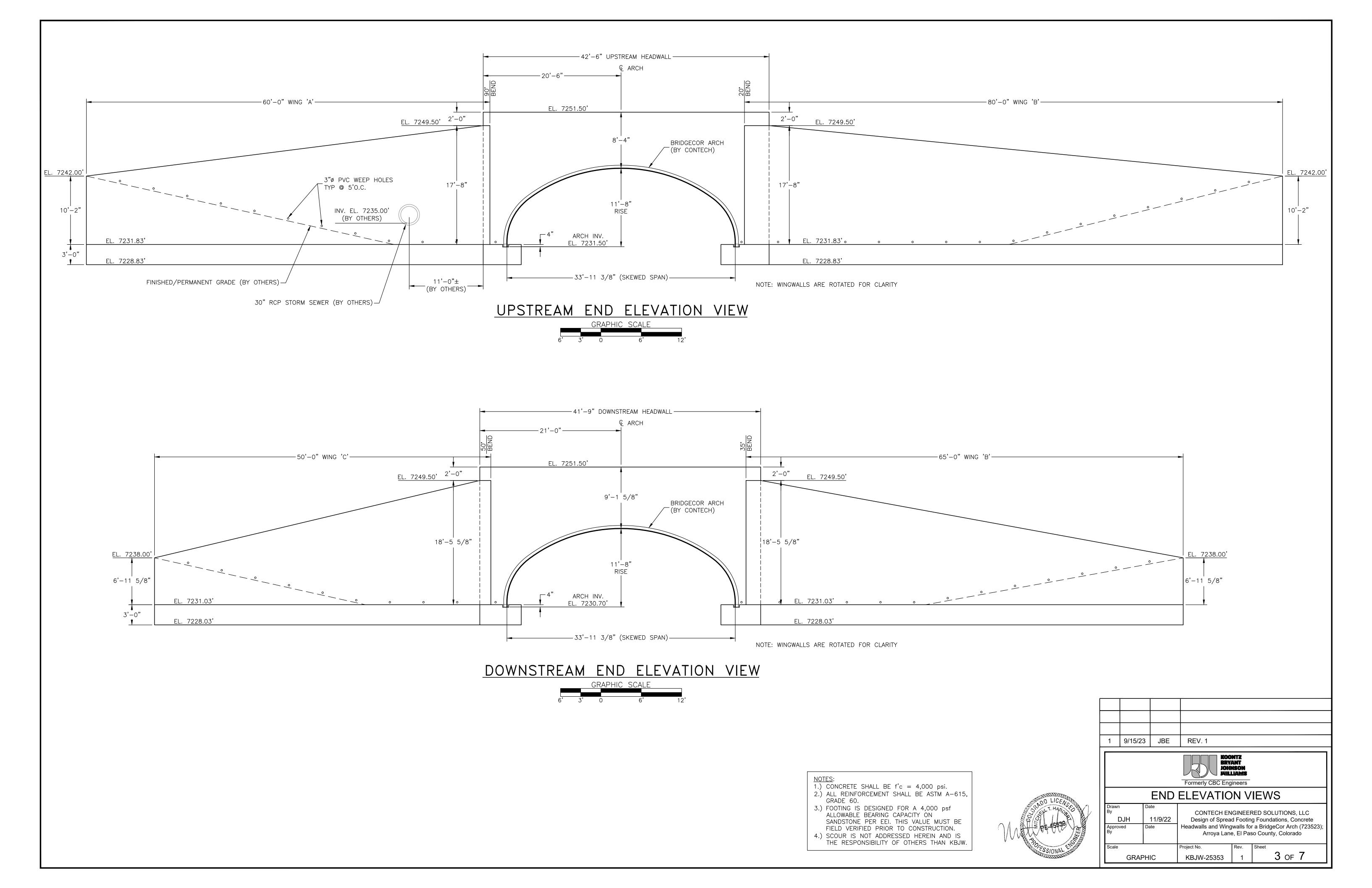
JOSHUA PALMER, P.E. COUNTY ENGINEER / ECM ADMINISTRATOR

