PRELIMINARY DRAINAGE PLAN

CREEKSIDE AT LORSON RANCH FILING NO. 1

AUGUST, 2018

PUD SP-18-X

PUDSP-18-005

Prepared for:

Lorson, LLC
212 N. Wahsatch Ave, Suite 301
Colorado Springs, Colorado 80903
(719) 635-3200

Prepared by:

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(719) 570-1100

Project No. 100.045

CORE
ENGINEERING GROUP

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Add a section Labeled
"Four Step Process"

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 El Paso County for drainage reports and said report is in conformity with the 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.

Richard L. Schindler, P.E. #33997 Date
For and on Behalf of Core Engineering Group, LLC

OWNER'S STATEMENT

I, the Owner, have read and will comply with all the requirements specified in the drainage report and plan.

Lorson, LLC Date

By
Jeff Mark

Title
Manager

Address
212 N. Wahsatch Avenue, Suite 301, Colorado Springs, CO 80903

FLOODPLAIN STATEMENT

To the best of my knowledge and belief, this development is located within a designated floodplain as shown on Flood Insurance Rate Map Panel No. 08041C0957 F, dated March 17, 1997 and modified by modified per LOMR Case No. 14-08-0534P. (See Appendix A, FEMA FIRM Exhibit)

Richard L. Schindler, #33997 Date

EL PASO COUNTY

Filed in accordance with the requirements of the El Paso County Land Development Code, Drainage Criteria Manual, Volume 1 and 2, and Engineering Criteria Manual, As Amended.

Jennifer Irvine Date
County Engineer/ECM Administrator

Conditions: ______________________________

1.0 LOCATION and DESCRIPTION

Creekside at Lorson Ranch Filing No. 1 is located north of the East Tributary of Jimmy Camp Creek (Etrib). The site is located on approximately 83.085 acres of vacant land. Future plans are to develop this site into single-family residential developments. Also included in this report and plan is the proposed layout for Creekside at Lorson Ranch Filing No. 1 which is located west and north of the East Tributary of Jimmy Camp Creek. The land is currently owned by Lorson LLC or its nominees for Lorson Ranch.

The site is located in the North 1/2 of Section 23, Township 15 South and Range 65 West of the 6th Principal Meridian. The property is bounded on the north by Lorson Boulevard, on the east by the Etrib, the west by Jimmy Camp Creek, and the south by unplatted land in Lorson Ranch. For reference, a vicinity map is included in Appendix A of this report.

Conformance with applicable Drainage Basin Planning Studies

There is an existing (unapproved) DBPS for Jimmy Camp Creek prepared by Wilson & Company in 1987, and is referenced in this report. The only major drainage improvements for this study area according to the 1987 Wilson study was the reconstruction of the East Tributary of Jimmy Camp Creek (East Tributary). In 2014 a portion of the East Tributary was reconstructed from Fontaine Boulevard south 2,800 feet in accordance with the 1987 study which is located within this project. This section of the East Tributary included a trapezoidal channel section with 6:1 side slopes and a sand bottom. On March 9, 2015 a new DBPS for Jimmy Camp Creek and the East Tributary was completed by Kiowa Engineering. The Kiowa Engineering DBPS for Jimmy Camp Creek has not been adopted by El Paso County but is allowed for concept design. The concept design for the remaining portions of the Etrib include an armoring concept and full spectrum detention pond requirements. The Kiowa DBPS did not calculate drainage fees so current El Paso County drainage/bridge fees apply to this development. Per the Kiowa DBPS concept the preferred channel improvements include selective channel armoring on outer bends and a low flow channel for the East Tributary. Channel improvements in the East Tributary are potentially reimbursable against drainage fees for future development but need to be processed through the county process for reimbursement.

Conformance with Lorson Ranch MDDP1 by Pentacor Engineering

Lorson Ranch MDDP1 (October 26, 2006) includes this preliminary plan area and the East Tributary. This PDR conforms to the MDDP1 for Lorson Ranch and is referenced in this report. The major infrastructure to be constructed in this PDR site includes the Etrib armoring from the south property line of Lorson Ranch east and north to the previously reconstructed Etrib completed in 2014 and construction of several on-site detention ponds. Kiowa Engineering is currently designing this section of the East Tributary and is included in the appendix of this report. Detention/WQ Pond C1-R (existing) and several proposed detention ponds are shown within this preliminary plan area and will be designed/constructed as part of Creekside at Lorson Ranch Filing No. 1.

Reconstruction of the East Tributary of Jimmy Camp Creek

The Kiowa DBPS shows the East Tributary to be protected using selective armoring (soil rip rap) at the outside stream bends (500' minimum radius) and a stabilized low flow channel. The East Tributary has been divided into three different sections, south, middle, and north. The first section (south) is from the south property line east and north to design point ET-3 (see drainage map) and is roughly 2,900 feet in length. The south section is within this preliminary plan area and will be armored in accordance with the Kiowa DBPS and is currently being designed by Kiowa Engineering. The Etrib construction plans will be submitted for approval before or in conjunction with this preliminary plan submittal. The 100-year flow rate for design is 5,500cfs for the south section. The middle section is from Design Point ET-3 north 2,800 feet to the future extension of Fontaine Boulevard. The channel for this section was reconstructed and stabilized in 2014 in accordance with the 1987 Wilson DBPS. LOMR Case No. 14-08-0534P was approved by FEMA for this middle section. The northern section is from Fontaine Boulevard and extends north to the north property line. The north section is under construction in 2018 in conformance with the Kiowa DBPS as part of Lorson Ranch East Filing No. 1 improvements. The channel consists of a stabilized low flow channel and soil rip rap armored outer bends. A CLOMR for the creek construction is approved by FEMA under Case No. 17-08-1043R. The 100-year flow rate for design is from FEMA FIS data and is from 4,400cfs to 4,750cfs for this section. The low flow channel is sized using 10% of the 100-yr FEMA flow rates and is from 440cfs to 475cfs.

Creekside at Lorson Ranch Filing No. 1 is located within the ***"Jimmy Camp Creek Drainage Basin"***, which is a fee basin in El Paso County.

2.0 DRAINAGE CRITERIA

The supporting drainage design and calculations were performed in accordance with the City of Colorado Springs and El Paso County "Drainage Criteria Manual (DCM)", dated November, 1991, the El Paso County "Engineering Criteria Manual", Chapter 6 and Section 3.2.1 Chapter 13 of the City of Colorado Springs Drainage Criteria Manual dated May 2014, and the UDFCD "Urban Storm Drainage Criteria Manual" Volumes 1, 2 and 3 for inlet sizing and full spectrum ponds. No deviations from these published criteria are requested for this site. The proposed improvements to the Lorson Ranch Development will be in substantial compliance with the "Jimmy Camp Creek Drainage Basin Planning Study", prepared by Kiowa Engineering Corp., Colorado Springs, CO.

The Rational Method as outlined in Section 6.3.0 of the May 2014 "Drainage Criteria Manual" and in Section 3.2.8.F of the El Paso County "Engineering Criteria Manual" was used for basins less than 130 acres to determine the rainfall and runoff conditions for the proposed development of the site. The runoff rates for the 5-year initial storm and 100-year major design storm were calculated.

Current updates to the Drainage Criteria manual for El Paso County states the if detention is necessary, Full Spectrum Detention will be included in the design, based on this criteria, Full Spectrum Detention will be required for this development

Add the section "Four Step Process" discuss the four steps utilized

3.0 EXISTING HYDROLOGICAL CONDITIONS

The site is currently undeveloped with native vegetation (grass with no shrubs) and slopes in a southerly direction to the East Tributary of Jimmy Camp Creek.

The Soil Conservation Service (SCS) classifies the soils within the Lorson Ranch East property as Blendon Sandy Loam (40%); Ellicott Loamy Coarse Sand (1%) Manzanst clay loam (59%) [3]. The sandy loams are considered hydrologic soil group A/B soils with moderate to moderately rapid permeability. The clay loams are considered hydrologic soil group C soils with slow permeability. For the purposes of this report the Ellicot Loamy Coarse Sand will not be used since it is only 1% of the site and is in an area that will not be disturbed. All of these soils are susceptible to erosion by wind and water, have low bearing strength, moderate shrink-swell potential, and high frost heave potential (see table 3.1 below). The clay loams are difficult to vegetate. These soils can be mitigated easily by limiting their use as topsoil.

Table 3.1: SCS Soils Survey.

Soil	Hydro. Group	Shrink/Swell Potential	Permeability	Surface Runoff Potential	Erosion Hazard
10-Blendon Sandy Loam (40%)	B	Low	Moderately Rapid	Slow	Moderate
28-Ellicott Loamy Coarse Sand (1%)	A	Low	Rapid	Slow	High
52Manzanst Clay Loam (59%)	C	Moderate to High	Slow	Medium	Moderate

Excerpts from the SCS "Soil Survey of El Paso County Area, Colorado" are provided in ***Appendix A*** for further reference.

For the purpose of preparing hydrologic calculations for this report, the soil of each basin are assumed to be wholly comprised of the majority soil hydrologic group.

Portions of the site are located within the delineated 100-year floodplain of the East Tributary of Jimmy Camp Creek per the Federal Emergency Management Agency (FEMA) Flood Rate Insurance Map (FIRM) number 08041C0957 F, effective March 17, 1997 [2]. Floodplain along Jimmy Camp Creek was modified per LOMR Case No. 06-08-B643P, effective August 29, 2007 (see appendix). Floodplain along the East Tributary was modified per LOMR Case No. 14-08-0534P, effective January 29, 2015 (see appendix). Floodplain designations include Zone AE and Zone X within the property boundary. A portion of this map is provided in ***Appendix A*** for reference. A CLOMR for the creek construction by Kiowa Engineering will not be necessary since BFE's are not changing.

Basin EX-B
This 35.5 acre basin includes the east portions of the site. Under existing conditions, this area flows overland south to the East Tributary contributes 17.6cfs and 94.0cfs for 5-year and 100-year events respectively.

Basin EX-C1
This 10.32 acre basin includes the middle portions of the site. Under existing conditions, this area flows overland south to the East Tributary contributes 5.3cfs and 29.7cfs for 5-year and 100-year events respectively.

Basin EX-D
This 29.29 acre basin includes the west portions of the site. Under existing conditions, this area flows overland south to the East Tributary contributes 8.6cfs and 57.5cfs for 5-year and 100-year events respectively. A very small portion of the runoff at the south property line of Lorson Ranch flows south onto the golf course property but was not calculated because the proposed Pond CR2 located next to the south property line will capture all the flow from the developed areas of the site.

4.0 DEVELOPED HYDROLOGICAL CONDITIONS

Hydrology for the **Creekside at Lorson Ranch Filing No. 1** drainage report was based on the City of Colorado Springs/El Paso County Drainage Criteria. Sub-basins that lie within this project were determined and the 5-year and 100-year peak discharges for the developed conditions have been presented in this report. Based on these flows, storm inlets will be added when the street capacity is exceeded.

This site can be broken into two soil types. The west portions are Soil Type B and the east portions are Soil Type C. See Appendix A for SCS Soils Map.

The time of concentration for each basin and sub-basin was developed using an overland, ditch, street and pipe flow components. The maximum overland flow length for developed conditions was limited to 100 feet. Travel time velocities ranged from 2 to 6 feet per second. The travel time calculations are included in the back of this report. Runoff coefficients for the various land uses were obtained from the City of Colorado Springs/El Paso County Drainage Criteria Manual.

Drainage concepts for each of the basins are briefly discussed as follow:

Basin C1.1
This basin consists of runoff from residential development. Runoff will be directed west in Kalama Drive to Design Point 1 in curb/gutter where it will be collected by a Type R inlet on Alsea Drive. The developed flow from this basin is 3.8cfs and 8.4cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.2
This basin consists of runoff from residential development. Runoff will be directed west in Castor Drive to Design Point 1 in curb/gutter where it will be collected by a Type R inlet on Alsea Drive. The developed flow from this basin is 5.4cfs and 12.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.3-C1.4
These basins consist of runoff from residential development. Runoff will be directed west in Kalama Drive to Design Point 2 in curb/gutter where it will be collected by a Type R inlet on Alsea Drive. The developed flow from these basins is 1.8cfs/ 4.0cfs for the 5/100-year storm event for Basin C1.3 and 4.5cfs/ 10.0cfs for the 5/100-year storm event for Basin C1.3. See the appendix for detailed calculations.

Basin C1.5
This basin consists of runoff from residential development. Runoff will be directed to Design Point 3 in curb/gutter where it will be collected by a Type R inlet on Alsea Drive. The developed flow from this basin is 0.4cfs and 1.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.6
This basin consists of runoff from residential development. Runoff will be directed west in Castor Drive to Design Point 6 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 1.5cfs and 3.3cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.7
This basin consists of runoff from residential development. Runoff will be directed east in Castor Drive to Design Point 6 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 3.1cfs and 6.8cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.8
This basin consists of runoff from residential development. Runoff will be directed east in Castor Drive to Design Point 6 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 1.6cfs and 3.5cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.9
This basin consists of runoff from residential development. Runoff will be directed west in Castor Drive to Design Point 10 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 3.5cfs and 7.8cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.10-C1.11
These basins consist of runoff from residential development. Runoff will be directed north in Maidford Drive to Design Point 2 in curb/gutter on Castor Drive. The developed flow from these basins is 0.4cfs/

0.8cfs for the 5/100-year storm event for Basin C1.10 and 0.4cfs/ 0.9cfs for the 5/100-year storm event for Basin C1.11. See the appendix for detailed calculations.

Basin C1.12

This basin consists of runoff from residential development. Runoff will be directed west in Castor Drive to Design Point 10 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 1.8cfs and 4.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.13

This basin consists of runoff from residential development. Runoff will be directed east in Castor Drive to Design Point 10 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 1.4cfs and 3.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.14

This basin consists of runoff from residential development. Runoff will be directed east in Castor Drive to Design Point 10 in curb/gutter where it will be collected by a Type R inlet on Castor Drive. The developed flow from this basin is 2.3cfs and 5.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.15

This basin consists of runoff from residential development. Runoff will be directed south in Maidford Drive Design Point 11 in curb/gutter where it will be collected by a Type R inlet. The developed flow from this basin is 1.6cfs and 3.5cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.16

This basin consists of runoff from residential development. Runoff will be directed south in Maidford Drive Design Point 11 in curb/gutter where it will be collected by a Type R inlet. The developed flow from this basin is 1.1cfs and 2.5cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.17

This basin consists of runoff from residential development. Runoff will be directed south overland to Design Point 12 where it will be collected by a CDOT Type D inlet. The developed flow from this basin is 2.9cfs and 6.3cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C1.18

This basin consists of runoff from residential development and open space areas draining directly to Pond C1-R. Runoff will be directed overland to Pond C1-R. The developed flow from this basin is 5.7cfs and 19.5cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C2

This basin consists of runoff from backyards of residential development and open space areas draining directly to the East Tributary. The developed flow from this basin is 11.6cfs and 25.9cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary. See the appendix for detailed calculations.

A Deviation request must be provided to allow developed portions of this area to not receive complete treatment.

Basin C3

This basin consists of runoff from backyards of residential development and open space areas draining directly to the East Tributary. The developed flow from this basin is 1.5cfs and 3.2cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary. See the appendix for detailed calculations.

This area should be collected in an area drain & routed to CR -1.

Basin C4

This basin consists of runoff from backyards of residential development and open space areas draining directly to the East Tributary. The developed flow from this basin is 4.1cfs and 9.2cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary. See the appendix for detailed calculations.

A Deviation request must be provided to allow developed portions of this area to not receive complete treatment..

Basin C5.1

This basin consists of runoff from residential development. Runoff will be directed south in Yazoo Drive Design Point 15 in curb/gutter where it will be collected by a Type R inlet. The developed flow from this basin is 2.2cfs and 3.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C5.2

This basin consists of runoff from backyards of residential development and open space areas draining to Pond CR3. The developed flow from this basin is 1.3cfs and 2.3cfs for the 5/100-year storm event. The runoff will be detained/treated in Pond CR3 prior to entering the East Tributary. See the appendix for detailed calculations.

Overall Basin C5

This overall basin consists of runoff from backyards of residential development and open space areas draining directly to Pond CR3. The developed flow from this overall basin is 3.5cfs and 6.0cfs for the 5/100-year storm event. The runoff will be detained/treated in Pond CR3 prior to entering the East Tributary. See the appendix for detailed calculations.

Basin C6

This basin consists of runoff from backyards of residential development and open space areas draining directly to the East Tributary. The developed flow from this basin is 1.7cfs and 3.8cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary. See the appendix for detailed calculations.

This area should be routed to CR 3.

Basin D1.1

This basin consists of runoff from backyards of residential development and open space areas draining south to an 18” end section at Design Point 16. The developed flow from this basin is 2.1cfs and 4.6cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.2

This basin consists of runoff from Lorson Boulevard west of Tensas Drive. The runoff flows east to Tensas Drive then flows south in Tensas Drive. The developed flow from this basin is 2.2cfs and 3.9cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.3

This basin consists of runoff from residential development. Runoff will be directed west in Castor Drive to Design Point 17 at Tensas Drive. The developed flow from this basin is 0.8cfs and 1.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.4

This basin consists of runoff from residential development. Runoff will be directed south in Castor Drive to Design Point 18. The developed flow from this basin is 2.1cfs and 4.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.5

This basin consists of runoff from residential development. Runoff will be directed south in Castor Drive to Design Point 23 in curb/gutter where it will be collected by a Type R inlet. The developed flow from this basin is 1.5cfs and 3.4cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.6
This basin consists of runoff from residential development. Runoff will be directed south in Castor Drive to Design Point 20 in curb/gutter. The developed flow from this basin is 2.2cfs and 4.8cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.7
This basin consists of runoff from residential development. Runoff will be directed southwest in Winnicut Drive to Design Point 20 in curb/gutter. The developed flow from this basin is 2.2cfs and 4.9cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.8
This basin consists of runoff from residential development. Runoff will be directed southwest in Winnicut Drive to Design Point 21 in curb/gutter. The developed flow from this basin is 1.7cfs and 3.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1.9
This basin consists of runoff from residential development. Runoff will be directed south in Castor Drive to Design Point 23 in curb/gutter where it will be collected by a Type R inlet. The developed flow from this basin is 0.5cfs and 1.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Overall Basin D1
This overall basin consists of runoff from backyards of residential development and open space areas draining directly to Pond CR2 and is the total flow in the storm sewer at Design Point 23. The developed flow from this overall basin is 12.1cfs and 26.1cfs for the 5/100-year storm event. The runoff will be detained/treated in Pond CR2 prior to entering the East Tributary. See the appendix for detailed calculations.

Basin D2
This basin consists of runoff from backyards of residential development and open space areas draining directly to Jimmy Camp Creek. The developed flow from this basin is 2.4cfs and 5.2cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering Jimmy Camp Creek. See the appendix for detailed calculations.

A Deviation request must be provided to allow developed portions of this area to not receive complete treatment..

Basin D3
This basin consists of runoff from backyards of residential development and open space areas draining directly to Jimmy Camp Creek. The developed flow from this basin is 0.5cfs and 2.2cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering Jimmy Camp Creek. See the appendix for detailed calculations.

Remove the backyard reference.

Basin D4
This basin consists of runoff from backyards of residential development and open space areas draining directly to the East Tributary. The developed flow from this basin is 3.6cfs and 8.0cfs for the 5/100-year storm event. The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary. See the appendix for detailed calculations.

The western area of this basin should be routed to CR 2.

Basin D5
This basin consists of runoff from backyards of residential development and open space areas draining directly to Pond D5. The developed flow from this basin is 1.1cfs and 3.8cfs for the 5/100-year storm event. ~~The backyard runoff will cross a grass buffer BMP prior to entering the East Tributary~~. See the appendix for detailed calculations.

Basin D6

This basin consists of runoff from open space areas draining south offsite onto the golf course as in existing conditions. No grading will be done in this basin and it will have the same drainage characteristics as in pre-developed conditions. The developed flow from this basin is 0.1cfs and 0.6cfs for the 5/100-year storm event. This flow is the same as pre-developed conditions. See the appendix for detailed calculations.

See the Developed Conditions Hydrology Calculations in the back of this report and the Developed Conditions Drainage Map (Map Pocket) for the 5-year and 100-year storm event amounts.

5.0 HYDRAULIC SUMMARY

The sizing of the hydraulic structures and detentions ponds were prepared by using the *StormSewers* and *Hydrographs* computer software programs developed by Intellisolve, which conforms to the methods outlined in the “City of Colorado Springs/El Paso County Drainage Criteria Manual”. Street capacities and Inlets were sized by Denver Urban Drainage’s xcel spreadsheet UD-Inlet.

It is the intent of this drainage report to use the proposed curb/gutter and storm sewer in the streets to convey runoff to detention and water quality ponds then to the East Tributary of Jimmy Camp Creek. Inlet size and location are preliminary only as shown on the storm sewer layout in the appendix. See Appendix C for detailed hydraulic calculations and the storm sewer model.

Table 1: Street Capacities (100-year capacity is only ½ of street)

	Residential Local		Residential Collector		Principal Arterial	
Street Slope	5-year	100-year	5-year	100-year	5-year	100-year
0.5%	6.3	26.4	9.7	29.3	9.5	28.5
0.6%	6.9	28.9	10.6	32.1	10.4	31.2
0.7%	7.5	31.2	11.5	34.6	11.2	33.7
0.8%	8.0	33.4	12.3	37.0	12.0	36.0
0.9%	8.5	35.4	13.0	39.3	12.7	38.2
1.0%	9.0	37.3	13.7	41.4	13.4	40.2
1.4%	10.5	44.1	16.2	49.0	15.9	47.6
1.8%	12.0	45.4	18.4	50.4	18.0	50.4
2.2%	13.3	42.8	19.4	47.5	19.5	47.5
2.6%	14.4	40.7	18.5	45.1	18.5	45.1
3.0%	15.5	39.0	17.7	43.2	17.8	43.2
3.5%	16.7	37.2	16.9	41.3	17.0	41.3
4.0%	17.9	35.7	16.2	39.7	16.3	29.7
4.5%	19.0	34.5	15.7	38.3	15.7	38.3
5.0%	19.9	33.4	15.2	37.1	15.2	37.1

Note: all flows are in cfs (cubic feet per second)

Drainage calculations for Lorson Boulevard can be found in Project CDR 18-006 and are not included in this report.

Design Point 1

Design Point 1 is located at a low point in Alsea Drive (east side)

(5-year storm)

Tributary Basins: C1.1-C1.2 **Inlet/MH Number:** Inlet DP-1
Upstream flowby: 0cfs **Total Street Flow:** 9.1cfs

Flow Intercepted: 9.1cfs **Flow Bypassed:** 0
Inlet Size: 15’ type R, sump

Street Capacity: Street slope = 1.5%, capacity = 10.9cfs, capacity okay

(100-year storm)

Tributary Basins: C1.1-C1.2 **Inlet/MH Number:** Inlet DP-1
Upstream flowby: 0cfs **Total Street Flow:** 20.2cfs

Flow Intercepted: 20.2cfs **Flow Bypassed:** 0
Inlet Size: 15’ type R, sump

Street Capacity: Street slope = 1.5%, capacity = 44.4cfs (half street) is okay

Design Point 2

Design Point 2 is located on Alsea Drive and is located north of Design Point 3. This design point was added to verify the street capacity of Alsea Drive on the north side of Inlet DP-3. The total street flow is 5.3cfs and 11.9cfs in the 5/100-year storm events from Basins C1.3 & C1.4. The street capacity of Alsea Drive at 1.7% slope is 11.3cfs (5-yr) and 44.8cfs (100-yr). The street capacity is not exceeded north of Inlet DP-3.

Design Point 3

Design Point 3 is located at a low point in Alsea Drive (west side)

(5-year storm)

Tributary Basins: C1.3-C1.5 **Inlet/MH Number:** Inlet DP-3
Upstream flowby: 0cfs **Total Street Flow:** 5.6cfs

Flow Intercepted: 5.6cfs **Flow Bypassed:** 0
Inlet Size: 10’ type R, sump

Street Capacity: Street slope = 1.5%, capacity = 10.9cfs, capacity okay

(100-year storm)

Tributary Basins: C1.3-C1.5 **Inlet/MH Number:** Inlet DP-3
Upstream flowby: 0cfs **Total Street Flow:** 12.6cfs

Flow Intercepted: 12.6cfs **Flow Bypassed:** 0
Inlet Size: 10’ type R, sump

Street Capacity: Street slope = 1.5%, capacity = 44.4cfs (half street) is okay

Design Point 4

Design Point 4 is the total pipe flow in storm sewer from Alsea Drive to Pond C1-R and is located west of Design Point 3. The total pipe flow is 14.7cfs and 32.7cfs in the 5/100-year storm events. Since there is a low point in Alsea Drive an emergency overflow swale must be constructed from Alsea Drive to Pond C1-R for 32.7cfs. The overflow swale has an 8' bottom, 4:1 side slopes, 1.3% slope, and flows at a 0.69' flow depth.

Design Point 5

Design Point 5 is located on the north side of Castor Drive and is located west of Design Point 6. This design point was added to verify the street capacity of Castor Drive on the north side of the street. The total street flow is 4.1cfs and 9.1cfs in the 5/100-year storm events from Basins C1.7 & C1.8. The street capacity of Castor Drive at 0.65% slope is 7.2cfs (5-yr) and 30.0cfs (100-yr). The street capacity is not exceeded west of Inlet DP-6.

