FINAL DRAINAGE PLAN

CREEKSIDE SOUTH AT LORSON RANCH FILING NO. 1

JUNE 15, 2020
REV 9/10/2020

SF-20-017

Prepared for:

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

Prepared by:

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Project No. 100.051

CORE ENGINEERING GROUP

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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
For and on Behalf of Core Engineering Group, LLC

Date 11-20-2020

COLORADO REGISTERED PROFESSIONAL ENGINEER RICHARD L. SCHINDLER 33997

OWNER'S STATEMENT

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

Lorson, LLC

11/20/20
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 G, dated December 7, 2018 and modified by modified per LOMR Case No. 19-08-0605P. (See Appendix A, FEMA FIRM Exhibit)

Richard L. Schindler, #33997

Date

COLORADO REGISTERED PROFESSIONAL ENGINEER RICHARD L. SCHINDLER 33997 11-20-2020

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
County Engineer/ECM Administrator

Date

APPROVED
Engineering Department
02/01/2021 1:25:52 PM
dsdnijkamp
EPC Planning & Community Development Department

Conditions: ______________________

1.0 LOCATION and DESCRIPTION

Creekside South at Lorson Ranch is located south of the East Tributary of Jimmy Camp Creek (Etrib). The site is located on approximately 64.257 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 South at Lorson Ranch which is located east and south 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 and west by the East Tributary of Jimmy Camp Creek, on the northeast by Lorson Ranch East Filing #4, single-family residential development and on the south by the Lorson Ranch South Boundary and Peaceful Valley Lake Estates. For reference, a vicinity map is included in Appendix A of this report.

<u>Conformance with applicable Drainage Basin Planning Studies</u>
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. Kiowa Engineering Corporation is nearing completion of construction plans for the East Tributary of Jimmy Camp Creek from the south property line of Lorson Ranch north and east to the channel improvements constructed by Lorson Ranch in 2014.

<u>Conformance with Lorson Ranch MDDP1 by Pentacor Engineering</u>
Lorson Ranch MDDP1 (October 26, 2006) includes this project area and the East Tributary. This PDR conforms to the MDDP1 for Lorson Ranch and is referenced in this report. The major infrastructure referenced in the MDDP 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 E2 (existing pond) will be increased in volume and will include a full spectrum outlet structure and emergency overflow structure. Proposed Detention/WQ pond J is shown as a full spectrum detention /WQ pond in the MDDP and will be designed/constructed as part of Creekside South at Lorson Ranch. Pond I and K as shown in the MDDP will not be constructed because the runoff has been redirected to Pond J reducing the number of ponds Lorson Ranch will have to maintain.

<u>Reconstruction of the East Tributary of Jimmy Camp Creek</u>
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 and construction will begin in

2020. 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 was constructed in 2018 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 and followup LOMR Case No. 19-08-0605P has been approved. 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 South at Lorson Ranch 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 indicates 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

3.0 EXISTING HYDROLOGICAL CONDITIONS

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

The Soil Conservation Service (SCS) classifies the soils within the Creekside South at Lorson Ranch site and the offsite drainage basin boundary as; 3-Ascalon Sandy Loam (2%), 10-Blendon Sandy Loam (1%), 52-Manzanst clay loam (11%), 54-Midway Clay Loam (10%), 56-Nelson-Tassel fine sandy loams (29%), 104-Vona Sandy Loam (12%), 108-Wiley silt loam (35%). 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/D soils with slow permeability. All of these soils are susceptible to erosion by wind and water, have low bearing strength, moderate to high shrink-swell potential, and high frost heave potential. The sandy loams are comprised of the hydrologic soil group B with moderate to moderately rapid permeability. The Manzanst and Midway clay loams are considered hydrologic soil group C and D soils with slow permeability. The clay loam soils are susceptible to erosion by wind and water, have low bearing strength, moderate to high shrink-swell potential, and high frost heave potential (see table 3.1 below). The clay loams are difficult to vegetate. 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
3-Ascalon Sandy Loam (2%)	B	Moderate	Moderate	Slow to Medium	Moderate
10-Blendon Sandy Loam (1%)	B	Low	Moderately Rapid	Slow	Moderate
52-Manzanst Clay Loam (11%)	C	Moderate to high	Slow	Medium	Moderate
54-Midway Clay Loam (10%)	D	High	Slow	Medium to Rapid	Moderate to High
56-Nelson-Tassel sandy loam (29%)	B	Moderate	Moderately Rapid	Slow	Moderate
104-Vona Sandy Loam (12%)	C	Moderate to High	Slow	Medium	Moderate
108-Vona Sandy Loam (35%)	B	Moderate	Moderate	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 was weighted and used for runoff calculations.

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 G, effective December 7, 2018 [2]. Floodplain along the East Tributary of Jimmy Camp Creek was modified per LOMR 19-08-0605P effective May 4, 2020 (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.

<u>Basin OS-E2 (overall offsite basin)</u>
This 43.18 acre offsite basin contributes flow from the Peaceful valley Lake Estates subdivision overland onto Basin EX-E2. It is estimated that the flow to Basin EX-E2 will be 18.0cfs and 78.4cfs for 5-year and 100-year events respectively.

<u>Basin EX-E2</u>
This 31.81 acre basin includes areas between the east tributary of Jimmy Camp Creek, Lorson Ranch East Filing #4 and the South Boundary of Lorson Ranch. Under existing conditions, this area flows northwesterly to the East Tributary and contributes 10.7cfs and 56.9cfs for 5-year and 100-year events respectively.

Design Point 3 (OS-E2.1 & EX-E2)
This 74.99 acre design point is comprised of Basins OS-E2.1 and EX-E2; runoff flows in a northwesterly direction to the East Tributary of Jimmy Camp Creek and generates a peak flow of 23.9cfs and 112.0cfs for 5-year and 100-year events respectively

Basin OS-I1.1
This 15.54 acre offsite basin contributes flow from the Peaceful Valley Lake Estates subdivision overland onto Basin EX-I1. It is estimated that the flow to Basin EX-I1 will be 6.7cfs and 30.7cfs for the 5-year and 100-year storm events respectively.

Basin EX-I1
This 12.78 acre basin includes that area between the east tributary of Jimmy Camp Creek and the South Boundary of Lorson Ranch. Under existing conditions, this area flows northwesterly to the East Tributary and contributes 3.4cfs and 22.7cfs for 5-year and 100-year events respectively.

Design Point 2 (OS-I1.1 & EX-I1)
This 28.32 acre design point is comprised of Basins OS-I1.1 and EX-I1; runoff flows in a northwesterly direction to the East Tributary of Jimmy Camp Creek and generates a peak flow of 8.7cfs and 46.1cfs for 5-year and 100-year events respectively

Basin OS-J1.1
This 11.55 acre offsite basin contributes flow from the Peaceful Valley Lake Estates subdivision and a portion of Apple Ridge Subdivision overland onto Basin EX-J1. It is estimated that the flow to Basin EX-J1 will be 5.7cfs and 23.0cfs for the 5-year and 100-year storm events respectively.

Basin EX-J1
This 15.29 acre basin includes that area between the east tributary of Jimmy Camp Creek and the South Boundary of Lorson Ranch. Under existing conditions, this area flows northwesterly to the East Tributary and contributes 6.9cfs and 31.6cfs for 5-year and 100-year events respectively.

Design Point 1 (OS-J1.1 & EX-J1)
This 26.84 acre design point is comprised of Basins OS-J1.1 and EX-J1; runoff flows in a northwesterly direction to the East Tributary of Jimmy Camp Creek and generates a peak flow of 10.8cfs and 46.6cfs for 5-year and 100-year events respectively

4.0 DEVELOPED HYDROLOGICAL CONDITIONS

Hydrology for the **Creekside South at Lorson Ranch** 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 westerly portion is Soil Type A/B and a small area on the east edge consists of Soil Type C/D. See Appendix A for SCS Soils Map.

The time of concentration for each basin and sub-basins were developed by using 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:

<u>Basin E7</u>
This basin consists of runoff from residential development. Runoff will be directed east to Trappe Drive and then routed north via existing curb and gutter to a low point in Trappe Drive where it will be collected by an existing Type R inlet constructed as part of Lorson Ranch East Filing No. 4. Basin E7 was included in the final drainage report for Lorson Ranch East Filing No. 4. The developed flow from this 0.60-acre basin E7 is 1.2cfs for the 5-year storm event and 2.7cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin E8.1</u>
This basin consists of runoff from 2.5-acre rural residential land and open space areas under the existing electric powerline. Runoff will be directed northwesterly to Horton Drive, then routed west in Horton Drive to Design Point 16 via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 4.00-acre is 4.0cfs for the 5-year storm event and 12.9cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin OS-E2.1</u>
This offsite basin consists of runoff from existing offsite 5.0-acre rural residential development located south of Lorson Ranch. These flows will be directed north through onsite basin E8.2 to Design Point 14 and intercepted by a storm sewer system in Horton Drive. The developed flow from this 36.66-acre basin OS-E2.1 is 15.3cfs for the 5-year storm event and 68.1cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin E8.2</u>
This basin consists of runoff from 2.5-acre rural residential development land. These flows will be directed north to Design Point 14 and intercepted by a storm sewer system in Horton Drive, and will also intercept overland runoff from basin OS-E2.1 The developed flow from this 1.70-acre basin E8.2 is 1.2cfs for the 5-year storm event and 4.9cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin OS-E2.2</u>
This offsite basin consists of runoff from offsite existing 5.0-acre rural residential development. These flows will be directed north through onsite basin E8.3 to Horton Drive, then southwesterly in Horton Drive to Design Point 15 and a Type R inlet. The developed flow from this 6.52-acre basin is 3.4cfs for the 5-year storm event and 15.4cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin E8.3</u>
This basin consists of runoff from on-site 2.5-acre lots and urban residential development. These flows will be directed north to Horton Drive, then southwesterly in Horton Drive to Design Point 15 via curb and gutter where it will be collected by a Type R inlet, and will also intercept overland runoff from basin OS-E2.2 The developed flow from this 5.37-acre basin E8.3 is 4.7cfs for the 5-year storm event and 14.3cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin E8.4</u>
This basin consists of runoff from the urban residential development. These flows will be directed south to Horton Drive, then southeasterly in Horton Drive to Shunka Lane at Design Point 17. The developed flow from this 1.20-acre basin is 1.9cfs for the 5-year storm event and 4.3cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin E8.5</u>
This basin consists of runoff from the urban residential development. These flows will be directed north and east to Shunka Lane, then northerly in Shunka Lane and Luneth Drive to Design Point 19 via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 4.27-acre basin is

6.8cfs for the 5-year storm event and 15.0cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin E8.6
This basin consists of runoff from the urban residential development. These flows will be directed west to Akela Lane, then northerly in Akela Lane and west in Luneth Drive to Design Point 19 via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 1.02-acre basin is 2.0cfs for the 5-year storm event and 4.5cfs for the 100-year storm event. See the appendix for detailed calculations

Basin E8.7
This basin consists of runoff from the urban residential development. These flows will be directed east to Akela Lane, then northerly in Akela Lane and west in Luneth Drive to Design Point 19 via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 0.71-acre basin is 1.4cfs for the 5-year storm event and 3.1cfs for the 100-year storm event. See the appendix for detailed calculations

Basin E8.8
This basin consists of runoff from the urban residential development. These flows will be directed south to Horton Drive and west to Shunka Lane, then northerly in Shunka Lane and west in Luneth Drive to Design Point 19 via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 2.43-acre basin is 4.1cfs for the 5-year storm event and 9.1cfs for the 100-year storm event. See the appendix for detailed calculations

Basin E8.9
This basin consists of runoff from the urban residential development. These flows will be directed south to Luneth Drive, then southeasterly in Luneth Drive to Design Point 20 via curb and gutter where it will be collected by Type R inlets. The developed flow from this 1.52-acre basin is 2.4cfs for the 5-year storm event and 5.4cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin E8.10
This basin consists of runoff from the urban residential development. These flows will be directed south to Luneth Drive, then southwesterly in Luneth Drive to Design Point 20 via curb and gutter where it will be collected by Type R inlets. The developed flow from this 0.38-acre basin is 0.9cfs for the 5-year storm event and 2.0cfs for the 100-year storm event. See the appendix for detailed calculations

Basin E8.11
This basin consists of runoff from backyards of urban residential development and open space areas draining directly to Pond E2. The developed flow from this 3.99-acre basin is 3.2cfs for the 5-year storm event and 12.2cfs for the 100-year storm event. See the appendix for detailed calculations

Basin OS-I1.1
This offsite basin consists of runoff from offsite existing 5.0-acre rural residential development. These flows will be directed north through onsite basin I1.2 to Design Point 4 and intercepted by a Type R inlet in Horton Drive. The developed flow from this 15.54-acre basin is 6.7cfs for the 5-year storm event and 30.7cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin I1.2
This basin consists of runoff from 2.5-acre rural residential development. These flows will be directed north to Design Point 4 and intercepted by a Type R inlet in Horton Drive, and will also intercept runoff from basin OS-I1.1 The developed flow from this 6.23-acre basin is 5.8cfs for the 5-year storm event and 17.6cfs for the 100-year storm event. See the appendix for detailed calculations

Overall Basin I1
This overall basin combines basins OS-I1.1 and I1.2 and develops peak flow from offsite 5.0-acre and 2.5-acre rural residential development. These flows will be directed north to Design Point 4 and intercepted by a Type R inlet in Horton Drive. The developed flow from this overall 21.77-acre basin is 11.1cfs for the 5-year storm event and 43.6cfs for the 100-year storm event. See the appendix for detailed calculations

Basin I2
This basin consists of runoff from the urban residential development. These flows will be directed south to Horton Drive, then east to Nakai Lane, then northerly in Nakai Lane and west in Luneth Drive to Design Point 5 via curb and gutter where it will be collected by a Type R inlet on a continuous grade. The developed flow from this 1.12-acre basin is 1.9cfs for the 5-year storm event and 4.2cfs for the 100-year storm event. See the appendix for detailed calculations

Basin I3
This basin consists of runoff from the urban residential development. These flows will be directed south to Horton Drive, then westerly and southwesterly in Horton Drive to Nakai Lane. The developed flow from this 1.01-acre basin is 1.7cfs for the 5-year storm event and 3.8cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin I4
This basin consists of runoff from the urban residential development. These flows will be directed north to Luneth Drive and west to Nakai Lane, then northerly in Nakai Lane and then continues west in Luneth Drive to Design Point 5 via curb and gutter where it will be collected by a Type R inlet on a continuous grade. The developed flow from this 3.45-acre basin is 5.6cfs for the 5-year storm event and 12.3cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin I5
This basin consists of runoff from the urban residential development. These flows will be directed north to Luneth Drive, runoff then continues west in Luneth Drive to Design Point 7 via curb and gutter where it will be collected by a Type R inlet on a continuous grade. The developed flow from this 1.44-acre basin is 2.7cfs for the 5-year storm event and 5.8cfs for the 100-year storm event. See the appendix for detailed calculations

Basin I6
This basin consists of runoff from the urban residential development. These flows will be directed northerly and westerly to Luneth Drive, runoff continues westerly and southerly in Luneth Drive to Design Point 8 via curb and gutter where it will be collected by a sump Type R inlet. The developed flow from this 1.18-acre basin is 2.0cfs for the 5-year storm event and 4.4cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin I7
This basin consists of runoff from the street and the urban residential development. These flows will be directed southerly to Luneth Drive, then westerly and southwesterly in Luneth Drive to via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 2.01-acre basin I7 is 2.8cfs for the 5-year storm event and 6.2cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin I8
This basin consists of runoff from backyards of urban residential development and open space areas draining directly to pond J. The developed flow from this 2.44-acre basin is 1.9cfs for the 5-year storm event and 6.6cfs for the 100-year storm event. See the appendix for detailed calculations

Overall Basin I

This basin contains a total area of 34.42 acres and runoff is routed to proposed detention pond J via overland, street and storm system from the I1 to I8 offsite and onsite sub-basins. The peak developed flow from basin is 24.3cfs for the 5-year storm event and 73.8cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin OS-J1.1

This offsite basin consists of runoff from existing offsite 5.0-acre rural residential development. These flows will be directed north through onsite basin J1.2 to Design Point 1 and intercepted by a storm sewer in Horton Drive. The developed flow from this 11.55-acre basin is 5.7cfs for the 5-year storm event and 23.0cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin J1.2

This basin consists of runoff from 2.5-acre rural residential development. These flows will be directed north to Design Point 1 and intercepted by a storm sewer system in Horton Drive, and will also intercept runoff from basin OS-J1.1 The developed flow from this 2.01-acre basin is 1.3cfs for the 5-year storm event and 5.3cfs for the 100-year storm event. See the appendix for detailed calculations

Overall Basin J1

This overall basin combines basins OS-J1.1 and J1.2 and develops peak flow from existing offsite 5.0-acre and 2.5-acre rural residential development. These flows will be directed north to Design Point 1 and intercepted by a storm sewer system in Horton Drive. The developed flow from this 13.56-acre basin is 6.4cfs for the 5-year storm event and 26.0cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin J2

This basin consists of runoff from the street and urban residential development. These flows will be directed north to Horton Drive, runoff then continues west in Luneth Drive to Design Point 2 via curb and gutter where it will be collected by a Type R inlet on a continuous grade. The developed flow from this 0.36-acre basin is 1.1cfs for the 5-year storm event and 2.2cfs for the 100-year storm event. See the appendix for detailed calculations

Basin J3

This basin consists of runoff from the urban residential development, open space, and offsite urban development. These flows will be directed north to Luneth Drive, then easterly and northeasterly in Luneth Drive to Design Point 2a. The developed flow from this 3.40-acre basin is 5.5cfs for the 5-year storm event and 12.0cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin J4

This basin consists of runoff from the urban residential development and open space. These flows will be directed southerly to Horton Drive and westerly to Luneth Drive, runoff continues westerly in Horton Drive and northeasterly in Luneth Drive to Design Point 3a via curb and gutter where it will be collected by a Type R inlet. The developed flow from this 1.93-acre basin is 3.4cfs for the 5-year storm event and 7.4cfs for the 100-year storm event. See the appendix for detailed calculations.

Basin J5

This basin consists of runoff from the urban residential development. These flows will be directed south to Luneth Drive, then easterly and northeasterly in Luneth Drive to Design Point 10 via curb and gutter where it will be collected by sump Type R inlets. The developed flow from this 1.93-acre basin is 3.1cfs for the 5-year storm event and 6.9cfs for the 100-year storm event. See the appendix for detailed calculations.