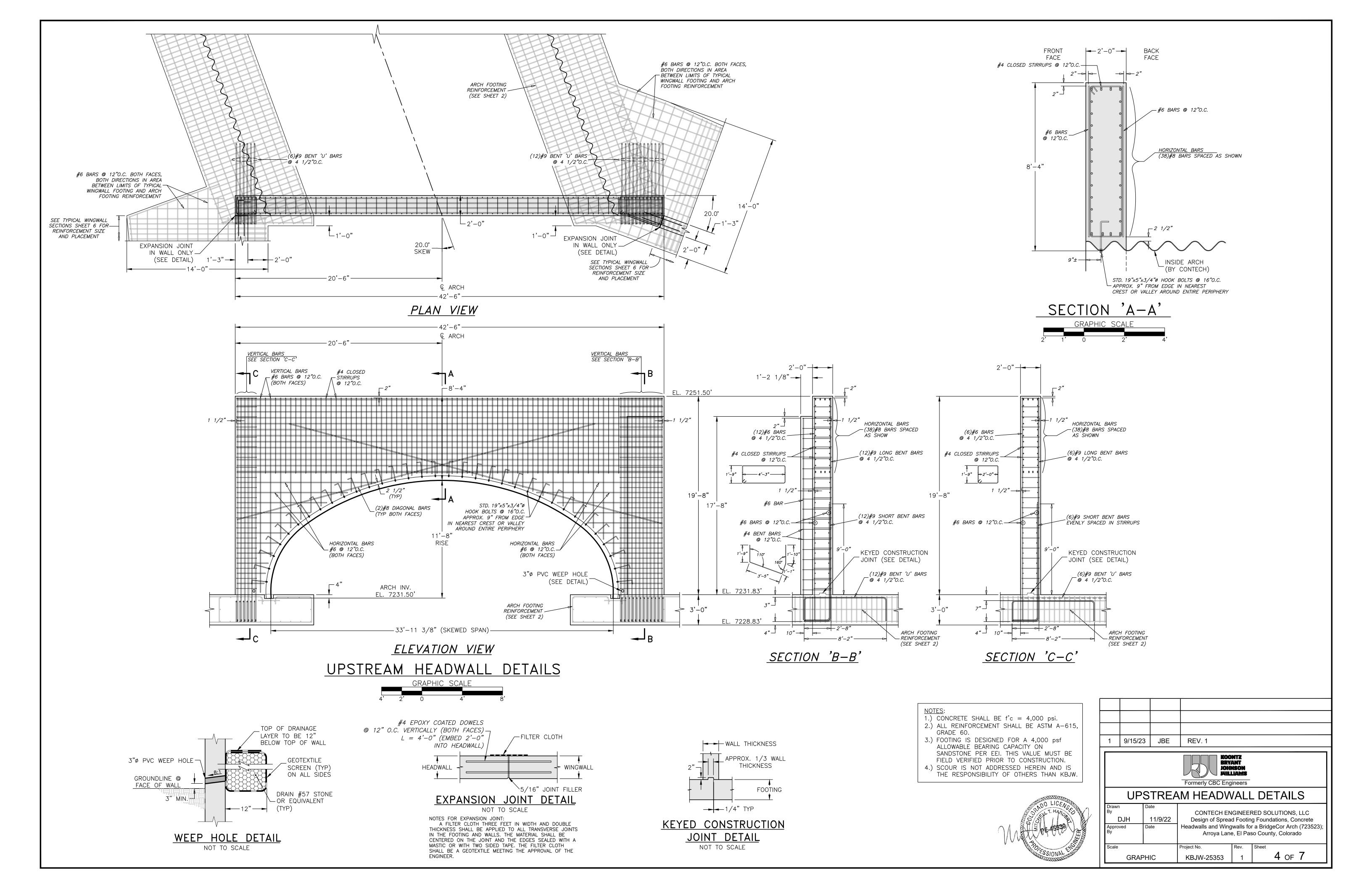
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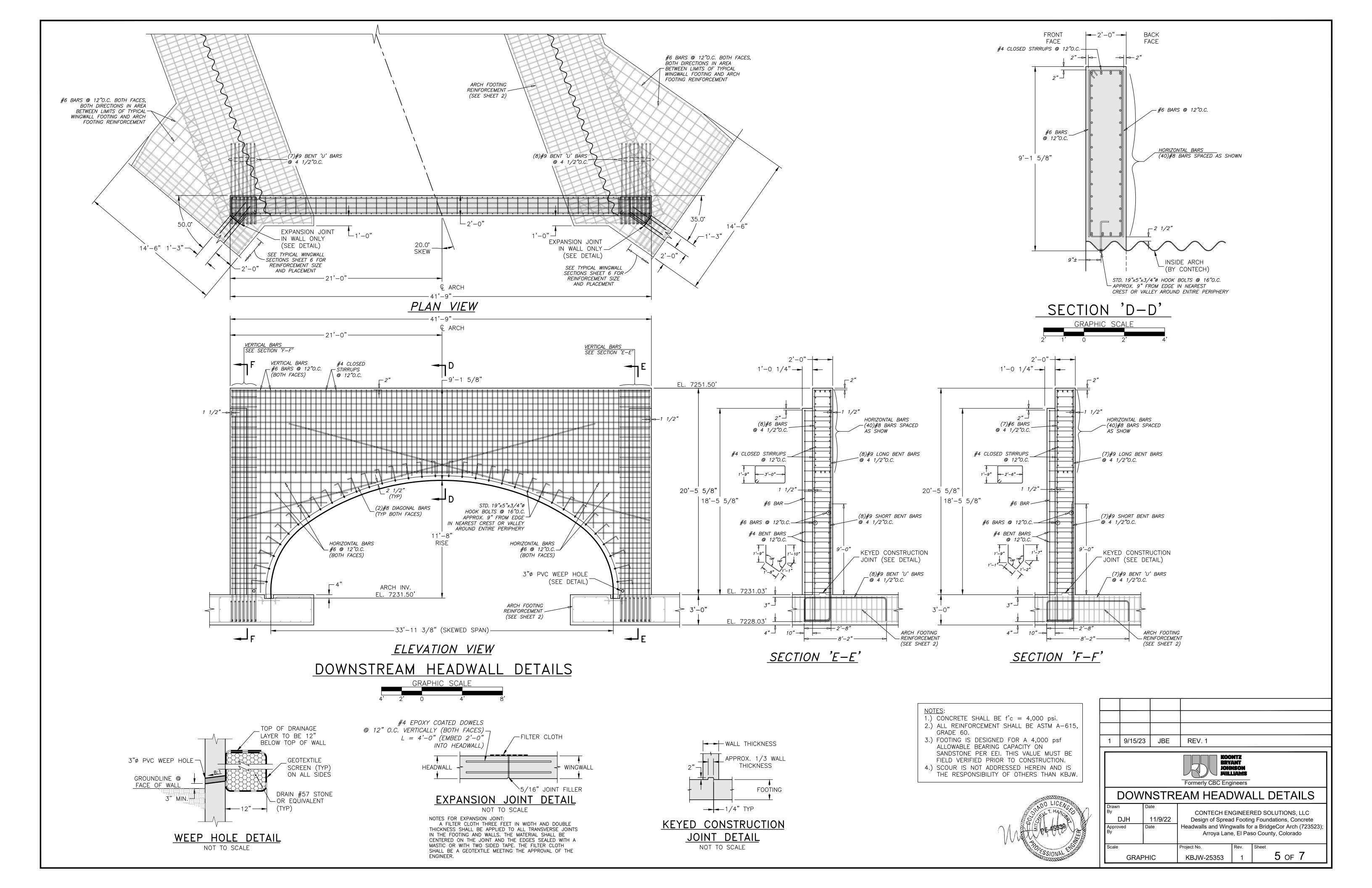