Design Point 6

Design Point 6 is located at a low point in Castor Drive adjacent to Pond C1-R (north side of street)

(5-year storm)

Tributary Basins: C1.6-C1.8	**Inlet/MH Number:** Inlet DP-6
Upstream flowby: 0cfs	**Total Street Flow:** 5.3cfs
Flow Intercepted: 5.3cfs	**Flow Bypassed:** 0
Inlet Size: 10' type R, sump	

Street Capacity: Street slope = 0.65%, capacity = 7.2cfs, capacity okay

(100-year storm)

Tributary Basins: C1.6-C1.8	**Inlet/MH Number:** Inlet DP-6
Upstream flowby: 0cfs	**Total Street Flow:** 11.8cfs
Flow Intercepted: 11.8cfs	**Flow Bypassed:** 0
Inlet Size: 10' type R, sump	

Street Capacity: Street slope = 0.65%, capacity = 30.0cfs (half street) is okay

Design Point 7

Design Point 7 is located on the south side of Castor Drive and is located west of Maidford Drive. This design point was added to verify the street capacity of Castor Drive on the south side of the street. The total street flow is 4.1cfs and 9.1cfs in the 5/100-year storm events from Basins C1.9 - C1.11. The street capacity of Castor Drive at 0.7% slope is 7.5cfs (5-yr) and 31.2cfs (100-yr). The street capacity is not exceeded at this design point.

Design Point 8

Design Point 8 is located on the south side of Castor Drive and is located east of Design Point 10. This design point was added to verify the street capacity of Castor Drive on the south side of the street on the east side of Inlet DP-10. The total street flow is 5.2cfs and 11.6cfs in the 5/100-year storm events from Basins C1.9 - C1.12. The street capacity of Castor Drive at 0.7% slope is 7.5cfs (5-yr) and 31.2cfs (100-yr). The street capacity is not exceeded at this design point.

Design Point 9

Design Point 9 is located on the south side of Castor Drive and is located west of Design Point 10. This design point was added to verify the street capacity of Castor Drive on the south side of the street on the west side of Inlet DP-10. The total street flow is 3.2cfs and 7.0cfs in the 5/100-year storm events from Basins C1.13 - C1.14. The street capacity of Castor Drive at 0.65% slope is 7.2cfs (5-yr) and 30.0cfs (100-yr). The street capacity is not exceeded at this design point.

Design Point 10

Design Point 10 is located at a low point in Castor Drive adjacent to Pond C1-R (south side of street)

(5-year storm)
Tributary Basins: C1.9-C1.14 **Inlet/MH Number:** Inlet DP-10
Upstream flowby: 0cfs **Total Street Flow:** 7.9cfs

Flow Intercepted: 7.9cfs **Flow Bypassed:** 0
Inlet Size: 10’ type R, sump

Street Capacity: Street slope = 0.65%, capacity = 7.2cfs, capacity okay since half flow from east

(100-year storm)
Tributary Basins: C1.9-C1.14 **Inlet/MH Number:** Inlet DP-10
Upstream flowby: 0cfs **Total Street Flow:** 17.6cfs

Flow Intercepted: 17.6cfs **Flow Bypassed:** 0
Inlet Size: 10’ type R, sump

Street Capacity: Street slope = 0.65%, capacity = 30.0cfs (half street) is okay

Design Point 11

Design Point 11 is located at a low point in Maidford Drive.

(5-year storm)
Tributary Basins: C1.15-C1.16 **Inlet/MH Number:** Inlet DP-11
Upstream flowby: 0cfs **Total Street Flow:** 2.5cfs

Flow Intercepted: 2.5cfs **Flow Bypassed:** 0
Inlet Size: 5’ type R, sump

Street Capacity: Street slope = 0.7%, capacity = 7.5cfs, capacity okay

(100-year storm)
Tributary Basins: C1.15-C1.16 **Inlet/MH Number:** Inlet DP-11

Upstream flowby: 0cfs **Total Street Flow:** 5.7cfs

Flow Intercepted: 5.7cfs **Flow Bypassed:** 0
Inlet Size: 5’ type R, sump

Street Capacity: Street slope = 0.7%, capacity = 31.2cfs (half street) is okay

Design Point 12

Design Point 12 is located south of Castor Drive and west of Maidford Drive and Design Point 11. This design point was added to verify flow to Inlet DP-12 from Basin C1.17. The total flow in the backyard swale is 2.9cfs and 6.3cfs in the 5/100-year storm events from Basins C1.17 . A CDOT type D inlet will capture the flow at this design point and convey it via storm sewer to Pond C1-R .

Design Point 13

Design Point 13 is located on the north of Castor Drive and is the total flow in storm sewer entering Pond C1-R from Design Point 11 & 12. The total flow in the storm sewer is 5.4cfs and 12.0cfs in the 5/100-year storm events from Basins C1.15 – C1.17.

Design Point 14

Design Point 14 is located on the north of Castor Drive and is the total flow in storm sewer entering Pond C1-R from Design Point 6 & 10. The total flow in the storm sewer is 13.2cfs and 29.4cfs in the 5/100-year storm events from Basins C1.6 – C1.14.

Design Point 14a

Design Point 14a is located on the south side of Castor Drive and is the total flow from the outlet structure for Pond C1-R. The total outflow is 8.9cfs and 133.8cfs in the 5/100-year storm events from Pond C1-R per the full spectrum EDB worksheets.

Design Point 15

Design Point 15 is located at a low point in Yazoo Drive.

(5-year storm)

Tributary Basins: C5.1	**Inlet/MH Number:** Inlet DP-15
Upstream flowby: 0cfs	**Total Street Flow:** 2.2cfs
Flow Intercepted: 2.2cfs	**Flow Bypassed:** 0
Inlet Size: 5’ type R, sump	

Street Capacity: Street slope = 0.7%, capacity = 7.5cfs, capacity okay

(100-year storm)

Tributary Basins: C5.1	**Inlet/MH Number:** Inlet DP-15
Upstream flowby: 0cfs	**Total Street Flow:** 3.7cfs
Flow Intercepted: 3.7cfs	**Flow Bypassed:** 0
Inlet Size: 5’ type R, sump	

Street Capacity: Street slope = 0.7%, capacity = 31.2cfs (half street) is okay

Design Point 15a

Design Point 15a is located south side of Yazoo Drive and is the total flow from the outlet structure for Pond CR3. The total outflow is 0.04cfs and 1.5cfs in the 5/100-year storm events from Pond CR3 per the full spectrum EDB/SFB worksheets.

Design Point 16

Design Point 16 is located south of Castor Drive and west of Winnicut Drive. This design point was added to verify flow to Design Point 16 from Basin D1.1 in a swale. The total flow in the backyard swale is 2.1cfs and 4.6cfs in the 5/100-year storm events from Basins D1.1. An 18” storm sewer and end section will capture the flow at this design point and convey it via south in storm sewer to Design Point 24 .

Design Point 17

Design Point 17 is located on the north side of Castor Drive and is west of Tensas Drive. This design point was added to verify the street capacity of Castor Drive. The total street flow is 2.8cfs and 5.3cfs in the 5/100-year storm events from Basins D1.2 & D1.3. The street capacity of Castor Drive at 0.85% slope is 8cfs (5-yr) and 33.4cfs (100-yr). The street capacity is not exceeded.

Design Point 18

Design Point 18 is located on the west side of Castor Drive and is southwest of Design Point 17. This design point was added to verify the street capacity of Castor Drive. The total street flow is 4.2cfs and 8.6cfs in the 5/100-year storm events from Basins D1.2 - D1.4. The street capacity of Castor Drive at 0.8% slope is 8.2cfs (5-yr) and 34.4cfs (100-yr). The street capacity is not exceeded.

Design Point 19

Design Point 19 is located on the south end of Castor Drive in the cul-de-sac. This design point was added to verify the street capacity of Castor Drive in the cul-de-sac from the west. The total street flow is 4.9cfs and 10.3cfs in the 5/100-year storm events from Basins D1.2 - D1.5. The street capacity of Castor Drive at 0.8% slope is 8cfs (5-yr) and 33.4cfs (100-yr). The street capacity is not exceeded.

Design Point 20

Design Point 20 is located on the north side of Winnicut Drive at Castor Drive south of Design Point 16. This design point was added to verify the street capacity of Castor/Winnicut Drive. The total street flow is 4.3cfs and 9.4cfs in the 5/100-year storm events from Basins D1.6 - D1.7. The street capacity at 0.8% slope is 8cfs (5-yr) and 33.4cfs (100-yr). The street capacity is not exceeded.

Design Point 21

Design Point 21 is located on the south side of Winnicut Drive at Castor Drive south of Design Point 20. This design point was added to verify the street capacity of Castor Drive. The total street flow is 5.9cfs and 12.9cfs in the 5/100-year storm events from Basins D1.6 - D1.8. The street capacity at 0.8% slope is 8cfs (5-yr) and 33.4cfs (100-yr). The street capacity is not exceeded.

Design Point 22

Design Point 22 is located on the south end of Castor Drive in the cul-de-sac. This design point was added to verify the street capacity of Castor Drive in the cul-de-sac from the east. The total street flow is 6.0cfs and 13.3cfs in the 5/100-year storm events from Basins D1.6 - D1.9. The street capacity of Castor Drive at 0.8% slope is 8cfs (5-yr) and 33.4cfs (100-yr). The street capacity is not exceeded.

Design Point 23

Design Point 23 is located at a low point in Castor Drive in the cul-de-sac at the very south end from Design Points 19 and 22.

(5-year storm)

Tributary Basins: D1.2-D1.9
Inlet/MH Number: Inlet DP-23
Upstream flowby: 0cfs
Total Street Flow: 10.5cfs

Flow Intercepted: 10.5cfs
Flow Bypassed: 0
Inlet Size: 15’ type R, sump

Street Capacity: Street slope = 0.8%, capacity = 8.0cfs, capacity okay since half is from each side

(100-year storm)

Tributary Basins: D1.2-D1.9
Inlet/MH Number: Inlet DP-23
Upstream flowby: 0cfs
Total Street Flow: 22.4cfs

Flow Intercepted: 22.4cfs
Flow Bypassed: 0
Inlet Size: 15’ type R, sump

Street Capacity: Street slope = 0.8%, capacity = 33.4cfs (half street) is okay

Design Point 24

Design Point 24 is located south of Castor Drive and Design Point 23. This design point was added to calculate the total flow from the “D1” basins in the storm sewer entering Pond CR2. The total flow in the storm sewer is 12.1cfs and 26.1cfs in the 5/100-year storm events from the Basins D1 basins. A 24” storm sewer at this design point will convey flow south in this storm sewer to Pond CR2.

Design Point 24a

Design Point 24a is located south of the Castor Drive cul-de-sac and is the total flow from the outlet structure for Pond CR2. The total outflow is 0.2cfs and 10.2cfs in the 5/100-year storm events from Pond CR2 per the full spectrum EDB worksheets.

6.0 DETENTION AND WATER QUALITY PONDS

Detention and Storm Water Quality for Creekside at Lorson Ranch Filing No. 1 is required per El Paso County criteria. We have implemented the Full Spectrum approach for detention for Creekside at Lorson Ranch Filing No. 1 per the Denver Urban Drainage Districts specifications. There is one existing detention pond, one proposed detention pond, and one sand filter basin with full spectrum detention for this project site. Nearly all runoff from this site will flow to ponds and will incorporate storm water quality features prior to discharge into the East Tributary. There are some area comprising of backyard runoff that will flow directly to Jimmy Camp Creek or the Etrib which will require a deviation for Water Quality Grass Buffer in the final plat process.

Full Spectrum Pond Construction Requirements

Design calculations for full spectrum ponds will include a 10' wide gravel access road on a 15' wide bench at a maximum 10% slope to the pond bottom. The final design of the full spectrum ponds consists of an outlet structure, storm sewer outfall to the East Tributary, concrete low flow channels (in new ponds), sediment forebays, and overflow weirs to the East Tributary. Soil borings, embankment, slope, and compaction requirements for detention ponds can be found in the geotechnical report for the Creekside prepared by RMG.

Detention Pond C1-R (Full Spectrum Design)

Pond C1-R formerly known as Pond C1 (Lorson Ranch MDDP1, Allegiant at Lorson Ranch), is an existing pond constructed in 2010 to serve residential subdivisions north of Lorson Boulevard. Pond C1-R included a traditional outlet structure, forebays, low flow channels, and was sized to accommodate residential areas north of Lorson Boulevard and most of the runoff from Creekside at Lorson Ranch Filing No. 1. Since full spectrum detention is now required on new developments we are proposing to remove the old outlet structure and construct a new full spectrum outlet structure to meet current detention requirements. The existing forebays, low flow channels will remain and new forebays/low flow channels will be constructed to accommodate additional storm sewer outfalls to the pond. Based on the overall tributary area to Pond C1-R and the existing as-built pond volumes it appears that the pond was built large enough in 2010 and does not need additional volume to serve the new drainage areas in Creekside. Pond C1-R is designed using the UDCF Full Spectrum spreadsheets. The outlet structure is a standard 22' long x 4' wide full spectrum sloped outlet structure to match pre-developed rates. The full spectrum print outs are in the appendix of this report. See map in appendix for watershed areas.

Add discussion that the emergency overflow over Castor Street meets FSD pond criteria for outfall armoring and crown depth per DCM Vol 1. Figure 13-12a. & Sec. 13-5.12.

- Watershed Ares: 117.5 acres
- Watershed Imperviousness: 55%
- Hydrologic Soils Group C (80%) and B (20%)
- Zone 1 WQCV: 1.989ac-ft, WSEL: 5686.87, 1.0cfs
- Zone 2 EURV: 5.664ac-ft, WSEL: 5688.67, Top EURV wall set at 5689.23, 22'x4' outlet with 4:1 slope, 4.9cfs
- (5-yr): 7.374ac-ft, WSEL: 5689.41, 8.9cfs
- Zone 3 (100-yr): 11.860ac-ft, WSEL: 5691.22, 133.8cfs
- Pipe Outlet: 54" RCP at 0.3% with restrictor plate 44" up.
- Overflow Spillway: overtops roadway, elevation=5693.60
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5683.80

Detention Pond CR2 (Full Spectrum Design)
This is an on-site permanent full spectrum extended detention pond that includes water quality and discharges directly into the East Tributary. Pond CR2 is designed using the UDCF Full Spectrum spreadsheets. The outlet structure is a standard 4'x4' full spectrum sloped outlet structure and the overflow spillway is a weir set above the outlet structure designed by the full spectrum spreadsheets to match pre-developed rates. The full spectrum print outs are in the appendix of this report. See map in appendix for watershed areas.

- Watershed Ares: 9.5 acres
- Watershed Imperviousness: 52%
- Hydrologic Soils Group B
- Forebay: 0.004ac-ft, 18" depth
- Zone 1 WQCV: 0.154ac-ft, WSEL: 5683.25, 0.1cfs
- Zone 2 EURV: 0.499ac-ft, WSEL: 5684.65, Top EURV wall set at 5684.88, 4'x4' outlet with 4:1 slope, 0.2cfs
- (5-yr): 0.552ac-ft, WSEL: 5684.83, 0.2cfs
- Zone 3 (100-yr): 0.902ac-ft, WSEL: 5685.89, 10.2cfs
- Pipe Outlet: 18" RCP at 1.0% with restrictor plate up 10"
- Overflow Spillway: 10' wide bottom, elevation=5687.00, 4:1 side slopes, flow depth=0.71'
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5681.00

Detention Pond CR3 (Full Spectrum Design, Sand Filter Basin)
This is an on-site permanent full spectrum sand filter basin pond that includes water quality, full spectrum detention, and discharges directly into the East Tributary. Pond CR3 is designed using the UDCF Full Spectrum spreadsheets. Water quality is provided by a Sand Filter Basin and full spectrum detention is provided by a CDOT Type C drainage structure modified to meet full spectrum requirements. The primary overflow structure is a CDOT Type D drainage structure connected to the full spectrum structure. The primary overflow structure will collect the incoming undetained developed flows of 5.1cfs at a depth of 0.4' deep and a top elevation of 5688.10 and convey it to the East Tributary via an 18" storm sewer pipe. The secondary overflow structure is a trapezoidal swale set at elevation 5688.50 and a top elevation of 5689.00. The full spectrum outlet structure and spreadsheets are designed to match pre-developed rates. The full spectrum print outs are in the appendix of this report. See map in appendix for watershed areas.

- Watershed Ares: 1.6 acres
- Watershed Imperviousness: 50%
- Hydrologic Soils Group B
- Forebay: 0.00165ac-ft
- Sand Filter Area: 492sf, 11/16" orifice for underdrain restrictor plate
- Zone 1 WQCV: 0.018ac-ft, WSEL: 5685.08, 0.02cfs
- Zone 2 EURV: 0.08ac-ft, WSEL: 5686.79, Top EURV wall set at 5687.30, 3'x3' CDOT Type C outlet, flat top, 0.04cfs
- EURV Orifice = 4.8" orifice, 2.3' below sand filter (5684.00)
- (5-yr): 0.088ac-ft, WSEL: 5686.95, 0.04cfs
- Zone 3 (100-yr): 0.167ac-ft, WSEL: 5688.17, 1.5cfs
- Pipe Outlet: 18" RCP at 1.56%
- Overflow Spillway: 10' wide bottom, elevation=5687.00, 4:1 side slopes, flow depth=0.71'
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5684.00

Water Quality Design

Water quality will be provided by two permanent extended detention basins (Pond C1-R, CR2) and one Sand Filter Basin (Pond CR3) for 98.6% of the 83.085acre site. Approximately 3.07acres (3.7% of the total 83.085-acre preliminary plan area) consists of backyards that drain directly to the East Tributary over grass buffers. Final platting of these areas may need to include a deviation from county criteria or a grass buffer bmp which will be determined at the final drainage report stage.

7.0 DRAINAGE AND BRIDGE FEES

Creekside at Lorson Ranch Filing No. 1 is located within the Jimmy Camp Creek drainage basin which is currently a fee basin in El Paso County. Current El Paso County regulations require drainage and bridge fees to be paid for platting of land as part of the plat recordation process. Lorson Ranch Metro District will be constructing the major drainage infrastructure as part of the district improvements.

Lorson Ranch Metro District will compile and submit to the county on a yearly basis the Drainage and bridge fees for the approved plats, and shall show all credits they have received for the same yearly time frame.

Creekside at Lorson Ranch Filing No. 1 contains approximately 83.085 acres. The 83.085 acres will be assessed Drainage, Bridge and Surety fees. The 2018 drainage fees are $17,197 per impervious acre, bridge fees are $804 per impervious acre, and Drainage Surety fees are $7,285 per impervious acre per Resolution 17-348. The drainage and bridge fees are calculated when the final plat is submitted. The fees are due at plat recordation.

Table 7.1: Public Drainage Facility Costs (non-reimbursable)

Item	Quantity	Unit	Unit Cost	Item Total
Rip Rap	200	CY	$50/CY	$10,000
Manholes	1	EA	$3000/EA	$3,000
18” Storm	1226	LF	$35	$42,910
24” Storm	286	LF	$40	$11,440
18” FES	1	EA	$200	$200
Inlets	8	EA	$3,000	$24,000
			Subtotal	$91,550
			Eng/Cont 15%)	$13,750
			Total Est. Cost	$105,300

Table 7.2: Lorson Ranch Metro District Drainage Facility Costs (non-reimbursable)

Item	Quantity	Unit	Unit Cost	Item Total
Full Spectrum Ponds and Outlet	2.5	EA	$70,000	$175,000
			Subtotal	$175,000
			Eng/Cont (15%)	$26,250
			Total Est. Cost	$201,250

Table 7.3: Lorson Ranch Metro District Drainage Facility Costs (Potential Reimbursable)

Item	Quantity	Unit	Unit Cost	Item Total
E. Tributary Channel Improvements-Kiowa	1	LS	$800,000	$800,000
			Subtotal	$800,000
			Total Est. Cost	$800,000

8.0 CONCLUSIONS

This drainage report has been prepared in accordance with the City of Colorado Springs/El Paso County Drainage Criteria Manual. The proposed development and drainage infrastructure will not cause adverse impacts to adjacent properties or properties located downstream. Several key aspects of the development discussed above are summarized as follows:

- Developed runoff will be conveyed via curb/gutter and storm sewer facilities
- The East Tributary of Jimmy Camp Creek will be reconstructed within this study area
- Detention and water quality for this preliminary plan area will be provided in two permanent ponds and one sand filter basin.

9.0 REFERENCES

1. City of Colorado Springs/El Paso County Drainage Criteria Manual DCM, dated November, 1991
2. Soil Survey of El Paso County Area, Colorado by USDA, SCS
3. Jimmy Camp Creek Drainage Basin Planning Study, Dated March 9, 2015, by Kiowa Engineering Corporation
4. City of Colorado Springs “Drainage Criteria Manual, Volume 2
5. El Paso County “Engineering Criteria Manual”
6. Lorson Ranch MDDP1, October 26, 2006 by Pentacor Engineering.
7. Final construction plans “East Fork Jimmy Camp Creek Channel Design”, Dated 2018, by Kiowa Engineering Corporation
8. El Paso County Resolution #15-042, El Paso County adoption of Chapter 6 and Section 3.2.1 of the City of Colorado Springs Drainage Criteria Manual dated May, 2014.

APPENDIX A – VICINTIY MAP, SOILS MAP, FEMA MAP

VICINITY MAP

NO SCALE

CORE
ENGINEERING GROUP

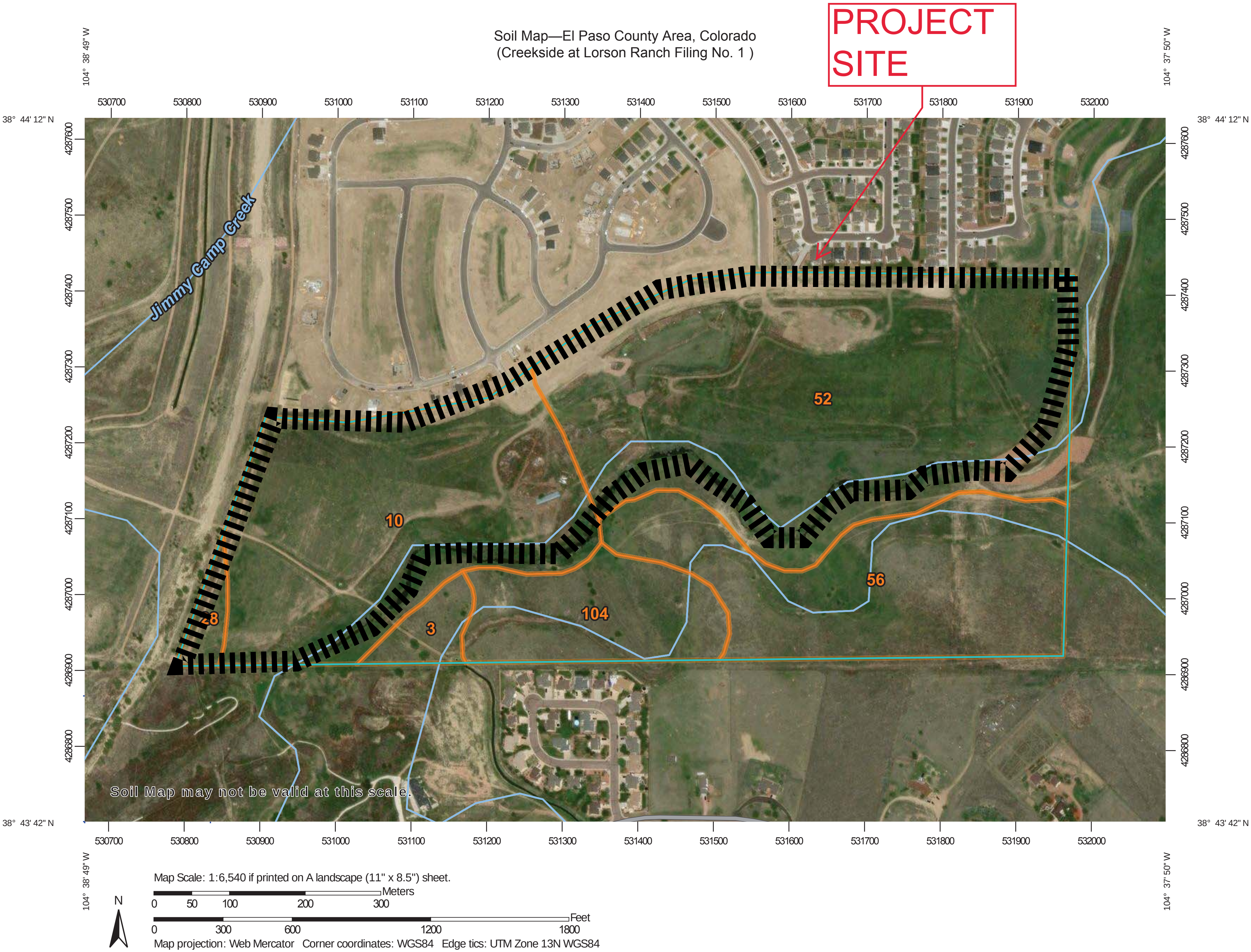
15004 1ST AVE. S.
BURNSVILLE, MN 55306
PH: 719.570.1100

CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

CREEKSIDE AT LORSON RANCH FILING NO. 1
VICINITY MAP

SCALE:	DATE:	FIGURE NO.
NTS	AUGUST, 2018	--

Soil Map—El Paso County Area, Colorado
(Creekside at Lorson Ranch Filing No. 1)



Map Scale: 1:6,540 if printed on A landscape (11" x 8.5") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N WGS84

MAP LEGEND

Area of Interest (AOI)

- ▭ Area of Interest (AOI)

Soils

- ▭ Soil Map Unit Polygons
- ~ Soil Map Unit Lines
- ■ Soil Map Unit Points

Special Point Features

- ◎ Blowout
- ⊠ Borrow Pit
- ✱ Clay Spot
- ◇ Closed Depression
- ✕ Gravel Pit
- ∴ Gravelly Spot
- ● Landfill
- Λ Lava Flow
- ⊥ Marsh or swamp
- ⚒ Mine or Quarry
- ◎ Miscellaneous Water
- ○ Perennial Water
- ∨ Rock Outcrop
- + Saline Spot
- ∷ Sandy Spot
- ⊖ Severely Eroded Spot
- ◇ Sinkhole
- ≫ Slide or Slip
- ∅ Sodic Spot
- ≡ Spoil Area
- ● Stony Spot
- ● Very Stony Spot
- ♆ Wet Spot
- △ Other
- ⋯ Special Line Features

Water Features

- ~ Streams and Canals

Transportation

- ┼┼┼ Rails
- ~ Interstate Highways
- ~ US Routes
- ~ Major Roads
- ~ Local Roads

Background

- ■ Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 15, Oct 10, 2017

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

Date(s) aerial images were photographed: Nov 7, 2015—Mar 9, 2017

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
3	Ascalon sandy loam, 3 to 9 percent slopes	2.4	2.0%
10	Blendon sandy loam, 0 to 3 percent slopes	31.3	26.0%
28	Ellicott loamy coarse sand, 0 to 5 percent slopes	1.5	1.2%
52	Manzanst clay loam, 0 to 3 percent slopes	51.4	42.7%
56	Nelson-Tassel fine sandy loams, 3 to 18 percent slopes	23.4	19.4%
104	Vona sandy loam, warm, 0 to 3 percent slopes	10.4	8.7%
Totals for Area of Interest		**120.5**	**100.0%**

Provide soil data for 10 and 52.