Overall Basin J

This basin contains a total area of 21.18 acres and runoff is routed to proposed detention pond J via overland, street and storm system from the J1 to the J5 offsite and onsite sub-basins. The peak developed flow from this overall basin is 15.4cfs for the 5-year storm event and 45.9cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Combined Basins I & J</u>
These basins contain a total area of 55.60 acres and runoff is routed to proposed detention Pond J via overland, street and storm system from the I1 to I8 offsite and onsite sub-basins. The peak developed flow from this overall basin is 38.7cfs for the 5-year storm event and 116.7cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin K1</u>
This basin consists of runoff from open space areas that sheet flows directly to the East Tributary. The developed flow from this 2.65-acre basin is 1.2cfs for the 5-year storm event and 6.6cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin K2</u>
This basin consists of runoff from open space areas that sheet flows directly to the East Tributary. The developed flow from this 2.14-acre basin is 1.2cfs for the 5-year storm event and 6.8cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin L</u>
This basin consists of runoff from open space areas that sheet flows directly to the East Tributary. The developed flow from this 2.30-acre basin is 1.4cfs for the 5-year storm event and 7.8cfs for the 100-year storm event. See the appendix for detailed calculations.

<u>Basin M</u>
This basin consists of runoff from open space areas that sheet flows directly to the East Tributary. The developed flow from this 0.48-acre basin is 0.3cfs for the 5-year storm event and 1.5cfs for the 100-year storm event. 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 excel 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)

Design Point 1

Design Point 1 is the overland runoff flowing north to a 24" storm sewer (end section) located at Horton Drive. The total flow into the storm sewer system is 6.4cfs and 26.0cfs in the 5/100-year storm events. The Hw/D is 1.69' for a 24" RCP end section in the 100-year storm event.

Design Point 2

Design Point 2 is located in Horton Drive (south side) east of Luneth Drive

(5-year storm)

Tributary Basins: J1 **Inlet/MH Number:** Inlet J2

Upstream flowby: 0cfs **Total Street Flow:** 1.1cfs

Flow Intercepted: 1.1cfs **Flow Bypassed:** 0

Inlet Size: 5' type R

Street Capacity: Street slope = 2%, capacity = 12cfs, capacity okay

(100-year storm)

Tributary Basins: J1 **Inlet/MH Number:** Inlet J2

Upstream flowby: 0cfs **Total Street Flow:** 2.2cfs

Flow Intercepted: 1.9cfs **Flow Bypassed:** 0.3cfs to Des. Pt 8

Inlet Size: 5'type R

Street Capacity: Street slope = 2%, capacity = 45.4cfs (half street) is okay

Design Point 3

Design Point 3 is the total pipe flow in a 24" RCP storm sewer in Horton Drive to Luneth Drive and is located west of Design Point 2. The total pipe flow is 7.5cfs and 27.9cfs in the 5/100-year storm events.

Design Point 3a

Design Point 3a is located in Luneth Drive (east side) south of the low point at Pond J

(5-year storm)

Tributary Basins: J3+J4 **Inlet/MH Number:** Inlet J4

Upstream flowby: 0cfs **Total Street Flow:** 8.6cfs

Flow Intercepted: 6.6cfs **Flow Bypassed:** 2.0cfs to Des. Pt. 8

Inlet Size: 10' type R

Street Capacity: Street slope = 1.9%, capacity = 12cfs, capacity okay

(100-year storm)

Tributary Basins: J3+J4 **Inlet/MH Number:** Inlet J4

Upstream flowby: 0.3cfs **Total Street Flow:** 19.2cfs

Flow Intercepted: 9.9cfs **Flow Bypassed:** 9.3cfs to Des. Pt 8

Inlet Size: 10'type R

Street Capacity: Street slope = 1.9%, capacity = 45.4cfs (half street) is okay

Design Point 3b

Design Point 3b is the total pipe flow in a 30" RCP storm sewer in Luneth Drive and is located south of Design Point 8. The total pipe flow is 14.1cfs and 37.8cfs in the 5/100-year storm events.

Design Point 4

Design Point 4 is located at a low point in Horton Drive (south side) at Nakai Lane

(5-year storm)

Tributary Basins: I-1 **Inlet/MH Number:** Inlet I-1

Upstream flowby: 0cfs **Total Street Flow:** 11.1cfs

Flow Intercepted: 11.1cfs **Flow Bypassed:** 0

Inlet Size: 20' type R, sump

Street Capacity: Street slope = 2.3%, capacity = 13.3cfs, capacity okay

(100-year storm)

Tributary Basins: I-1 **Inlet/MH Number:** Inlet I-1

Upstream flowby: 0cfs **Total Street Flow:** 43.6cfs

Flow Intercepted: 15.5cfs **Flow Bypassed:** 28.1cfs to Des. Pt. 5

Inlet Size: 20' type R, sump

Street Capacity: Street slope = 2.3%, capacity = 32.8cfs (half street). Flow uses north half of street

Design Point 5

Design Point 5 is located in Luneth Drive (south side) west of Nakai Lane

(5-year storm)
Tributary Basins: I2, I3, I4 **Inlet/MH Number:** Inlet I-2
Upstream flowby: 0cfs **Total Street Flow:** 8.7cfs

Flow Intercepted: 3.6cfs **Flow Bypassed:** 5.1cfs to Des. Pt. 7
Inlet Size: 5' type R, on-grade

Street Capacity: Street slope = 0.8%, capacity = 8.0cfs, capacity okay

(100-year storm)
Tributary Basins: I2, I3, I4 **Inlet/MH Number:** Inlet I-2
Upstream flowby: 28.1cfs **Total Street Flow:** 47.4cfs

Flow Intercepted: 7.1cfs **Flow Bypassed:** 40.3cfs to Des. Pt 7
Inlet Size: 5' type R, on-grade

Street Capacity: Street slope = 0.8%, capacity = 33.4cfs (half street). Flow overtops crown and flows on north side of Luneth Dr. Only flow from Basin I7 (6.2cfs) is on north side. Capacity okay

Design Point 6

Design Point 6 is the total pipe flow in a 30" RCP storm sewer in Luneth Drive and is located west of Nakai Lane. The total pipe flow is 14.7cfs and 22.6cfs in the 5/100-year storm events.

Design Point 7

Design Point 7 is located in Luneth Drive (south side) west of Nakai Lane and Des. Pt. 5

(5-year storm)
Tributary Basins: I5 **Inlet/MH Number:** Inlet I-5
Upstream flowby: 5.1cfs **Total Street Flow:** 7.8cfs

Flow Intercepted: 3.5cfs **Flow Bypassed:** 4.3cfs to Des. Pt. 8
Inlet Size: 5' type R, on-grade

Street Capacity: Street slope = 0.8%, capacity = 8.0cfs, capacity okay

(100-year storm)
Tributary Basins: I5 **Inlet/MH Number:** Inlet I-5
Upstream flowby: 40.3cfs **Total Street Flow:** 46.1cfs

Flow Intercepted: 7.0cfs **Flow Bypassed:** 24.1cfs to Des. Pt 8
15.0cfs to Des. Pt.10

Inlet Size: 5' type R, on-grade

Street Capacity: Street slope = 0.8%, capacity = 33.4cfs (half street). Flow overtops crown and flows on north side of Luneth Dr to Des. Pt.10. Only flow from Basin I7 (6.2cfs) is on north side. Capacity okay

Design Point 7a

Design Point 7a is the total pipe flow in a 30" RCP storm sewer in Luneth Drive and is located west of Design Point 7. The total pipe flow is 18.2cfs and 29.6cfs in the 5/100-year storm events.

Design Point 8

Design Point 8 is located in Luneth Drive (east side) at a low point east of Pond J

(5-year storm)

Tributary Basins: I6

Inlet/MH Number: Inlet I6

Upstream flowby: 2.0+4.3 = 6.3cfs

Total Street Flow: 8.3cfs

Flow Intercepted: 8.3cfs

Flow Bypassed: 0

Inlet Size: 10' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, capacity okay since flow is from both ways

(100-year storm)

Tributary Basins: I6

Inlet/MH Number: Inlet I6

Upstream flowby: 0.3+9.3+24.1=33.7cfs

Total Street Flow: 38.1cfs

Flow Intercepted: 15.5cfs

Flow Bypassed: 22.6cfs to Des. Pt 10

Inlet Size: 10' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 37.3cfs (half street). Flow overtops crown and flows on west side of Luneth Dr. Only flow from Basin J5 (6.9cfs) is on westside. Capacity okay

Design Point 9

Design Point 9 is the total pipe flow in a 36" RCP storm sewer in Luneth Drive and is located west of Design Point 8. The total pipe flow is 22.4cfs and 53.3cfs in the 5/100-year storm events.

Design Point 10
Design Point 10 is located in Luneth Drive (west side) at a low point east of Pond J

(5-year storm)
Tributary Basins: J5+I7
Inlet/MH Number: Inlet I7
Upstream flowby: 0
Total Street Flow: 5.0cfs

Flow Intercepted: 5.0cfs
Flow Bypassed: 0
Inlet Size: 45' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, capacity okay since flow is from both ways

(100-year storm)
Tributary Basins: J5+I7
Inlet/MH Number: Inlet I7
Upstream flowby: 37.6cfs
Total Street Flow: 50.7cfs

Flow Intercepted: 50.7cfs
Flow Bypassed: 0
Inlet Size: 45' type R, sump (flow depth = 7.8" at flowline curb, flow spreads to ROW line and does not inundate lots

Design Point 11
Design Point 11 is the total pipe flow in a 42" RCP storm sewer in Luneth Drive and is located west of Design Point 10. The total pipe flow is 27.4cfs and 104.0cfs in the 5/100-year storm events.

Design Point 12
Design Point 12 is the total pipe flow in a 48" RCP storm sewer in Luneth Drive and is located west of Design Point 11. The total pipe flow is 45.6cfs and 133.6cfs in the 5/100-year storm events and outlets into Pond J.

Design Point 13
Design Point 13 is located west of Pond J and is the total flow from the outlet structure for Pond J in a 36" RCP storm sewer. The total outflow is 1.2cfs and 55.5cfs in the 5/100-year storm events from the Pond J full spectrum EDB worksheets. The Pond Outflow Channel is a trapezoidal swale with a 5' bottom, 3:1 side slopes, 0.5% channel slope, and is lined with 12" D50 rip rap. This flow enters the low flow channel of the East Tributary of Jimmy Camp Creek in a rip rap swale. Kiowa Engineering has accommodated the low flow channel for this swale.

Design Point 14
Design Point 14 is the overland runoff flowing north to a 36" storm sewer (end section) located at Horton Drive and Shunka Lane. The total flow into the storm sewer system is 15.8cfs and 70.0cfs in the 5/100-year storm events. The Hw/D is 1.72' for a 36" RCP end section in the 100-year storm event.

Design Point 15
Design Point 15 is located on the south side of Horton Drive at Shunka Lane. This design point was added to verify the street capacity of Horton Drive on the south side of the street from the west. The total street flow is 6.2cfs and 22.9cfs in the 5/100-year storm events from Basins OS E2.2 & E8.3. The street capacity of Horton 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 from the west.

Design Point 16

Design Point 16 is located in Horton Drive (south side) at a low point at Shunka Lane

(5-year storm)

Tributary Basins: E8.1 + Des. Pt 15 | **Inlet/MH Number:** Inlet E8.1

Upstream flowby: 0 | **Total Street Flow:** 9.4cfs

Flow Intercepted: 9.4cfs | **Flow Bypassed:** 0

Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 10.5cfs, capacity okay

(100-year storm)

Tributary Basins: E8.1+ Des. Pt 15 | **Inlet/MH Number:** Inlet E8.1

Upstream flowby: 0 | **Total Street Flow:** 33.0cfs

Flow Intercepted: 18.9cfs | **Flow Bypassed:** 14.1cfs to Des. Pt.20

Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 44.1cfs, capacity okay

Design Point 17

Design Point 17 is the total pipe flow in a 36" RCP storm sewer in Shunka Lane. The total pipe flow is 25.2cfs and 85.5cfs in the 5/100-year storm events.

Design Point 18

Design Point 18 is located on the south side of Luneth Drive at Akela Lane. This design point was added to verify the street capacity of Luneth Drive on the south side of the street from the east. The total street flow is 3.4cfs and 7.5cfs in the 5/100-year storm events from Basins E8.6 & E8.7. The street capacity of Luneth Drive at 1.7% slope is 10.5cfs (5-yr) and 44.1cfs (100-yr). The street capacity is not exceeded at this design point.

Design Point 19

Design Point 19 is located in Luneth Drive (south side) at a low point at Shunka Lane

(5-year storm)

Tributary Basins: E8.4-E8.8 | **Inlet/MH Number:** Inlet E8.5

Upstream flowby: 0 | **Total Street Flow:** 15.0cfs

Flow Intercepted: 15.0cfs | **Flow Bypassed:** 0

Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 10.5cfs, capacity okay since half flow is from the west

(100-year storm)

Tributary Basins: E8.1+ Des. Pt 15 | **Inlet/MH Number:** Inlet E8.5

Upstream flowby: 0 | **Total Street Flow:** 33.1cfs

Flow Intercepted: 32.0cfs | **Flow Bypassed:** 1.1cfs to Des. Pt.20

Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 44.1cfs, capacity okay

Design Point 20
Design Point 20 is located in Luneth Drive (north side) at a low point at Shunka Lane

(5-year storm)
Tributary Basins: E8.9+E8.10 **Inlet/MH Number:** Inlet E8.9
Upstream flowby: 0 **Total Street Flow:** 3.1cfs

Flow Intercepted: 3.1cfs **Flow Bypassed:** 0
Inlet Size: double 10' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 10.5cfs, capacity okay

(100-year storm)
Tributary Basins: E8.9+E8.10 **Inlet/MH Number:** Inlet E8.9
Upstream flowby: 1.1+14.1=15.2cfs **Total Street Flow:** 22.0cfs

Flow Intercepted: 22.0cfs **Flow Bypassed:** 0
Inlet Size: double 10' type R, sump

Street Capacity: Street slope = 1.5%, capacity = 44.1cfs (half street). Capacity okay

Design Point 21
Design Point 21 is the total pipe flow in a 30" RCP storm sewer in Luneth Drive connecting Inlets E8.9 to a 48" storm sewer flowing to Pond E2. The total pipe flow is 18.1cfs and 54.0cfs in the 5/100-year storm events.

Design Point 22
Design Point 22 is the total pipe flow in a 48" RCP storm sewer in from Luneth Drive flowing north into Pond E2. The total pipe flow is 43.3cfs and 139.5cfs in the 5/100-year storm events and outlets into Pond E2.

Design Point 22a
Design Point 22a is the total inflow from this development and Lorson Ranch East Filing No. 4 into Pond E2. The total pond inflow is 102.8cfs and 333.2cfs in the 5/100-year storm events and outlets into Pond E2.

Design Point 23
Design Point 23 is located west of Pond E2 and is the total flow from the outlet structure for Pond E2 in a 48" RCP storm sewer. The pond outlet structure is built as part of Lorson Ranch East Filing No. 4 but will be modified to account for development in Creekside South. The total pond outflow is 4.5cfs and 151.3cfs in the 5/100-year storm events from the Pond E2 full spectrum EDB worksheets. The outlet structure, outflow pipe, and creek outfall are all constructed as part of the Lorson Ranch East Filing 4 development. This flow enters the reconstructed portion of the East Tributary of Jimmy Camp Creek.

6.0 DETENTION AND WATER QUALITY PONDS

Detention and Storm Water Quality for Creekside South at Lorson Ranch is required per El Paso County criteria. We have implemented the Full Spectrum approach for detention for Creekside South at Lorson Ranch per the Denver Urban Drainage Districts specifications. There is one existing detention pond to be expanded and one proposed detention pond for this project site. All developed runoff from this site will flow to ponds and will incorporate storm water quality features prior to discharge into the East Tributary. Open space areas (no development) will be allowed to flow directly to the East Tributary of Jimmy Camp Creek.

Full Spectrum Pond Construction Requirements

Design calculations for full spectrum ponds will include a 15' 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, 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 South prepared by RMG.

Detention Pond E2 (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 E2 was partially constructed as part of Lorson Ranch East Filing No. 4 with a full spectrum outlet structure, forebay, and concrete low flow channel designed using the UDCF Full Spectrum spreadsheets. The existing outlet structure is a 6.7'x6.7' full spectrum sloped outlet structure with an interim overflow structure. Creekside South will add more developed area to the pond's tributary area requiring the pond to be expanded and the outlet structure to be modified. This development will expand the existing pond, update the existing outlet structure, construct additional forebay, construct additional concrete low flow channel, and construct a new concrete weir overflow spillway 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: 125 acres (full buildout of watershed including offsite 5-acre rural residential lots)
- Watershed Imperviousness: 35%
- Hydrologic Soils Group B (40%) and C/D (60%)
- Forebay: 0.0256ac-ft (see spreadsheet in appendix). (one existing and one new forebay)
- Zone 1 WQCV: 1.617ac-ft, WSEL: 5696.05, 0.8cfs
- Zone 2 EURV: 3.868ac-ft, WSEL: 5697.57, Top EURV set at 5698.85, 6.7'x17' outlet with flat top, 3.5cfs
- (5-yr): 5.072ac-ft, WSEL: 5698.34, 4.5cfs
- Zone 3 (100-yr): 9.855ac-ft, WSEL: 5701.16, 151.3cfs
- Pipe Outlet: 48" RCP at 0.5% with no restrictor plate
- Overflow Spillway: 55' wide bottom, elevation=5701.20, 4:1 side slopes
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5693.00

Existing Pond E2 and Existing Pond E2 Outlet Structure Modifications:

- Expand pond size to accommodate full buildout area
- Construct one additional concrete forebay at new 48" RCP storm sewer
- Construct new concrete low flow channel from new forebay to existing outlet structure
- Construct additional access road to new forebay
- Construct additional 11' length to 6.7'x6.7' outlet structure
- Modify Zone 1 WQ plate to larger size openings
- Modify Zone 2 square opening to 4"x18"
- Construct permanent concrete overflow weir

Detention Pond J (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 J is designed using the UDCF Full Spectrum spreadsheets. The outlet structure is a standard full spectrum sloped outlet structure and the overflow spillway is a concrete 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: 54 acres including offsite 5-acre rural residential lots
- Watershed Imperviousness: 26%
- Hydrologic Soils Group B (90%), C/D (10%)
- Forebay: 0.024ac-ft, 24” depth, std concrete forebay
- Zone 1 WQCV: 0.575ac-ft, WSEL: 5684.13, 0.3cfs
- Zone 2 EURV: 1.272ac-ft, WSEL: 5685.10, Top EURV wall set at 5685.37, 6’x6’ outlet with 6:1 slope, 1.1cfs
- (5-yr): 1.442ac-ft, WSEL: 5685.31, 1.2cfs
- Zone 3 (100-yr): 3.545ac-ft, WSEL: 5687.67, 55.5cfs
- Pipe Outlet: 36” RCP at 0.5% with restrictor plate up 25”
- Overflow Spillway: 25’ wide bottom, elevation=5687.77, 4:1 side slopes, flow depth=1.2’
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5681.67

Water Quality Design
Water quality will be provided by two permanent extended detention basins (Pond E2, J) for all of the developed areas of this site.