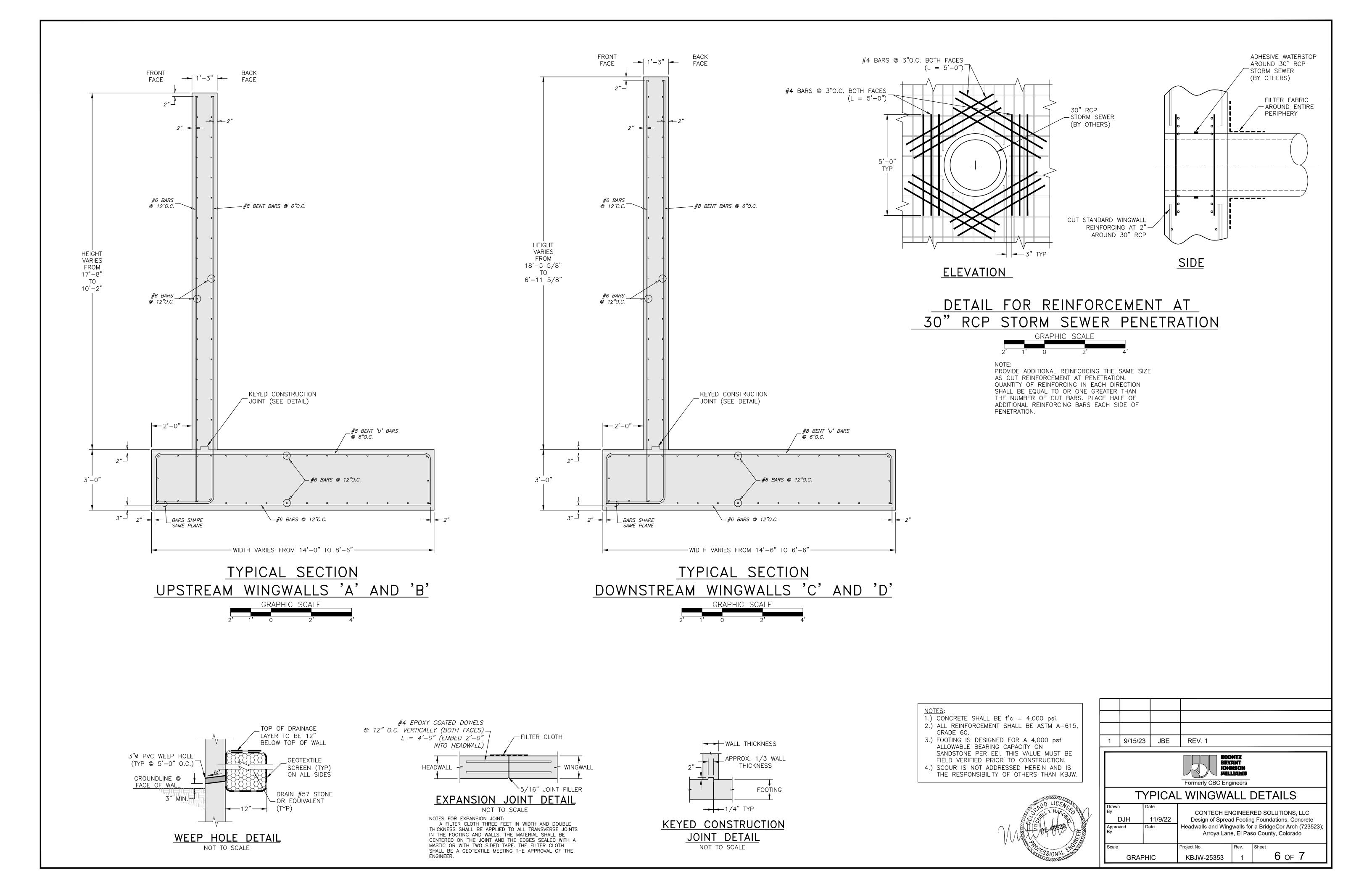
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Drawn By DJH		Date 11/9/22	CONTECH ENGINEERED SOLUTIONS, LLC Design of Spread Footing Foundations, Concrete		
Appro By	oved	Date	Headwalls and Wingwalls for a BridgeCor Arch (723523 Arroya Lane, El Paso County, Colorado		
Scale GRAPHIC			Project No. KBJW-25353	Rev.	Sheet 1 OF 7