Legend

▓	1% annual chance (100-Year) Floodplain
▨	1% annual chance (100-Year) Floodway
░	0.2% annual chance (500-Year) Floodplain

APPROXIMATE SCALE IN FEET
500 0 500

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

EL PASO COUNTY,
COLORADO AND
INCORPORATED AREAS

REVISED TO
REFLECT LOMR
EFFECTIVE: January 29, 2015

PANEL 957 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)

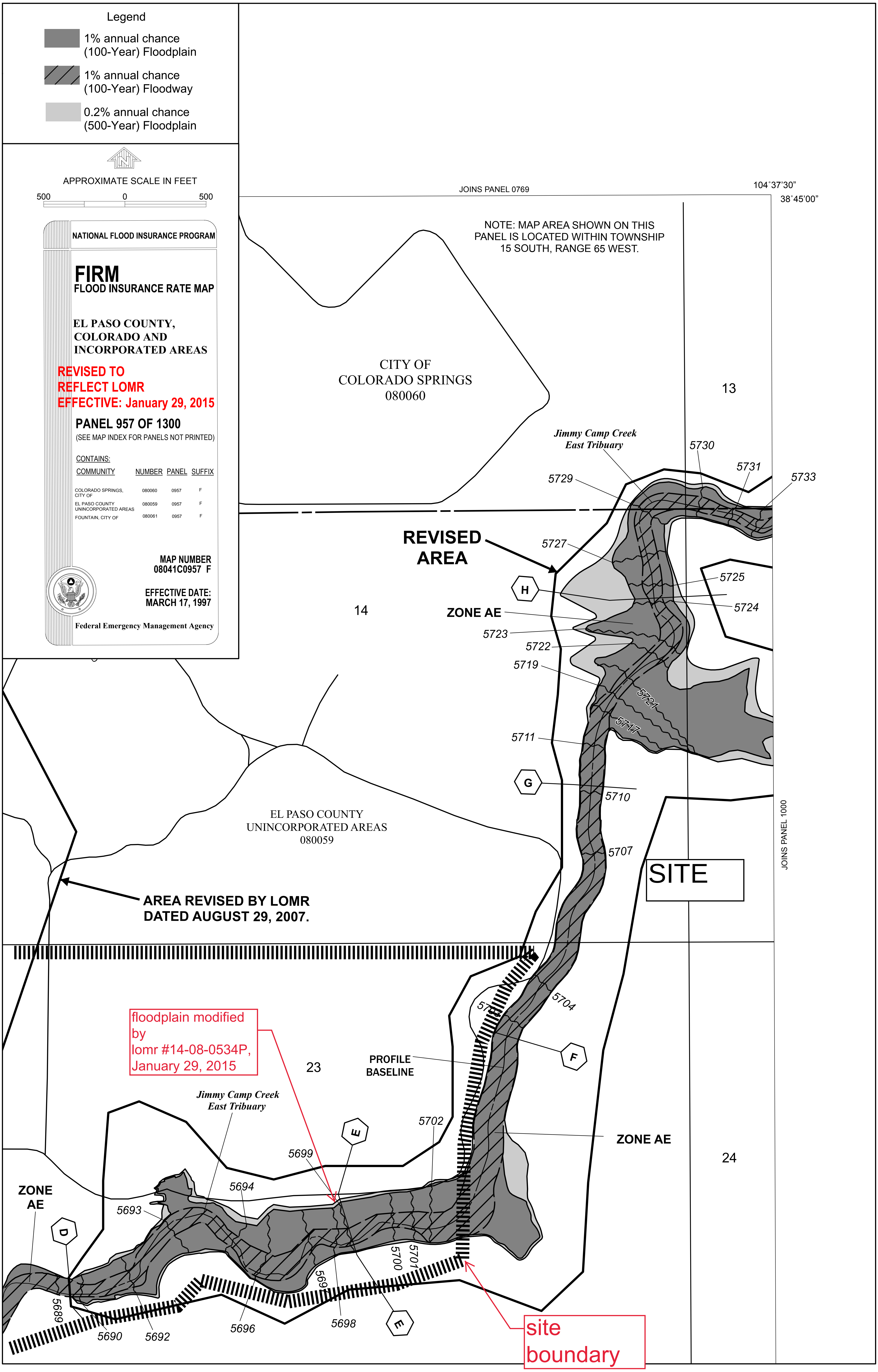
CONTAINS:

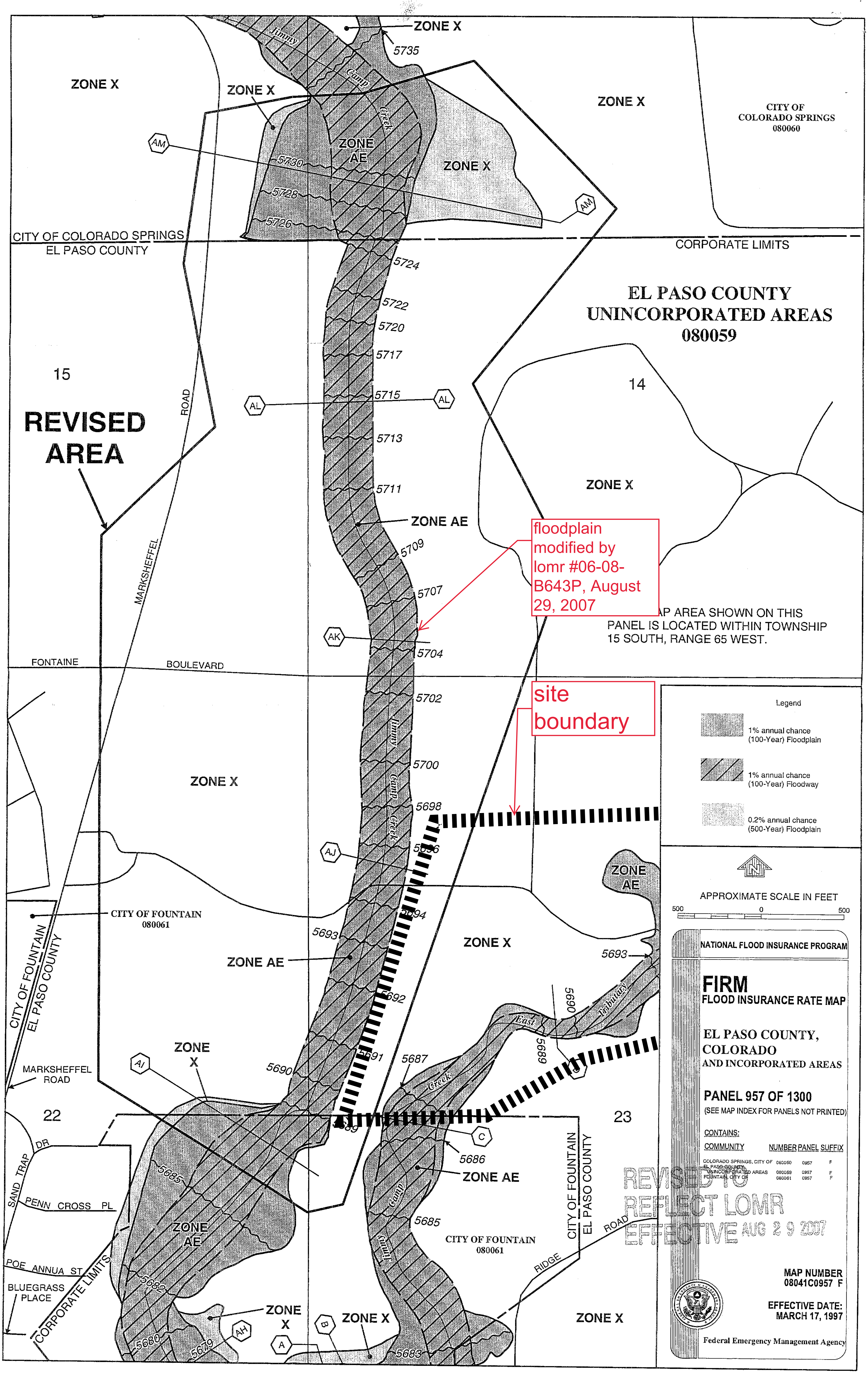
COMMUNITY	NUMBER	PANEL	SUFFIX
COLORADO SPRINGS, CITY OF	080060	0957	F
EL PASO COUNTY UNINCORPORATED AREAS	080059	0957	F
FOUNTAIN, CITY OF	080061	0957	F

MAP NUMBER
08041C0957 F

EFFECTIVE DATE:
MARCH 17, 1997

Federal Emergency Management Agency





floodplain modified by lomr #06-08-B643P, August 29, 2007

site boundary

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

EL PASO COUNTY, COLORADO AND INCORPORATED AREAS

PANEL 957 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
COLORADO SPRINGS, CITY OF	080060	0957	F
EL PASO COUNTY, UNINCORPORATED AREAS	080059	0957	F
FOUNTAIN, CITY OF	080061	0957	F

REVISED TO REFLECT LOMR EFFECTIVE AUG 2 9 2007

MAP NUMBER
08041C0957 F

EFFECTIVE DATE:
MARCH 17, 1997

Federal Emergency Management Agency

APPENDIX B – HYDROLOGY CALCULATIONS

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: July 17, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **5 - Year Event, Existing Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
EX-B			35.50	0.16	19.9	5.68	3.09	17.6													
EX-C1			10.32	0.15	16.0	1.55	3.42	5.3													
EX-D			29.29	0.09	18.0	2.64	3.25	8.6													

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

Calculated By: Leonard Beasley
Date: July 17, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **100 - Year Event, Existing Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
EX-B			35.50	0.51	19.9	18.11	5.19	94.0													
EX-C1			10.32	0.50	16.0	5.16	5.75	29.7													
EX-D			29.29	0.36	18.0	10.54	5.45	57.5													

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: June 29, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
C1.1			2.27	0.49	16.46	1.11	3.38	3.8					1.0%	3.8							
C1.2			3.35	0.49	17.36	1.64	3.30	5.4					0.9%	5.4							
(C1.1&C1.2)	**1**		5.62						17.4	2.75	3.30	9.1	L.P.	9.1	9.1	1.0%	24"	35'	5.3	0.1	
C1.3			0.90	0.49	10.47	0.44	4.06	1.8					1.0%	1.8							
C1.4			2.41	0.49	12.59	1.18	3.78	4.5					1.1%	4.5							
(C1.3&C1.4)	**2**		3.31						17.5	1.62	3.29	5.3	L.P.	5.3							
C1.5			0.19	0.49	6.56	0.09	4.76	0.4					1.3%	0.4							
(C1.3-C1.5)	**3**		3.50						17.5	1.72	3.29	5.6	L.P.	5.6							
(C1.1-C1.5)	**4**		9.12						17.5	4.47	3.29	14.7	L.P.	14.7	14.7	2.3%	24"	132'	6.5	0.3	
C1.6			0.73	0.49	9.81	0.36	4.16	1.5					0.8%	1.5							
C1.7			1.92	0.45	14.53	0.86	3.57	3.1					0.6%	3.1							
C1.8			0.77	0.47	8.47	0.36	4.38	1.6					1.0%	1.6							
(C1.7&C1.8)	**5**		2.69						16.6	1.23	3.37	4.1	L.P.	4.1							
(C1.6-C1.8)	**6**		3.42						16.6	1.58	3.37	5.3	L.P.	5.3							
C1.9			2.10	0.49	16.04	1.03	3.42	3.5					0.8%	3.5							
C1.10			0.18	0.49	9.30	0.09	4.24	0.4					0.8%	0.4							
C1.11			0.17	0.49	6.72	0.08	4.73	0.4													

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: June 29, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
(C1.9-C1.11)	**7**		2.45						16.0	1.20	3.42	4.1	0.8%	4.1							
C1.12			1.05	0.49	14.53	0.51	3.57	1.8					0.9%	1.8							
(C1.9-C1.12)	**8**		3.50						20.8	1.72	3.03	5.2	L.P.	5.2							
C1.13			0.71	0.45	9.25	0.32	4.25	1.4					0.7%	1.4							
C1.14			1.27	0.46	11.74	0.58	3.89	2.3					0.7%	2.3							
(C1.13&C1.14)	**9**		1.98						15.3	0.90	3.49	3.2	L.P.	3.2							
(C1.9-C1.14)	**10**		5.48						20.8	2.62	3.03	7.9	L.P.	7.9							
C1.15			0.80	0.49	10.96	0.39	3.99	1.6					1.0%	1.6							
C1.16			0.50	0.49	7.61	0.25	4.54	1.1					1.3%	1.1							
(C1.15&C1.16)	**11**		1.30						11.0	0.64	3.99	2.5	L.P.	2.5							
C1.17	**12**		1.38	0.49	9.44	0.68	4.22	2.9													
(C1.15-C1.17)			2.68						11.4	1.31	3.94	5.2			5.2	1.6%	18"	185'	2.9	1.1	
C1.18			5.81	0.27	13.91	1.57	3.63	5.7													
C2			5.44	0.49	8.54	2.67	4.37	11.6													
C3			0.69	0.49	8.94	0.34	4.30	1.5													
C4			1.84	0.47	6.48	0.86	4.78	4.1													

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: June 29, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
C5.1	**15**		1.14	0.45	9.02	0.51	4.28	2.2					L.P.	1.3	1.3	1.0%	18"	34'	1.2	0.0	
C5.2			0.72	0.45	9.85	0.32	4.15	1.3													
C5			1.86						9.9	0.84	4.15	3.5									
C6			0.92	0.45	9.85	0.41	4.15	1.7													
D1.1	**16**		1.21	0.45	12.00	0.54	3.86	2.1							2.1	1.0%	18"	385'	1.2	5.3	
D1.2			0.55	0.90	8.36	0.50	4.40	2.2													
D1.3			0.42	0.45	10.41	0.19	4.07	0.8													
(D1.2&D1.3)	**17**		0.97						10.4	0.68	4.07	2.8	1.1%	2.8							
D1.4			1.13	0.45	9.53	0.51	4.20	2.1					1.3%	2.1							
(D1.2-D1.4)	**18**		2.10						14.9	1.19	3.53	4.2	1.0%	4.2							
D1.5			0.87	0.45	11.63	0.39	3.90	1.5					0.9%	1.5							
(D1.2-D1.5)	**19**		2.97						19.6	1.58	3.12	4.9	L.P.	4.9	12.1	3.0%	24"	50'	3.9	0.2	
D1.6			1.26	0.45	12.39	0.57	3.81	2.2					1.1%	2.2							
D1.7			1.39	0.45	14.42	0.63	3.58	2.2					0.7%	2.2							
(D1.6&D1.7)	**20**		2.65						14.4	1.19	3.58	4.3	0.7%	4.3							
D1.8			1.05	0.45	14.94	0.47	3.53	1.7					0.8%	1.7							
(D1.6-D1.8)	**21**		3.70						14.9	1.67	3.53	5.9									

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ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: June 29, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
D1.9			0.24	0.45	6.68	0.11	4.73	0.5					1.1%	0.5							
(D1.6-D1.9)	**22**		3.94						16.3	1.77	3.40	6.0	L.P.	6.0	12.1	3.0%	24"	50'	3.9	0.2	
(D1.2-D1.9)	**23**		6.91						19.6	3.36	3.12	10.5	L.P.	12.1	12.1	3.0%	24"	50'	3.9	0.2	
D1	**24**		8.12						19.7	3.90	3.11	12.1									
D2			1.16	0.45	7.68	0.52	4.53	2.4													
D3			0.79	0.16	10.79	0.13	4.02	0.5													
D4			1.68	0.45	6.38	0.76	4.80	3.6													
D5			1.30	0.22	11.76	0.29	3.89	1.1													
D6			0.23	0.16	10.56	0.04	4.05	0.1													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: July 10, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **100 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	∑ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
C1.1			2.27	0.65	16.46	1.48	5.68	8.4					1.0%	8.4							
C1.2			3.35	0.65	17.36	2.18	5.54	12.1					0.9%	12.1							
(C1.1&C1.2)	**1**		5.62						17.4	3.65	5.54	20.2	L.P.	20.2	20.2	1.0%	24"	35'	7.5	0.1	
C1.3			0.90	0.65	10.47	0.59	6.82	4.0					1.0%	4.0							
C1.4			2.41	0.65	12.59	1.57	6.35	10.0					1.1%	10.0							
(C1.3&C1.4)	**2**		3.31						17.5	2.15	5.52	11.9	L.P.	11.9							
C1.5			0.19	0.65	6.56	0.12	7.99	1.0					1.3%	1.0							
(C1.3-C1.5)	**3**		3.50						17.5	2.28	5.52	12.6	L.P.	12.6							
(C1.1-C1.5)	**4**		9.12						17.5	5.93	5.52	32.7	L.P.	32.7	32.7	2.3%	24"	132'	10.4	0.2	
C1.6			0.73	0.65	9.81	0.47	6.98	3.3					0.8%	3.3							
C1.7			1.92	0.59	14.53	1.13	5.99	6.8					0.6%	6.8							
C1.8			0.77	0.62	8.47	0.48	7.35	3.5					1.0%	3.5							
(C1.7&C1.8)	**5**		2.69						16.6	1.61	5.65	9.1	L.P.	9.1							
(C1.6-C1.8)	**6**		3.42						16.6	2.08	5.65	11.8	L.P.	11.8							
C1.9			2.10	0.65	16.04	1.37	5.74	7.8					0.8%	7.8							
C1.10			0.18	0.65	9.30	0.12	7.12	0.8					0.8%	0.9							
C1.11			0.17	0.65	6.72	0.11	7.93	0.9													

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: July 10, 2018
Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **100 - Year Event, Proposed Conditions**

		Direct Runoff							Total Runoff				Street		Pipe			Travel Time			
Street or Basin	Design Point	Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	Remarks
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
(C1.9-C1.11)	**7**		2.45						16.0	1.59	5.74	9.1	0.8%	9.1							
C1.12			1.05	0.65	14.53	0.68	5.99	4.1					0.9%	4.1							
(C1.9-C1.12)	**8**		3.50						20.8	2.28	5.08	11.6	L.P.	11.6							
C1.13			0.71	0.59	9.25	0.42	7.13	3.0					0.7%	3.0							
C1.14			1.27	0.61	11.74	0.77	6.53	5.1					0.7%	5.1							
(C1.13&C1.14)	**9**		1.98						15.3	1.19	5.86	7.0	L.P.	7.0							
(C1.9-C1.14)	**10**		5.48						20.8	3.47	5.08	17.6	L.P.	16.7							
C1.15			0.80	0.65	10.96	0.52	6.70	3.5					1.0%	3.5							
C1.16			0.50	0.65	7.61	0.33	7.62	2.5					1.3%	2.5							
(C1.15&C1.16)	**11**		1.30						11.0	0.85	6.70	5.7	L.P.	5.7							
C1.17	**12**		1.38	0.65	9.44	0.90	7.08	6.3													
(C1.15-C1.17)			2.68						11.4	1.74	6.61	11.5			11.5	1.6%	18"	185'	6.5	0.5	
C1.18			5.81	0.55	13.91	3.20	6.10	19.5													
C2			5.44	0.65	8.54	3.54	7.33	25.9													
C3			0.69	0.65	8.94	0.45	7.21	3.2													
C4			1.84	0.62	6.48	1.14	8.03	9.2													

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

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Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **100 - Year Event, Proposed Conditions**

		Direct Runoff							Total Runoff				Street		Pipe			Travel Time			
Street or Basin	Design Point	Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	Remarks
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
C5.1	**15**		1.14	0.45	9.02	0.51	7.19	3.7					L.P.	2.2	2.2	1.0%	18"	34'	2.7	0.0	
C5.2			0.72	0.45	9.85	0.32	6.97	2.3													
C5			1.86						9.0	0.84	7.19	6.0									
C6			0.92	0.59	9.85	0.54	6.97	3.8													
D1.1	**16**		1.21	0.59	12.00	0.71	6.47	4.6							4.6	1.0%	18"	385'	2.6	2.5	
D1.2			0.55	0.96	8.36	0.53	7.38	3.9													
D1.3			0.42	0.59	10.41	0.25	6.83	1.7													
(D1.2&D1.3)	**17**		0.97						10.4	0.78	6.83	5.3	1.1%	5.3							
D1.4			1.13	0.59	9.53	0.67	7.05	4.7					1.3%	4.7							
(D1.2-D1.4)	**18**		2.10						14.9	1.44	5.93	8.6	1.0%	8.6							
D1.5			0.87	0.59	11.63	0.51	6.55	3.4					0.9%	3.4							
(D1.2-D1.5)	**19**		2.97						19.6	1.96	5.24	10.3	L.P.	10.3	26.1	3.0%	24"	50'	8.3	0.1	
D1.6			1.26	0.59	12.39	0.74	6.39	4.8					1.1%	4.8							
D1.7			1.39	0.59	14.42	0.82	6.01	4.9					0.7%	4.9							
(D1.6&D1.7)	**20**		2.65						14.4	1.56	6.01	9.4	0.7%	9.4							
D1.8			1.05	0.59	14.94	0.62	5.92	3.7					0.8%	3.7							
(D1.6-D1.8)	**21**		3.70						14.9	2.18	5.92	12.9									

CORE
ENGINEERING GROUP

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

Calculated By: Leonard Beasley
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Checked By: Leonard Beasley

Job No: 100.045
Project: Creekside Filing No. 1
Design Storm: **100 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	∑ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
D1.9			0.24	0.59	6.68	0.14	7.95	1.1					1.1%	1.1							
(D1.6-D1.9)	**22**		3.94						16.3	2.32	5.71	13.3	L.P.	13.3	26.1	3.0%	24"	50'	8.3	0.1	
(D1.2-D1.9)	**23**		6.91						19.6	4.28	5.24	22.4	L.P.	26.1	26.1	3.0%	24"	50'	8.3	0.1	
D1	**24**		8.12						19.7	4.99	5.22	26.1									
D2			1.16	0.59	7.68	0.68	7.60	5.2													
D3			0.79	0.41	10.79	0.32	6.74	2.2													
D4			1.68	0.59	6.38	0.99	8.07	8.0													
D5			1.30	0.45	11.76	0.59	6.52	3.8													
D6			0.23	0.41	10.56	0.09	6.80	0.6													

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Job No: 100.045
Project: Creekside Filing No. 1

Sub-Basin Data				Initial Overland Time (t_i)				Travel Time (t_t)					t_c Check (urbanized Basins)		Final t_c
BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc tc=(L/180)+10 minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
C1.1	0.49	2.27	20	86.00	2.10%	0.18	8.00	1076.0	1.05%	2.05	8.75	16.75	1162.00	16.46	16.46
C1.2	0.49	3.35	20	59.00	1.90%	0.14	6.84	1265.0	0.94%	1.94	10.87	17.72	1324.00	17.36	17.36
DP-1	0.49	5.62	20	59.00	1.90%	0.14	6.84	1265.0	0.94%	1.94	10.87	17.72	1324.00	17.36	17.36
C1.3	0.49	0.90	20	76.00	2.00%	0.17	7.64	340.0	1.00%	2.00	2.83	10.47	416.00	12.31	10.47
C1.4	0.49	2.41	20	36.00	2.80%	0.13	4.70	1010.0	1.14%	2.14	7.88	12.59	1046.00	15.81	12.59
DP-2	0.49	3.31	20	76.00	2.00%	0.17	7.64	1280.0	1.00%	2.00	10.67	18.30	1356.00	17.53	17.53
C1.5	0.49	0.19	20	45.00	2.00%	0.13	5.88	93.0	1.29%	2.27	0.68	6.56	138.00	10.77	6.56
DP-3	0.49	3.50	20	76.00	2.00%	0.17	7.64	1280.0	1.00%	2.00	10.67	18.30	1356.00	17.53	17.53
C1.6	0.49	0.73	20	28.00	2.00%	0.10	4.64	559.0	0.81%	1.80	5.18	9.81	587.00	13.26	9.81
C1.7	0.45	1.92	20	100.00	2.00%	0.18	9.34	716.0	0.63%	1.59	7.52	16.85	816.00	14.53	14.53
C1.8	0.47	0.77	20	20.00	2.00%	0.08	4.05	520.0	0.96%	1.96	4.42	8.47	540.00	13.00	8.47
DP-5	0.46	2.69	20	100.00	2.00%	0.18	9.19	1093.0	0.73%	1.71	10.66	19.85	1193.00	16.63	16.63
C1.9	0.49	2.10	20	50.00	2.00%	0.13	6.20	1057.0	0.80%	1.79	9.85	16.04	1107.00	16.15	16.04
C1.10	0.49	0.18	20	100.00	2.30%	0.20	8.37	100.0	0.80%	1.79	0.93	9.30	200.00	11.11	9.30
C1.11	0.49	0.17	20	42.00	2.00%	0.12	5.68	116.0	0.86%	1.85	1.04	6.72	158.00	10.88	6.72
C1.12	0.49	1.05	20	98.00	2.45%	0.20	8.11	717.0	0.71%	1.69	7.09	15.20	815.00	14.53	14.53
DP-8	0.49	3.50	20	50.00	2.00%	0.13	6.20	1902.0	0.76%	1.74	18.18	24.38	1952.00	20.84	20.84