7.0 DRAINAGE AND BRIDGE FEES

Creekside South at Lorson Ranch 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 South at Lorson Ranch Filing No. 1 contains 64.257 acres. This project consists of 13.037 acres of open space (2% impervious), 30.458 acres residential (57% impervious based on 4600sf lots), 14.753 acres of large lots (5% impervious). Tract A (6.009 acres) which is a future development tract will defer fees until it is platted into lots. The 2020 drainage fees are $19,084, bridge fees are $893 and Drainage Surety fees are $7,285 per impervious acre per Resolution 18-470. The drainage and bridge fees are calculated when the final plat is submitted. The fees are due at plat recordation. The following table details the drainage fees for the platted area. Lorson Ranch intends to use the Drainage Fee Pre-credits for the surety, drainage fee, and the bridge fee credits for the bridge fee.

Table 1: Drainage/Bridge Fees

Type of Land Use	Total Area (ac)	Imperviousness	Drainage Fee	Bridge Fee	Surety Fee
Residential Area	30.458	51%	$296,442	$13,871	$113,162

Large Lots	14.753	11%	$30,970	$1,449	$11,822
Open Space, Landscape Tracts,	13.037	2%	$4,975	$232	$1,899
		Total	$332,387	$15,552	$126,883

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	11	EA	$3000/EA	$33,000
Inlets	12	EA	$3,000	$36,000
18” Storm	54	LF	$35	$1,890
24” Storm	760	LF	$40	$30,400
30” Storm	775	LF	$45	$34,875
36” Storm	560	EA	$50	$28,000
42” Storm	29	EA	$55	$1,595
48” Storm	202	EA	$60	$12,120
24” FES	1	EA	$250	$250
36” FES	1	EA	$300	$300
			Subtotal	$188,430
			Eng/Cont 15%)	$28,264
			Total Est. Cost	$216,694

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	LS	$100,000	$100,000
			Subtotal	$100,000
			Eng/Cont (15%)	$15,000
			Total Est. Cost	$115,000

8.0 FOUR STEP PROCESS

The site has been developed to minimize wherever possible the rate of developed runoff that will leave the site and to provide water quality management for the runoff produced by the site as proposed on the development plan. The following four step process should be considered and incorporated into the storm water collection system and storage facilities where applicable.

Step 1: Employ Runoff Reduction Practices
Creekside South at Lorson Ranch has employed several methods of reducing runoff.

- The street configuration was laid out to minimize the length of streets. Many streets are straight and perpendicular resulting in lots with less wasted space.
- A buffer tract added along the East Tributary which reduces impervious areas
- Larger 2.5-acre lots added on south side to reduce impervious area
- Lorson Ranch Metro District requires homeowners to maintain landscaping on lots
- Full Spectrum Detention Pond J and E2 will be constructed. The full spectrum detention mimics existing storm discharges

Step 2: Implement BMP's that Slowly Release the Water Quality Capture Volume
Treatment and slow release of the water quality capture volume (WQCV) is required. Creekside South at Lorson Ranch will utilize Pond J and E2 which are full spectrum stormwater detention ponds which includes Water Quality Volumes and WQ outlet structures.

Step 3: Stabilize Drainageways
East Tributary of Jimmy Camp Creek is a major drainageway located west of this site. In 2020 the East Tributary of JCC adjacent to this site will be reconstructed and stabilized per county criteria. The design included a low flow channel bottom and selectively armored sides.

Step 4: Implement Site Specific & Source Control BMP's
There are no potential sources of contaminants that could be introduced to the County's MS4. During construction source control will be provided with the proper installation of erosion control BMPs to limit erosion and transport of sediment. Area disturbed by construction will be seeded and mulched. Cut and fill slopes will be reseeded, and the slopes equal to or greater than three-to-one will be protected with erosion control fabric. Silt fences will be placed at the bottom of re-vegetated and rough graded slopes. Inlet protection will be used around proposed inlets. In addition, temporary sediment basins will be constructed so runoff will be treated prior to discharge. Construction BMPs in the form of vehicle tracking control, sediment basins, concrete washout area, rock socks, buffers, and silt fences will be utilized to protect receiving waters.

9.0 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:

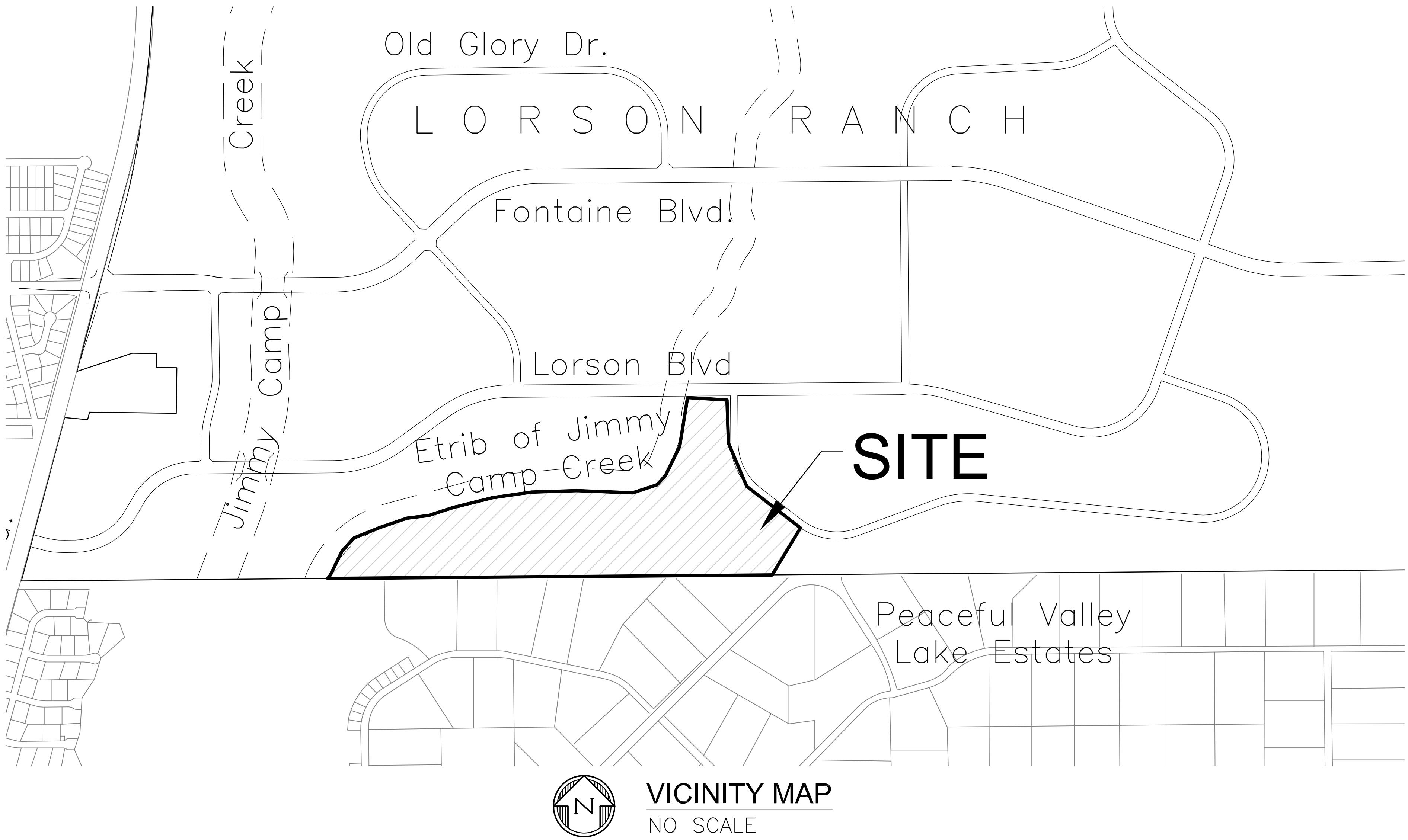
- Developed runoff will be conveyed via curb/gutter and storm sewer facilities
- The East Tributary of Jimmy Camp Creek will be reconstructed within this study area
- Detention and water quality for this preliminary plan area will be provided in two permanent ponds

10.0 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 East MDDP, June 30, 2017 by Core Engineering.
7. Final construction plans “East Fork Jimmy Camp Creek Channel Design”, Dated April 1, 2020 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.
9. Final Drainage Report for Lorson Ranch East Filing No. 4 by Core Engineering, approved September 12, 2019 (SF19-008)

APPENDIX A – VICINTIY MAP, SOILS MAP, FEMA MAP



VICINITY MAP

NO SCALE

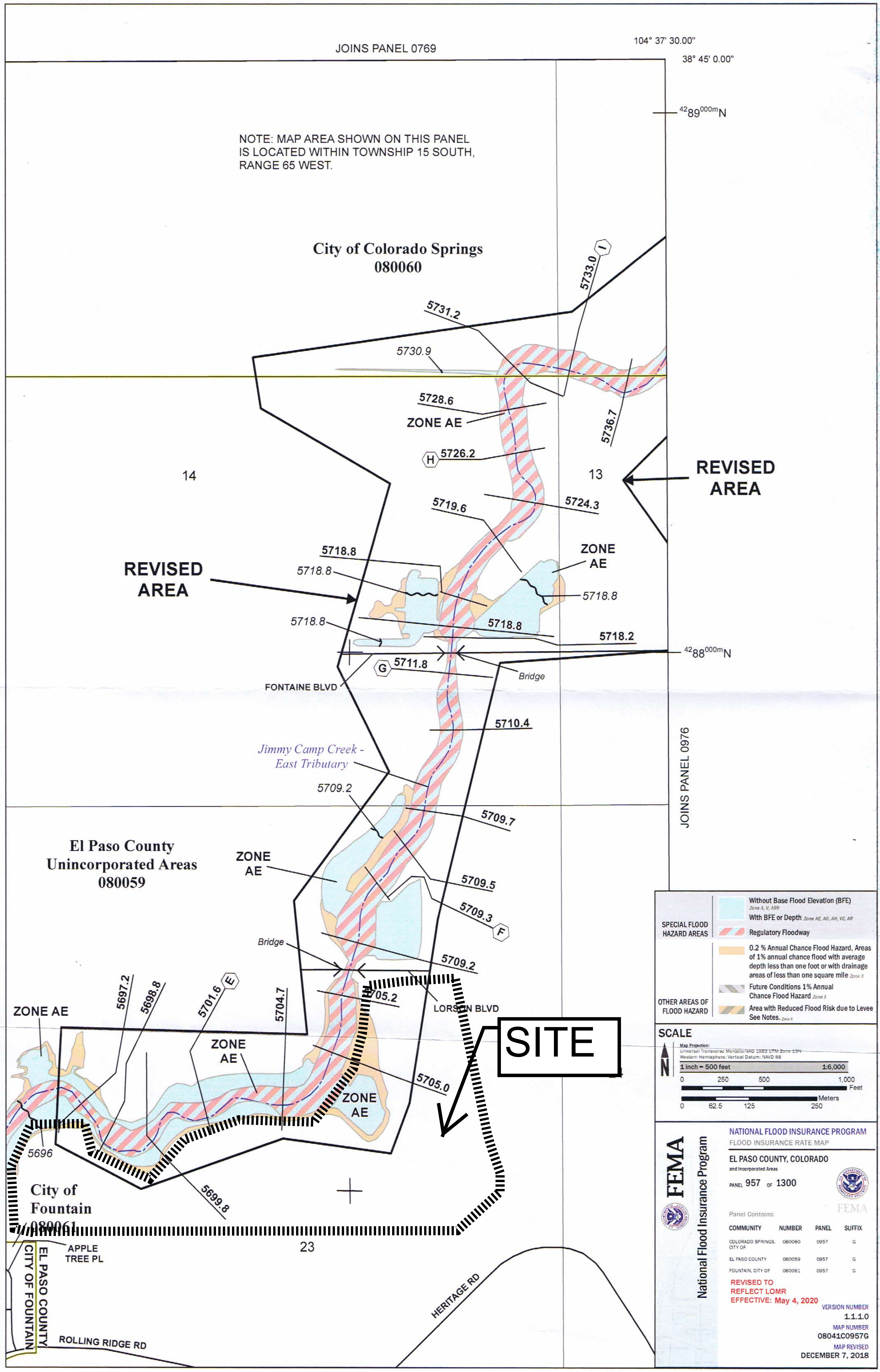
CORE ENGINEERING GROUP

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

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

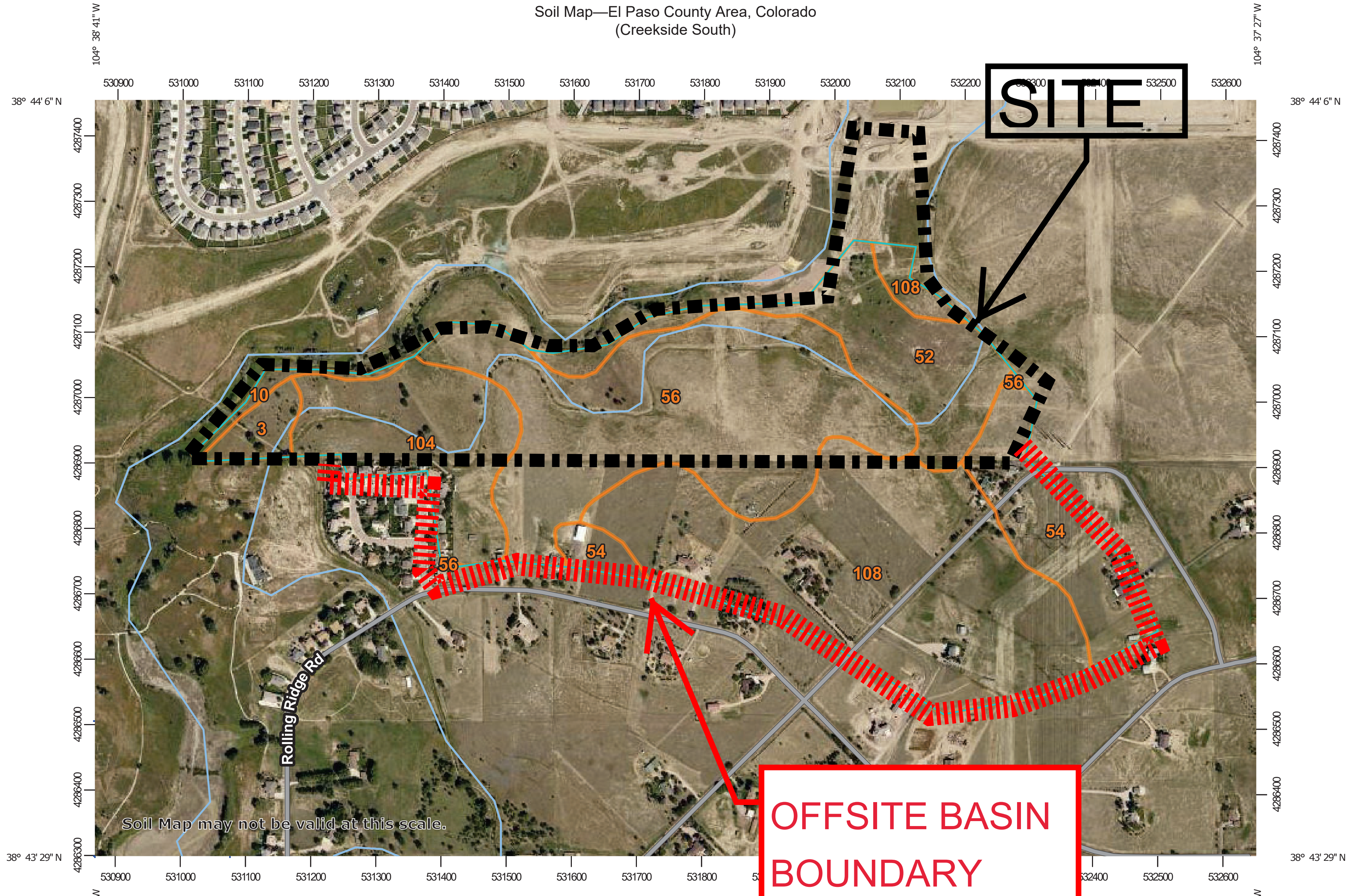
CREEKSIDE SOUTH AT LORSON RANCH
VICINITY MAP

SCALE:	DATE:	FIGURE NO.
NTS	JANUARY, 2020	--



JOINS PANEL 0769
104° 37' 30.00"
38° 45' 0.00"
4289000mN
NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 15 SOUTH, RANGE 65 WEST.
City of Colorado Springs 080060
5733.0
I
5731.2
5730.9
5728.6
ZONE AE
5736.7
H
5726.2
14
13
REVISED AREA
5724.3
5719.6
ZONE AE
5718.8
5718.8
REVISED AREA
5718.8
5718.8
5718.8
5718.2
4288000mN
G
5711.8
Bridge
FONTAINE BLVD
5710.4
Jimmy Camp Creek - East Tributary
JOINS PANEL 0976
5709.2
5709.7
El Paso County Unincorporated Areas 080059
ZONE AE
5709.5
5709.3
F
Bridge
5709.2
5697.2
5698.8
5701.6
E
5704.7
5705.2
LORSON BLVD
ZONE AE
ZONE AE
SITE
5705.0
ZONE AE
5696
5699.8
City of Fountain 080061
APPLE TREE PL
23
EL PASO COUNTY
CITY OF FOUNTAIN
HERITAGE RD
ROLLING RIDGE RD
SPECIAL FLOOD HAZARD AREAS
Without Base Flood Elevation (BFE) Zone A, V, A99
With BFE or Depth Zone AE, AO, AH, VE, AR
Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD
0.2 % Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile Zone X
Future Conditions 1% Annual Chance Flood Hazard Zone X
Area with Reduced Flood Risk due to Levee See Notes. Zone X
SCALE
Map Projection:
Universal Transverse Mercator NAD 1983 UTM Zone 13N
Western Hemisphere; Vertical Datum: NAVD 88
1 inch = 500 feet
1:6,000
0
250
500
1,000
Feet
Meters
0
62.5
125
250
FEMA
National Flood Insurance Program
NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP
EL PASO COUNTY, COLORADO
and Incorporated Areas
PANEL 957 OF 1300
Panel Contains:
COMMUNITY NUMBER PANEL SUFFIX
COLORADO SPRINGS, CITY OF 080060 0957 G
EL PASO COUNTY 080059 0957 G
FOUNTAIN, CITY OF 080061 0957 G
REVISED TO REFLECT LOMR EFFECTIVE: May 4, 2020
VERSION NUMBER
1.1.1.0
MAP NUMBER
08041C0957G
MAP REVISED
DECEMBER 7, 2018