## I - GENERAL

## STANDARDS AND DEFINITIONS

- 1.1 STANDARDS All standards refer to latest edition unless otherwise noted.
  - 1.1.1 ASTM D-698-70 (Method C) "Standard Test Methods for Moisture. Density Relations of Soils and Soil Aggregate Mixtures Using 5.5-lb (2.5 kg.) Rammer and 12-inch (305mm) Drop".
  - 1.1.2 ASTM D-2922 "Standard Test Method for Density of Soil and Soil Aggregate in Place by Nuclear methods (Shallow Depth)".
  - 1.1.3 ASTM D-1556 "Standard Test Method for Density of Soil in place by the Sand-Cone
  - 1.1.4 ASTM D-1557 "Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort."
  - 1.1.5 All construction and materials shall be in accordance with the latest AASHTO LRFD Bridge Design Specifications and DOT requirements.

#### 1.2 **DEFINITIONS**

- **1.2.1** Owner In these specifications the word "Owner" shall mean El Paso County, Colorado
- 1.2.2 Engineer In these specifications the word "Engineer" shall mean the Owner designated engineer.
- 1.2.3 Design Engineer In these specifications the words "Design Engineer" shall mean KBJW, Inc.
- 1.2.4 Contractor In these specifications the word "Contractor" shall mean the firm or corporation undertaking the execution of any work under the terms of these specifications.
- 1.2.5 Approved In these specifications the word "approved" shall refer to the approval of the Engineer or his designated representative.
- **1.2.6** As Directed In these specifications the words "as directed" shall refer to the directions to the Contractor from the Owner or his designated representative.

## **GENERAL CONDITIONS**

2.1 The Contractor shall furnish all labor, material and equipment and perform all work and services except those set out and furnished by the Owner, necessary to complete in a satisfactory manner the site preparation, excavation, filling, compaction, grading, footing construction, endwall construction as shown on the plans and as described therein.

This work shall consist of all mobilization clearing and grading, grubbing, stripping, removal of existing material unless otherwise stated, preparation of the land to be filled, filling of the land, spreading and compaction of the fill, and all subsidiary work necessary to complete the grading of the cut and fill areas to conform with the lines, grades, slopes, and specifications.

This work is to be accomplished under the observation of the Owner or his designated

Prior to bidding the work, the Contractor shall examine, investigate and inspect the construction site as to the nature and location of the work, and the general and local conditions at the construction site, including, without limitation, the character of surface or subsurface conditions and obstacles to be encountered on and around the construction site; and shall make such additional investigation as he may deem necessary for the planning and proper execution of the

If conditions other than those indicated are discovered by the Contractor, the Owner should be notified immediately. The material which the Contractor believes to be a changed condition should not be disturbed so that the owner can investigate the condition.

The construction shall be performed under the direction of an experienced engineer who is familiar with the design plan.

# II – BRIDGECOR FOOTINGS

# EXCAVATION FOR FOOTINGS

- 1.1 Footing excavation shall consist of the removal of all material, of whatever nature, necessary for the construction of foundations.
- 1.2 It shall be the responsibility of the Contractor to identify and relocate all existing utilities which conflict with the proposed footing locations shown on the plan. The Contractor must call the appropriate utility company at least 48 hours before any excavation to request exact field location of utilities, and coordinate removal and installation of all utilities with the respective utility company.
- The side of all excavations shall be cut to prevent sliding or caving of the material above the
- Excavated material shall be disposed in accordance with the plan established by the Engineer.
- The footings are designed for a net allowable bearing capacity of 4,000 psf on sandstone bedrock in accordance with the recommendations by Entech Engineering, Inc. A friction factor of 0.45 has also been utilized. These shall be verified in the field before construction. The evaluation and design of any required foundation improvement to achieve the recommended bearing capacity and friction factor, and to protect against frost and scour and settlement, is the responsibility of others than KBJW. All recommendations in the project geotechnical report shall be followed during construction.
- 1.6 Care must be exercised to ensure that heavy equipment is not operated adjacent to the footings. Heavy equipment shall not be allowed any closer than 5 feet to the footings.