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BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc $t_c=(L/180)+10$ minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
C1.13	0.45	0.71	20	42.00	3.33%	0.14	5.11	400.0	0.65%	1.61	4.13	9.25	442.00	12.46	9.25
C1.14	0.46	1.27	20	34.00	2.00%	0.11	5.36	641.0	0.70%	1.67	6.38	11.74	675.00	13.75	11.74
DP-9	0.46	1.98	20	42.00	3.33%	0.14	5.03	1002.0	0.66%	1.62	10.28	15.31	1044.00	15.80	15.31
C1.15	0.49	0.80	20	85.00	2.47%	0.19	7.53	401.0	0.95%	1.95	3.43	10.96	486.00	12.70	10.96
C1.16	0.49	0.50	20	37.00	2.00%	0.12	5.33	315.0	1.33%	2.31	2.28	7.61	352.00	11.96	7.61
C1.17	0.49	1.38	15	77.00	3.25%	0.20	6.55	300.0	1.33%	1.73	2.89	9.44	377.00	12.09	9.44
DP-12	0.49	2.68	20	85.00	2.47%	0.19	7.53	401.0	0.95%	1.95	3.43				
			18" RCP					185.0	1.62%	7.57	0.41	11.37	671.00	13.73	11.37
C1.18	0.27	5.81	15	100.00	3.00%	0.16	10.43	38.0	23.68%	7.30	0.09				
			20					565.0	0.50%	1.41	6.66	17.17	703.00	13.91	13.91
C1	0.49	26.51	20	50.00	2.00%	0.13	6.20	1902.0	0.76%	1.74	18.18	24.38	1952.00	20.84	20.84
C2	0.49	5.44	15	100.00	4.00%	0.24	6.97	150.0	1.13%	1.59	1.57	8.54	250.00	11.39	8.54
C3	0.49	0.69	15	100.00	2.00%	0.19	8.76	26.0	2.70%	2.46	0.18	8.94	126.00	10.70	8.94
C4	0.47	1.84	15	30.00	2.00%	0.10	4.96	236.0	2.97%	2.59	1.52	6.48	266.00	11.48	6.48
C5.1	0.45	1.14	20	80.00	2.50%	0.17	7.76	197.0	1.68%	2.59	1.27	9.02	277.00	11.54	9.02
C5.2	0.45	0.72	15	100.00	2.00%	0.18	9.34	79.0	6.33%	3.77	0.35				
			15					58.0	15.52%	5.91	0.16	9.85	237.00	11.32	9.85

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BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc tc=(L/180)+10 minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
C5	0.45	1.86	15	100.00	2.00%	0.18	9.34	79.0	6.33%	3.77	0.35				
			15					58.0	15.52%	5.91	0.16	9.85	237.00	11.32	9.85
C6	0.45	0.92	15	100.00	2.00%	0.18	9.34	120.0	6.67%	3.87	0.52	9.85	220.00	11.22	9.85
D1.1	0.45	1.21	15	90.00	2.67%	0.19	8.05	445.0	1.57%	1.88	3.95	12.00	535.00	12.97	12.00
D1.2	0.90	0.55	20	30.00	2.00%	0.32	1.57	681.0	0.70%	1.67	6.78	8.36	711.00	13.95	8.36
D1.3	0.45	0.42	20	100.00	2.00%	0.18	9.34	135.0	1.10%	2.10	1.07	10.41	235.00	11.31	10.41
D1.4	0.45	1.13	20	46.00	3.26%	0.14	5.39	556.0	1.25%	2.24	4.14	9.53	602.00	13.34	9.53
DP-16	0.57	2.10	20	30.00	2.00%	0.12	4.17	1289.0	1.01%	2.01	10.69	14.86	1319.00	17.33	14.86
D1.5	0.45	0.87	20	61.00	1.64%	0.13	7.79	433.0	0.88%	1.88	3.85	11.63	494.00	12.74	11.63
DP-17	0.53	2.97	20	30.00	2.00%	0.11	4.48	1771.0	0.96%	1.96	15.06	19.55	1801.00	20.01	19.55
D1.6	0.45	1.26	20	47.00	2.00%	0.12	6.40	736.0	1.05%	2.05	5.99	12.39	783.00	14.35	12.39
D1.7	0.45	1.39	20	100.00	3.50%	0.21	7.76	696.0	0.72%	1.70	6.84	14.60	796.00	14.42	14.42
DP-18	0.45	2.65	20	100.00	3.50%	0.21	7.76	696.0	0.72%	1.70	6.84	14.60	796.00	14.42	14.42
D1.8	0.45	1.05	20	100.00	2.00%	0.18	9.34	789.0	0.79%	1.78	7.40	16.73	889.00	14.94	14.94
DP-19	0.45	3.70	20	100.00	2.00%	0.18	9.34	789.0	0.79%	1.78	7.40	16.73	889.00	14.94	14.94
D1.9	0.45	0.24	20	39.00	3.08%	0.13	5.06	206.0	1.12%	2.12	1.62	6.68	245.00	11.36	6.68
DP-20	0.45	3.94	20	100.00	2.00%	0.18	9.34	1029.0	0.86%	1.85	9.25	18.58	1129.00	16.27	16.27

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BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc tc=(L/180)+10 minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
D1	0.48	8.12	20	30.00	2.00%	0.10	4.88	1771.0	0.99%	1.99	14.83	19.71	1801.00	20.01	19.71
D2	0.45	1.16	15	50.00	16.00%	0.25	3.32	314.0	0.64%	1.20	4.36	7.68	364.00	12.02	7.68
D3	0.16	0.79	7	100.00	1.00%	0.10	16.97	43.0	1.00%	0.70	1.02	18.00	143.00	10.79	10.79
D4	0.45	1.68	20	60.00	3.33%	0.16	6.11	67.0	4.48%	4.23	0.26	6.38	127.00	10.71	6.38
D5	0.22	1.30	20	95.00	3.37%	0.15	10.37	81.0	11.11%	6.67	0.20				
			15					140.0	1.00%	1.50	1.56	12.13	316.00	11.76	11.76
D6	0.16	0.23	15	100.00	2.00%	0.12	13.50					13.50	100.00	10.56	10.56

APPENDIX C – HYDRAULIC CALCULATIONS

Channel Report

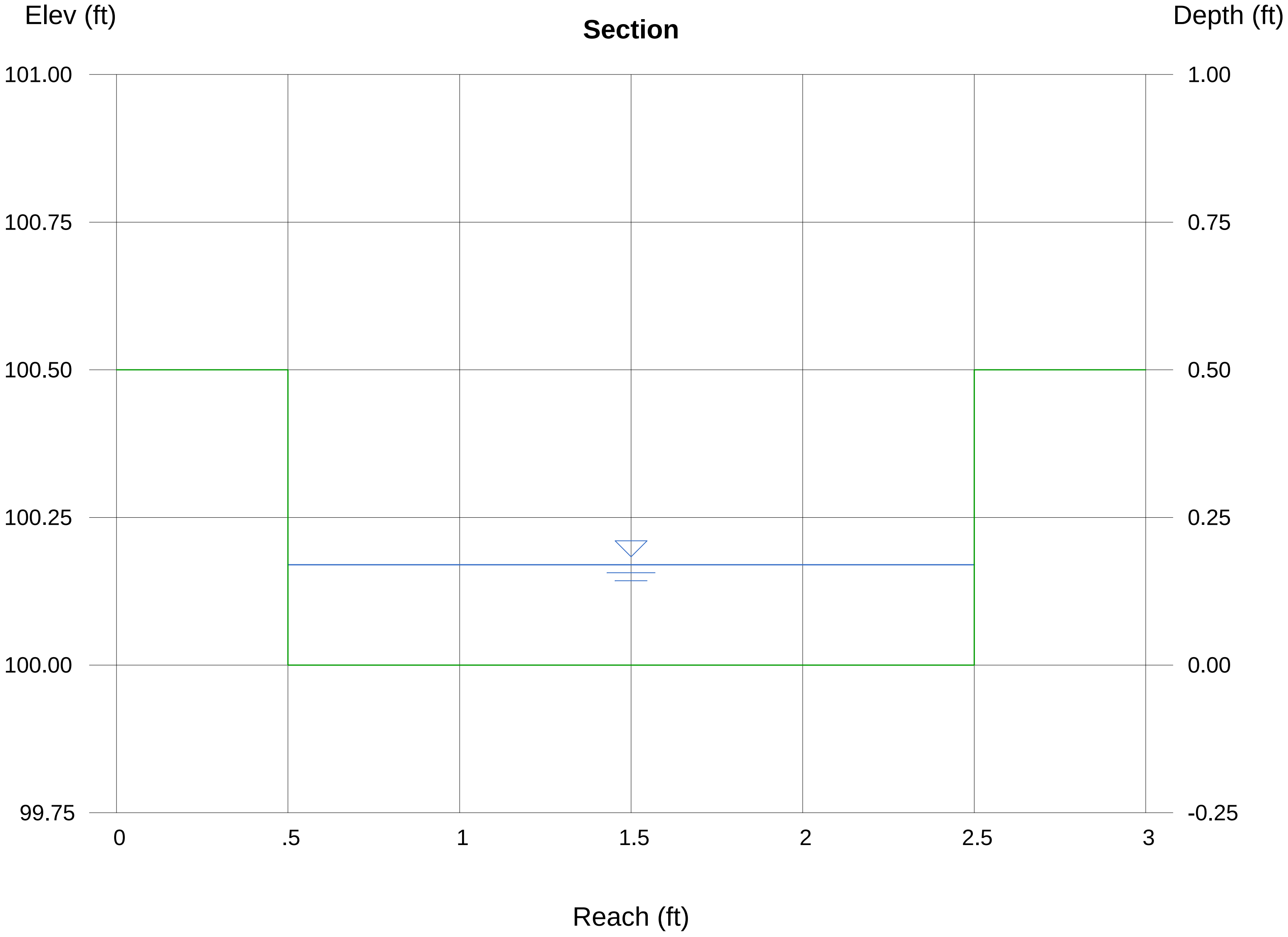
trickle channel pond cr2

Rectangular
Botom Width (ft) = 2.00
Total Depth (ft) = 0.50

Invert Elev (ft) = 100.00
Slope (%) = 1.00
N-Value = 0.013

Calculations
Compute by: Known Q
Known Q (cfs) = 1.00

Highlighted
Depth (ft) = 0.17
Q (cfs) = 1.000
Area (sqft) = 0.34
Velocity (ft/s) = 2.94
Wetted Perim (ft) = 2.34
Crit Depth, Yc (ft) = 0.20
Top Width (ft) = 2.00
EGL (ft) = 0.30

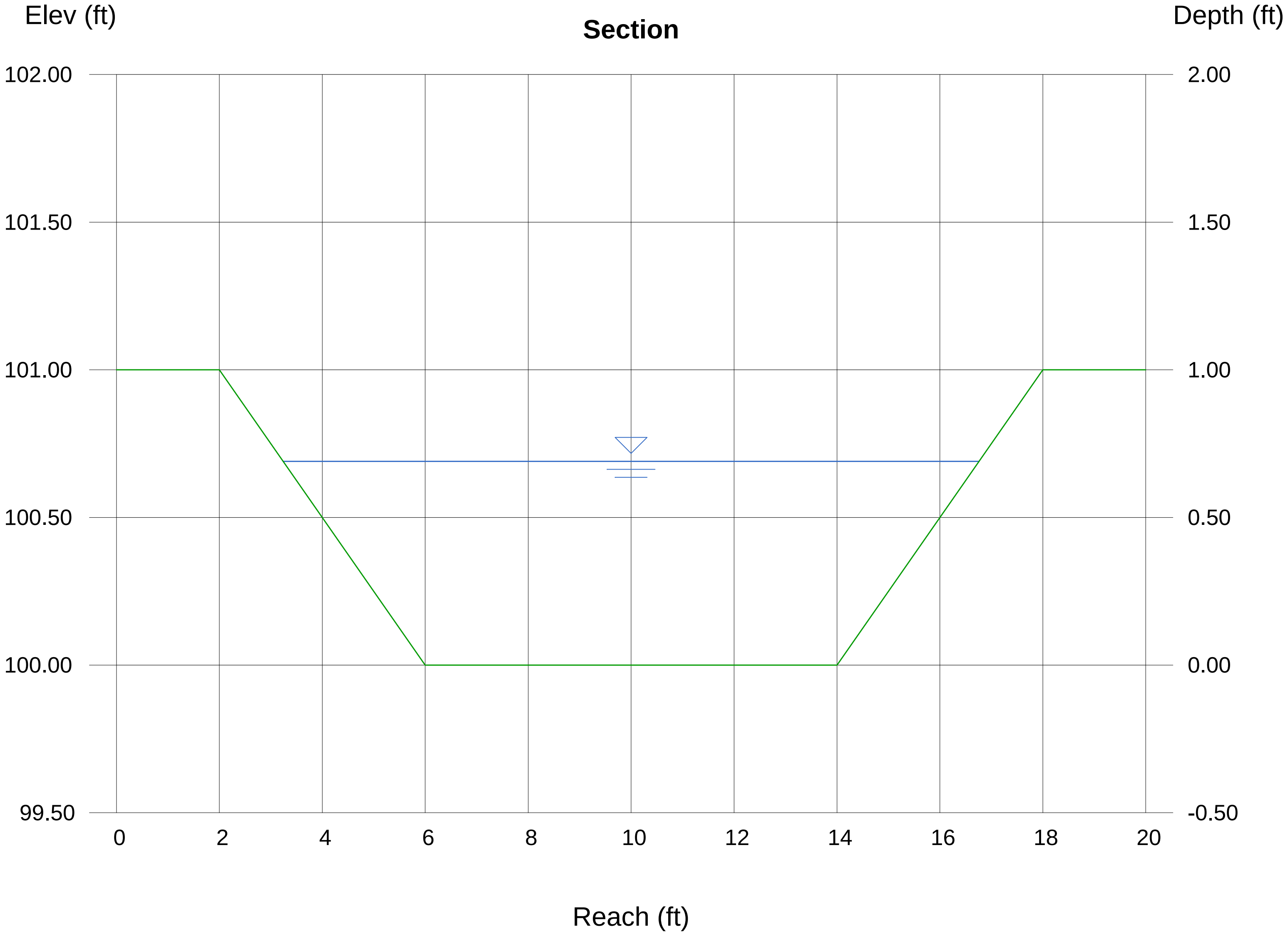


Channel Report

Overflow from Des. Pt 4 (Alsea Dr) to Pond C1-R

Trapezoidal	
Botom Width (ft)	= 8.00
Side Slope (z:1)	= 4.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.30
N-Value	= 0.025
Calculations	
Compute by:	Known Q
Known Q (cfs)	= 32.80

Highlighted	
Depth (ft)	= 0.69
Q (cfs)	= 32.80
Area (sqft)	= 7.42
Velocity (ft/s)	= 4.42
Wetted Perim (ft)	= 13.69
Crit Depth, Yc (ft)	= 0.72
Top Width (ft)	= 13.52
EGL (ft)	= 0.99



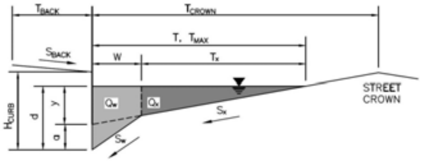
Version 4.05 Released March 2017

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

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-1**



Gutter Geometry (Enter data in the blue cells)			
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

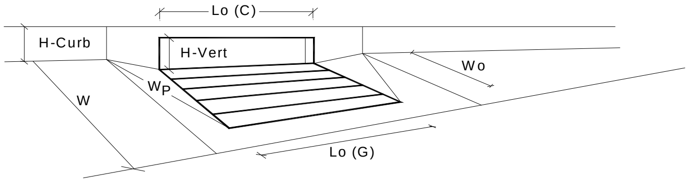
Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.60	4.08	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.51	1.51	inches
Water Depth at Gutter Flowline	d =	5.11	5.59	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	13.0	15.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.397	0.350	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	31.2	46.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	29.2	44.2	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.186	0.123	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

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	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.9	8.0	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	15.00	15.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.31	1.31	
Clogging Factor for Multiple Units	Clog =	0.04	0.04	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	9.5	21.2	cfs
Interception with Clogging	Q_{wa} =	9.1	20.2	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	20.8	26.8	cfs
Interception with Clogging	Q_{oa} =	19.8	25.7	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	13.1	22.2	cfs
Interception with Clogging	Q_{ma} =	12.5	21.2	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**9.1**	**20.2**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	15.00	15.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	18.1	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.3	2.4	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.32	0.50	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.55	0.75	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.78	0.89	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**9.1**	**20.2**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	9.1	20.2	cfs

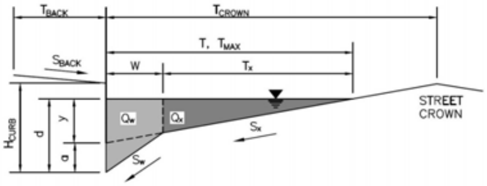
Version 4.05 Released March 2017

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

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

Project: Creekside Filing No. 1, Lorson Ranch, El Paso County, CO #100.045

Inlet ID: Inlet #DP-3



Gutter Geometry (Enter data in the blue cells)			
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

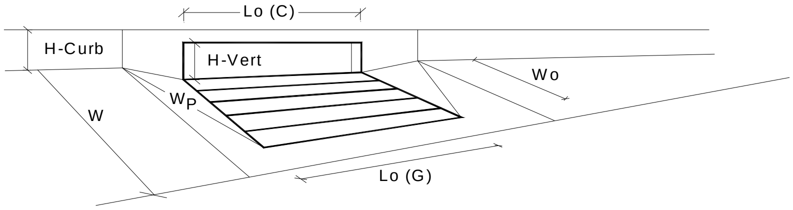
Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.60	4.08	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.51	1.51	inches
Water Depth at Gutter Flowline	d =	5.11	5.59	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	13.0	15.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.397	0.350	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	31.2	46.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	29.2	44.2	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.186	0.123	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor ($d \geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

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	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.2	7.1	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.25	1.25	
Clogging Factor for Multiple Units	Clog =	0.06	0.06	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	6.0	13.5	cfs
Interception with Clogging	Q_{wa} =	5.6	12.6	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	12.3	16.3	cfs
Interception with Clogging	Q_{oa} =	11.5	15.3	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	8.0	13.8	cfs
Interception with Clogging	Q_{ma} =	7.5	12.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**5.6**	**12.6**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	15.4	23.3	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	1.5	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.27	0.43	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.49	0.67	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.88	0.99	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**5.6**	**12.6**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	5.6	12.6	cfs

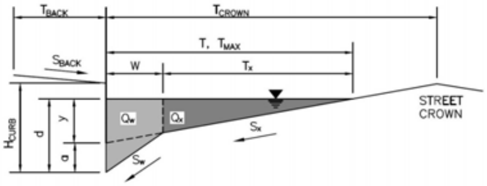
Version 4.05 Released March 2017

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

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-6**

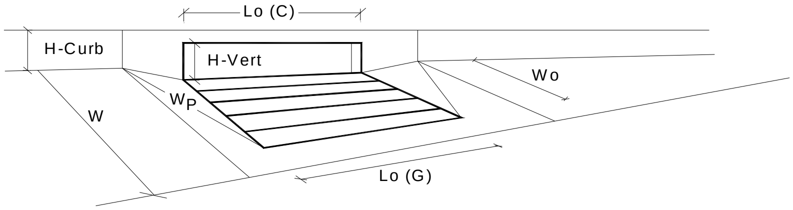


Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0		ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.200		ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015		
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00		inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0		ft
Gutter Width	W =	2.00		ft
Street Transverse Slope	S_X =	0.020		ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083		ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000		ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016		
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	
		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.1	6.9	inches
Grate Information		MINOR	MAJOR	☑ Override
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.25	1.25	
Clogging Factor for Multiple Units	Clog =	0.06	0.06	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	5.7	12.6	cfs
Interception with Clogging	Q_{wa} =	5.3	11.8	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	12.1	16.0	cfs
Interception with Clogging	Q_{oa} =	11.3	15.0	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	7.7	13.2	cfs
Interception with Clogging	Q_{ma} =	7.2	12.4	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**5.3**	**11.8**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	15.0	22.5	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	1.3	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.26	0.41	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.48	0.65	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.88	0.98	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**5.3**	**11.8**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	5.3	11.8	cfs

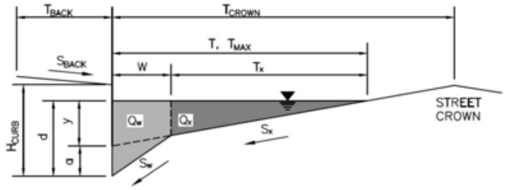
Version 4.05 Released March 2017

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

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-10**



Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

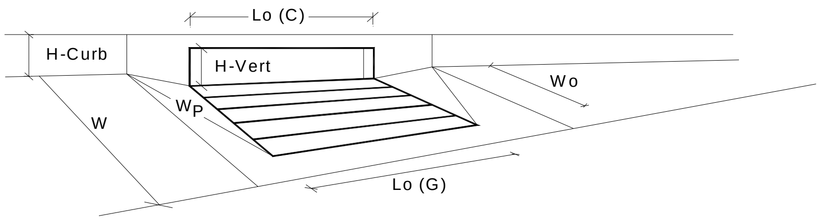
Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.60	4.08	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.51	1.51	inches
Water Depth at Gutter Flowline	d =	5.11	5.59	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	13.0	15.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.397	0.350	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	31.2	46.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	29.2	44.2	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.186	0.123	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

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	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.9	8.9	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.25	1.25	
Clogging Factor for Multiple Units	Clog =	0.06	0.06	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	8.5	21.4	cfs
Interception with Clogging	Q_{wa} =	7.9	20.0	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	13.9	19.4	cfs
Interception with Clogging	Q_{oa} =	13.1	18.1	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	10.1	18.9	cfs
Interception with Clogging	Q_{ma} =	9.5	17.7	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**7.9**	**17.7**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	18.3	30.8	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.3	3.3	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.33	0.58	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.56	0.84	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.93	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**7.9**	**17.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	7.9	17.6	cfs

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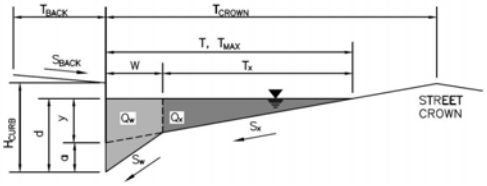
ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-11**



Gutter Geometry (Enter data in the blue cells)

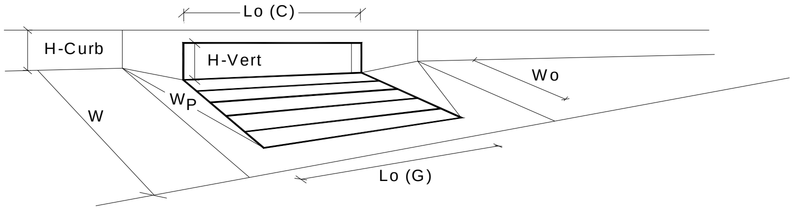
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	4.4	6.2	inches
Grate Information		MINOR	MAJOR	☑ Override
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	2.8	6.4	cfs
Interception with Clogging	Q_{wa} =	2.5	5.8	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	5.1	7.3	cfs
Interception with Clogging	Q_{oa} =	4.6	6.5	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	3.5	6.4	cfs
Interception with Clogging	Q_{ma} =	3.1	5.7	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**2.5**	**5.7**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	12.0	19.5	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	0.6	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.20	0.35	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.56	0.79	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	1.00	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**2.5**	**5.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.5	5.7	cfs

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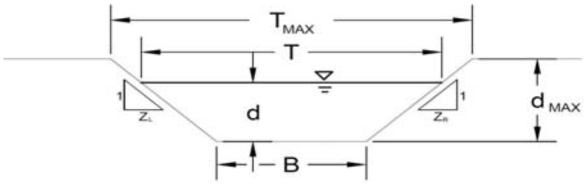
AREA INLET IN A SWALE

Creekside Filing No. 1, Lorson Ranch, El Paso County, CO **#100.045**

Inlet #DP-12 (C1.17)



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)	A, B, C, D or E	C	
Manning's n (Leave cell D16 blank to manually enter an n value)	n =	see details below	
Channel Invert Slope	S_O =	0.0133	ft/ft
Bottom Width	B =	0.00	ft
Left Side Slope	Z1 =	30.00	ft/ft
Right Side Slope	Z2 =	30.00	ft/ft

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

Choose One:
● **Non-Cohesive**
○ **Cohesive**
○ **Paved**

		Minor Storm	Major Storm	
Max. Allowable Top Width of Channel for Minor & Major Storm	T_{MAX} =	**60.00**	**60.00**	**feet**
Max. Allowable Water Depth in Channel for Minor & Major Storm	d_{MAX} =	**0.80**	**1.00**	**feet**