Soil Map—El Paso County Area, Colorado
(Creekside South)



Map Scale: 1:8,150 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 100, 200, 400, 600

Feet: 0, 350, 700, 1400, 2100

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
- 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 17, Sep 13, 2019

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

Date(s) aerial images were photographed: Aug 14, 2018—Sep 23, 2018

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
3	Ascalon sandy loam, 3 to 9 percent slopes	2.6	2.0%
10	Blendon sandy loam, 0 to 3 percent slopes	1.2	0.9%
52	Manzanst clay loam, 0 to 3 percent slopes	14.5	11.2%
54	Midway clay loam, 3 to 25 percent slopes	12.9	10.0%
56	Nelson-Tassel fine sandy loams, 3 to 18 percent slopes	37.6	29.2%
104	Vona sandy loam, warm, 0 to 3 percent slopes	15.7	12.2%
108	Wiley silt loam, 3 to 9 percent slopes	44.5	34.5%
Totals for Area of Interest		**128.8**	**100.0%**

APPENDIX B – HYDROLOGY CALCULATIONS

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: December 17, 2019
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **5-Year Event (Current)**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time		
		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
OS-E2.1			43.18	0.17	30.5	7.34	2.46	18.0												
EX-E2			31.81	0.13	27.7	4.14	2.60	10.7												
(E2 & E2.1)	**3**	74.99							39.1	11.48	2.08	23.9								
OS-I1.1			15.54	0.15	23.1	2.33	2.87	6.7												
EX-I1			12.78	0.09	22.1	1.15	2.94	3.4												
(I1 & I1.1)	**2**	28.32							29.6	3.48	2.50	8.7								
OS-J1.1			11.55	0.19	28.1	2.19	2.58	5.7												
EX-J1			15.29	0.15	21.2	2.29	3.00	6.9												
(J1 & J1.1)	**1**	26.84							31.7	4.49	2.40	10.8								

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: October 29, 2019
Checked By: Leonard Beasley

Job No: 100.050
Project: Creekside South at Lorson Ranch
Design Storm: **100-Year Event (Current)**

Street or Basin	Design Point	Direct Runoff: Area Design	Direct Runoff: Area (A)	Direct Runoff: Runoff Coeff. (C)	Direct Runoff: tc	Direct Runoff: CA	Direct Runoff: i	Direct Runoff: Q	Total Runoff: tc	Total Runoff: Σ (CA)	Total Runoff: i	Total Runoff: Q	Street: Slope	Street: Street Flow	Pipe: Design Flow	Pipe: Slope	Pipe: Pipe Size	Travel Time: Length	Travel Time: Velocity	Travel Time: tt	Remarks
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
OS-E2.1			43.18	0.44	30.5	19.00	4.13	78.4													
EX-E2			31.81	0.41	27.7	13.04	4.36	56.9													
(E2 & E2.21	3	74.99							39.1	32.04	3.50	112.0									
OS-I1.1			15.54	0.41	23.1	6.37	4.82	30.7													
EX-I1			12.78	0.36	22.1	4.60	4.94	22.7													
(I1 & I1.1)	2	28.32							29.6	10.97	4.20	46.1									
OS-J1.1			11.55	0.46	28.1	5.31	4.33	23.0													
EX-J1			15.29	0.41	21.2	6.27	5.04	31.6													
(J1 & J1.1)	1	26.84							31.7	11.58	4.03	46.6									

CORE ENGINEERING GROUP
15004 1st Avenue South
Burnsville, MN 55306

PROJECT NAME: Creekside South at Lorson Ranch
PROJECT NUMBER: 100.051
ENGINEER: LAB
DATE: December 16, 2019

Preliminary Drainage Plan
CURRENT CONDITIONS COEFFICIENT "C" CALCULATIONS

BASIN	Soil No.	Hydro Group	Area	Cover (%)	C5	Wtd. C5	C100	Wtd. C100	Impervious	Type of Cover
OS-E2.1	108	B	32.10	74.34%	0.15	0.11	0.41	0.30	10%	5 Ac. Tracts
	54	D	11.08	25.66%	0.22	0.06	0.53	0.14	10%	5 Ac. Tracts
			43.18	100.00%		0.17		0.44		
EX-E2	56/108	B	24.15	75.92%	0.09	0.07	0.36	0.27	2%	Undeveloped
	52	C	2.08	6.54%	0.49	0.03	0.65	0.04	2%	Undeveloped
	52	C	5.58	17.54%	0.16	0.03	0.51	0.09	65%	Single Family
			31.81	100.00%		0.13		0.41		
OS-J1.1	56	B	1.10	9.52%	0.30	0.03	0.50	0.05	40%	0.25 Ac. Tracts
	56	B	6.25	54.11%	0.15	0.08	0.41	0.22	10%	5 Ac. Tracts
	54	D	4.20	36.36%	0.22	0.08	0.53	0.19	10%	5 Ac. Tracts
			11.55	100.00%		0.19		0.46		
DP-1	56	B	1.29	4.73%	0.30	0.01	0.50	0.02	100%	5 Ac. Tracts
	56	B	6.34	23.23%	0.15	0.03	0.41	0.10	80%	0.25 Ac. Tracts
	54	D	4.37	16.01%	0.22	0.04	0.53	0.08	95%	5 Ac. Tracts
	104	A	15.29	56.03%	0.09	0.05	0.36	0.20	40%	Yard Area
			27.29	100.00%		0.13		0.41		
DP-2	56/108	B	15.54	54.87%	0.15	0.08	0.41	0.22	100%	Hard Surface
	56	B	12.78	45.13%	0.09	0.04	0.36	0.16	80%	Light Industrial
			28.32	100.00%		0.12		0.39		
DP-3	108	B	32.19	42.80%	0.15	0.06	0.41	0.18	10%	5 Ac. Tracts
	54	D	11.21	14.90%	0.22	0.03	0.53	0.08	10%	5 Ac. Tracts
	56/108	B	24.15	32.11%	0.09	0.03	0.36	0.12	2%	Undeveloped
	52	C	2.08	2.77%	0.49	0.01	0.65	0.02	2%	Undeveloped
	52	C	5.58	7.42%	0.16	0.01	0.51	0.04	65%	Single Family
			75.21	100.00%		0.15		0.43		

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

CORE ENGINEERING GROUP

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **5 - Year Event (Proposed)**

Street or Basin	Design Point	Area Design	Direct Runoff						Total Runoff				Street		Pipe			Travel Time			Remarks
			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	
E7			0.60	0.49	10.1	0.29	4.12	1.2													
E8.1			4.00	0.30	16.6	1.20	3.37	4.0													
OS-E2.1			36.66	0.17	30.5	6.23	2.46	15.3													
E8.2			1.70	0.19	12.8	0.32	3.76	1.2													
OS-E2.1 - E8.2	**14**	38.36							31.4	6.56	2.41	15.8									
OS-E2.2			6.52	0.15	16.0	0.98	3.43	3.4													
E8.3			5.37	0.26	16.6	1.40	3.37	4.7													
OS-E2.2 - E8.3	**15**	11.89							27.2	2.37	2.63	6.2									
E8.1 & DP-15	**16**	15.89							27.2	3.57	2.63	9.4									
DP-14 thru DP-16		54.25							31.4	10.13	2.41	24.4									
E8.4			1.20	0.45	14.3	0.54	3.59	1.9													
E8.5			4.27	0.45	14.7	1.92	3.55	6.8													
E8.6			1.02	0.49	10.5	0.50	4.06	2.0													
E8.7			0.71	0.47	9.4	0.33	4.22	1.4													
E8.6 & E8.7	**18**	1.73							10.5	0.83	4.06	3.4									
E8.8			2.43	0.45	12.8	1.09	3.76	4.1													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **5 - Year Event (Proposed)**

Street or Basin	Design Point	Direct Runoff: Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	Total Runoff: tc	Σ (CA)	i	Q	Street: Slope	Street Flow	Pipe: Design Flow	Slope	Pipe Size	Travel Time: Length	Velocity	tt	Remarks
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
E8.4 - E8.8	**19**	9.63							16.1	4.39	3.42	15.0									
E8.1 - E8.8		63.88							32.2	14.52	2.38	34.5									
E8.9			1.52	0.45	14.5	0.68	3.57	2.4													
E8.10			0.38	0.47	5.2	0.18	5.12	0.9													
E8.9 - E8.10	**20**	1.90							14.5	0.86	3.57	3.1									
OS-E2.1 - E8.10		65.78							31.8	15.38	2.40	36.8									
E8.11			3.99	0.21	12.4	0.84	3.80	3.2													
E8		69.77							31.8	16.22	2.40	38.8									
OS-I1.1			15.54	0.15	23.1	2.33	2.87	6.7													
I1.2			6.23	0.27	16.0	1.68	3.43	5.8													
I1	**4**	21.77							25.0	4.01	2.76	11.1									
I2			1.12	0.45	12.7	0.50	3.77	1.9													
I3			1.01	0.45	12.5	0.45	3.79	1.7													
I4			3.45	0.45	14.2	1.55	3.61	5.6													
I2 - I4		5.58							15.4	2.51	3.48	8.7									
OS-I1.1-I4		27.35							25.2	6.52	2.74	17.9									

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **5 - Year Event (Proposed)**

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	
I5			1.44	0.45	10.3	0.65	4.09	2.7													
OS-I1.1-I5		28.79							25.7	7.17	2.71	19.5									
I6			1.18	0.45	12.6	0.53	3.78	2.0													
OS-I1.1-I6		29.97							26.6	7.70	2.66	20.5									
DP-2a & DP-8		35.30							28.6	10.10	2.55	25.8									
I7			2.01	0.45	19.4	0.90	3.13	2.8													
I7 & J5	**10**	3.94							19.4	1.77	3.13	5.6									
DP-9 & DP-10	**11**	39.24							28.6	11.87	2.55	30.3									
I8			2.44	0.21	13.7	0.51	3.66	1.9													
I	**12**	34.42							26.6	9.12	2.66	24.3									
OS-J1.1			11.55	0.19	28.1	2.19	2.58	5.7													
J1.2			2.01	0.17	12.8	0.34	3.76	1.3													
J1	**1**	13.56							29.4	2.54	2.51	6.4									
J2			0.36	0.62	6.0	0.22	4.90	1.1													
OS-J1.1 - J2	**2**	13.92							29.4	2.76	2.51	6.9									
J3			3.40	0.45	14.5	1.53	3.57	5.5													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **5 - Year Event (Proposed)**

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	
J4			1.93	0.45	11.8	0.87	3.89	3.4													
J3-J4		5.33							14.5	2.40	3.57	8.6									
J5			1.93	0.45	14.1	0.87	3.62	3.1													
J		21.18							28.6	6.03	2.55	15.4									
I & J		55.60							28.6	15.15	2.55	38.7									
K1			2.65	0.12	12.5	0.32	3.79	1.2													
K2			2.14	0.12	6.2	0.26	4.84	1.2													
L			2.30	0.12	5.0	0.28	5.17	1.4													
M			0.48	0.12	6.3	0.06	4.83	0.3													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **100 - Year Event (Proposed)**

Street or Basin	Design Point	Area Design	Direct Runoff						Total Runoff				Street		Pipe			Travel Time			Remarks
			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	
E7			0.60	0.65	10.1	0.39	6.91	2.7													
E8.1			4.00	0.57	16.6	2.28	5.66	12.9													
OS-E2.1			36.66	0.45	30.5	16.50	4.13	68.1													
E8.2			1.70	0.46	12.8	0.78	6.31	4.9													
OS-E2.1 - E8.2	**14**	38.36							31.4	17.28	4.05	70.0									
OS-E2.2			6.52	0.41	16.0	2.67	5.75	15.4													
E8.3			5.37	0.47	16.6	2.52	5.65	14.3													
OS-E2.2 - E8.3	**15**	11.89							27.2	5.20	4.42	22.9									
E8.1 & DP-15	**16**	15.89							27.2	7.48	4.42	33.0									
DP-14 thru DP-16		54.25							31.4	24.76	4.05	100.2									
E8.4			1.20	0.59	14.3	0.71	6.03	4.3													
E8.5			4.27	0.59	14.7	2.52	5.96	15.0													
E8.6			1.02	0.65	10.5	0.66	6.81	4.5													
E8.7			0.71	0.62	9.4	0.44	7.08	3.1													
E8.6 & E8.7	**18**	1.73							10.5	1.10	6.81	7.5									
E8.8			2.43	0.59	12.8	1.43	6.31	9.1													
E8.4 - E8.8	**19**	9.63							16.1	5.76	5.73	33.1									
E8.1 - E8.8		63.88							32.2	30.52	3.99	121.7									

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **100 - Year Event (Proposed)**

Street or Basin	Design Point	Area Design	Direct Runoff: Area (A)	Direct Runoff: Runoff Coeff. (C)	Direct Runoff: tc	Direct Runoff: CA	Direct Runoff: i	Direct Runoff: Q	Total Runoff: tc	Total Runoff: Σ (CA)	Total Runoff: i	Total Runoff: Q	Street: Slope	Street: Street Flow	Pipe: Design Flow	Pipe: Slope	Pipe: Pipe Size	Travel Time: Length	Travel Time: Velocity	Travel Time: tt	Remarks
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
E8.9			1.52	0.59	14.5	0.90	5.99	5.4													
E8.10			0.38	0.62	5.2	0.24	8.59	2.0													
E8.9 - E8.10	**20**	1.90							14.5	1.13	5.99	6.8									
OS-E2.1 - E8.10		65.78							31.8	31.65	4.02	127.2									
E8.11			3.99	0.48	12.4	1.92	6.38	12.2													
E8		69.77							34.1	33.57	3.84	129.0									
OS-I1.1			15.54	0.41	23.1	6.37	4.82	30.7													
I1.2			6.23	0.49	16.0	3.05	5.75	17.6													
I1	**4**	21.77							25.0	9.42	4.62	43.6									
I2			1.12	0.59	12.7	0.66	6.34	4.2													
I3			1.01	0.59	12.5	0.60	6.37	3.8													
I4			3.45	0.59	14.2	2.04	6.05	12.3													
I2 - I4		5.58							15.4	3.29	5.85	19.3									
OS-I1.1-I4		27.35							25.2	12.72	4.61	58.6									
I5			1.44	0.59	10.3	0.85	6.87	5.8													
OS-I1.1-I5		28.79							25.7	13.57	4.55	61.8									
I6			1.18	0.59	12.6	0.70	6.35	4.4													
OS-I1.1-I6		29.97							26.6	14.26	4.47	63.7									
DP-2a & DP-8		35.30							28.6	23.83	4.29	102.2									

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **100 - Year Event (Proposed)**

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	
I7			2.01	0.59	19.4	1.19	5.26	6.2													
I7 & J5		3.94							19.4	2.32	5.26	12.2									
DP-9 & DP-10		39.24							28.6	26.16	4.29	112.1									
I8			2.44	0.44	13.7	1.07	6.14	6.6													
I		34.42							26.6	16.52	4.47	73.8									
OS-J1.1			11.55	0.46	28.1	5.31	4.33	23.0													
J1.2			2.01	0.42	12.8	0.84	6.32	5.3													
J1	**1**	13.56							29.4	6.16	4.22	26.0									
J2			0.36	0.75	6.0	0.27	8.23	2.2													
OS-J1.1 - J2	**2**	13.92							29.4	6.43	4.22	27.1									
J3			3.40	0.59	14.5	2.01	5.99	12.0													
J4			1.93	0.59	11.8	1.14	6.52	7.4													
J3-J4		5.33							14.5	3.14	6.00	18.9									
J5			1.93	0.59	14.1	1.14	6.08	6.9													
J		21.18							28.6	10.71	4.29	45.9									
I & J		55.60							28.6	27.23	4.29	116.7									
K1			2.65	0.39	12.5	1.03	6.37	6.6													
K2			2.14	0.39	6.2	0.83	8.13	6.8													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: January 15, 2020
Checked By: Leonard Beasley

Job No: 100.051
Project: Creekside South at Lorson Ranch
Design Storm: **100 - Year Event (Proposed)**

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	
L			2.30	0.39	5.0	0.90	8.68	7.8													
M			0.48	0.39	6.3	0.19	8.11	1.5													