# CONCRETE FOOTING DIMENSIONS

**2.1** The footings shall be reinforced in accordance with the construction drawings.

#### III - HEADWALLS AND WINGWALLS

- 1.0 The headwalls and wingwalls shall consist of reinforced concrete conforming to Chapter IV of these specifications, and AASHTO LRFD Bridge Design Specifications having a minimum compression
- 2.0 Reinforcing steel shall conform to ASTM A-615, Grade 60, having minimum yield strength of 60,000
- 3.0 The headwalls shall be anchored to the BridgeCor arch in the manner shown on the plans and shall be formed and poured in accordance with the plan dimensions.
- 4.0 Round weep holes spaced not over 5 feet on center shall be placed in the walls above finished grade as shown on the construction drawings. A granular envelope, consisting of #57 stone (clean 3/4" aggregate) or equivalent, shall be placed behind each weep hole for a distance of approximately 1 foot from all edges of the weep hole. A free-draining geotextile screen shall be placed between the weep hole and the stone to prevent erosion of the stone.
- 5.0 The select backfill behind the headwalls must be a well-graded, angular, durable granular material conforming to the select backfill specifications for the BridgeCor arch placed and compacted to achieve a minimum internal friction angle of 36 degrees and a maximum unit weight of 120 pcf. The material must be placed in strict conformance with the project specifications, the manufacturer's requirements, and industry standards. The select backfill behind the wingwalls must be a well-graded, angular, durable granular material placed and compacted to achieve a minimum internal friction angle of 36 degrees and a maximum unit weight of 120 pcf. These values to be field verified.
- 6.0 All Federal, State, and Local regulations shall be strictly adhered to relative to excavation side-slope geometry and any required excavation shoring.

## IV – CONCRETE FOR FOOTINGS AND ENDWALLS

### 1.0 CODES AND STANDARDS

1.1 Reinforced concrete shall conform to the requirements of Colorado DOT Standards and Section V of the AASHTO LRFD Bridge Design Specifications having a minimum compressive strength of 4,000 psi.

## 2.0 STANDARDS FOR MATERIALS

- **2.1** Portland Cement Conforming to ASTM Specification C-150, Type II.
- 2.2 Water The water shall be drinkable, clean free from injurious amounts of oils, acids, alkalis, organic materials, or deleterious substances
- Aggregates Fine and coarse aggregates shall conform to current ASTM Specification C-33 "Specification for Concrete Aggregates" except that local aggregates which have been shown by tests and by actual service to produce satisfactory qualities may be used when approved by the Engineer.
- 2.4 Submittals Test data and/or certifications to the Owner shall be furnished upon request.

# 3.0 PROPORTIONING OF CONCRETE

# 3.1 COMPOSITION

- **3.1.1** The concrete shall be composed of cement, fine aggregate, coarse aggregate and water.
- **3.1.2** The concrete shall be homogeneous, readily placeable and uniformly workable and shall be proportioned in accordance with ACI-211.1.
- 3.1.3 Proportions shall be established on the basis of field experience with the materials to be employed. The amount of water used shall not exceed the maximum 0.45 water/cement ratio, and shall be reduced as necessary to produce concrete of the specified consistency at the time of placement.
- **3.1.4** An air-entraining admixture, conforming to the requirements of ASTM C260, shall be used in all concrete furnished under this contract. The quantity of admixture shall be such as to produce an air content in the freshly mixed concrete of 6 percent plus or minus 1 percent as determined in accordance with ASTM C231 or C173.
- 3.2 Qualities Required As indicated in the table below:

# TABLE IV-1

# QUALITIES REQUIRED

ITEM	QUALITY REQUIRED		
AASHTO Class	A		
Type of Cement	II		
Compressive Strength fc @ 28 days	4,000 psi		
Slump, inches	2 - 4 in.		

- Maximum Size of Coarse Aggregates Maximum size of coarse aggregates shall not be larger than 19 mm (3/4 inches).
- 3.4 Rate of Hardening of Concrete Concrete mix shall be adjusted to produce the required rate of hardening for varied climatic conditions:

Under 40°F Ambient Temperature – All work to be in accordance with the recommendations of ACI-306R "Cold Weather Concreting."

# 4.0 MIXING AND PLACING

- 4.1 Equipment Ready Mix Concrete shall be used and shall conform to the "Specifications for Ready-Mix Concrete," ASTM C-94. Approval is required prior to using job mixed concrete.
- 4.2 Preparation All work shall be in accordance with ACI-304, "Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete." All construction debris and extraneous matter shall be removed from within the forms. Concrete shall be placed on clean surfaces, free from water. Concrete that has to be dropped four (4) feet or more shall be placed through a tremie.
- 4.3 All concrete shall be consolidated by internal mechanical vibration immediately after placement. Vibrators shall be of a size appropriate for the work, capable of transmitting vibration to concrete at frequencies of not less than 4,500 impulses per minute.