Maximum Channel Capacity Based On Allowable Top Width		Minor Storm	Major Storm	
Max. Allowable Top Width	T_{MAX} =	**60.00**	**60.00**	**ft**
Water Depth	d =	1.00	1.00	ft
Flow Area	A =	30.00	30.00	sq ft
Wetted Perimeter	P =	60.03	60.03	ft
Hydraulic Radius	R =	0.50	0.50	ft
Manning's n based on NRCS Vegetal Retardance	n =	0.215	0.215	
Flow Velocity	V =	0.50	0.50	fps
Velocity-Depth Product	VR =	0.25	0.25	ft^2/s
Hydraulic Depth	D =	0.50	0.50	ft
Froude Number	Fr =	0.13	0.13	
Max. Flow Based On Allowable Top Width	Q_T =	**15.1**	**15.1**	**cfs**

Maximum Channel Capacity Based On Allowable Water Depth		Minor Storm	Major Storm	
Max. Allowable Water Depth	d_{MAX} =	**0.80**	**1.00**	**feet**
Top Width	T =	48.00	60.00	feet
Flow Area	A =	19.20	30.00	square feet
Wetted Perimeter	P =	48.03	60.03	feet
Hydraulic Radius	R =	0.40	0.50	feet
Manning's n based on NRCS Vegetal Retardance	n =	0.430	0.215	
Flow Velocity	V =	0.22	0.50	fps
Velocity-Depth Product	VR =	0.09	0.25	ft^2/s
Hydraulic Depth	D =	0.40	0.50	feet
Froude Number	Fr =	0.06	0.13	
Max. Flow Based On Allowable Water Depth	Q_d =	**4.2**	**15.1**	**cfs**

Allowable Channel Capacity Based On Channel Geometry		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	**4.2**	**15.1**	**cfs**
MAJOR STORM Allowable Capacity is based on Depth Criterion	d_{allow} =	**0.80**	**1.00**	**ft**

Water Depth in Channel Based On Design Peak Flow				
Design Peak Flow	Q_o =	**2.9**	**6.3**	**cfs**
Water Depth	**d =**	**0.70**	**0.91**	**feet**
Top Width	T =	41.91	54.69	feet
Flow Area	A =	14.64	24.92	square feet
Wetted Perimeter	P =	41.93	54.72	feet
Hydraulic Radius	R =	0.35	0.46	feet
Manning's n based on NRCS Vegetal Retardance	n =	0.430	0.402	
Flow Velocity	V =	0.20	0.25	fps
Velocity-Depth Product	VR =	0.07	0.12	ft^2/s
Hydraulic Depth	D =	0.35	0.46	feet
Froude Number	Fr =	0.06	0.07	

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'
Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Version 4.05 Released March 2017

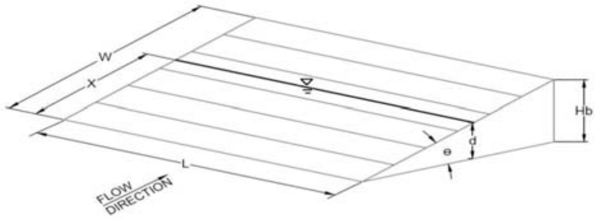
AREA INLET IN A SWALE

Creekside Filing No. 1, Lorson Ranch, El Paso County, CO **#100.045**

Inlet #DP-12 (C1.17)

Inlet Design Information (Input)

Type of Inlet CDOT Type C (Depressed) ▾ Inlet Type = CDOT Type C (Depressed)



Angle of Inclined Grate (must be <= 30 degrees)	θ =		0.00	degrees
Width of Grate	W =		3.00	feet
Length of Grate	L =		3.00	feet
Open Area Ratio	A_{RATIO} =		0.70	
Height of Inclined Grate	H_B =		0.00	feet
Clogging Factor	C_f =		0.50	
Grate Discharge Coefficient	C_d =		0.84	
Orifice Coefficient	C_o =		0.56	
Weir Coefficient	C_w =		1.81	
		MINOR	MAJOR	
Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)	d =	1.70	1.91	
Grate Capacity as a Weir				
Submerged Side Weir Length	X =	3.00	3.00	feet
Inclined Side Weir Flow	Q_{ws} =	21.0	25.1	cfs
Base Weir Flow	Q_{wb} =	30.0	35.8	cfs
Interception without Clogging	Q_{wi} =	72.0	85.9	cfs
Interception with Clogging	Q_{wa} =	36.0	43.0	cfs
Grate Capacity as an Orifice				
Interception without Clogging	Q_{oi} =	37.1	39.3	cfs
Interception with Clogging	Q_{oa} =	18.5	19.7	cfs
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**18.5**	**19.7**	**cfs**
	Bypassed Flow, Q_b =	**0.0**	**0.0**	**cfs**
	Capture Percentage = Q_a/Q_o = C%	**100**	**100**	**%**

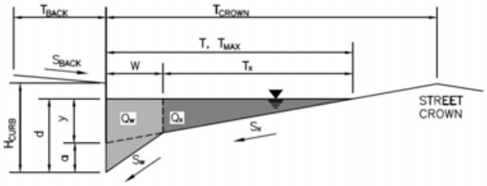
Version 4.05 Released March 2017

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

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-15**



Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

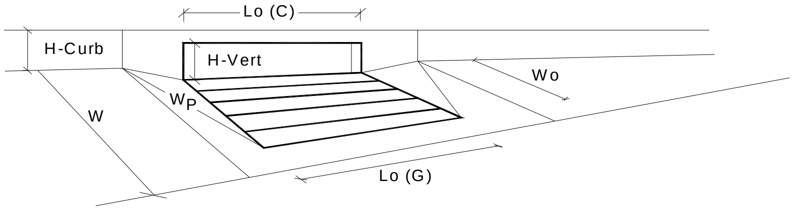
Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.60	4.08	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.51	1.51	inches
Water Depth at Gutter Flowline	d =	5.11	5.59	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	13.0	15.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.397	0.350	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	31.2	46.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	29.2	44.2	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.186	0.123	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



		MINOR	MAJOR	
Design Information (Input)	CDOT Type R Curb Opening			
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	4.2	5.1	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	2.5	4.2	cfs
Interception with Clogging	Q_{wa} =	2.2	3.7	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	4.8	6.1	cfs
Interception with Clogging	Q_{oa} =	4.3	5.5	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	3.2	4.7	cfs
Interception with Clogging	Q_{ma} =	2.9	4.2	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**2.2**	**3.7**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	11.3	15.1	ft
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	0.0	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.19	0.26	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.54	0.66	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	1.00	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**2.2**	**3.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.2	3.7	cfs

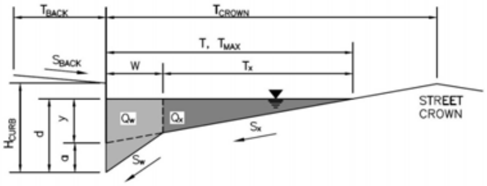
Version 4.05 Released March 2017

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

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

Project: **Creekside Filing No. 1, Lorson Ranch, El Paso County, CO** **#100.045**

Inlet ID: **Inlet #DP-23**



Gutter Geometry (Enter data in the blue cells)			
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.015	
Height of Curb at Gutter Flow Line	H_{CURB} =	9.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	17.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	15.0	17.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	12.6	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

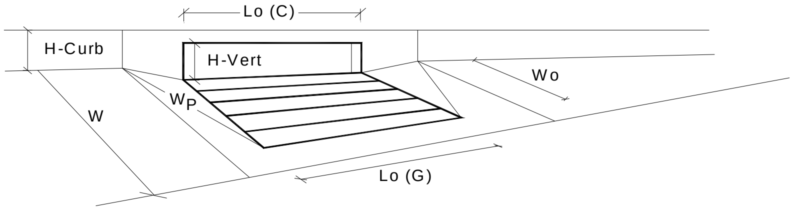
Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.60	4.08	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.51	1.51	inches
Water Depth at Gutter Flowline	d =	5.11	5.59	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	13.0	15.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.397	0.350	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	Q_T =	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	31.2	46.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	29.2	44.2	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.186	0.123	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	Q_d =	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	**SUMP**	**SUMP**	**cfs**

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	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.2	8.4	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	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_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	15.00	15.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.31	1.31	
Clogging Factor for Multiple Units	Clog =	0.04	0.04	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	11.0	23.5	cfs
Interception with Clogging	Q_{wa} =	10.5	22.4	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	21.8	27.7	cfs
Interception with Clogging	Q_{oa} =	20.8	26.5	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	14.4	23.7	cfs
Interception with Clogging	Q_{ma} =	13.8	22.7	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	**Q_{Curb} =**	**10.5**	**22.4**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	15.00	15.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	19.5	28.5	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.6	2.8	inches
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.35	0.53	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.58	0.79	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.80	0.91	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**10.5**	**22.4**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	Q PEAK REQUIRED =	10.5	22.4	cfs

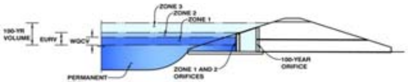
APPENDIX D – POND CALCULATIONS

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch

Basin ID: Pond C1-R



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type = **EDB**
Watershed Area = 117.50 acres
Watershed Length = 3,000 ft
Watershed Slope = 0.009 ft/ft
Watershed Imperviousness = 55.00% percent
Percentage Hydrologic Soil Group A = 0.0% percent
Percentage Hydrologic Soil Group B = 20.0% percent
Percentage Hydrologic Soil Groups C/D = 80.0% percent
Desired WQCV Drain Time = 40.0 hours
Location for 1-hr Rainfall Depths = User Input
Water Quality Capture Volume (WQCV) = 2.158 acre-feet
Excess Urban Runoff Volume (EURV) = 6.321 acre-feet

			Optional User Override 1-hr Precipitation	
2-yr Runoff Volume (P1 = 1.19 in.) =	5.795	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	8.318	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	10.616	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	14.187	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	16.883	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	20.271	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches

Approximate 2-yr Detention Volume = 5.434 acre-feet
Approximate 5-yr Detention Volume = 7.833 acre-feet
Approximate 10-yr Detention Volume = 9.170 acre-feet
Approximate 25-yr Detention Volume = 9.887 acre-feet
Approximate 50-yr Detention Volume = 10.240 acre-feet
Approximate 100-yr Detention Volume = 11.444 acre-feet

Stage-Storage Calculation

Zone 1 Volume (WQCV) = 2.158 acre-feet
Zone 2 Volume (EURV - Zone 1) = 4.163 acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) = 6.202 acre-feet
Total Detention Basin Volume = 12.523 acre-feet
Initial Surcharge Volume (ISV) = user ft^3
Initial Surcharge Depth (ISD) = user ft
Total Available Detention Depth (H_{total}) = user ft
Depth of Trickle Channel (H_{TC}) = user ft
Slope of Trickle Channel (S_{TC}) = user ft/ft
Slopes of Main Basin Sides (S_{main}) = user H:V
Basin Length-to-Width Ratio ($R_{L/W}$) = user

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

Depth Increment = 0.2 ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	40	0.001		
5684	--	0.20	--	--	--	50	0.001	9	0.000
5685	--	1.20	--	--	--	11,456	0.263	5,648	0.130
5686	--	2.20	--	--	--	44,890	1.031	33,935	0.779
5687	--	3.20	--	--	--	82,996	1.905	97,877	2.247
5688	--	4.20	--	--	--	91,041	2.090	184,896	4.245
5689	--	5.20	--	--	--	99,130	2.276	279,981	6.427
5690	--	6.20	--	--	--	106,283	2.440	382,688	8.785
5691	--	7.20	--	--	--	113,531	2.606	492,595	11.308
5692	--	8.20	--	--	--	120,991	2.778	609,856	14.000
5693	--	9.20	--	--	--	128,724	2.955	734,713	16.867
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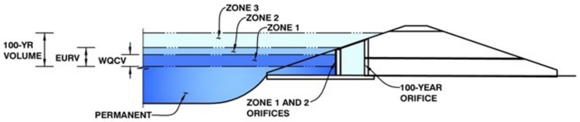
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch

Basin ID: Pond C1-R



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	3.16	2.158	Orifice Plate
Zone 2 (EURV)	5.16	4.163	Rectangular Orifice
(100+1/2WQCV)	7.66	6.202	Weir&Pipe (Restrict)
		12.523	**Total**

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

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

Calculated Parameters for Underdrain

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

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

Invert of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	3.16	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	13.00	inches
Orifice Plate: Orifice Area per Row =	7.10	sq. inches (use rectangular openings)

Calculated Parameters for Plate

WQ Orifice Area per Row =	4.931E-02	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	1.10	2.20					
Orifice Area (sq. inches)	7.10	7.10	7.10					

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

User Input: Vertical Orifice (Circular or Rectangular)

	Zone 2 Rectangular	Not Selected	
Invert of Vertical Orifice =	3.30	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	5.16	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	8.00	N/A	inches
Vertical Orifice Width =	12.00		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.67	N/A	ft^2
Vertical Orifice Centroid =	0.33	N/A	feet

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	5.43	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	22.00	N/A	feet
Overflow Weir Slope =	4.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	6.43	N/A	feet
Over Flow Weir Slope Length =	4.12	N/A	feet
Grate Open Area / 100-yr Orifice Area =	4.58	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	63.50	N/A	ft^2
Overflow Grate Open Area w/ Debris =	31.75	N/A	ft^2

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

	Zone 3 Restrictor	Not Selected	
Depth to Invert of Outlet Pipe =	0.00	N/A	ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter =	54.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	44.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	13.88	N/A	ft^2
Outlet Orifice Centroid =	1.99	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	2.25	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	10.00	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	150.00	feet
Spillway End Slopes =	10.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Calculated Parameters for Spillway

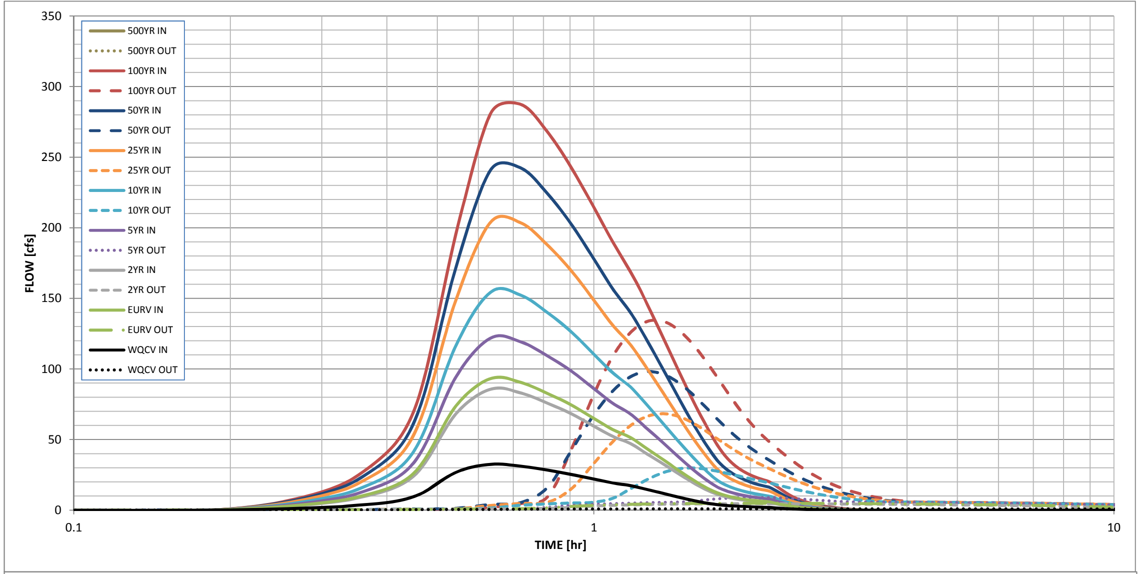
Spillway Design Flow Depth=	0.71	feet
Stage at Top of Freeboard =	11.71	feet
Basin Area at Top of Freeboard =	2.96	acres

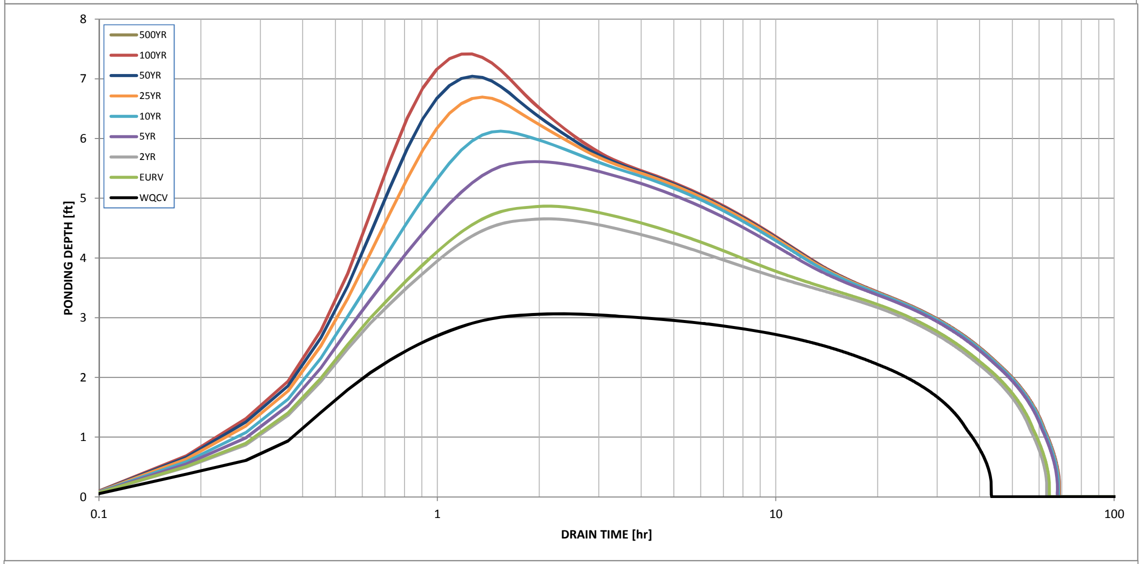
Routed Hydrograph Results

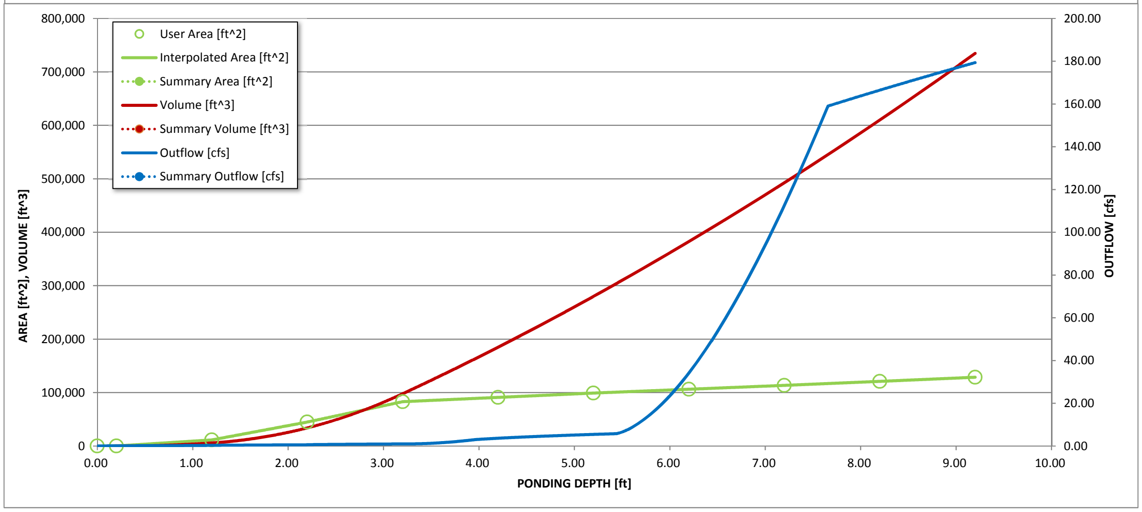
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	2.158	6.321	5.795	8.318	10.616	14.187	16.883	20.271	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	2.157	6.309	5.789	8.314	10.610	14.173	16.869	20.260	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.09	0.29	0.71	0.94	1.24	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	1.5	11.1	33.6	83.3	110.9	145.8	0.0
Peak Inflow Q (cfs) =	32.5	93.2	85.7	122.0	154.5	204.2	242.3	287.4	#N/A
Peak Outflow Q (cfs) =	1.0	4.9	4.6	8.9	29.9	68.3	98.1	133.8	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.8	0.9	0.8	0.9	0.9	#N/A
Structure Controlling Flow =	Plate	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	#N/A
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.0	0.4	1.0	1.4	2.0	#N/A
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) =	39	55	55	57	56	53	52	50	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**42**	60	59	63	62	61	60	59	#N/A
Maximum Ponding Depth (ft) =	3.07	4.87	4.66	5.61	6.13	6.70	7.05	7.42	#N/A
Area at Maximum Ponding Depth (acres) =	1.78	2.21	2.17	2.34	2.43	2.52	2.58	2.64	#N/A
Maximum Volume Stored (acre-ft) =	1.989	5.664	5.204	7.374	8.591	10.001	10.894	11.860	#N/A

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)







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

Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs **UD-Detention, Version 3.07 (February 2017)**

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

	SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	#N/A
Time Interval	TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
5.43 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:05:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Hydrograph Constant	0:10:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:16:17	1.39	3.70	3.44	4.64	5.61	6.94	7.84	8.65	#N/A
0.920	0:21:43	3.81	10.50	9.71	13.44	16.63	21.27	24.58	27.85	#N/A
	0:27:09	9.79	26.96	24.94	34.52	42.73	54.71	63.28	71.96	#N/A
	0:32:35	26.87	73.84	68.31	94.41	116.73	149.20	172.36	195.72	#N/A
	0:38:01	32.50	93.24	85.74	121.96	154.49	204.16	241.17	281.22	#N/A
	0:43:26	31.14	90.44	83.00	119.18	152.19	203.48	242.32	287.39	#N/A
	0:48:52	28.34	82.66	75.79	109.23	139.88	187.74	224.18	267.88	#N/A
	0:54:18	25.45	74.52	68.32	98.54	126.25	169.54	202.50	242.59	#N/A
	0:59:44	22.15	65.39	59.91	86.64	111.22	149.71	179.07	215.85	#N/A
	1:05:10	19.25	57.19	52.38	75.84	97.42	131.20	156.98	190.21	#N/A
	1:10:35	17.45	51.45	47.16	68.07	87.23	117.11	139.84	168.76	#N/A
	1:16:01	14.56	43.35	39.68	57.59	74.12	100.05	119.88	145.61	#N/A
	1:21:27	12.01	36.00	32.94	47.88	61.68	83.36	99.96	122.43	#N/A
	1:26:53	9.43	28.71	26.23	38.35	49.60	67.35	80.99	100.45	#N/A
	1:32:19	7.20	22.30	20.36	29.89	38.75	52.80	63.61	80.11	#N/A
	1:37:44	5.30	16.78	15.29	22.58	29.38	40.23	48.64	62.59	#N/A
	1:43:10	4.02	12.49	11.41	16.74	21.69	29.75	36.07	47.36	#N/A
	1:48:36	3.27	9.98	9.12	13.32	17.18	23.39	28.22	36.17	#N/A
	1:54:02	2.76	8.38	7.67	11.17	14.39	19.51	23.46	29.64	#N/A
	1:59:28	2.41	7.27	6.66	9.67	12.44	16.82	20.19	25.31	#N/A
	2:04:53	2.16	6.49	5.94	8.62	11.07	14.94	17.91	22.32	#N/A
	2:10:19	1.98	5.94	5.44	7.87	10.10	13.60	16.28	20.17	#N/A
	2:15:45	1.46	4.48	4.09	6.00	7.78	10.64	12.85	16.13	#N/A
	2:21:11	1.06	3.23	2.96	4.33	5.61	7.66	9.27	11.72	#N/A
	2:26:37	0.78	2.40	2.19	3.21	4.16	5.68	6.86	8.61	#N/A
	2:32:02	0.58	1.78	1.63	2.39	3.09	4.22	5.09	6.42	#N/A
	2:37:28	0.42	1.31	1.20	1.76	2.29	3.13	3.78	4.78	#N/A
	2:42:54	0.30	0.95	0.86	1.27	1.65	2.27	2.74	3.52	#N/A
	2:48:20	0.22	0.69	0.62	0.92	1.20	1.64	1.98	2.56	#N/A
	2:53:46	0.15	0.48	0.44	0.65	0.85	1.18	1.43	1.89	#N/A
	2:59:11	0.09	0.32	0.29	0.43	0.57	0.79	0.96	1.32	#N/A
	3:04:37	0.05	0.18	0.17	0.25	0.34	0.48	0.59	0.85	#N/A
	3:10:03	0.02	0.09	0.08	0.12	0.17	0.24	0.31	0.48	#N/A
	3:15:29	0.00	0.03	0.02	0.04	0.06	0.09	0.12	0.22	#N/A
	3:20:55	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.06	#N/A
	3:26:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:31:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:37:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:42:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:48:04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:53:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:58:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:04:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:09:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:15:13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:20:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:26:04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:31:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:36:56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:42:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:47:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:53:13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:58:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:04:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:09:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:14:56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:20:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:25:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:31:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:36:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:42:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:47:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:52:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:58:23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:03:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:09:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:14:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:20:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:25:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:30:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A

Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.07, March 2018)

Sheet 1 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **July 3, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond C1-R**

1. Basin Storage Volume

A) Effective Imperviousness of Tributary Area, I_a — I_a = 55.0 %

B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) — i = 0.550

C) Contributing Watershed Area — Area = 117.200 ac

D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm — d_6 = in

E) Design Concept
(Select EURV when also designing for flood control)

Choose One
● Water Quality Capture Volume (WQCV)
○ Excess Urban Runoff Volume (EURV)

F) Design Volume (WQCV) Based on 40-hour Drain Time
($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area)$ — V_{DESIGN}= 2.153 ac-ft