APPENDIX C – HYDRAULIC CALCULATIONS

Channel Report

DES.PT. 1 SWALE

Trapezoidal

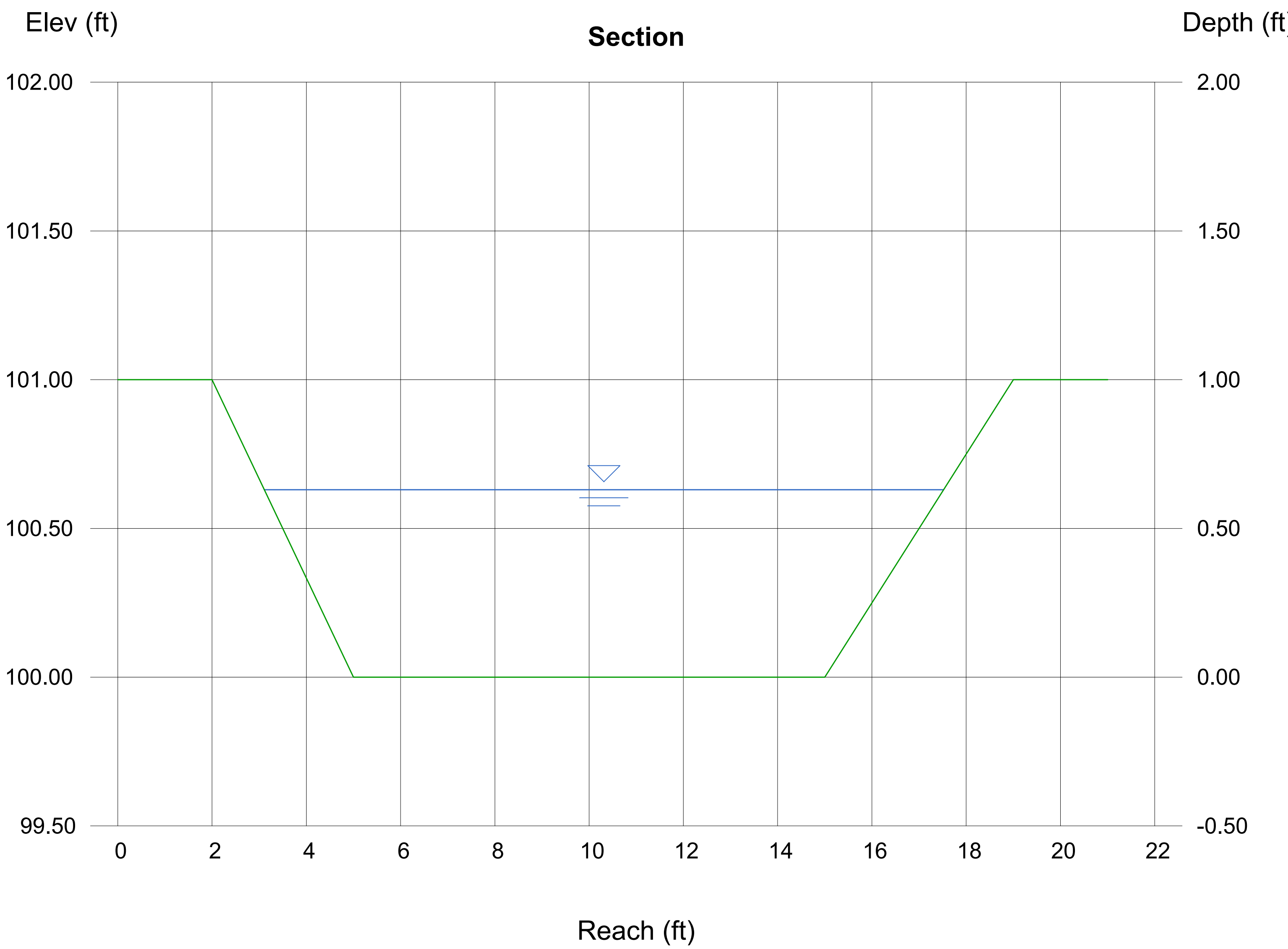
Botom Width (ft)	=	10.00
Side Slope (z:1)	=	3.00
Total Depth (ft)	=	1.00
Invert Elev (ft)	=	100.00
Slope (%)	=	0.50
N-Value	=	0.020

Calculations

Compute by:		Known Q
Known Q (cfs)	=	26.00

Highlighted

Depth (ft)	=	0.63
Q (cfs)	=	26.00
Area (sqft)	=	7.69
Velocity (ft/s)	=	3.38
Wetted Perim (ft)	=	14.59
Crit Depth, Yc (ft)	=	0.56
Top Width (ft)	=	14.41
EGL (ft)	=	0.81



Culvert Report

End Section Des. Pt. 1

Invert Elev Dn (ft)	=	100.00
Pipe Length (ft)	=	23.00
Slope (%)	=	1.00
Invert Elev Up (ft)	=	100.23
Rise (in)	=	24.0
Shape	=	Cir
Span (in)	=	24.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Projecting
Coeff. K,M,c,Y,k	=	0.0045, 2, 0.0317, 0.69, 0.5

Embankment

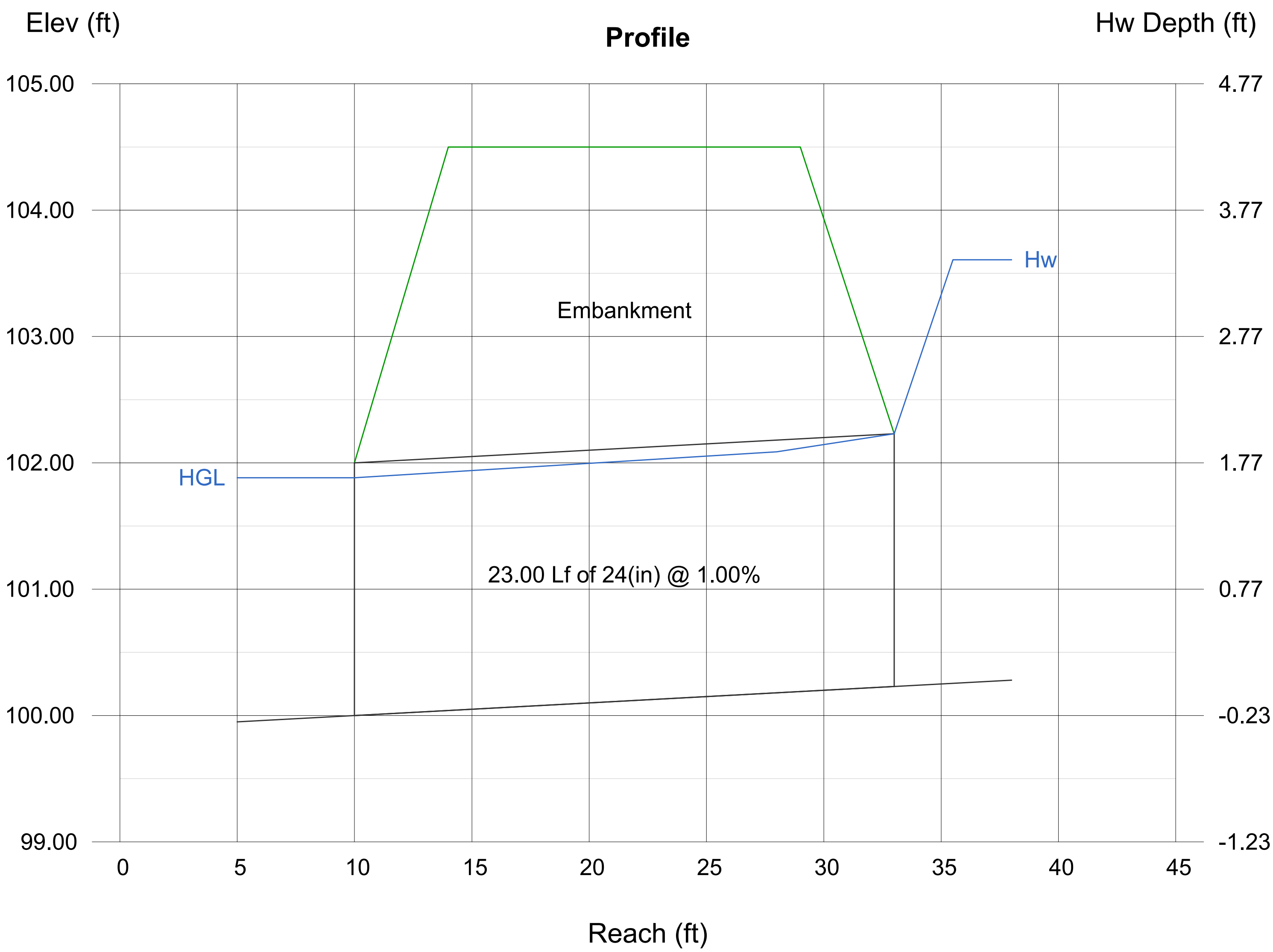
Top Elevation (ft)	=	104.50
Top Width (ft)	=	15.00
Crest Width (ft)	=	100.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	30.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	25.00
Qpipe (cfs)	=	25.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	9.65
Veloc Up (ft/s)	=	9.55
HGL Dn (ft)	=	101.88
HGL Up (ft)	=	102.14
Hw Elev (ft)	=	103.61
Hw/D (ft)	=	1.69
Flow Regime	=	Inlet Control



Culvert Report

Hydraflow Express by Intelisolve

Wednesday, Jan 22 2020, 4:59 PM

36inch end section -Horton-Shunka

Invert Elev Dn (ft)	=	99.58
Pipe Length (ft)	=	50.00
Slope (%)	=	0.84
Invert Elev Up (ft)	=	100.00
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Projecting
Coeff. K,M,c,Y,k	=	0.0045, 2, 0.0317, 0.69, 0.5

Embankment

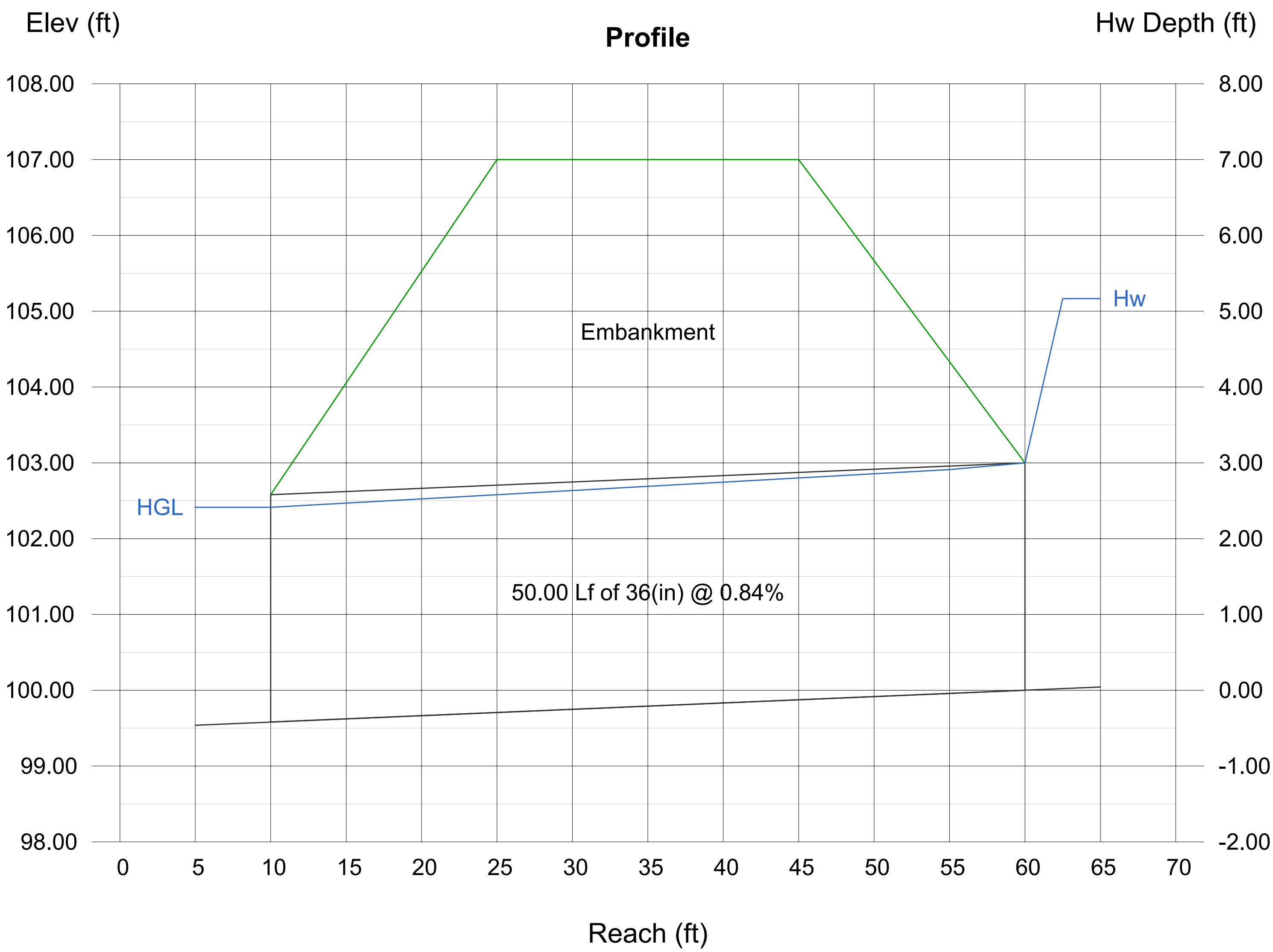
Top Elevation (ft)	=	107.00
Top Width (ft)	=	20.00
Crest Width (ft)	=	100.00

Calculations

Qmin (cfs)	=	70.00
Qmax (cfs)	=	70.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	70.00
Qpipe (cfs)	=	70.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	10.13
Veloc Up (ft/s)	=	9.92
HGL Dn (ft)	=	102.41
HGL Up (ft)	=	102.97
Hw Elev (ft)	=	105.17
Hw/D (ft)	=	1.72
Flow Regime	=	Inlet Control



Channel Report

pond J outflow channel

Trapezoidal

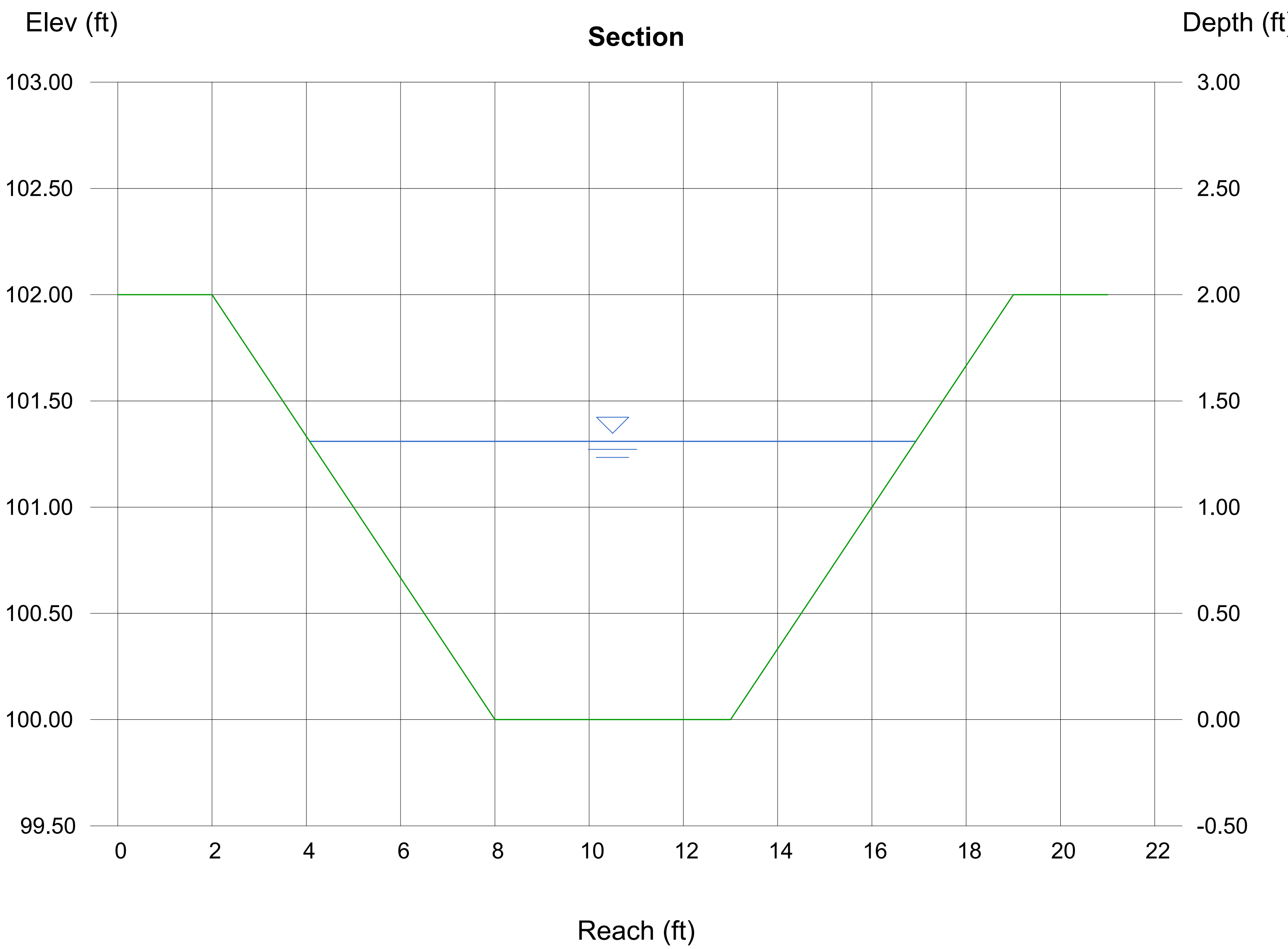
Botom Width (ft)	= 5.00
Side Slope (z:1)	= 3.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 100.00
Slope (%)	= 0.50
N-Value	= 0.020

Calculations

Compute by:	Known Q
Known Q (cfs)	= 56.00

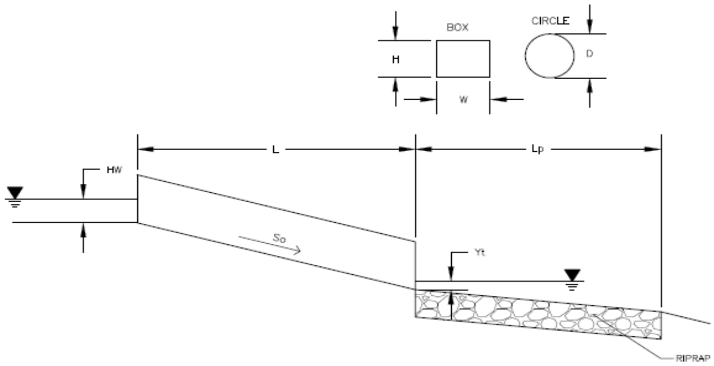
Highlighted

Depth (ft)	= 1.31
Q (cfs)	= 56.00
Area (sqft)	= 11.70
Velocity (ft/s)	= 4.79
Wetted Perim (ft)	= 13.29
Crit Depth, Yc (ft)	= 1.23
Top Width (ft)	= 12.86
EGL (ft)	= 1.67



Determination of Culvert Headwater and Outlet Protection

Project: Creekside South
Basin ID: Rip Rap sizing for outlet pipe from Pond J to Etrib