## FORM WORK

- 5.1 Forms shall be of wood, steel or other approved material and shall be set and held true to the dimensions, lines and grades of the structure prior to and during the placement of concrete.
- 5.2 Forms shall not be removed until the concrete has sufficient strength to prevent concrete damage and/or drainage.

#### 6.0 CURING

6.1 Fresh concrete shall be protected from rains, flowing water and mechanical injury for a period of seven (7) days. Loads shall not be placed on the concrete until it has reached its design

## 7.0 REINFORCING STEEL

#### 7.1 MATERIAL

7.1.1 All reinforcing bars shall be deformed bars (ASTM-A615) Grade 60.

### 7.2 BENDING AND SPLICING

- 7.2.1 Bar reinforcement shall be cut and bent to the shapes shown on the plans. Fabrication tolerances shall be in accordance with ACI 315. All bars shall be bent cold, unless otherwise permitted.
- 7.2.2 All reinforcement shall be furnished in the full lengths indicated on the plans unless otherwise permitted. Except for splices shown on the plans and splices for No. 5 or smaller bars, splicing of bars will not be permitted without written approval. Splices shall be staggered as far as possible.
- 7.2.3 In lapped splices, the bars shall be placed and wired in such a manner as to maintain the minimum distance to the surface of the concrete shown on the plans.
- 7.2.4 Substitution of different size bars will be permitted only when authorized by the engineer. The substituted bars shall have an area equivalent to the design area, or larger.

## 7.3 PLACING AND FASTENING

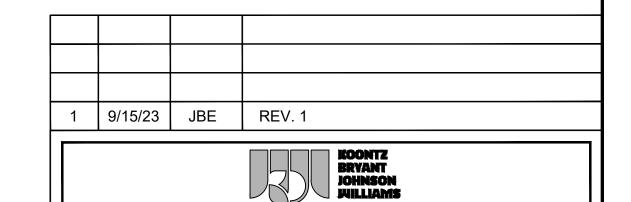
- 7.3.1 Steel reinforcement shall be accurately placed as shown on the plans and firmly held in position during the placing and setting of concrete. Bars shall be tied at all intersections around the perimeter of each mat and at not less than 2-foot centers or at every intersection, whichever is greater, elsewhere. Welding of cross bars (tack welding) will not be permitted for assembly of reinforcement.
- 7.3.2 Reinforcing steel shall be supported in its proper position by use of mortar blocks, wire bar supports, supplementary bars or other approved devices. Such devices shall be of such height and placed at sufficiently frequent intervals so as to maintain the distance between the reinforcing and the formed surface or the top surface within 1/4 inch of that indicated on the plans.

## V - FILTER FABRIC (GEOTEXTILE SCREEN)

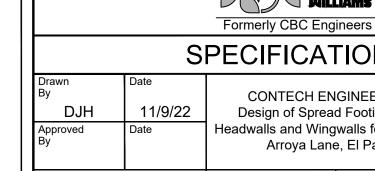
- 1.0 Filter fabric shall be placed at all locations shown on the construction drawings, and as necessary between all dissimilar materials to prevent soil migration and to maintain a soil-tight system.
- 2.0 Filter fabric cloth shall conform to Contech specification for C60-NW or equivalent and shall meet the following ASTM tests:
  - 2.1 ASTM D4751 Apparent opening size equal to #70 U.S. Standard Sieve Size.
  - 2.2 ASTM D4632 (Grab Tensile Test) Minimum Strength = 160 pounds.
  - **2.3** ASTM D4632 (Grab Elongation) 30-70%.
  - 2.4 ASTM D4533 (Trapezoidal Tear) Minimum Strength = 60 pounds.
  - 2.5 ASTM D4355 (Stabilized for Heat and Ultra-Violet Degradation) 70% strength retained.
- The minimum fabric coefficient of permeability (ASTM D4491) shall be 0.24 cm/sec.
- The fabric shall be non-woven with a minimum thickness (ASTM D5199) of 60 mils

5.0 Fabric shall not be placed over sharp or angular rocks that could tear or puncture it.

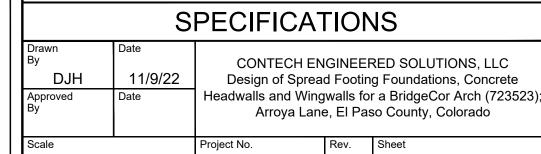
Care should be exercised to prevent any puncturing or rupture of the filter fabric. Should such rupture occur, the damaged area should be covered with a patch of filter fabric using an overlap minimum of