G) For Watersheds Outside of the Denver Region,
Water Quality Capture Volume (WQCV) Design Volume
($V_{WQCV\ OTHER} = (d_6*(V_{DESIGN}/0.43))$ — $V_{DESIGN\ OTHER}$= ac-ft

H) User Input of Water Quality Capture Volume (WQCV) Design Volume
(Only if a different WQCV Design Volume is desired) — $V_{DESIGN\ USER}$= ac-ft

I) NRCS Hydrologic Soil Groups of Tributary Watershed
i) Percentage of Watershed consisting of Type A Soils — HSG_A = %
ii) Percentage of Watershed consisting of Type B Soils — HSG_B = %
iii) Percentage of Watershed consisting of Type C/D Soils — $HSG_{C/D}$ = %

J) Excess Urban Runoff Volume (EURV) Design Volume
For HSG A: $EURV_A = 1.68 * i^{1.28}$
For HSG B: $EURV_B = 1.36 * i^{1.08}$
For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$
$EURV_{DESIGN}$ = ac-f t

K) User Input of Excess Urban Runoff Volume (EURV) Design Volume
(Only if a different EURV Design Volume is desired) — $EURV_{DESIGN\ USER}$= ac-f t

2. Basin Shape: Length to Width Ratio
(A basin length to width ratio of at least 2:1 will improve TSS reduction.) — L : W = 2.0 : 1

3. Basin Side Slopes

A) Basin Maximum Side Slopes
(Horizontal distance per unit vertical, 4:1 or flatter preferred) — Z = 4.00 ft / ft

4. Inlet

A) Describe means of providing energy dissipation at concentrated inflow locations:

5. Forebay

A) Minimum Forebay Volume
(V_{FMIN} = 3% of the WQCV) — V_{FMIN} = 0.065 ac-ft

B) Actual Forebay Volume — V_F = 0.070 ac-ft

C) Forebay Depth
(D_F = 30 inch maximum) — D_F = 30.0 in

D) Forebay Discharge

i) Undetained 100-year Peak Discharge — Q_{100} = 288.00 cfs

ii) Forebay Discharge Design Flow
($Q_F = 0.02 * Q_{100}$) — Q_F = 5.76 cfs

E) Forebay Discharge Design

Choose One
● Berm With Pipe
○ Wall with Rect. Notch
○ Wall with V-Notch Weir

ROUND UP TO NEAREST PIPE SIZE

F) Discharge Pipe Size (minimum 8-inches) — Calculated D_P = 13 in

G) Rectangular Notch Width — Calculated W_N = in

Design Procedure Form: Extended Detention Basin (EDB)

Sheet 2 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **July 3, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond C1-R**

6. Trickle Channel

A) Type of Trickle Channel

Choose One
○ Concrete
● Soft Bottom

F) Slope of Trickle Channel

S = 0.0050 ft / ft

PROVIDE A CONSISTENT LONGITUDINAL SLOPE FROM FOREBAY TO MICROPOOL WITH NO MEANDERING. RIPRAP AND SOIL RIPRAP LINED CHANNELS ARE NOT RECOMMENDED.
MINIMUM DEPTH OF 1.5 FEET

7. Micropool and Outlet Structure

A) Depth of Micropool (2.5-feet minimum)

D_M = 2.5 ft

B) Surface Area of Micropool (10 ft^2 minimum)

A_M = 65 sq ft

C) Outlet Type

Choose One
● Orifice Plate
○ Other (Describe):

D) Smallest Dimension of Orifice Opening Based on Hydrograph Routing (Use UD-Detention)

$D_{orifice}$ = 2.60 inches

E) Total Outlet Area

A_{ot} = 20.34 square inches

8. Initial Surcharge Volume

A) Depth of Initial Surcharge Volume
(Minimum recommended depth is 4 inches)

D_{IS} = 4 in

B) Minimum Initial Surcharge Volume
(Minimum volume of 0.3% of the WQCV)

V_{IS} = 281 cu ft

C) Initial Surcharge Provided Above Micropool

V_s= 21.7 cu ft

9. Trash Rack

A) Water Quality Screen Open Area: $A_t = A_{ot} * 38.5*(e^{-0.095D})$

A_t = 612 square inches

B) Type of Screen (If specifying an alternative to the materials recommended in the USDCM, indicate "other" and enter the ratio of the total open are to the total screen are for the material specified.)

Other (Please describe below)

wellscreen stainless

Other (Y/N): y

C) Ratio of Total Open Area to Total Area (only for type 'Other')

User Ratio = 0.6

D) Total Water Quality Screen Area (based on screen type)

A_{total} = 1020 sq. in. **Based on type 'Other' screen ratio**

E) Depth of Design Volume (EURV or WQCV)
(Based on design concept chosen under 1E)

H= 3.16 feet

F) Height of Water Quality Screen (H_{TR})

H_{TR}= 65.92 inches

G) Width of Water Quality Screen Opening ($W_{opening}$)
(Minimum of 12 inches is recommended)

$W_{opening}$ = 15.5 inches

Design Procedure Form: Extended Detention Basin (EDB)

Sheet 3 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **July 3, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond C1-R**

10. Overflow Embankment

A) Describe embankment protection for 100-year and greater overtopping:

B) Slope of Overflow Embankment
(Horizontal distance per unit vertical, 4:1 or flatter preferred)

Ze = ☐ ft / ft

11. Vegetation

Choose One
○ Irrigated
○ Not Irrigated

12. Access

A) Describe Sediment Removal Procedures

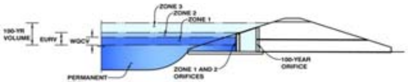
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch

Basin ID: Pond CR2



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type = **EDB**
Watershed Area = 9.50 acres
Watershed Length = 1,000 ft
Watershed Slope = 0.013 ft/ft
Watershed Imperviousness = 52.00% percent
Percentage Hydrologic Soil Group A = 0.0% percent
Percentage Hydrologic Soil Group B = 100.0% percent
Percentage Hydrologic Soil Groups C/D = 0.0% percent
Desired WQCV Drain Time = 40.0 hours
Location for 1-hr Rainfall Depths = User Input
Water Quality Capture Volume (WQCV) = 0.168 acre-feet
Excess Urban Runoff Volume (EURV) = 0.530 acre-feet

Optional User Override 1-hr Precipitation

2-yr Runoff Volume (P1 = 1.19 in.) = 0.429 acre-feet | 1.19 inches
5-yr Runoff Volume (P1 = 1.5 in.) = 0.584 acre-feet | 1.50 inches
10-yr Runoff Volume (P1 = 1.75 in.) = 0.783 acre-feet | 1.75 inches
25-yr Runoff Volume (P1 = 2 in.) = 1.078 acre-feet | 2.00 inches
50-yr Runoff Volume (P1 = 2.25 in.) = 1.284 acre-feet | 2.25 inches
100-yr Runoff Volume (P1 = 2.52 in.) = 1.554 acre-feet | 2.52 inches
500-yr Runoff Volume (P1 = 0 in.) = 0.000 acre-feet | inches
Approximate 2-yr Detention Volume = 0.401 acre-feet
Approximate 5-yr Detention Volume = 0.548 acre-feet
Approximate 10-yr Detention Volume = 0.722 acre-feet
Approximate 25-yr Detention Volume = 0.788 acre-feet
Approximate 50-yr Detention Volume = 0.823 acre-feet
Approximate 100-yr Detention Volume = 0.914 acre-feet

Stage-Storage Calculation

Zone 1 Volume (WQCV) = 0.168 acre-feet
Zone 2 Volume (EURV - Zone 1) = 0.362 acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) = 0.468 acre-feet
Total Detention Basin Volume = 0.998 acre-feet
Initial Surcharge Volume (ISV) = user ft^3
Initial Surcharge Depth (ISD) = user ft
Total Available Detention Depth (H_{total}) = user ft
Depth of Trickle Channel (H_{TC}) = user ft
Slope of Trickle Channel (S_{TC}) = user ft/ft
Slopes of Main Basin Sides (S_{main}) = user H:V
Basin Length-to-Width Ratio ($R_{L/W}$) = user

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

Depth Increment = 0.2 ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	40	0.001		
5681.33	--	0.33	--	--	--	57	0.001	15	0.000
5682	--	1.00	--	--	--	500	0.011	198	0.005
5683	--	2.00	--	--	--	8,344	0.192	4,541	0.104
5684	--	3.00	--	--	--	10,785	0.248	14,189	0.326
5685	--	4.00	--	--	--	13,382	0.307	26,272	0.603
5686	--	5.00	--	--	--	16,130	0.370	41,028	0.942
5687	--	6.00	--	--	--	19,029	0.437	58,608	1.345
5688	--	7.00	--	--	--	22,079	0.507	79,162	1.817
5689	--	8.00	--	--	--	25,280	0.580	102,841	2.361
5690	--	9.00	--	--	--	28,675	0.658	129,819	2.980
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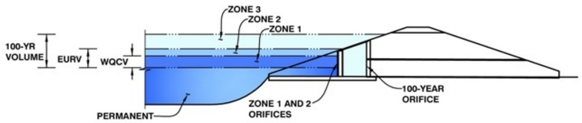
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch
Basin ID: Pond CR2



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.31	0.168	Orifice Plate
Zone 2 (EURV)	3.76	0.362	Rectangular Orifice
(100+1/2WQCV)	5.16	0.468	Weir&Pipe (Restrict)
		0.998	**Total**

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

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

Calculated Parameters for Underdrain

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

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

Invert of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	2.31	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	9.10	inches
Orifice Plate: Orifice Area per Row =	0.57	sq. inches (diameter = 13/16 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	3.958E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	0.77	1.54					
Orifice Area (sq. inches)	0.57	0.57	0.57					

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

User Input: Vertical Orifice (Circular or Rectangular)

	Zone 2 Rectangular	Not Selected	
Invert of Vertical Orifice =	2.31	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	3.76	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	2.10	N/A	inches
Vertical Orifice Width =	1.00		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.01	N/A	ft^2
Vertical Orifice Centroid =	0.09	N/A	feet

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	3.88	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	4.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	4.88	N/A	feet
Over Flow Weir Slope Length =	4.12	N/A	feet
Grate Open Area / 100-yr Orifice Area =	11.45	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	11.54	N/A	ft^2
Overflow Grate Open Area w/ Debris =	5.77	N/A	ft^2

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

	Zone 3 Restrictor	Not Selected	
Depth to Invert of Outlet Pipe =	0.00	N/A	ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter =	18.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	10.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	1.01	N/A	ft^2
Outlet Orifice Centroid =	0.48	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	1.68	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	6.00	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	10.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	2.29	feet

Calculated Parameters for Spillway

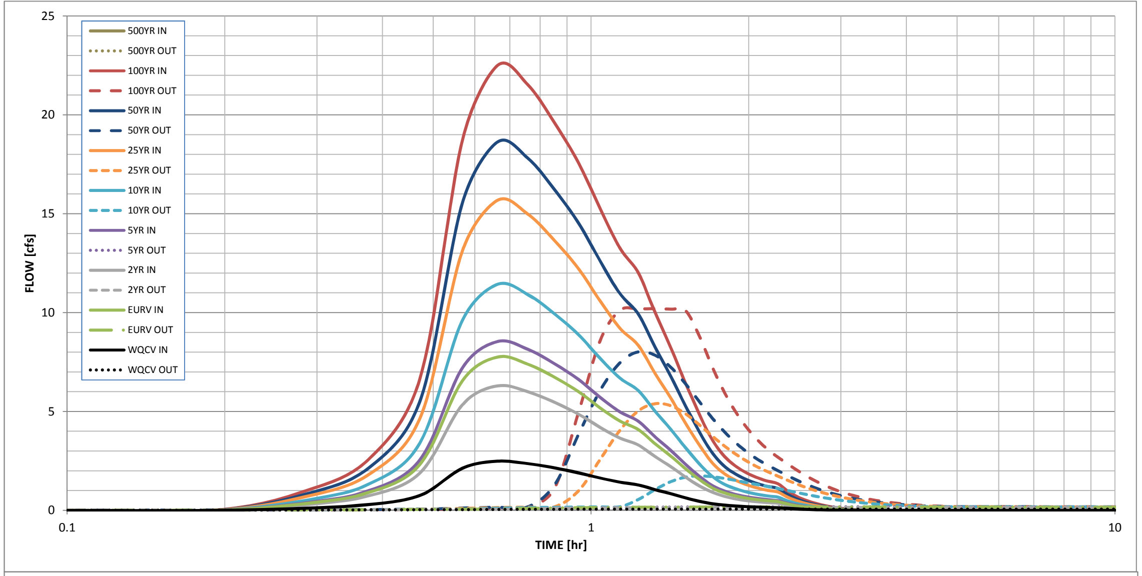
Spillway Design Flow Depth=	0.71	feet
Stage at Top of Freeboard =	9.00	feet
Basin Area at Top of Freeboard =	0.66	acres

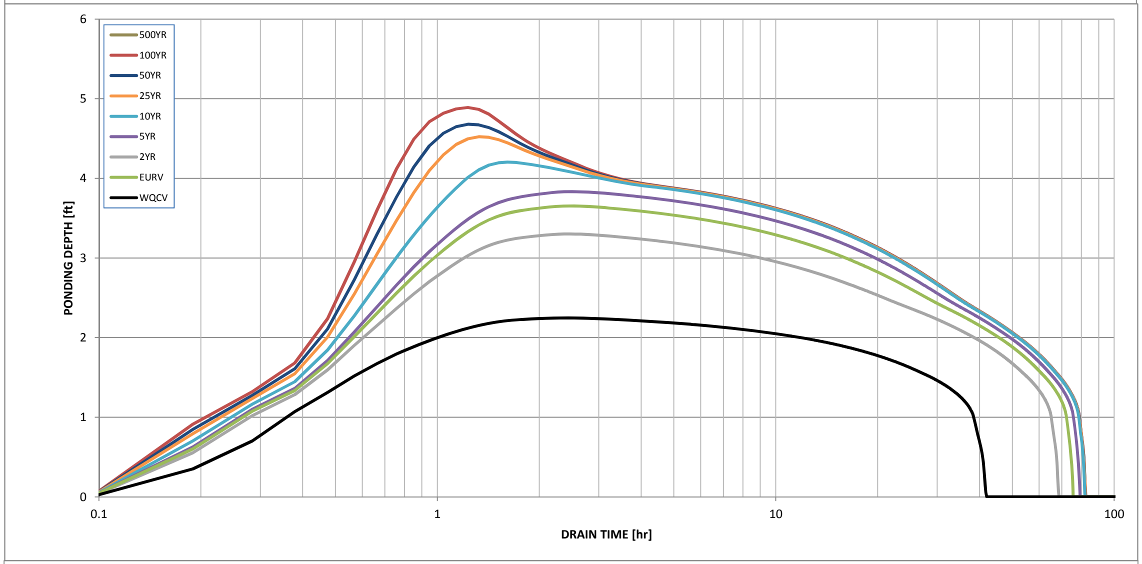
Routed Hydrograph Results

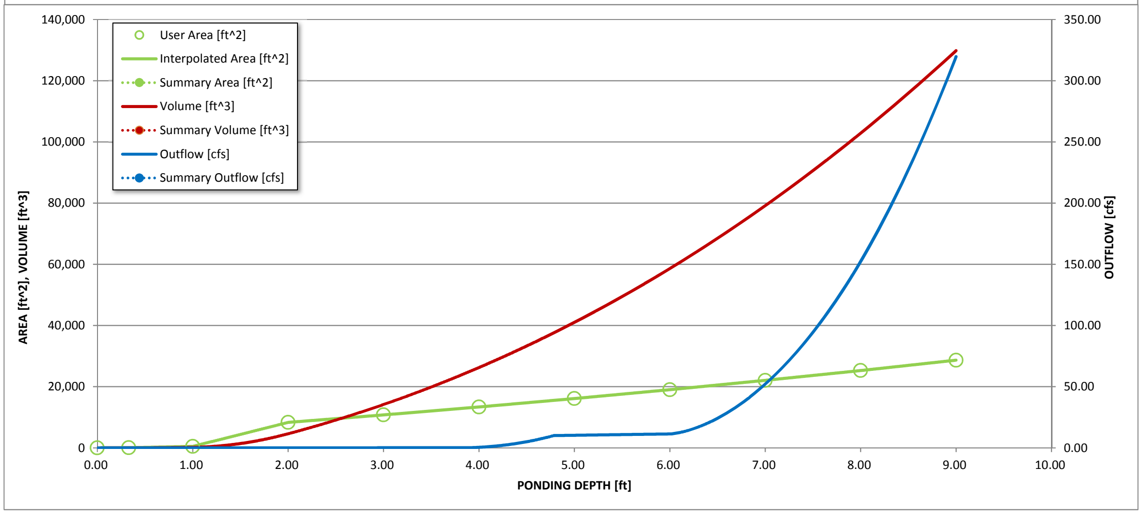
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.168	0.530	0.429	0.584	0.783	1.078	1.284	1.554	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.167	0.529	0.429	0.583	0.784	1.079	1.285	1.555	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.02	0.17	0.57	0.80	1.08	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	0.1	0.2	1.6	5.5	7.6	10.2	0.0
Peak Inflow Q (cfs) =	2.5	7.8	6.3	8.5	11.4	15.7	18.6	22.5	#N/A
Peak Outflow Q (cfs) =	0.1	0.2	0.2	0.2	1.7	5.4	8.0	10.2	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.1	1.1	1.0	1.1	1.0	#N/A
Structure Controlling Flow =	Plate	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	#N/A
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	0.1	0.4	0.7	0.9	#N/A
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) =	38	68	62	71	72	69	68	66	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**40**	72	65	75	77	77	76	75	#N/A
Maximum Ponding Depth (ft) =	2.25	3.65	3.30	3.83	4.20	4.52	4.68	4.89	#N/A
Area at Maximum Ponding Depth (acres) =	0.21	0.29	0.27	0.30	0.32	0.34	0.35	0.36	#N/A
Maximum Volume Stored (acre-ft) =	0.154	0.499	0.403	0.552	0.666	0.771	0.827	0.902	#N/A

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)







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

Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs UD-Detention, Version 3.07 (February 2017)

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

	SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	#N/A
Time Interval	TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
5.68 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:05:41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Hydrograph Constant	0:11:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:17:02	0.11	0.35	0.28	0.38	0.50	0.69	0.81	0.97	#N/A
0.881	0:22:43	0.30	0.93	0.75	1.02	1.36	1.86	2.20	2.65	#N/A
	0:28:24	0.77	2.38	1.94	2.62	3.50	4.77	5.66	6.81	#N/A
	0:34:05	2.13	6.55	5.33	7.20	9.60	13.11	15.54	18.71	#N/A
	0:39:46	2.49	7.75	6.30	8.54	11.43	15.68	18.63	22.50	#N/A
	0:45:26	2.36	7.39	6.00	8.15	10.92	14.99	17.81	21.53	#N/A
	0:51:07	2.15	6.73	5.46	7.42	9.94	13.64	16.22	19.60	#N/A
	0:56:48	1.90	6.01	4.87	6.62	8.88	12.22	14.53	17.57	#N/A
	1:02:29	1.63	5.18	4.19	5.71	7.68	10.58	12.60	15.25	#N/A
	1:08:10	1.42	4.51	3.66	4.98	6.69	9.21	10.95	13.25	#N/A
	1:13:50	1.29	4.09	3.31	4.51	6.06	8.35	9.93	12.02	#N/A
	1:19:31	1.05	3.37	2.72	3.72	5.01	6.92	8.25	10.00	#N/A
	1:25:12	0.84	2.75	2.21	3.03	4.10	5.68	6.78	8.23	#N/A
	1:30:53	0.63	2.11	1.69	2.33	3.17	4.41	5.28	6.43	#N/A
	1:36:34	0.46	1.56	1.25	1.73	2.37	3.32	3.99	4.88	#N/A
	1:42:14	0.34	1.14	0.91	1.26	1.71	2.40	2.90	3.57	#N/A
	1:47:55	0.27	0.88	0.71	0.97	1.32	1.85	2.22	2.72	#N/A
	1:53:36	0.22	0.73	0.58	0.80	1.09	1.52	1.82	2.22	#N/A
	1:59:17	0.19	0.62	0.50	0.68	0.92	1.28	1.54	1.88	#N/A
	2:04:58	0.17	0.54	0.44	0.60	0.81	1.13	1.35	1.64	#N/A
	2:10:38	0.15	0.49	0.39	0.54	0.73	1.01	1.21	1.47	#N/A
	2:16:19	0.14	0.45	0.36	0.50	0.67	0.93	1.11	1.35	#N/A
	2:22:00	0.10	0.33	0.27	0.36	0.49	0.68	0.82	1.00	#N/A
	2:27:41	0.07	0.24	0.20	0.27	0.36	0.50	0.60	0.73	#N/A
	2:33:22	0.05	0.18	0.14	0.20	0.26	0.37	0.44	0.54	#N/A
	2:39:02	0.04	0.13	0.10	0.14	0.20	0.27	0.33	0.40	#N/A
	2:44:43	0.03	0.09	0.07	0.10	0.14	0.20	0.23	0.29	#N/A
	2:50:24	0.02	0.07	0.05	0.07	0.10	0.14	0.17	0.20	#N/A
	2:56:05	0.01	0.05	0.04	0.05	0.07	0.10	0.12	0.15	#N/A
	3:01:46	0.01	0.03	0.02	0.03	0.05	0.07	0.08	0.10	#N/A
	3:07:26	0.00	0.02	0.01	0.02	0.03	0.04	0.05	0.06	#N/A
	3:13:07	0.00	0.01	0.01	0.01	0.01	0.02	0.03	0.03	#N/A
	3:18:48	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	#N/A
	3:24:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:30:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:35:50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:41:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:47:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:52:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:58:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:04:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:09:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:15:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:21:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:26:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:32:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:38:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:44:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:49:41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:55:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:01:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:06:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:12:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:18:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:23:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:29:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:35:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:40:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:46:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:52:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:57:50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:03:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:09:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:14:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:20:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:26:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:31:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:37:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:43:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:48:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A

Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.07, March 2018)

Sheet 1 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **June 28, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond CR2**

1. Basin Storage Volume

A) Effective Imperviousness of Tributary Area, I_a — I_a = 52.0 %

B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) — i = 0.520

C) Contributing Watershed Area — Area = 9.500 ac

D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm — d_6 = in

E) Design Concept
(Select EURV when also designing for flood control)

Choose One
● Water Quality Capture Volume (WQCV)
○ Excess Urban Runoff Volume (EURV)

F) Design Volume (WQCV) Based on 40-hour Drain Time
($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area$) — V_{DESIGN}= 0.168 ac-ft

G) For Watersheds Outside of the Denver Region,
Water Quality Capture Volume (WQCV) Design Volume
($V_{WQCV\ OTHER} = (d_6*(V_{DESIGN}/0.43))$ — $V_{DESIGN\ OTHER}$= ac-ft

H) User Input of Water Quality Capture Volume (WQCV) Design Volume
(Only if a different WQCV Design Volume is desired) — $V_{DESIGN\ USER}$= ac-ft

I) NRCS Hydrologic Soil Groups of Tributary Watershed
i) Percentage of Watershed consisting of Type A Soils — HSG_A = %
ii) Percentage of Watershed consisting of Type B Soils — HSG_B = %
iii) Percentage of Watershed consisting of Type C/D Soils — $HSG_{C/D}$ = %

J) Excess Urban Runoff Volume (EURV) Design Volume
For HSG A: $EURV_A = 1.68 * i^{1.28}$
For HSG B: $EURV_B = 1.36 * i^{1.08}$
For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$ — $EURV_{DESIGN}$ = ac-f t

K) User Input of Excess Urban Runoff Volume (EURV) Design Volume
(Only if a different EURV Design Volume is desired) — $EURV_{DESIGN\ USER}$= ac-f t

2. Basin Shape: Length to Width Ratio
(A basin length to width ratio of at least 2:1 will improve TSS reduction.) — L : W = 2.0 : 1

3. Basin Side Slopes

A) Basin Maximum Side Slopes
(Horizontal distance per unit vertical, 4:1 or flatter preferred) — Z = 4.00 ft / ft

4. Inlet

A) Describe means of providing energy dissipation at concentrated inflow locations:

5. Forebay

A) Minimum Forebay Volume
(V_{FMIN} = 2% of the WQCV) — V_{FMIN} = 0.003 ac-ft

B) Actual Forebay Volume — V_F = 0.004 ac-ft

C) Forebay Depth
(D_F = 18 inch maximum) — D_F = 18.0 in

D) Forebay Discharge

i) Undetained 100-year Peak Discharge — Q_{100} = 22.50 cfs

ii) Forebay Discharge Design Flow
($Q_F = 0.02 * Q_{100}$) — Q_F = 0.45 cfs

E) Forebay Discharge Design

Choose One
○ Berm With Pipe
● Wall with Rect. Notch
○ Wall with V-Notch Weir

Flow too small for berm w/ pipe

F) Discharge Pipe Size (minimum 8-inches) — Calculated D_P = in

G) Rectangular Notch Width — Calculated W_N = 4.5 in

Design Procedure Form: Extended Detention Basin (EDB)

Sheet 2 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **June 28, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond CR2**