Soil Type:
Choose One:
(●) Sandy
( ) Non-Sandy

Design Information (Input):

Design Discharge	Q =	55.5	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	36	inches
Inlet Edge Type (Choose from pull-down list)		Square End Projection	
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	100	ft
Outlet Elevation **OR** Slope	Elev OUT =	99.75	ft
Culvert Length	L =	50	ft
Manning's Roughness	n =	0.013	
Bend Loss Coefficient	k_b =	0	
Exit Loss Coefficient	k_x =	1	
Tailwater Surface Elevation	Elev Y_t =	101	ft
Max Allowable Channel Velocity	V =	5	ft/s

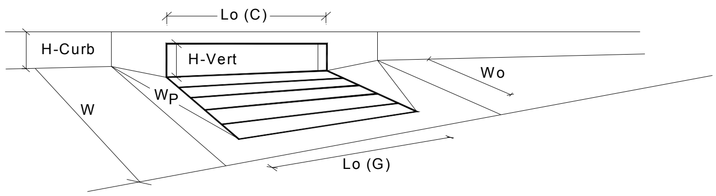
Required Protection (Output):

Tailwater Surface Height	Y_t =	1.25	ft
Flow Area at Max Channel Velocity	A_t =	11.10	ft^2
Culvert Cross Sectional Area Available	A =	7.07	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	0.36	
Sum of All Losses Coefficients	k_s =	1.86	ft
Culvert Normal Depth	Y_n =	2.11	ft
Culvert Critical Depth	Y_c =	2.42	ft
Tailwater Depth for Design	d =	2.71	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	-	ft
Expansion Factor	1/(2*tan(Θ)) =	4.16	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	3.56	$ft^{0.5}$/s
Froude Number	Fr =	-	**Pressure flow!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.42	
Inlet Control Headwater	HW_I =	4.51	ft
Outlet Control Headwater	HW_O =	4.24	
Design Headwater Elevation	**HW =**	**104.51**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**1.50**	**HW/D > 1.5!**
Minimum Theoretical Riprap Size	d_{50} =	8	in
Nominal Riprap Size	d_{50} =	9	in
UDFCD Riprap Type	**Type =**	**L**	
Length of Protection	L_p =	**25**	**ft**
Width of Protection	**T =**	**10**	**ft**

INLET E8.1

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet: CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.4	6.5	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) =	25.00	25.00	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	

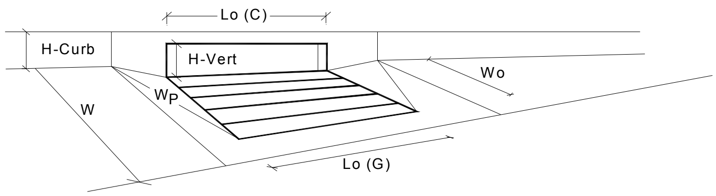
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.29	0.38	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.51	0.61	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.75	0.82	
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 =	**11.5**	**18.9**	**cfs**
WARNING: Inlet Capacity less than Q Peak for Major Storm	$Q_{PEAK\ REQUIRED}$ =	9.4	33.0	cfs

INLET E8.5

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017

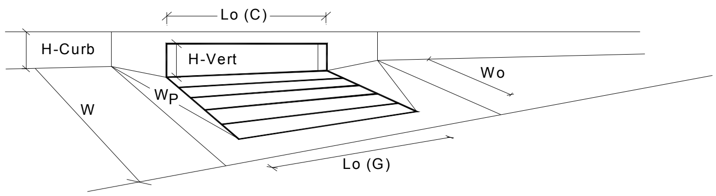


Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	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) =	25.00	25.00	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	
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	0.50	ft
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	0.57	0.76	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	0.79	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 =	**15.2**	**32.0**	**cfs**
WARNING: Inlet Capacity less than Q Peak for Major Storm		$Q_{PEAK\ REQUIRED}$ =	15.0	33.1	cfs

INLET E8.9

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet: CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	2	2	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	4.2	7.2	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	

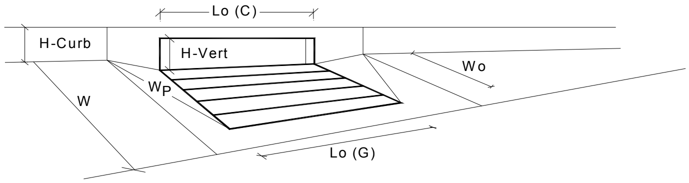
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.18	0.43	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.39	0.68	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.65	0.85	
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 =	**4.7**	**23.1**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	3.1	22.0	cfs

INLET I1

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017

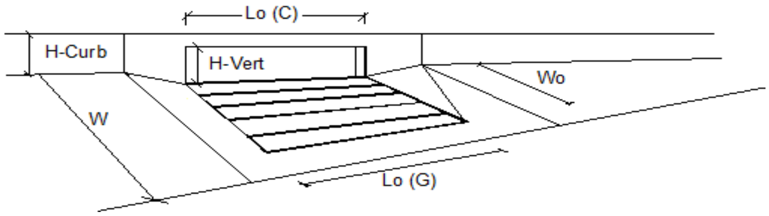


		MINOR	MAJOR	
Design Information (Input) CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	6.5	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) =	20.00	20.00	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 =	1.33	1.33	
Clogging Factor for Multiple Units	Clog =	0.03	0.03	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	12.9	16.0	cfs
Interception with Clogging	Q_{wa} =	12.5	15.5	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	39.0	40.5	cfs
Interception with Clogging	Q_{oa} =	37.7	39.2	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	20.9	23.7	cfs
Interception with Clogging	Q_{ma} =	20.2	22.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**12.5**	**15.5**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	20.00	20.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	18.7	20.8	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.4	0.9	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	0.38	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.57	0.61	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.79	0.82	
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 =	**12.5**	**15.5**	**cfs**
WARNING: Inlet Capacity less than Q Peak for Major Storm	$Q_{PEAK\ REQUIRED}$ =	11.1	43.4	cfs

INLET I2

INLET ON A CONTINUOUS GRADE

Version 4.05 Released March 2017

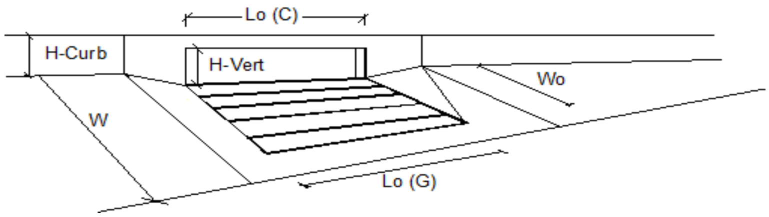


Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	5.00	5.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C_f-C =	0.10	0.10	
Street Hydraulics: WARNING: Q > ALLOWABLE Q FOR MAJOR STORM		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)	**Q_o =**	**8.7**	**47.4**	cfs
Water Spread Width	T =	16.7	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	5.5	9.4	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	3.8	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.358	0.205	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	5.6	33.6	cfs
Discharge within the Gutter Section W	Q_w =	3.1	8.7	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	5.1	cfs
Flow Area within the Gutter Section W	A_W =	0.75	1.40	sq ft
Velocity within the Gutter Section W	V_W =	4.1	6.2	fps
Water Depth for Design Condition	d_{LOCAL} =	8.5	12.4	inches
Grate Analysis (Calculated)		MINOR	MAJOR	
Total Length of Inlet Grate Opening	L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow	$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition		MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Interception Capacity	Q_i =	N/A	N/A	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet	GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet	GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Actual Interception Capacity	**Q_a =**	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)	**Q_b =**	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)		MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)	S_e =	0.087	0.059	ft/ft
Required Length L_T to Have 100% Interception	L_T =	17.33	46.62	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	5.00	5.00	ft
Interception Capacity	Q_i =	4.0	7.8	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.00	1.00	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.10	0.10	
Effective (Unclogged) Length	L_e =	4.50	4.50	ft
Actual Interception Capacity	**Q_a =**	**3.6**	**7.1**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	**Q_b =**	**5.1**	**40.3**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**3.6**	**7.1**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**5.1**	**40.3**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**42**	**15**	**%**

INLET I5

INLET ON A CONTINUOUS GRADE

Version 4.05 Released March 2017

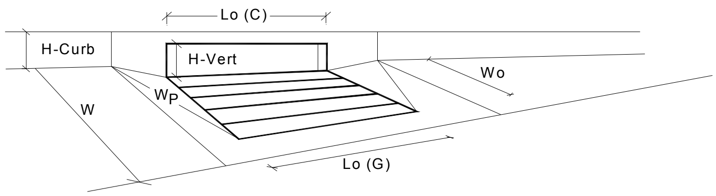


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	5.00	5.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C_f-C =	0.10	0.10	
Street Hydraulics: WARNING: Q > ALLOWABLE Q FOR MAJOR STORM		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)	**Q_o =**	**7.8**	**46.1**	cfs
Water Spread Width	T =	16.0	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	5.3	9.3	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	3.7	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.374	0.206	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	4.9	32.8	cfs
Discharge within the Gutter Section W	Q_w =	2.9	8.5	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	4.8	cfs
Flow Area within the Gutter Section W	A_W =	0.72	1.38	sq ft
Velocity within the Gutter Section W	V_W =	4.0	6.2	fps
Water Depth for Design Condition	d_{LOCAL} =	8.3	12.3	inches
Grate Analysis (Calculated)		MINOR	MAJOR	
Total Length of Inlet Grate Opening	L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow	$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition		MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Interception Capacity	Q_i =	N/A	N/A	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet	GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet	GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Actual Interception Capacity	**Q_a =**	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)	**Q_b =**	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)		MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)	S_e =	0.090	0.059	ft/ft
Required Length L_T to Have 100% Interception	L_T =	16.13	45.96	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	5.00	5.00	ft
Interception Capacity	Q_i =	3.8	7.7	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.00	1.00	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.10	0.10	
Effective (Unclogged) Length	L_e =	4.50	4.50	ft
Actual Interception Capacity	**Q_a =**	**3.5**	**7.0**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	**Q_b =**	**4.3**	**39.1**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**3.5**	**7.0**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**4.3**	**39.1**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**44**	**15**	**%**

INLET I6

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



		MINOR	MAJOR	
Design Information (Input)				
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	7.8	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	

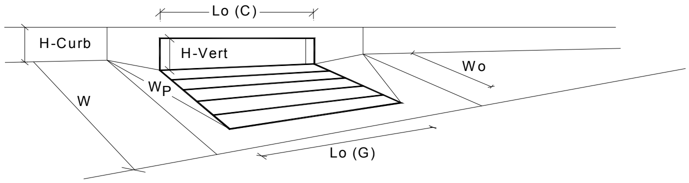
		MINOR	MAJOR	
Low Head Performance Reduction (Calculated)				
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.33	0.48	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.57	0.74	
Curb Opening Performance Reduction Factor 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 =	**8.3**	**15.5**	**cfs**
WARNING: Inlet Capacity less than Q Peak for Minor and Major Storms	$Q_{PEAK\ REQUIRED}$ =	8.3	38.1	cfs

INLET I7

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017

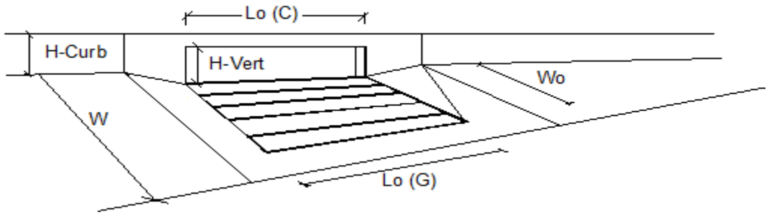


		MINOR	MAJOR	
Design Information (Input) CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	4.6	7.8	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) =	45.00	45.00	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 =	1.33	1.33	
Clogging Factor for Multiple Units	Clog =	0.01	0.01	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	11.8	52.1	cfs
Interception with Clogging	Q_{wa} =	11.6	51.3	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	77.0	99.5	cfs
Interception with Clogging	Q_{oa} =	75.9	98.0	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	28.0	66.9	cfs
Interception with Clogging	Q_{ma} =	27.6	65.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**11.6**	**51.3**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	45.00	45.00	feet
Resultant Street Flow Spread (based on street geometry from above)	T =	12.7	26.2	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	2.2	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} =	0.21	0.48	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.43	0.74	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	0.68	0.88	
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 =	**11.6**	**51.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	5.0	50.7	cfs

INLET J2

INLET ON A CONTINUOUS GRADE

Version 4.05 Released March 2017

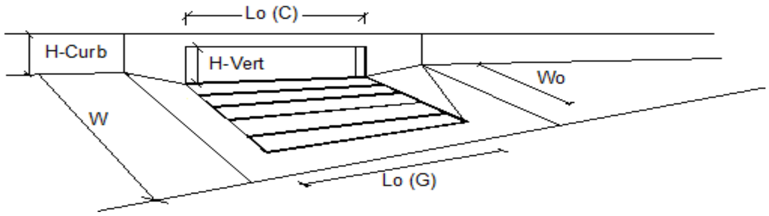


Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')		a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)		No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)		L_o =	5.00	5.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)		W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)		C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)		C_f-C =	0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity'			MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)		Q_o =	**1.1**	**2.2**	cfs
Water Spread Width		T =	5.0	7.5	ft
Water Depth at Flowline (outside of local depression)		d =	2.7	3.3	inches
Water Depth at Street Crown (or at T_{MAX})		d_{CROWN} =	0.0	0.0	inches
Ratio of Gutter Flow to Design Flow		E_o =	0.893	0.722	
Discharge outside the Gutter Section W, carried in Section T_x		Q_x =	0.1	0.6	cfs
Discharge within the Gutter Section W		Q_w =	1.0	1.6	cfs
Discharge Behind the Curb Face		Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W		A_W =	0.28	0.39	sq ft
Velocity within the Gutter Section W		V_W =	3.5	4.1	fps
Water Depth for Design Condition		d_{LOCAL} =	5.7	6.3	inches
Grate Analysis (Calculated)			MINOR	MAJOR	
Total Length of Inlet Grate Opening		L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow		$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition			MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins		V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow		R_f =	N/A	N/A	
Interception Rate of Side Flow		R_x =	N/A	N/A	
Interception Capacity		Q_i =	N/A	N/A	cfs
Under Clogging Condition			MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet		GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet		GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet		L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins		V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow		R_f =	N/A	N/A	
Interception Rate of Side Flow		R_x =	N/A	N/A	
Actual Interception Capacity		Q_a =	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)		Q_b =	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)			MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)		S_e =	0.188	0.155	ft/ft
Required Length L_T to Have 100% Interception		L_T =	4.45	6.92	ft
Under No-Clogging Condition			MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)		L =	4.45	5.00	ft
Interception Capacity		Q_i =	1.1	2.0	cfs
Under Clogging Condition			MINOR	MAJOR	
Clogging Coefficient		CurbCoef =	1.00	1.00	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet		CurbClog =	0.10	0.10	
Effective (Unclogged) Length		L_e =	4.50	4.50	ft
Actual Interception Capacity		Q_a =	**1.1**	**1.9**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a		Q_b =	**0.0**	**0.3**	cfs
Summary			MINOR	MAJOR	
Total Inlet Interception Capacity		**Q =**	**1.1**	**1.9**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)		**Q_b =**	**0.0**	**0.3**	**cfs**
Capture Percentage = Q_a/Q_o =		**C% =**	**100**	**85**	**%**

INLET J4

INLET ON A CONTINUOUS GRADE

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')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	10.00	10.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C_f-C =	0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)	Q_o =	**8.6**	**19.2**	cfs
Water Spread Width	T =	13.9	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	4.9	6.1	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	0.5	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.429	0.309	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	4.9	13.3	cfs
Discharge within the Gutter Section W	Q_w =	3.7	5.9	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	0.64	0.86	sq ft
Velocity within the Gutter Section W	V_W =	5.7	6.9	fps
Water Depth for Design Condition	d_{LOCAL} =	7.9	9.1	inches
Grate Analysis (Calculated)		MINOR	MAJOR	
Total Length of Inlet Grate Opening	L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow	$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition		MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Interception Capacity	Q_i =	N/A	N/A	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet	GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet	GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Actual Interception Capacity	Q_a =	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)	Q_b =	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)		MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)	S_e =	0.100	0.078	ft/ft
Required Length L_T to Have 100% Interception	L_T =	16.97	28.69	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	10.00	10.00	ft
Interception Capacity	Q_i =	6.9	10.3	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.25	1.25	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.06	0.06	
Effective (Unclogged) Length	L_e =	8.75	8.75	ft
Actual Interception Capacity	Q_a =	**6.6**	**9.9**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	**2.0**	**9.3**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	**6.6**	**9.9**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q_b =	**2.0**	**9.3**	**cfs**
Capture Percentage = Q_a/Q_o =	C% =	**77**	**52**	**%**

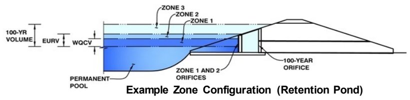
APPENDIX D – POND CALCULATIONS

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Creekside South FDR

Basin ID: POND E2



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	125.00	acres		
Watershed Length =	2,900	ft		
Watershed Slope =	0.030	ft/ft		
Watershed Imperviousness =	35.00%	percent		
Percentage Hydrologic Soil Group A =	0.0%	percent		
Percentage Hydrologic Soil Group B =	40.0%	percent		
Percentage Hydrologic Soil Groups C/D =	60.0%	percent		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =	User Input			
Water Quality Capture Volume (WQCV) =	1.732	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	4.232	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	3.645	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	5.556	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	7.847	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	12.045	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	14.974	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	18.724	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	3.411	acre-feet		
Approximate 5-yr Detention Volume =	5.233	acre-feet		
Approximate 10-yr Detention Volume =	6.508	acre-feet		
Approximate 25-yr Detention Volume =	7.293	acre-feet		
Approximate 50-yr Detention Volume =	7.634	acre-feet		
Approximate 100-yr Detention Volume =	9.039	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	1.732	acre-feet
Zone 2 Volume (EURV - Zone 1) =	2.500	acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) =	5.673	acre-feet
Total Detention Basin Volume =	9.905	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	--	--	--	20	0.000		
5693.33	--	0.33	--	--	--	50	0.001	11	0.000
5694	--	1.00	--	--	--	2,250	0.052	760	0.017
5695	--	2.00	--	--	--	35,024	0.804	19,070	0.438
5696	--	3.00	--	--	--	62,057	1.425	67,959	1.560
5697	--	4.00	--	--	--	65,120	1.495	131,548	3.020
5698	--	5.00	--	--	--	68,248	1.567	198,232	4.551
5699	--	6.00	--	--	--	71,443	1.640	268,077	6.154
5700	--	7.00	--	--	--	74,705	1.715	341,151	7.832
5701	--	8.00	--	--	--	78,040	1.792	417,524	9.585
5702	--	9.00	--	--	--	81,442	1.870	497,265	11.416
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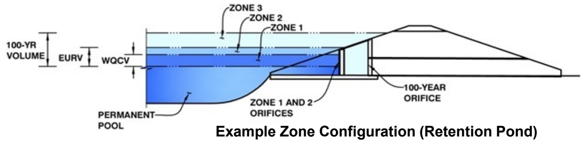
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Creekside South FDR