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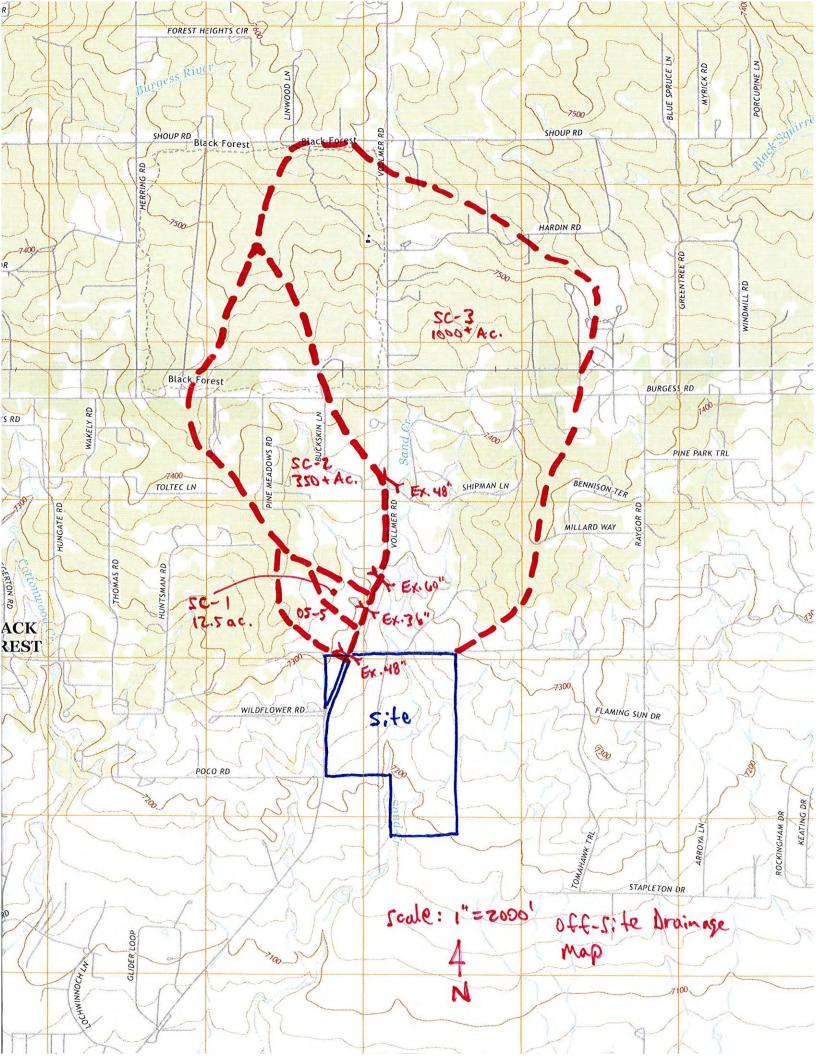
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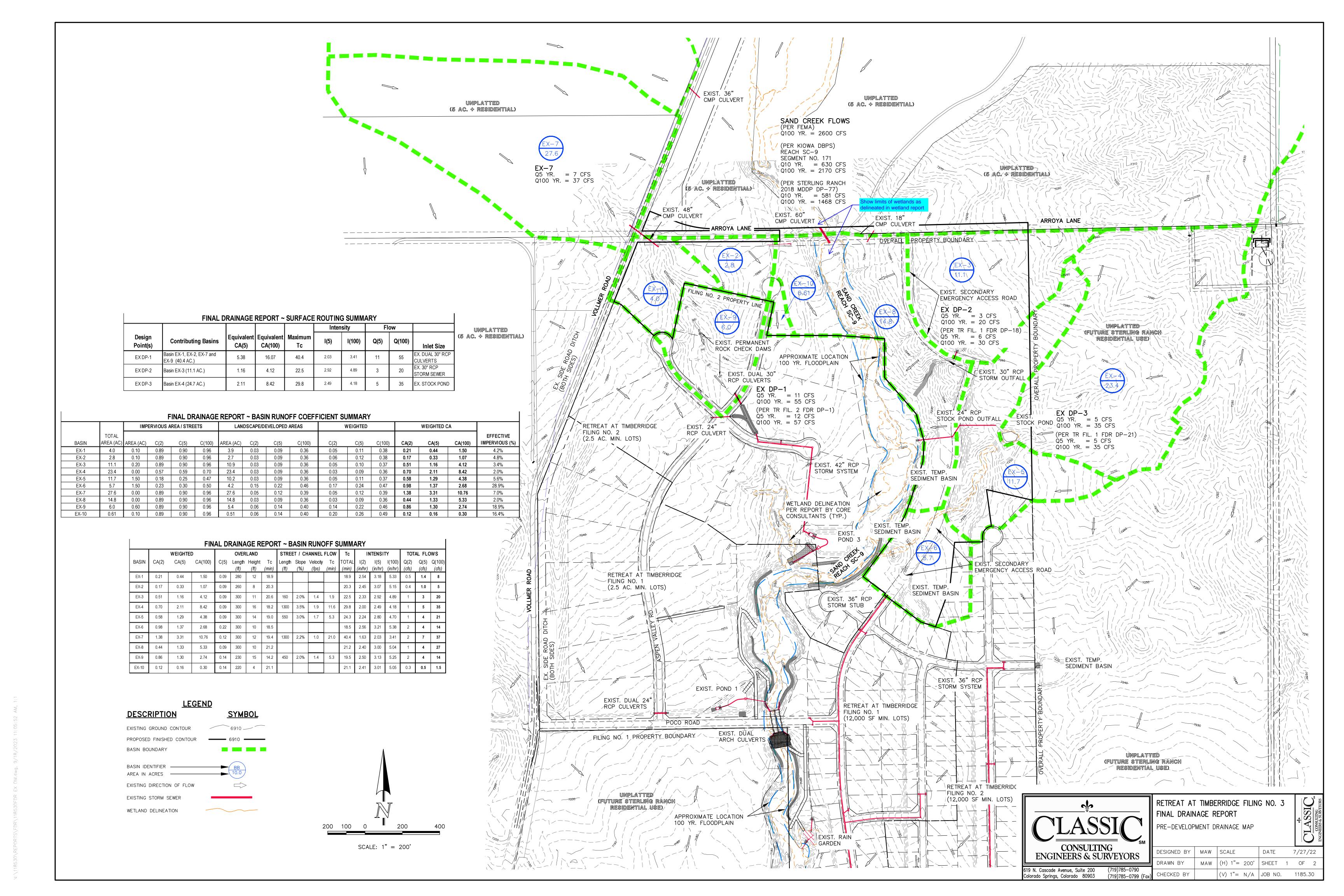