6. Trickle Channel	
A) Type of Trickle Channel	Choose One ● Concrete ○ Soft Bottom
F) Slope of Trickle Channel	S = 0.0100 ft / ft
7. Micropool and Outlet Structure	
A) Depth of Micropool (2.5-feet minimum)	D_M = 2.5 ft
B) Surface Area of Micropool (10 ft^2 minimum)	A_M = 56 sq ft
C) Outlet Type	Choose One ● Orifice Plate ○ Other (Describe):
D) Smallest Dimension of Orifice Opening Based on Hydrograph Routing (Use UD-Detention)	$D_{orifice}$ = 0.57 inches
E) Total Outlet Area	A_{ot} = 1.71 square inches
8. Initial Surcharge Volume	
A) Depth of Initial Surcharge Volume (Minimum recommended depth is 4 inches)	D_{IS} = 4 in
B) Minimum Initial Surcharge Volume (Minimum volume of 0.3% of the WQCV)	V_{IS} = cu ft
C) Initial Surcharge Provided Above Micropool	V_s= 18.7 cu ft
9. Trash Rack	
A) Water Quality Screen Open Area: $A_t = A_{ot} * 38.5*(e^{-0.095D})$	A_t = 62 square inches
B) Type of Screen (If specifying an alternative to the materials recommended in the USDCM, indicate "other" and enter the ratio of the total open are to the total screen are for the material specified.)	*Other (Please describe below)* wellscreen stainless
Other (Y/N): y	
C) Ratio of Total Open Area to Total Area (only for type 'Other')	User Ratio = 0.6
D) Total Water Quality Screen Area (based on screen type)	A_{total} = 104 sq. in. **Based on type 'Other' screen ratio**
E) Depth of Design Volume (EURV or WQCV) (Based on design concept chosen under 1E)	H= 2.23 feet
F) Height of Water Quality Screen (H_{TR})	H_{TR}= 54.76 inches
G) Width of Water Quality Screen Opening ($W_{opening}$) (Minimum of 12 inches is recommended)	$W_{opening}$ = 12.0 inches **VALUE LESS THAN RECOMMENDED MIN. WIDTH. WIDTH HAS BEEN SET TO 12 INCHES.**

Design Procedure Form: Extended Detention Basin (EDB)

Sheet 3 of 3

Designer: **Richard Schindler**
Company: **Core Engineering Group**
Date: **June 28, 2018**
Project: **Creekside at Lorson Ranch Filing No. 1**
Location: **Pond CR2**

10. Overflow Embankment

A) Describe embankment protection for 100-year and greater overtopping:

B) Slope of Overflow Embankment
(Horizontal distance per unit vertical, 4:1 or flatter preferred)

Ze = ______ ft / ft

11. Vegetation

Choose One
○ Irrigated
○ Not Irrigated

12. Access

A) Describe Sediment Removal Procedures

Notes:

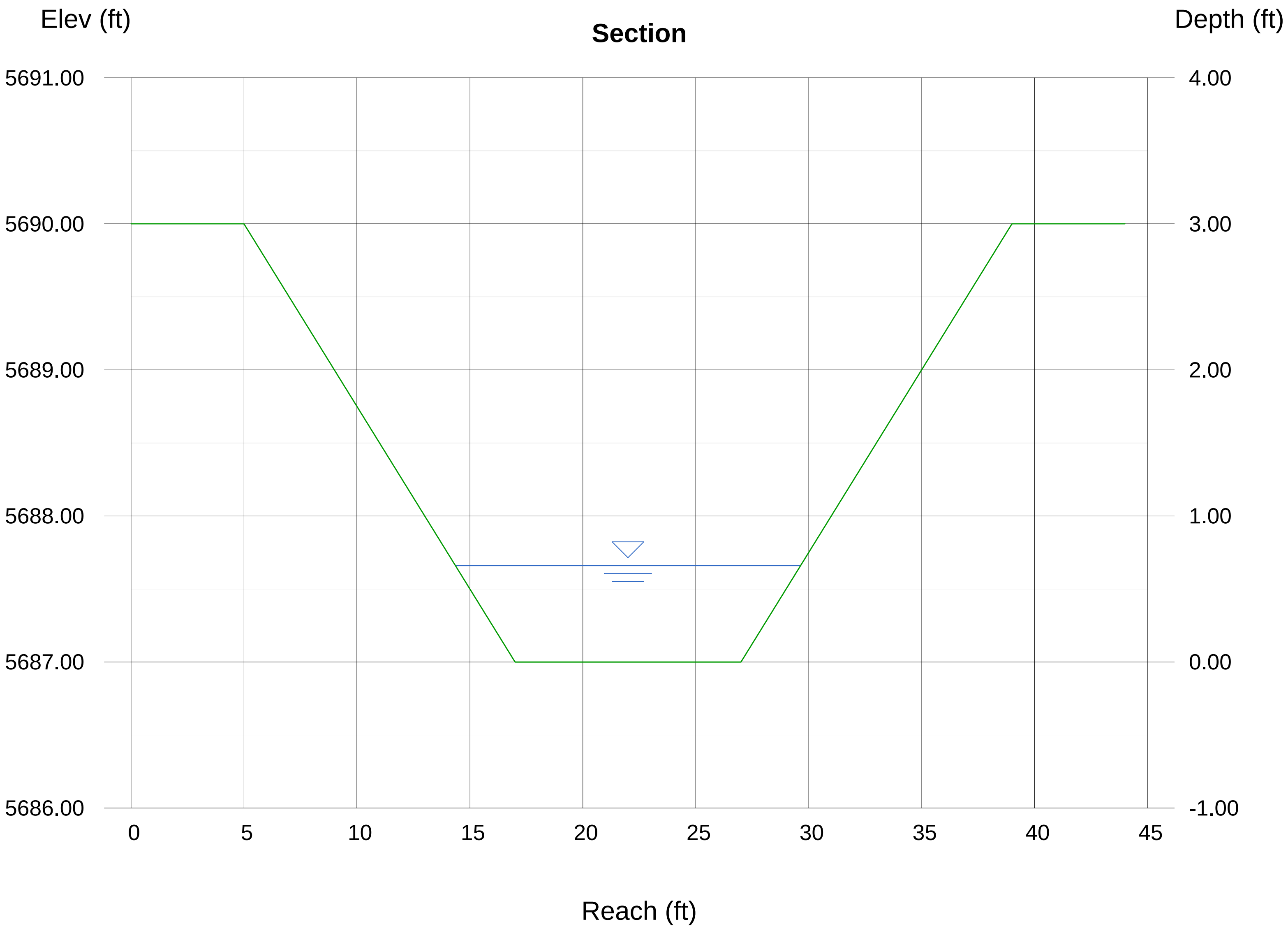
Channel Report

POND CR2 OVERFLOW CHANNEL

Trapezoidal
Botom Width (ft) = 10.00
Side Slope (z:1) = 4.00
Total Depth (ft) = 3.00
Invert Elev (ft) = 5687.00
Slope (%) = 0.50
N-Value = 0.025

Calculations
Compute by: Known Q
Known Q (cfs) = 23.00

Highlighted
Depth (ft) = 0.66
Q (cfs) = 23.00
Area (sqft) = 8.34
Velocity (ft/s) = 2.76
Wetted Perim (ft) = 15.44
Crit Depth, Yc (ft) = 0.52
Top Width (ft) = 15.28
EGL (ft) = 0.78

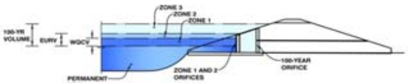


DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch

Basin ID: Pond CR3



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type = **SF**
Watershed Area = 1.60 acres
Watershed Length = 300 ft
Watershed Slope = 0.025 ft/ft
Watershed Imperviousness = 50.00% percent
Percentage Hydrologic Soil Group A = 0.0% percent
Percentage Hydrologic Soil Group B = 100.0% percent
Percentage Hydrologic Soil Groups C/D = 0.0% percent
Desired WQCV Drain Time = 12.0 hours
Location for 1-hr Rainfall Depths = User Input
Water Quality Capture Volume (WQCV) = 0.022 acre-feet
Excess Urban Runoff Volume (EURV) = 0.086 acre-feet

Optional User Override
1-hr Precipitation

2-yr Runoff Volume (P1 = 1.19 in.) = 0.069 acre-feet | 1.19 inches
5-yr Runoff Volume (P1 = 1.5 in.) = 0.094 acre-feet | 1.50 inches
10-yr Runoff Volume (P1 = 1.75 in.) = 0.127 acre-feet | 1.75 inches
25-yr Runoff Volume (P1 = 2 in.) = 0.178 acre-feet | 2.00 inches
50-yr Runoff Volume (P1 = 2.25 in.) = 0.212 acre-feet | 2.25 inches
100-yr Runoff Volume (P1 = 2.52 in.) = 0.258 acre-feet | 2.52 inches
500-yr Runoff Volume (P1 = 0 in.) = 0.000 acre-feet | inches
Approximate 2-yr Detention Volume = 0.065 acre-feet
Approximate 5-yr Detention Volume = 0.088 acre-feet
Approximate 10-yr Detention Volume = 0.117 acre-feet
Approximate 25-yr Detention Volume = 0.128 acre-feet
Approximate 50-yr Detention Volume = 0.134 acre-feet
Approximate 100-yr Detention Volume = 0.150 acre-feet

Stage-Storage Calculation

Zone 1 Volume (WQCV) = 0.022 acre-feet
Zone 2 Volume (EURV - Zone 1) = 0.064 acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) = 0.075 acre-feet
Total Detention Basin Volume = 0.161 acre-feet
Initial Surcharge Volume (ISV) = N/A ft^3
Initial Surcharge Depth (ISD) = N/A ft
Total Available Detention Depth (H_{total}) = 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_{main}) = user H:V
Basin Length-to-Width Ratio ($R_{L/W}$) = user

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

Depth Increment = 0.1 ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Media Surface	--	0.00	--	--	--	492	0.011		
5685	--	1.00	--	--	--	968	0.022	720	0.017
5686	--	2.00	--	--	--	1,571	0.036	1,984	0.046
5687	--	3.00	--	--	--	2,305	0.053	3,937	0.090
5688	--	4.00	--	--	--	3,296	0.076	6,738	0.155
5689	--	5.00	--	--	--	4,270	0.098	10,521	0.242
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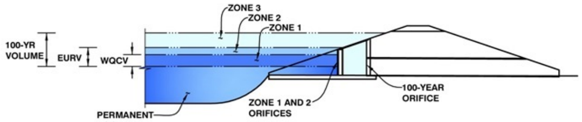
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Creekside at Lorson Ranch

Basin ID: Pond CR3



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	1.23	0.022	Filtration Media
Zone 2 (EURV)	2.91	0.064	Rectangular Orifice
(100+1/2WQCV)	4.08	0.075	Weir&Pipe (Circular)
		0.161	Total

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

Underdrain Orifice Invert Depth =	1.97	ft (distance below the filtration media surface)
Underdrain Orifice Diameter =	0.69	inches

Calculated Parameters for Underdrain

Underdrain Orifice Area =	0.0	ft^2
Underdrain Orifice Centroid =	0.03	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)

Invert of Lowest Orifice =	N/A	ft (relative to basin bottom at Stage = 0 ft)
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
Orifice Plate: Orifice Area per Row =	N/A	inches

Calculated Parameters for Plate

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

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

	Row 1 (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	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

	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)

	Zone 2 Rectangular	Not Selected	
Invert of Vertical Orifice =	1.23	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	2.91	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	1.50	N/A	inches
Vertical Orifice Width =	0.25		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.00	N/A	ft^2
Vertical Orifice Centroid =	0.06	N/A	feet

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	3.30	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	3.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	3.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	3.30	N/A	feet
Over Flow Weir Slope Length =	3.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	50.13	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	6.30	N/A	ft^2
Overflow Grate Open Area w/ Debris =	3.15	N/A	ft^2

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

	Zone 3 Circular	Not Selected	
Depth to Invert of Outlet Pipe =	2.30	N/A	ft (distance below basin bottom at Stage = 0 ft)
Circular Orifice Diameter =	4.80	N/A	inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Circular	Not Selected	
Outlet Orifice Area =	0.13	N/A	ft^2
Outlet Orifice Centroid =	0.20	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	N/A	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	4.50	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	6.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	0.40	feet

Calculated Parameters for Spillway

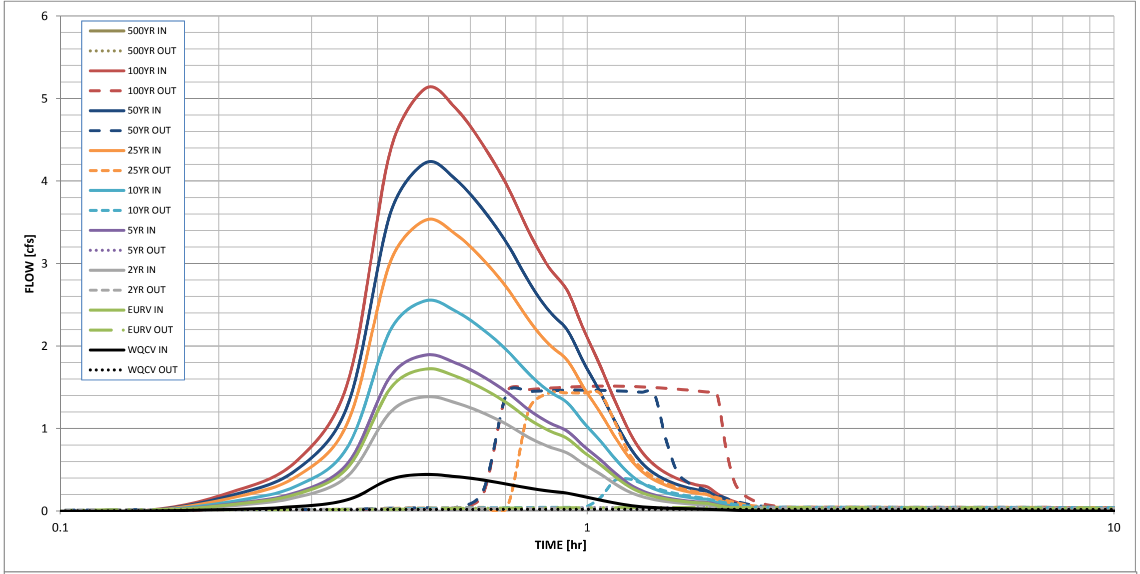
Spillway Design Flow Depth=	0.38	feet
Stage at Top of Freeboard =	5.28	feet
Basin Area at Top of Freeboard =	0.10	acres

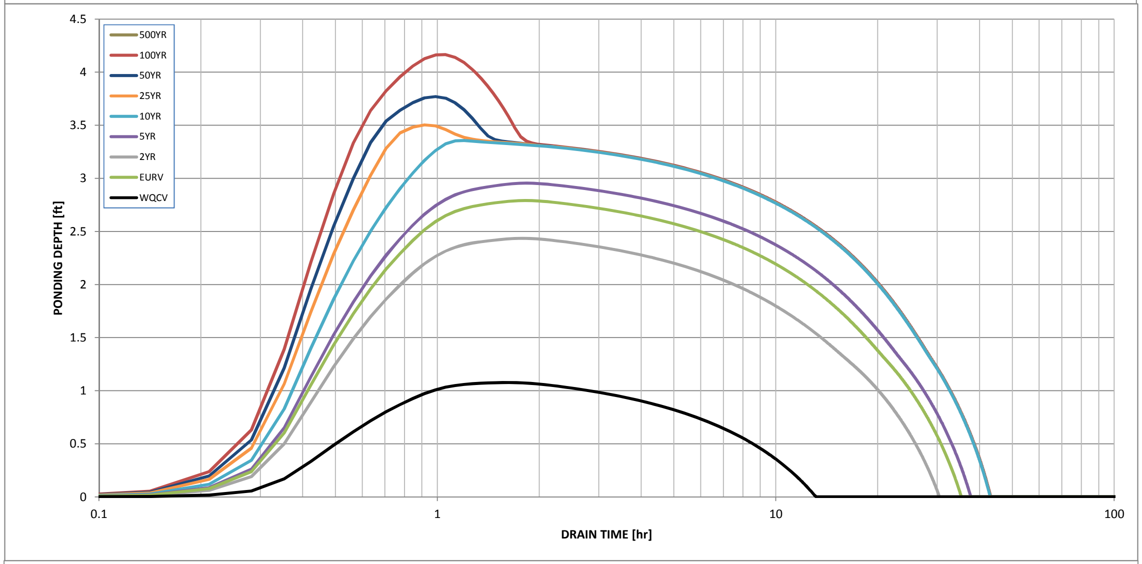
Routed Hydrograph Results

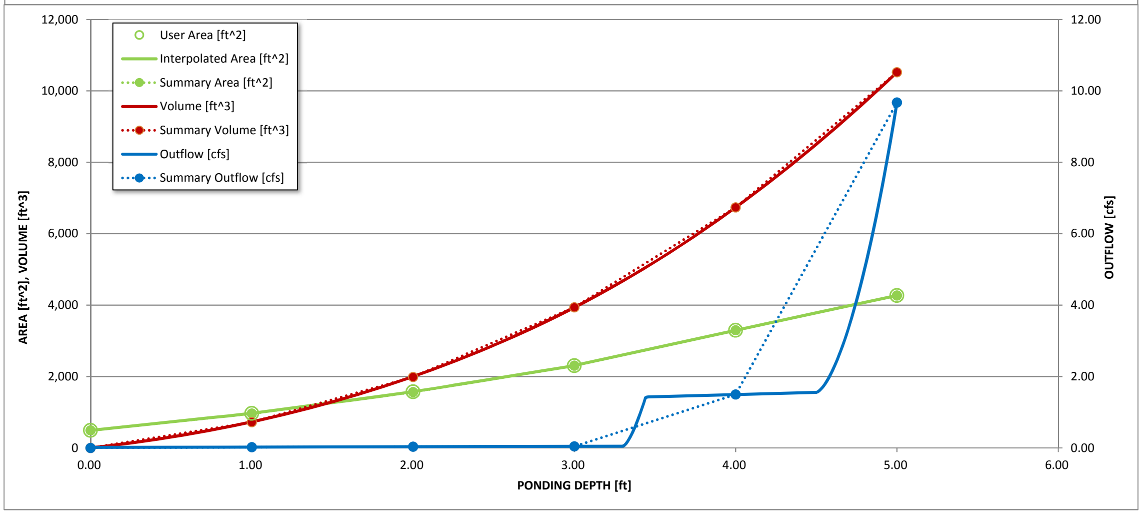
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.022	0.086	0.069	0.094	0.127	0.178	0.212	0.258	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.021	0.085	0.068	0.094	0.127	0.177	0.212	0.258	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.02	0.03	0.26	0.81	1.12	1.50	0.00
Predevelopment Peak Q (cfs) =	0.00	0.00	0.02	0.041	0.4	1.3	1.8	2.4	0.0
Peak Inflow Q (cfs) =	0.4	1.7	1.4	1.9	2.5	3.5	4.2	5.1	#N/A
Peak Outflow Q (cfs) =	0.02	0.04	0.04	0.04	0.4	1.4	1.5	1.5	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.1	1.0	1.1	0.8	0.6	#N/A
Structure Controlling Flow =	Filtration Media	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Outlet Plate 1	#N/A
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	0.1	0.2	0.2	0.2	#N/A
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) =	13	34	29	36	41	40	39	38	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**13**	35	30	37	42	42	42	42	#N/A
Maximum Ponding Depth (ft) =	1.08	2.79	2.44	2.95	3.36	3.50	3.77	4.17	#N/A
Area at Maximum Ponding Depth (acres) =	0.02	0.05	0.04	0.05	0.06	0.06	0.07	0.08	#N/A
Maximum Volume Stored (acre-ft) =	0.018	0.080	0.063	0.088	0.110	0.120	0.138	0.167	#N/A

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)







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

Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs UD-Detention, Version 3.07 (February 2017)

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

Time Interval
4.23 min

Hydrograph Constant
1.182

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	#N/A
TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:04:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:08:28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:12:41	0.02	0.08	0.06	0.09	0.12	0.16	0.19	0.23	#N/A
0:16:55	0.05	0.21	0.17	0.23	0.31	0.43	0.51	0.62	#N/A
0:21:09	0.14	0.54	0.43	0.59	0.80	1.10	1.31	1.59	#N/A
0:25:23	0.39	1.49	1.20	1.63	2.19	3.02	3.61	4.37	#N/A
0:29:37	0.44	1.72	1.38	1.89	2.55	3.53	4.22	5.12	#N/A
0:33:50	0.42	1.63	1.31	1.79	2.42	3.35	4.02	4.88	#N/A
0:38:04	0.38	1.48	1.19	1.63	2.20	3.05	3.65	4.44	#N/A
0:42:18	0.33	1.31	1.05	1.44	1.95	2.70	3.24	3.94	#N/A
0:46:32	0.28	1.11	0.89	1.23	1.66	2.31	2.78	3.38	#N/A
0:50:46	0.24	0.98	0.78	1.07	1.45	2.02	2.43	2.96	#N/A
0:54:59	0.22	0.88	0.70	0.97	1.31	1.83	2.20	2.67	#N/A
0:59:13	0.17	0.71	0.57	0.78	1.07	1.49	1.79	2.19	#N/A
1:03:27	0.14	0.57	0.45	0.63	0.86	1.20	1.45	1.77	#N/A
1:07:41	0.10	0.42	0.33	0.47	0.64	0.90	1.09	1.34	#N/A
1:11:55	0.07	0.30	0.24	0.33	0.46	0.66	0.80	0.98	#N/A
1:16:08	0.05	0.22	0.18	0.25	0.34	0.48	0.59	0.72	#N/A
1:20:22	0.04	0.18	0.14	0.20	0.27	0.38	0.46	0.56	#N/A
1:24:36	0.04	0.15	0.12	0.16	0.22	0.31	0.38	0.46	#N/A
1:28:50	0.03	0.13	0.10	0.14	0.19	0.27	0.32	0.40	#N/A
1:33:04	0.03	0.11	0.09	0.12	0.17	0.24	0.28	0.35	#N/A
1:37:17	0.02	0.10	0.08	0.11	0.15	0.21	0.26	0.31	#N/A
1:41:31	0.02	0.09	0.08	0.10	0.14	0.20	0.24	0.29	#N/A
1:45:45	0.02	0.07	0.06	0.08	0.10	0.15	0.17	0.21	#N/A
1:49:59	0.01	0.05	0.04	0.06	0.08	0.11	0.13	0.16	#N/A
1:54:13	0.01	0.04	0.03	0.04	0.06	0.08	0.09	0.11	#N/A
1:58:26	0.01	0.03	0.02	0.03	0.04	0.06	0.07	0.08	#N/A
2:02:40	0.00	0.02	0.01	0.02	0.03	0.04	0.05	0.06	#N/A
2:06:54	0.00	0.01	0.01	0.01	0.02	0.03	0.03	0.04	#N/A
2:11:08	0.00	0.01	0.01	0.01	0.01	0.02	0.02	0.03	#N/A
2:15:22	0.00	0.01	0.00	0.01	0.01	0.01	0.01	0.02	#N/A
2:19:35	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	#N/A
2:23:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:28:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:32:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:36:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:40:44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:44:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:49:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:53:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:57:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:01:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:06:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:10:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:14:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:18:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:23:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:27:16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:31:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:35:44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:39:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:44:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:48:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:52:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:56:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:01:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:05:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:09:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:13:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:18:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:22:16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:26:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:30:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:34:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:39:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:43:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:47:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:51:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:56:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:00:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:04:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

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 [ft^2]	Area [acres]	Volume [ft^3]	Volume [ac-ft]	Total Outflow [cfs]
	0.00	492	0.011	0	0.000	0.00
	1.00	963	0.022	720	0.017	0.02
	2.00	1,565	0.036	1,984	0.046	0.04
	3.00	2,305	0.053	3,937	0.090	0.04
	4.00	3,296	0.076	6,738	0.155	1.49
	5.00	4,270	0.098	10,521	0.242	9.67

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'.

Also include the inverts of all outlets (e.g. vertical orifice, overflow grate, and spillway, where applicable).

Design Procedure Form: Sand Filter (SF)

UD-BMP (Version 3.07, March 2018)

Sheet 1 of 2

Designer: **Richard Schindler**
Company: **Core Engineering**
Date: **July 10, 2018**
Project: **Creekside**
Location: **Pond CR3**

1. Basin Storage Volume

A) Effective Imperviousness of Tributary Area, I_a (100% if all paved and roofed areas upstream of sand filter)	I_a =	50.0	%
B) Tributary Area's Imperviousness Ratio ($i = I_a/100$)	i =	0.500	
C) Water Quality Capture Volume (WQCV) Based on 12-hour Drain Time $WQCV= 0.8 * (0.91* i^3 - 1.19 * i^2 + 0.78 * i)$	WQCV =	0.17	watershed inches
D) Contributing Watershed Area (including sand filter area)	Area =	69,696	sq ft
E) Water Quality Capture Volume (WQCV) Design Volume V_{WQCV} = WQCV / 12 * Area	V_{WQCV} =	958	cu ft
F) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 =		in
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume	$V_{WQCV\ OTHER}$ =		cu ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{WQCV\ USER}$ =		cu ft

2. Basin Geometry

A) WQCV Depth	D_{WQCV} =	1.23	ft
B) Sand Filter Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred). Use "0" if sand filter has vertical walls.	Z =	4.00	ft / ft
C) Minimum Filter Area (Flat Surface Area)	A_{Min} =	436	sq ft
D) Actual Filter Area	A_{Actual} =	492	sq ft
E) Volume Provided	V_T =	960	cu ft

3. Filter Material

Choose One
● 18" CDOT Class B or C Filter Material
○ Other (Explain):

4. Underdrain System

A) Are underdrains provided?

Choose One
● YES
○ NO

B) Underdrain system orifice diameter for 12 hour drain time

i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y =	1.8	ft
ii) Volume to Drain in 12 Hours	Vol_{12} =	958	cu ft
iii) Orifice Diameter, 3/8" Minimum	D_O =	3/4	in

Design Procedure Form: Sand Filter (SF)

Sheet 2 of 2

Designer: **Richard Schindler**
Company: **Core Engineering**
Date: **July 10, 2018**
Project: **Creekside**
Location: **Pond CR3**

5. Impermeable Geomembrane Liner and Geotextile Separator Fabric

A) Is an impermeable liner provided due to proximity of structures or groundwater contamination?