Basin ID: POND E2



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	3.13	1.732	Orifice Plate
Zone 2 (EURV)	4.80	2.500	Rectangular Orifice
(100+1/2WQCV)	8.18	5.673	Weir&Pipe (Restrict)
		9.905	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 =	3.12	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	12.00	inches
Orifice Plate: Orifice Area per Row =	5.25	sq. inches (use rectangular openings)

Calculated Parameters for Plate

WQ Orifice Area per Row =	3.646E-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	1.00	2.00	3.00				
Orifice Area (sq. inches)	5.25	5.25	5.25	5.25				

	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 =	3.50	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	4.80	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	4.00	N/A	inches
Vertical Orifice Width =	18.00		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.50	N/A	ft^2
Vertical Orifice Centroid =	0.17	N/A	feet

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

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	5.85	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	17.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	6.70	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 =	5.85	N/A	feet
Over Flow Weir Slope Length =	6.70	N/A	feet
Grate Open Area / 100-yr Orifice Area =	6.34	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	79.73	N/A	ft^2
Overflow Grate Open Area w/ Debris =	39.87	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 =	0.10	N/A	ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter =	48.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	48.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	12.57	N/A	ft^2
Outlet Orifice Centroid =	2.00	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	3.14	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	8.20	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	55.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	0.50	feet

Calculated Parameters for Spillway

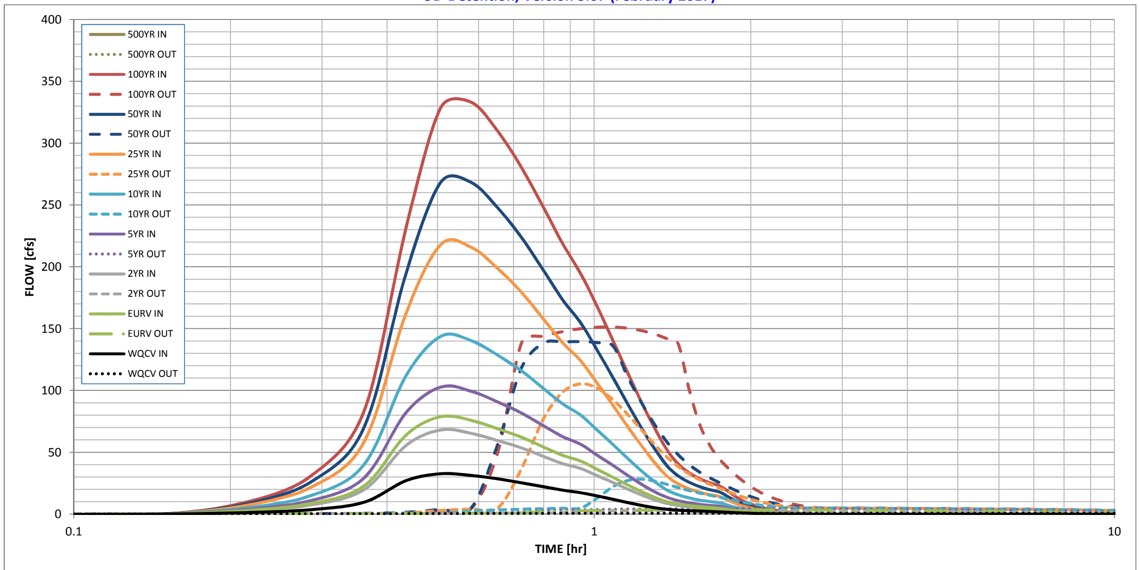
Spillway Design Flow Depth=	1.51	feet
Stage at Top of Freeboard =	10.21	feet
Basin Area at Top of Freeboard =	1.87	acres

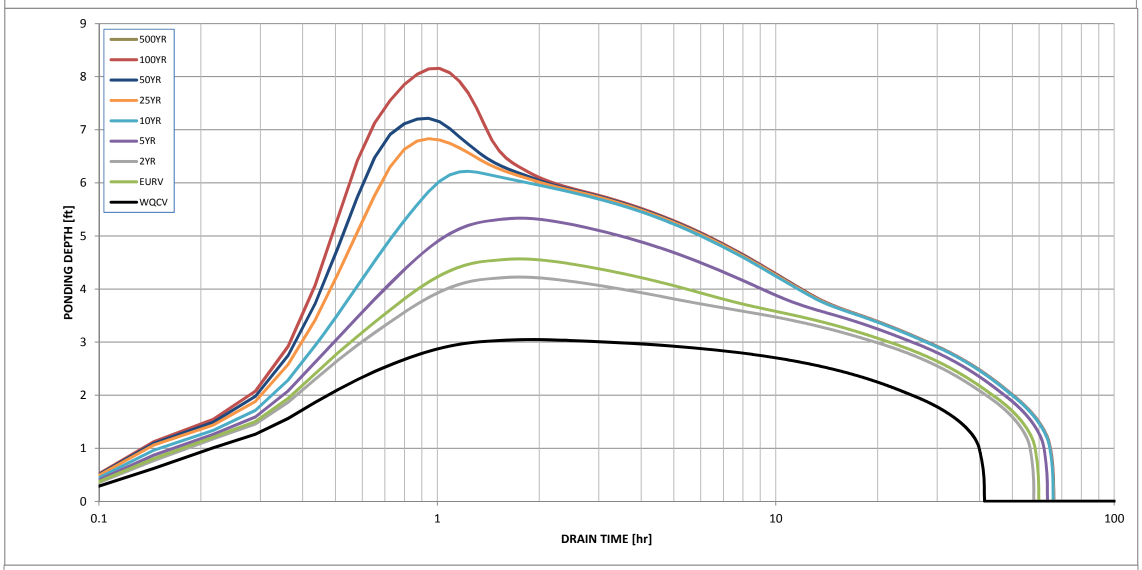
Routed Hydrograph Results

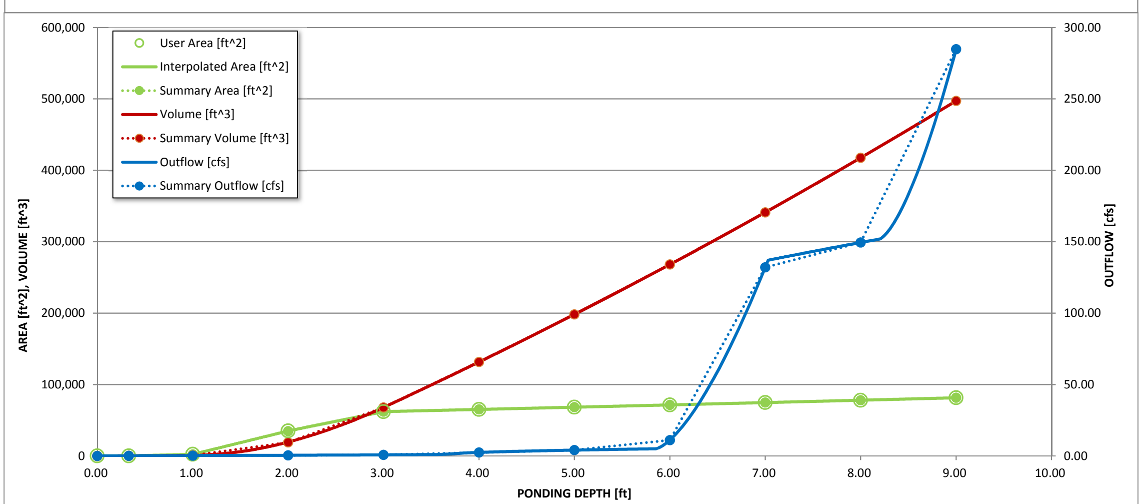
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	1.732	4.232	3.645	5.556	7.847	12.045	14.974	18.724	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	1.732	4.233	3.646	5.554	7.843	12.049	14.970	18.731	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.02	0.10	0.34	0.87	1.17	1.53	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	2.0	12.4	42.6	108.8	146.0	191.3	0.0
Peak Inflow Q (cfs) =	32.7	78.8	68.1	102.8	143.9	218.1	268.6	333.2	#N/A
Peak Outflow Q (cfs) =	0.8	3.5	2.9	4.5	28.3	105.4	139.5	151.3	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.4	0.7	1.0	1.0	0.8	#N/A
Structure Controlling Flow =	Plate	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	#N/A
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	0.3	1.2	1.7	1.8	#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	54	53	56	56	53	50	47	#N/A
Time to Drain 99% of Inflow Volume (hours) =	40	57	56	60	62	60	59	58	#N/A
Maximum Ponding Depth (ft) =	3.05	4.57	4.23	5.34	6.22	6.83	7.22	8.16	#N/A
Area at Maximum Ponding Depth (acres) =	1.43	1.54	1.51	1.59	1.66	1.70	1.73	1.80	#N/A
Maximum Volume Stored (acre-ft) =	1.617	3.868	3.351	5.072	6.500	7.541	8.194	9.855	#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 Outflow 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	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]
4.35 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:04:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Hydrograph	0:08:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Constant	0:13:03	1.41	3.27	2.85	4.14	5.52	7.70	9.01	10.37	#N/A
1.150	0:17:24	3.85	9.08	7.89	11.67	15.93	23.16	27.80	33.01	#N/A
	0:21:45	9.88	23.32	20.26	29.97	40.92	59.52	71.52	85.08	#N/A
	0:26:06	27.12	63.93	55.56	82.12	111.96	162.49	194.97	231.60	#N/A
	0:30:27	32.68	78.81	68.07	102.79	143.86	218.07	268.59	328.95	#N/A
	0:34:48	31.28	75.85	65.39	99.41	140.34	215.86	268.33	333.22	#N/A
	0:39:09	28.47	69.09	59.51	90.75	128.54	198.71	247.77	309.79	#N/A
	0:43:30	25.54	62.24	53.59	81.79	115.94	179.40	223.77	280.34	#N/A
	0:47:51	22.18	54.46	46.84	71.70	101.90	158.21	197.69	248.77	#N/A
	0:52:12	19.27	47.54	40.86	62.68	89.18	138.61	173.27	218.72	#N/A
	0:56:33	17.48	42.91	36.92	56.45	80.09	123.94	154.56	194.41	#N/A
	1:00:54	14.56	35.97	30.91	47.48	67.69	105.56	132.19	167.27	#N/A
	1:05:15	11.99	29.81	25.59	39.40	56.27	87.89	110.16	140.09	#N/A
	1:09:36	9.38	23.63	20.24	31.36	45.03	70.81	89.08	114.35	#N/A
	1:13:57	7.13	18.25	15.60	24.32	35.07	55.41	69.87	90.57	#N/A
	1:18:18	5.23	13.64	11.62	18.26	26.47	42.08	53.31	70.11	#N/A
	1:22:39	3.98	10.23	8.73	13.63	19.64	31.02	39.45	52.59	#N/A
	1:27:00	3.24	8.22	7.04	10.91	15.64	24.52	30.97	40.57	#N/A
	1:31:21	2.74	6.92	5.93	9.17	13.12	20.51	25.81	33.46	#N/A
	1:35:42	2.40	6.02	5.16	7.96	11.36	17.71	22.23	28.67	#N/A
	1:40:03	2.15	5.38	4.62	7.11	10.13	15.75	19.75	25.34	#N/A
	1:44:24	1.98	4.93	4.23	6.50	9.26	14.36	17.97	22.96	#N/A
	1:48:45	1.46	3.68	3.15	4.89	7.04	11.13	14.09	18.26	#N/A
	1:53:06	1.06	2.66	2.28	3.53	5.08	8.02	10.16	13.22	#N/A
	1:57:27	0.78	1.97	1.68	2.62	3.77	5.95	7.53	9.76	#N/A
	2:01:48	0.58	1.46	1.25	1.95	2.80	4.42	5.58	7.26	#N/A
	2:06:09	0.42	1.07	0.92	1.43	2.06	3.27	4.14	5.39	#N/A
	2:10:30	0.30	0.77	0.66	1.03	1.49	2.37	3.00	3.95	#N/A
	2:14:51	0.22	0.56	0.48	0.75	1.08	1.71	2.17	2.86	#N/A
	2:19:12	0.15	0.39	0.33	0.52	0.76	1.22	1.56	2.09	#N/A
	2:23:33	0.09	0.25	0.21	0.34	0.50	0.82	1.05	1.44	#N/A
	2:27:54	0.05	0.14	0.12	0.20	0.30	0.49	0.64	0.91	#N/A
	2:32:15	0.02	0.07	0.05	0.09	0.14	0.25	0.33	0.50	#N/A
	2:36:36	0.00	0.02	0.01	0.03	0.05	0.08	0.12	0.21	#N/A
	2:40:57	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	#N/A
	2:45:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	2:49:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	2:54:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	2:58:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:02:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:07:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:11:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:15:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:20:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:24:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:28:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:33:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:37:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:41:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:46:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:50:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:54:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:59:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:03:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:07:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:12:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:16:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:21:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:25:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:29:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:34:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:38:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:42:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:47:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:51:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:55:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:00:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:04:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:08:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:13:12	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]
5693	0.00	20	0.000	0	0.000	0.00
5693.33	0.33	49	0.001	11	0.000	0.10
5694	1.00	2,217	0.051	760	0.017	0.17
5695	2.00	34,696	0.797	19,070	0.438	0.42
5696	3.00	62,057	1.425	67,959	1.560	0.73
5697	4.00	65,120	1.495	131,548	3.020	2.47
5698	5.00	68,248	1.567	198,232	4.551	4.08
5699	6.00	71,443	1.640	268,077	6.154	11.09
5700	7.00	74,705	1.715	341,151	7.832	132.07
5701	8.00	78,040	1.792	417,524	9.585	149.44
5702	9.00	81,442	1.870	497,265	11.416	284.79

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: Extended Detention Basin (EDB)

UD-BMP (Version 3.06, November 2016)

Designer: Richard Schindler
Company: Core Engineering Group
Date: February 16, 2019
Project: Lorson Ranch East 4 FDR - Pond E2 forebay design (ultimate)
Location:

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a	I_a = 35.0 %
B) Tributary Area's Imperviousness Ratio (i = I_a / 100)	i = 0.350
C) Contributing Watershed Area	Area = 125.000 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}= 1.732 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) Predominant Watershed NRCS Soil Group	Choose One: ( ) A ( ) B ( ) C / D — WQCV selected. Soil group not required.
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 = 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 = 0.33 ft / ft **TOO STEEP (< 3)**
4. Inlet	
A) Describe means of providing energy dissipation at concentrated inflow locations:	

Design Procedure Form: Extended Detention Basin (EDB)

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Company: Core Engineering Group
Date: February 16, 2019
Project: Lorson Ranch East 4 FDR - Pond E2 forebay design (ultimate)
Location:

5. Forebay

A) Minimum Forebay Volume
(V_{FMIN} = 3% of the WQCV) — V_{FMIN} = 0.052 ac-ft

B) Actual Forebay Volume — V_F = 0.055 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} = 333.00 cfs

ii) Forebay Discharge Design Flow
(Q_F = 0.02 * Q_{100}) — Q_F = 6.66 cfs

E) Forebay Discharge Design

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

use 7" wide notch in this forebay. existing forebay has a 6" wide notch

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

G) Rectangular Notch Width — Calculated W_N = 12.1 in

6. Trickle Channel

A) Type of Trickle Channel

Choose One
- ● Concrete
- ○ Soft Bottom

F) Slope of Trickle Channel — S = 0.0050 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) — $D_{orifice}$ = 2.30 inches

E) Total Outlet Area — A_{ot} = 21.00 square inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Richard Schindler
Company: Core Engineering Group
Date: February 16, 2019
Project: Lorson Ranch East 4 FDR - Pond E2 forebay design (ultimate)
Location:

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} = 226.3 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 = 650 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)* well screen stainless steel
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} = 1083 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 feet
F) Height of Water Quality Screen (H_{TR})	H_{TR}= 64 inches
G) Width of Water Quality Screen Opening ($W_{opening}$) (Minimum of 12 inches is recommended)	$W_{opening}$ = 16.9 inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Richard Schindler
Company: Core Engineering Group
Date: February 16, 2019
Project: Lorson Ranch East 4 FDR - Pond E2 forebay design (ultimate)
Location:

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)

11. Vegetation

Choose One
- ○ Irrigated
- ○ Not Irrigated

12. Access

A) Describe Sediment Removal Procedures

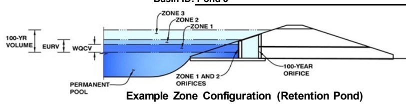
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Creekside South at Lorson Ranch