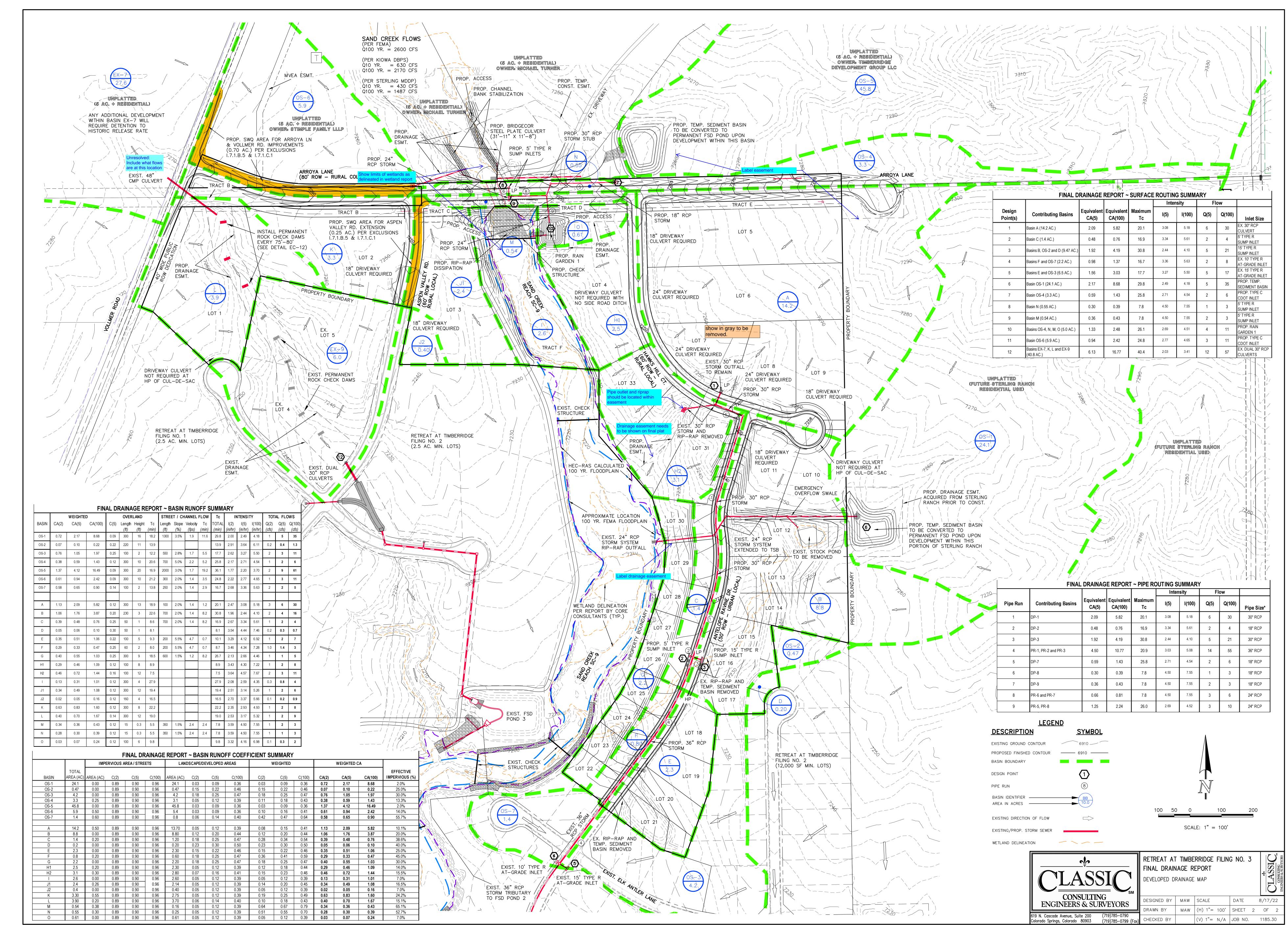
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## **DRAINAGE MAPS**









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