Choose One
○ YES ● NO

6. Inlet / Outlet Works

A) Describe the type of energy dissipation at inlet points and means of conveying flows in excess of the WQCV through the outlet

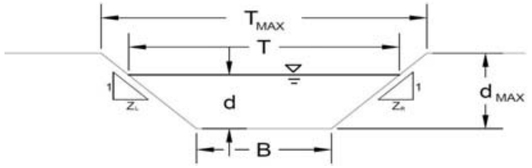
Notes:

Version 4.05 Released March 2017

AREA INLET IN A SWALE

Creekside

Pond CR3 type D Emergency Overflow Structure



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)	A, B, C, D or E	A	
Manning's n (Leave cell D16 blank to manually enter an n value)	n =	see details below	
Channel Invert Slope	S_O =	0.0050	ft/ft
Bottom Width	B =	27.00	ft
Left Side Slope	Z1 =	4.00	ft/ft
Right Side Slope	Z2 =	4.00	ft/ft

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

Choose One:
- ● Non-Cohesive
- ○ Cohesive
- ○ Paved

		Minor Storm	Major Storm	
Max. Allowable Top Width of Channel for Minor & Major Storm	T_{MAX} =	**60.00**	**70.00**	**feet**
Max. Allowable Water Depth in Channel for Minor & Major Storm	d_{MAX} =	**0.60**	**0.70**	**feet**

Allowable Channel Capacity Based On Channel Geometry		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	**4.3**	**5.3**	**cfs**
MAJOR STORM Allowable Capacity is based on Depth Criterion	d_{allow} =	**0.60**	**0.70**	**ft**

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow	Q_o =	**1.9**	**5.2**	**cfs**
Water Depth	d =	**0.36**	**0.69**	**feet**

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Version 4.05 Released March 2017

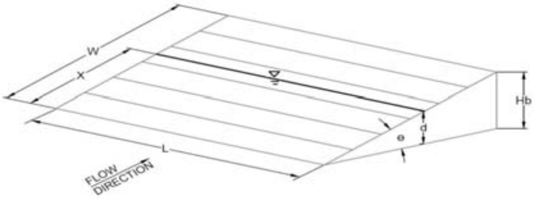
AREA INLET IN A SWALE

Creekside

Pond CR3 type D Emergency Overflow Structure

Inlet Design Information (Input)

Type of Inlet: CDOT TYPE D (Parallel) ▾ Inlet Type = CDOT TYPE D (Parallel)



Angle of Inclined Grate (must be <= 30 degrees)	θ =	0.00	degrees
Width of Grate	W =	6.00	feet
Length of Grate	L =	3.00	feet
Open Area Ratio	A_{RATIO} =	0.70	
Height of Inclined Grate	H_B =	0.00	feet
Clogging Factor	C_f =	0.38	
Grate Discharge Coefficient	C_d =	0.76	
Orifice Coefficient	C_o =	0.50	
Weir Coefficient	C_w =	1.62	

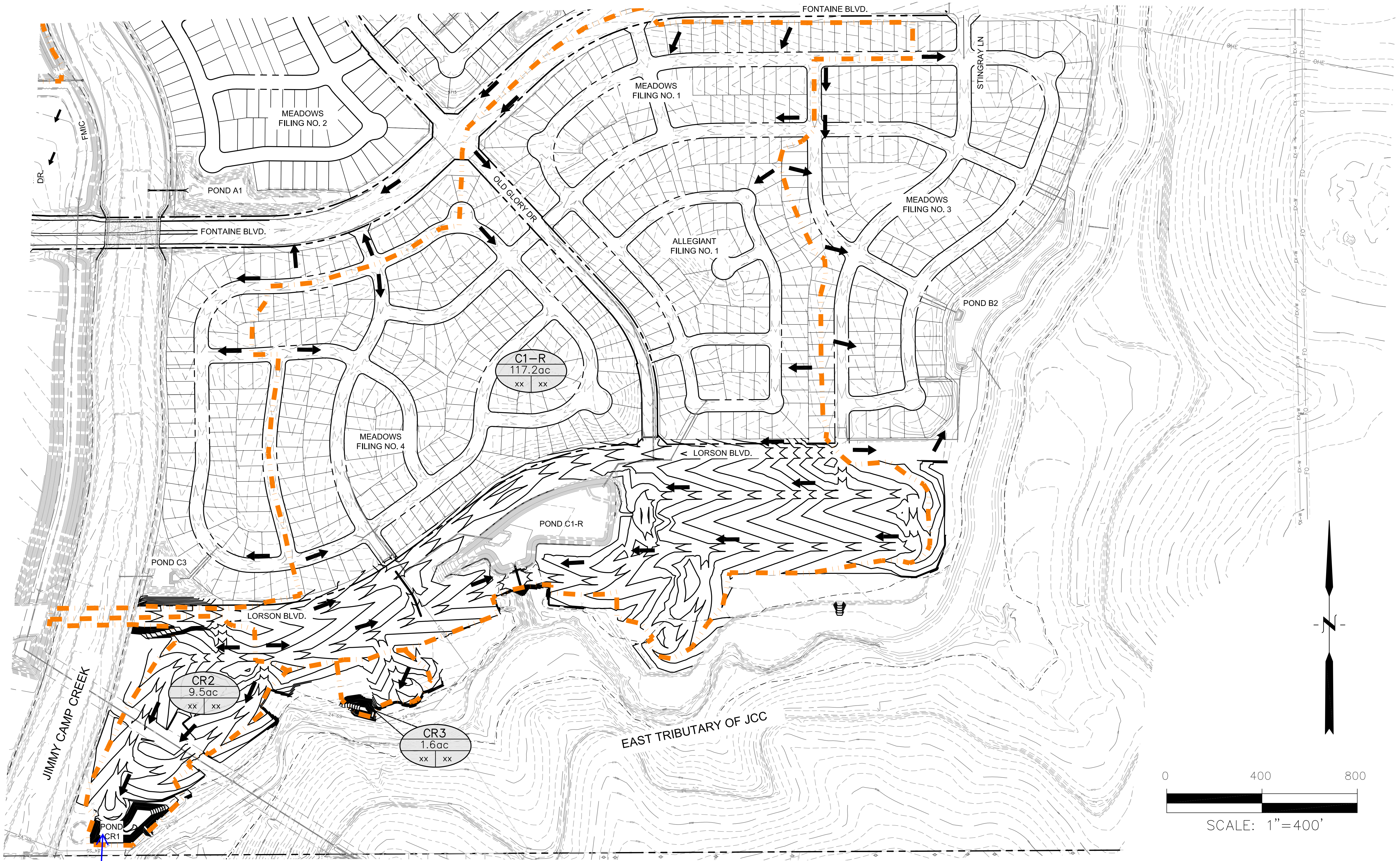
		MINOR	MAJOR	
Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)	d =	0.36	0.69	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**5.5**	**14.7**	**cfs**
	Bypassed Flow, Q_b =	**0.0**	**0.0**	**cfs**
	Capture Percentage = Q_a/Q_o = C%	**100**	**100**	**%**

Warning 02: Depth (d) exceeds USDCM Volume I recommendation.

APPENDIX E– STORM SEWER SCHEMATIC AND HYDRAFLOW STORM SEWER CALCS

APPENDIX F–KIOWA ENGINEERING CHANNEL DESIGN REPORT

MAP
POCKET



Pond CR2

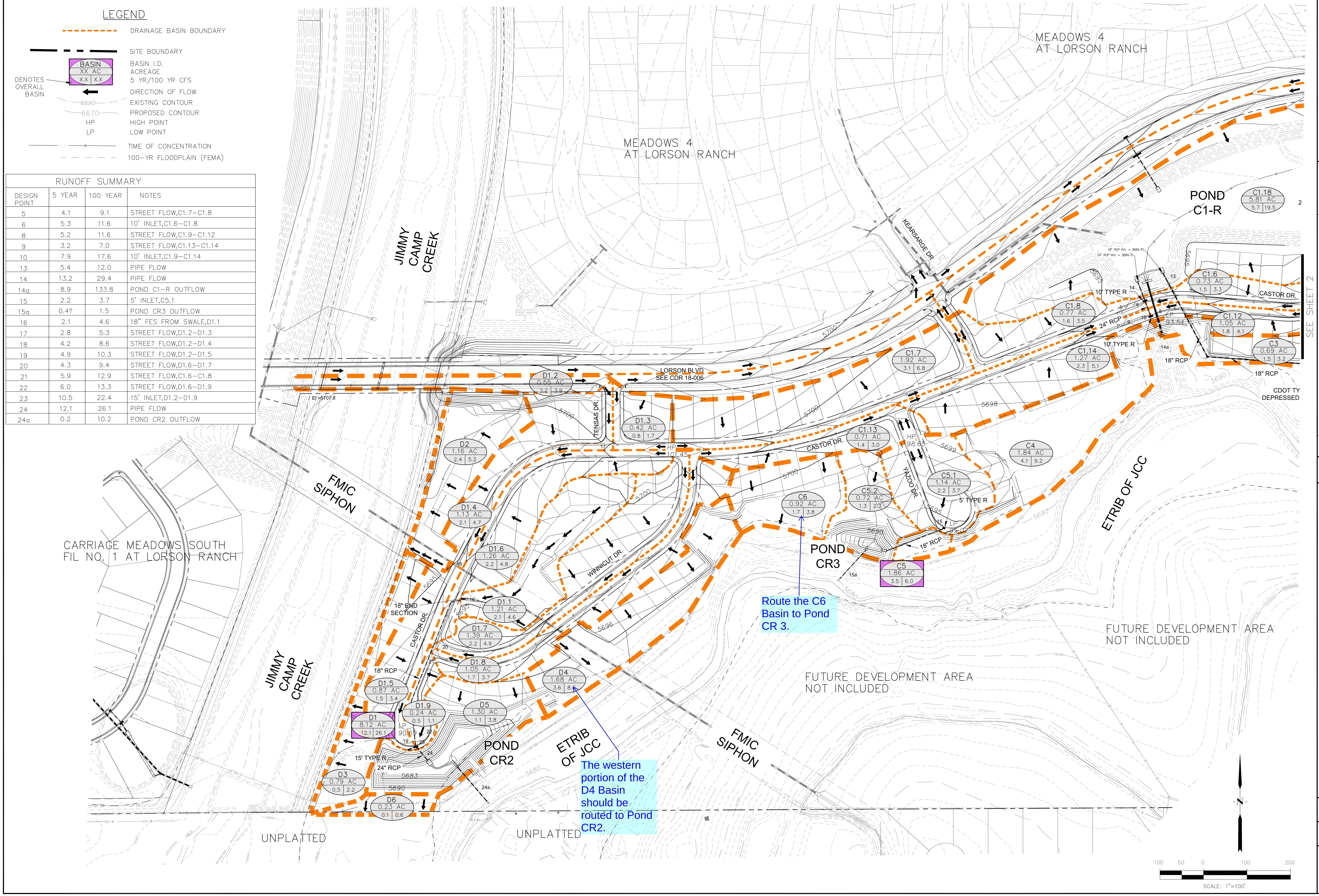
CORE

ENGINEERING GROUP

15004 1ST AVENUE S.
BURNSVILLE, MN 55306
PH: 719.570.1100
CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

CREEKSIDE AT LORSON RANCH FILING NO. 1
WATER QUALITY & POND TRIBUTARY AREAS

SCALE:	DATE:	FIGURE NO.
NTS	AUGUST, 2018	1



Route the C6 Basin to Pond CR 3.

The western portion of the D4 Basin should be routed to Pond CR2.

LEGEND

▬ ▬ ▬	DRAINAGE BASIN BOUNDARY
▬ ▬ ▬	SITE BOUNDARY
BASIN XX AC X.X X.X (DENOTES OVERALL BASIN)	BASIN I.D. ACREAGE 5 YR/100 YR CFS
←	DIRECTION OF FLOW
6690	EXISTING CONTOUR
6670	PROPOSED CONTOUR
HP	HIGH POINT
LP	LOW POINT
→	TIME OF CONCENTRATION
— · —	100-YR FLOODPLAIN (FEMA)

RUNOFF SUMMARY

DESIGN POINT	5 YEAR	100 YEAR	NOTES
5	4.1	9.1	STREET FLOW,C1.7–C1.8
6	5.3	11.6	10' INLET,C1.6–C1.8
8	5.2	11.6	STREET FLOW,C1.9–C1.12
9	3.2	7.0	STREET FLOW,C1.13–C1.14
10	7.9	17.6	10' INLET,C1.9–C1.14
13	5.4	12.0	PIPE FLOW
14	13.2	29.4	PIPE FLOW
14a	8.9	133.8	POND C1–R OUTFLOW
15	2.2	3.7	5' INLET,C5.1
15a	0.4?	1.5	POND CR3 OUTFLOW
16	2.1	4.6	18" FES FROM SWALE,D1.1
17	2.8	5.3	STREET FLOW,D1.2–D1.3
18	4.2	8.6	STREET FLOW,D1.2–D1.4
19	4.9	10.3	STREET FLOW,D1.2–D1.5
20	4.3	9.4	STREET FLOW,D1.6–D1.7
21	5.9	12.9	STREET FLOW,C1.6–C1.8
22	6.0	13.3	STREET FLOW,D1.6–D1.9
23	10.5	22.4	15' INLET,D1.2–D1.9
24	12.1	26.1	PIPE FLOW
24a	0.2	10.2	POND CR2 OUTFLOW

CORE
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NO.	DESCRIPTION	DATE

PREPARED FOR:
LORSON, LLC
212 N. WAHSATCH AVE., SUITE 301
COLORADO SPRINGS, COLORADO 80903
(719) 635-3200
CONTACT: JEFF MARK

PROJECT:
CREEKSIDE FIL NO. 1
LORSON BLVD – OLD GLORY DRIVE
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

DEVELOPED CONDITIONS
CREEKSIDE FILING NO. 1

DATE
AUGUST, 2018

PROJECT NO.
100.045

SHEET NUMBER
1

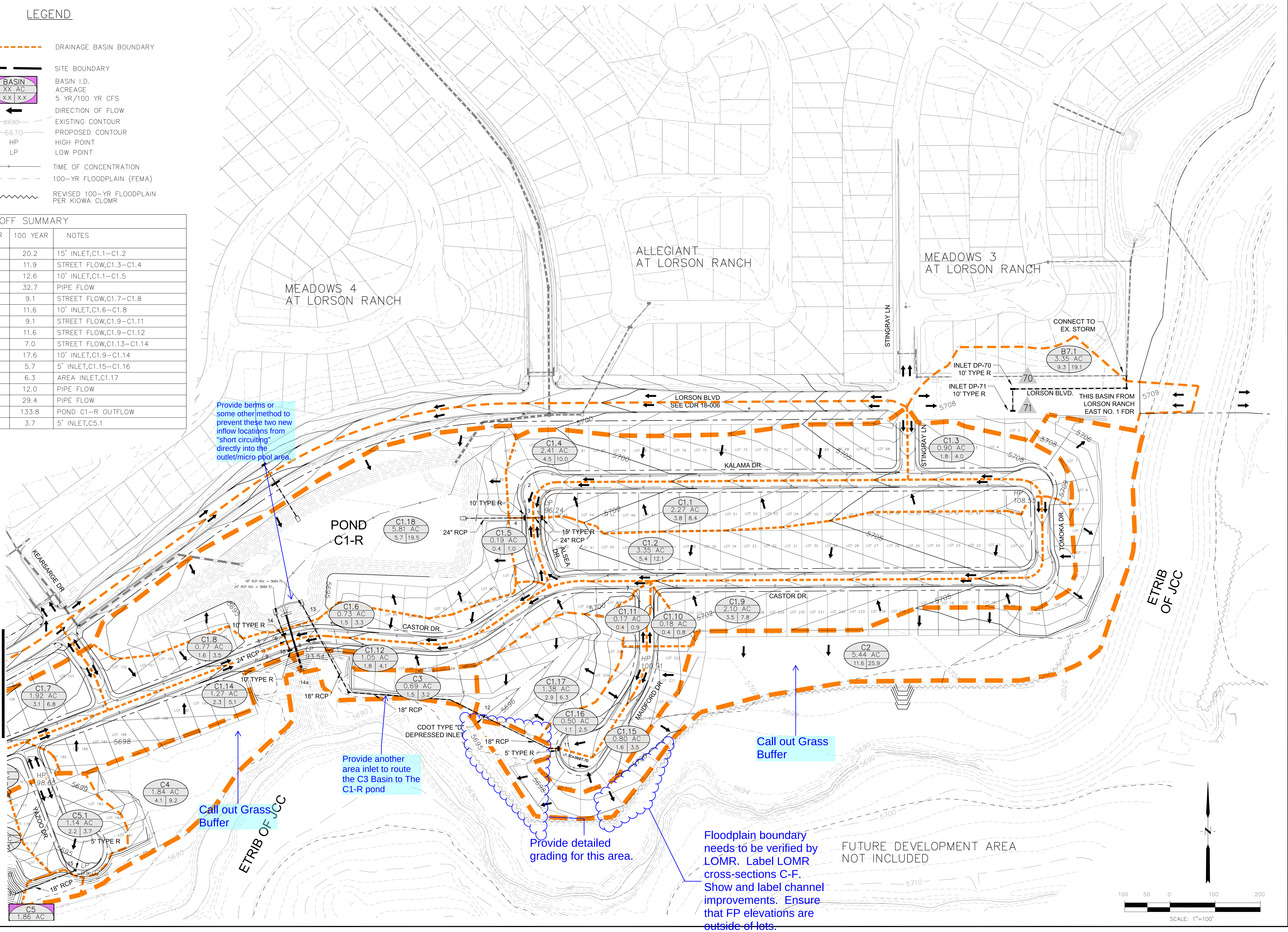
TOTAL SHEETS: 2

LEGEND

DRAINAGE BASIN BOUNDARY
SITE BOUNDARY
BASIN I.D. / ACREAGE / 5 YR/100 YR CFS (BASIN / XX AC / X.X X.X) — DENOTES OVERALL BASIN
DIRECTION OF FLOW
6690 EXISTING CONTOUR
6670 PROPOSED CONTOUR
HP HIGH POINT
LP LOW POINT
TIME OF CONCENTRATION
100–YR FLOODPLAIN (FEMA)
REVISED 100–YR FLOODPLAIN PER KIOWA CLOMR

RUNOFF SUMMARY

DESIGN POINT	5 YEAR	100 YEAR	NOTES
1	9.1	20.2	15' INLET,C1.1–C1.2
2	5.3	11.9	STREET FLOW,C1.3–C1.4
3	5.6	12.6	10' INLET,C1.1–C1.5
4	14.7	32.7	PIPE FLOW
5	4.1	9.1	STREET FLOW,C1.7–C1.8
6	5.3	11.6	10' INLET,C1.6–C1.8
7	4.1	9.1	STREET FLOW,C1.9–C1.11
8	5.2	11.6	STREET FLOW,C1.9–C1.12
9	3.2	7.0	STREET FLOW,C1.13–C1.14
10	7.9	17.6	10' INLET,C1.9–C1.14
11	2.5	5.7	5' INLET,C1.15–C1.16
12	2.9	6.3	AREA INLET,C1.17
13	5.4	12.0	PIPE FLOW
14	13.2	29.4	PIPE FLOW
14a	8.9	133.8	POND C1–R OUTFLOW
15	2.2	3.7	5' INLET,C5.1



Provide berms or some other method to prevent these two new inflow locations from "short circuiting" directly into the outlet/micro pool area.

Provide another area inlet to route the C3 Basin to The C1-R pond

Call out Grass Buffer

Call out Grass Buffer

Provide detailed grading for this area.

Floodplain boundary needs to be verified by LOMR. Label LOMR cross-sections C-F. Show and label channel improvements. Ensure that FP elevations are outside of lots.

CORE ENGINEERING GROUP
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BURNSVILLE, MN 55306
PH: 719.570.1100
CONTACT: RICHARD L. SCHINDLER, P.E.
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NO.	DESCRIPTION	DATE

PREPARED FOR:
LORSON, LLC
212 N. WAHSATCH AVE., SUITE 301
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CREEKSIDE FIL NO. 1
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EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

DEVELOPED CONDITIONS CREEKSIDE FILING NO. 1

DATE
AUGUST, 2018

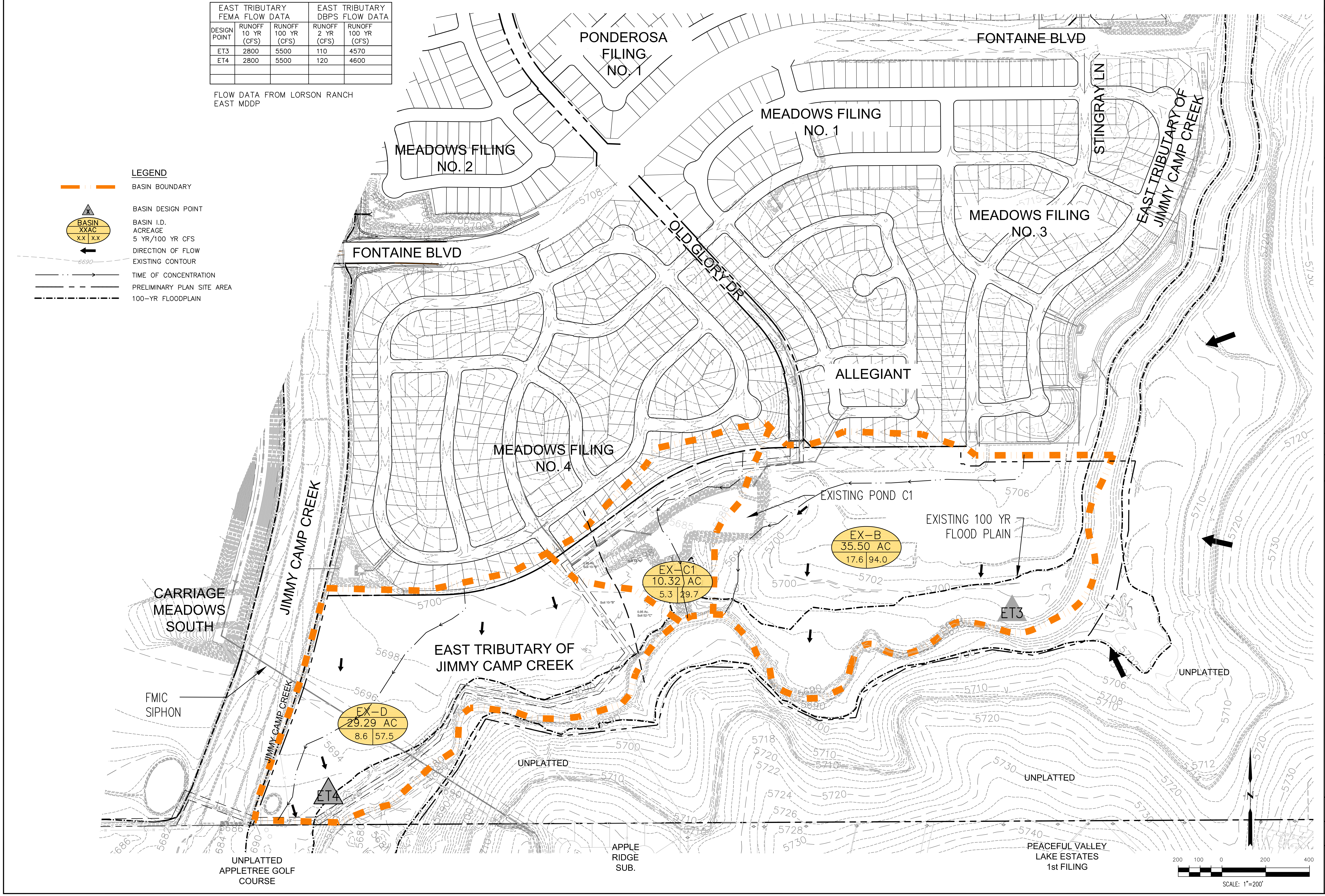
PROJECT NO.
100.045

SHEET NUMBER
2

TOTAL SHEETS: 2

EAST TRIBUTARY FEMA FLOW DATA			EAST TRIBUTARY DBPS FLOW DATA	
DESIGN POINT	RUNOFF 10 YR (CFS)	RUNOFF 100 YR (CFS)	RUNOFF 2 YR (CFS)	RUNOFF 100 YR (CFS)
ET3	2800	5500	110	4570
ET4	2800	5500	120	4600

FLOW DATA FROM LORSON RANCH EAST MDDP



LEGEND

BASIN BOUNDARY

BASIN DESIGN POINT

BASIN XXAC x.x | x.x — BASIN I.D. ACREAGE 5 YR/100 YR CFS

DIRECTION OF FLOW

6690 — EXISTING CONTOUR

TIME OF CONCENTRATION

PRELIMINARY PLAN SITE AREA

100–YR FLOODPLAIN

CORE
ENGINEERING GROUP
15004 1ST AVENUE S.
BURNSVILLE, MN 55306
PH: 719.570.1100
CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

NO.	DESCRIPTION	DATE

PREPARED FOR:
LORSON LLC
212 NORTH WAHSATCH AVE, SUITE 301
COLORADO SPRINGS, COLORADO 80903 (719) 635–3200
CONTACT: JEFF MARK

PROJECT:
CREEKSIDE FILING NO. 1
LORSON BLVD – OLD GLORY DRIVE
EL PASO COUNTY, COLORADO

DRAWN:	LAB
DESIGNED:	LAB
CHECKED:	RLS

EXISTING CONDITIONS
CREEKSIDE AT LORSON RANCH FIL. NO. 1

DATE:
AUGUST, 2018

PROJECT NO.
100.045

SHEET NUMBER
1

TOTAL SHEETS: 1