Basin ID: Pond J



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	EDB			
Watershed Area =	54.00	acres		
Watershed Length =	2,200	ft		
Watershed Slope =	0.020	ft/ft		
Watershed Imperviousness =	26.00%	percent		
Percentage Hydrologic Soil Group A =	0.0%	percent		
Percentage Hydrologic Soil Group B =	90.0%	percent		
Percentage Hydrologic Soil Groups C/D =	10.0%	percent		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =	User Input			
Water Quality Capture Volume (WQCV) =	0.623	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	1.408	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	1.084	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	1.592	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	2.509	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	4.406	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	5.648	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	7.267	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	1.011	acre-feet		
Approximate 5-yr Detention Volume =	1.495	acre-feet		
Approximate 10-yr Detention Volume =	2.192	acre-feet		
Approximate 25-yr Detention Volume =	2.587	acre-feet		
Approximate 50-yr Detention Volume =	2.728	acre-feet		
Approximate 100-yr Detention Volume =	3.285	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.623	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.785	acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) =	2.188	acre-feet
Total Detention Basin Volume =	3.596	acre-feet
Initial Surcharge Volume (ISV) =	user	ft^3
Initial Surcharge Depth (ISD) =	user	ft
Total Available Detention Depth (H_{total}) =	user	ft
Depth of Trickle Channel (H_{TC}) =	user	ft
Slope of Trickle Channel (S_{TC}) =	user	ft/ft
Slopes of Main Basin Sides (S_{main}) =	user	H:V
Basin Length-to-Width Ratio ($R_{L/W}$) =	user	
Initial Surcharge Area (A_{ISV}) =	user	ft^2
Surcharge Volume Length (L_{ISV}) =	user	ft
Surcharge Volume Width (W_{ISV}) =	user	ft
Depth of Basin Floor (H_{FLOOR}) =	user	ft
Length of Basin Floor (L_{FLOOR}) =	user	ft
Width of Basin Floor (W_{FLOOR}) =	user	ft
Area of Basin Floor (A_{FLOOR}) =	user	ft^2
Volume of Basin Floor (V_{FLOOR}) =	user	ft^3
Depth of Main Basin (H_{MAIN}) =	user	ft
Length of Main Basin (L_{MAIN}) =	user	ft
Width of Main Basin (W_{MAIN}) =	user	ft
Area of Main Basin (A_{MAIN}) =	user	ft^2
Volume of Main Basin (V_{MAIN}) =	user	ft^3
Calculated Total Basin Volume (V_{total}) =	user	acre-feet

Depth Increment = ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	49	0.001		
5682	--	0.33	--	--	--	50	0.001	16	0.000
5683	--	1.33	--	--	--	7,000	0.161	3,472	0.080
5684	--	2.33	--	--	--	29,000	0.666	21,541	0.495
5685	--	3.33	--	--	--	32,800	0.753	52,441	1.204
5686	--	4.33	--	--	--	37,100	0.852	87,391	2.006
5687	--	5.33	--	--	--	40,720	0.935	126,301	2.899
5688	--	6.33	--	--	--	44,410	1.020	168,866	3.877
5689	--	7.33	--	--	--	48,200	1.107	215,171	4.940
5690	--	8.33	--	--	--	52,430	1.204	265,486	6.095
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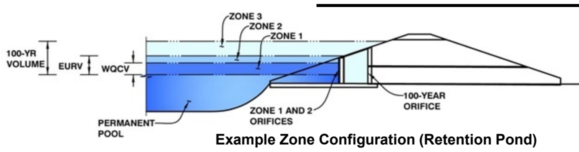
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Creekside South
Basin ID: Pond J



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.52	0.623	Orifice Plate
Zone 2 (EURV)	3.60	0.785	Rectangular Orifice
(100+1/2WQCV)	6.06	2.188	Weir&Pipe (Restrict)
		3.596	Total

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

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

Calculated Parameters for Underdrain

Underdrain Orifice Area =		ft^2
Underdrain Orifice Centroid =		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 =	2.52	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	9.00	inches
Orifice Plate: Orifice Area per Row =	2.15	sq. inches (diameter = 1-5/8 inches)

Calculated Parameters for Plate

WQ Orifice Area per Row =	1.493E-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	0.80	1.60					
Orifice Area (sq. inches)	2.15	2.15	2.15					

	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 =	2.52	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	3.60	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	2.00	N/A	inches
Vertical Orifice Width =	12.13		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.17	N/A	ft^2
Vertical Orifice Centroid =	0.08	N/A	feet

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

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	3.70	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	6.00	N/A	feet
Overflow Weir Slope =	6.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	6.00	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 =	4.70	N/A	feet
Over Flow Weir Slope Length =	6.08	N/A	feet
Grate Open Area / 100-yr Orifice Area =	4.88	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	25.55	N/A	ft^2
Overflow Grate Open Area w/ Debris =	12.77	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 =	36.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	25.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	5.24	N/A	ft^2
Outlet Orifice Centroid =	1.16	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	1.97	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	6.10	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	25.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Calculated Parameters for Spillway

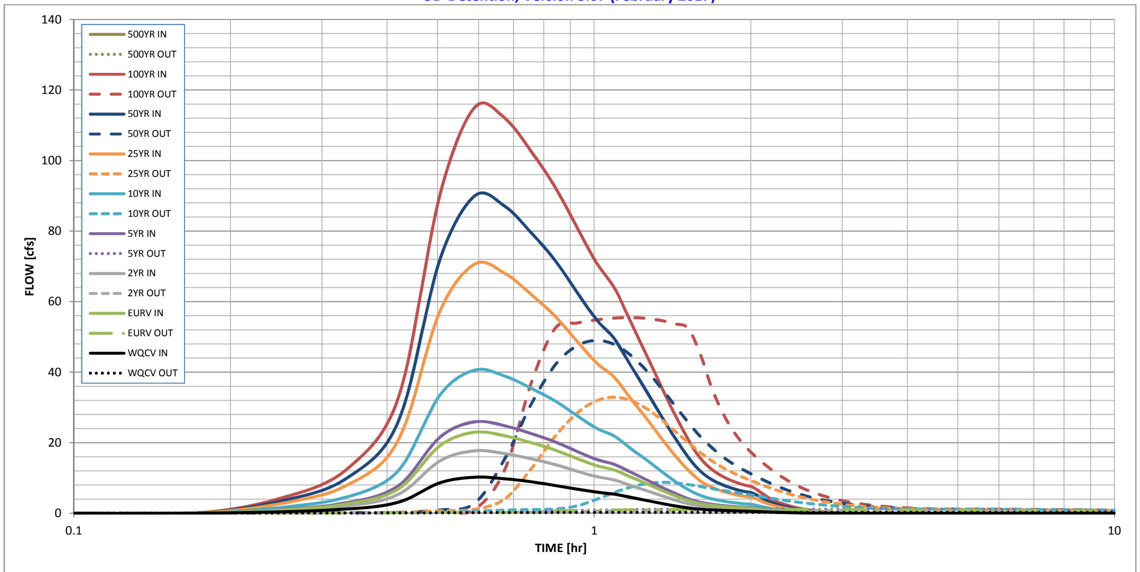
Spillway Design Flow Depth=	1.20	feet
Stage at Top of Freeboard =	8.30	feet
Basin Area at Top of Freeboard =	1.20	acres

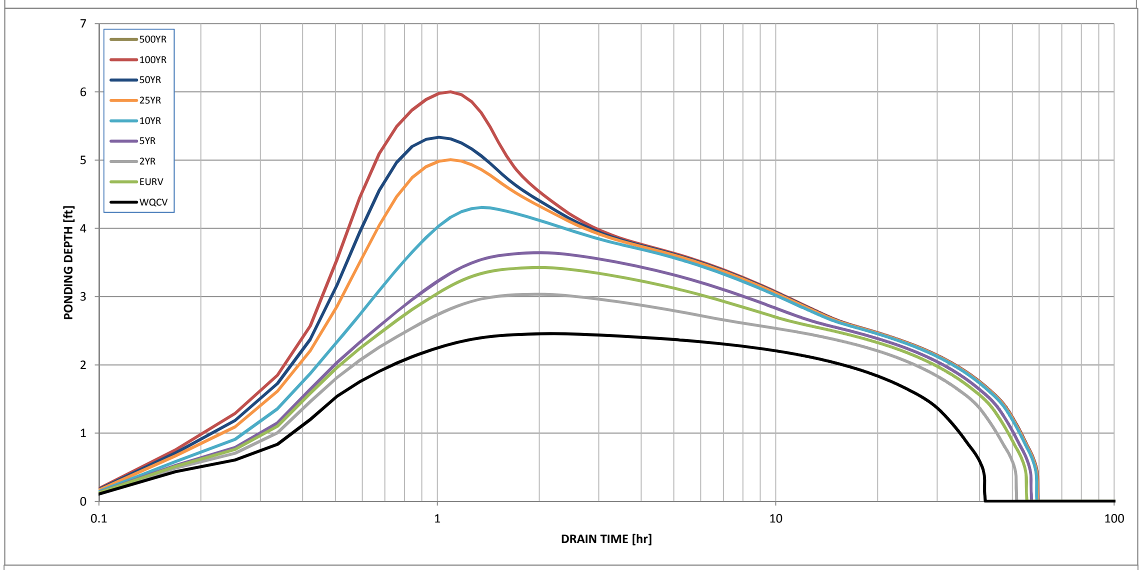
Routed Hydrograph Results

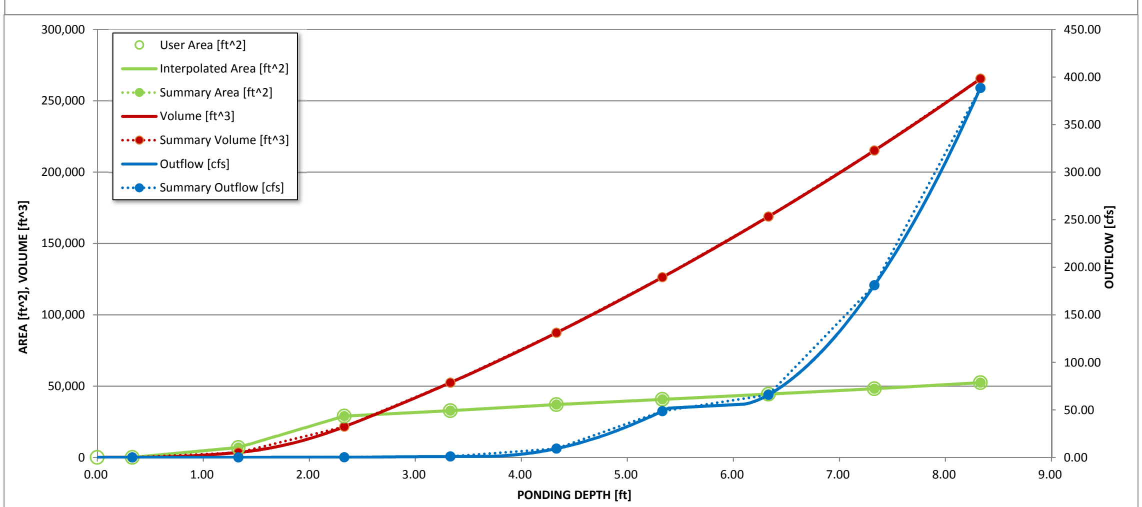
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.623	1.408	1.084	1.592	2.509	4.406	5.648	7.267	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.622	1.409	1.084	1.593	2.510	4.409	5.647	7.267	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.03	0.21	0.67	0.92	1.24	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	0.7	1.7	11.4	36.1	49.8	66.7	0.0
Peak Inflow Q (cfs) =	10.2	23.0	17.7	25.9	40.6	70.6	90.0	115.1	#N/A
Peak Outflow Q (cfs) =	0.3	1.1	0.9	1.2	8.8	32.9	49.0	55.5	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.7	0.8	0.9	1.0	0.8	#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	0.3	1.2	1.9	2.1	#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	48	46	49	49	44	42	39	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**40**	52	49	53	54	52	51	50	#N/A
Maximum Ponding Depth (ft) =	2.46	3.43	3.03	3.64	4.31	5.01	5.34	6.00	#N/A
Area at Maximum Ponding Depth (acres) =	0.68	0.76	0.73	0.78	0.85	0.91	0.93	0.99	#N/A
Maximum Volume Stored (acre-ft) =	0.575	1.272	0.982	1.442	1.981	2.596	2.899	3.545	#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]
5.05 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	0:05:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Hydrograph	0:10:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
Constant	0:15:09	0.45	1.00	0.78	1.12	1.73	2.92	3.62	4.47	#N/A
0.991	0:20:12	1.22	2.71	2.10	3.05	4.75	8.13	10.21	12.82	#N/A
	0:25:15	3.14	6.96	5.39	7.84	12.19	20.86	26.21	32.93	#N/A
	0:30:18	8.63	19.12	14.82	21.54	33.46	57.20	71.82	90.14	#N/A
	0:35:21	10.24	22.96	17.72	25.91	40.59	70.65	89.98	115.07	#N/A
	0:40:24	9.77	21.96	16.94	24.80	38.91	68.04	87.06	112.01	#N/A
	0:45:27	8.89	19.99	15.42	22.57	35.41	61.99	79.48	102.51	#N/A
	0:50:30	7.94	17.92	13.81	20.25	31.83	55.85	71.64	92.45	#N/A
	0:55:33	6.85	15.55	11.96	17.58	27.73	48.88	62.81	81.21	#N/A
	1:00:36	5.97	13.51	10.40	15.27	24.13	42.68	54.91	71.05	#N/A
	1:05:39	5.41	12.25	9.43	13.85	21.85	38.51	49.45	63.85	#N/A
	1:10:42	4.46	10.19	7.82	11.53	18.24	32.30	41.59	53.91	#N/A
	1:15:45	3.64	8.38	6.42	9.49	15.07	26.77	34.52	44.79	#N/A
	1:20:48	2.81	6.53	4.98	7.41	11.85	21.23	27.48	35.80	#N/A
	1:25:51	2.09	4.94	3.75	5.62	9.08	16.41	21.32	27.86	#N/A
	1:30:54	1.51	3.61	2.72	4.12	6.71	12.27	16.01	21.01	#N/A
	1:35:57	1.17	2.76	2.09	3.14	5.07	9.19	11.94	15.61	#N/A
	1:41:00	0.96	2.25	1.71	2.56	4.11	7.39	9.56	12.44	#N/A
	1:46:03	0.82	1.91	1.45	2.16	3.47	6.22	8.03	10.44	#N/A
	1:51:06	0.72	1.67	1.27	1.89	3.03	5.41	6.98	9.05	#N/A
	1:56:09	0.65	1.50	1.14	1.70	2.71	4.83	6.23	8.07	#N/A
	2:01:12	0.60	1.38	1.05	1.56	2.49	4.42	5.70	7.37	#N/A
	2:06:15	0.44	1.01	0.77	1.15	1.84	3.30	4.28	5.59	#N/A
	2:11:18	0.32	0.74	0.57	0.84	1.34	2.39	3.10	4.04	#N/A
	2:16:21	0.24	0.54	0.42	0.62	0.99	1.77	2.29	2.99	#N/A
	2:21:24	0.17	0.40	0.31	0.46	0.73	1.32	1.70	2.22	#N/A
	2:26:27	0.12	0.29	0.22	0.33	0.53	0.96	1.25	1.64	#N/A
	2:31:30	0.09	0.21	0.16	0.23	0.38	0.69	0.90	1.18	#N/A
	2:36:33	0.06	0.15	0.11	0.17	0.28	0.50	0.65	0.86	#N/A
	2:41:36	0.04	0.10	0.08	0.12	0.19	0.35	0.46	0.61	#N/A
	2:46:39	0.02	0.06	0.05	0.07	0.12	0.23	0.30	0.40	#N/A
	2:51:42	0.01	0.03	0.02	0.04	0.07	0.13	0.17	0.23	#N/A
	2:56:45	0.00	0.01	0.01	0.01	0.03	0.06	0.08	0.11	#N/A
	3:01:48	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.03	#N/A
	3:06:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:11:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:16:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:22:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:27:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:32:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:37:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:42:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:47:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:52:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	3:57:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:02:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:07:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:12:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:17:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:22:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:27:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:32:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:37:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:42:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:47:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:52:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	4:57:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:03:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:08:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:13:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:18:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:23:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:28:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:33:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:38:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:43:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:48:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:53:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	5:58:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
	6:03:36	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]
5682	0.33	50	0.001	16	0.000	0.04
5683	1.33	6,931	0.159	3,472	0.080	0.13
5684	2.33	29,000	0.666	21,541	0.495	0.26
5685	3.33	32,800	0.753	52,441	1.204	1.03
5686	4.33	37,100	0.852	87,391	2.006	9.30
5687	5.33	40,720	0.935	126,301	2.899	48.73
5688	6.33	44,410	1.020	168,866	3.877	65.85
5689	7.33	48,200	1.107	215,171	4.940	181.06
5690	8.33	52,430	1.204	265,486	6.095	388.58

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: Extended Detention Basin (EDB)

UD-BMP (Version 3.07, March 2018)

Designer:	**Richard Schindler**
Company:	**Core Engineering Group**
Date:	**January 9, 2020**
Project:	**Creekside South at Lorson Ranch**
Location:	**Pond J**

1. Basin Storage Volume

A) Effective Imperviousness of Tributary Area, I_a — I_a = 26.0 %

B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) — i = 0.260

C) Contributing Watershed Area — Area = 54.000 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} = 0.623 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 — $EURV_{DESIGN}$ = ___ ac-f t
- For HSG A: $EURV_A = 1.68 * i^{1.28}$
- For HSG B: $EURV_B = 1.36 * i^{1.08}$
- For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$

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 = 3.00 ft / ft

DIFFICULT TO MAINTAIN, INCREASE WHERE POSSIBLE

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} = 0.019 ac-ft

B) Actual Forebay Volume — V_F = 0.024 ac-ft

C) Forebay Depth (D_F = 18 inch maximum) — D_F = 24.0 in **DF > DF MAXIMUM**

D) Forebay Discharge
- i) Undetained 100-year Peak Discharge — Q_{100} = 115.00 cfs
- ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$) — Q_F = 2.30 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 = 7.7 in

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	**Richard Schindler**
Company:	**Core Engineering Group**
Date:	**January 9, 2020**
Project:	**Creekside South at Lorson Ranch**
Location:	**Pond J**

6. Trickle Channel

A) Type of Trickle Channel

Choose One
- (●) Concrete
- ( ) Soft Bottom

F) Slope of Trickle Channel — S = 0.0050 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 = 50 sq ft

C) Outlet Type

Choose One
- (●) Orifice Plate
- ( ) Other (Describe):

D) Smallest Dimension of Orifice Opening Based on Hydrograph Routing (Use UD-Detention) — $D_{orifice}$ = 1.63 inches

E) Total Outlet Area — A_{ot} = 6.45 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} = 81 cu ft

C) Initial Surcharge Provided Above Micropool — V_s= 16.7 cu ft

9. Trash Rack

A) Water Quality Screen Open Area: $A_t = A_{ot} * 38.5*(e^{-0.095D})$ — A_t = 213 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} = 355 sq. in. **Based on type 'Other' screen ratio**

E) Depth of Design Volume (EURV or WQCV) (Based on design concept chosen under 1E) — H= 2.52 feet

F) Height of Water Quality Screen (H_{TR}) — H_{TR}= 58.24 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)

Designer:	**Richard Schindler**
Company:	**Core Engineering Group**
Date:	**January 9, 2020**
Project:	**Creekside South at Lorson Ranch**
Location:	**Pond J**

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

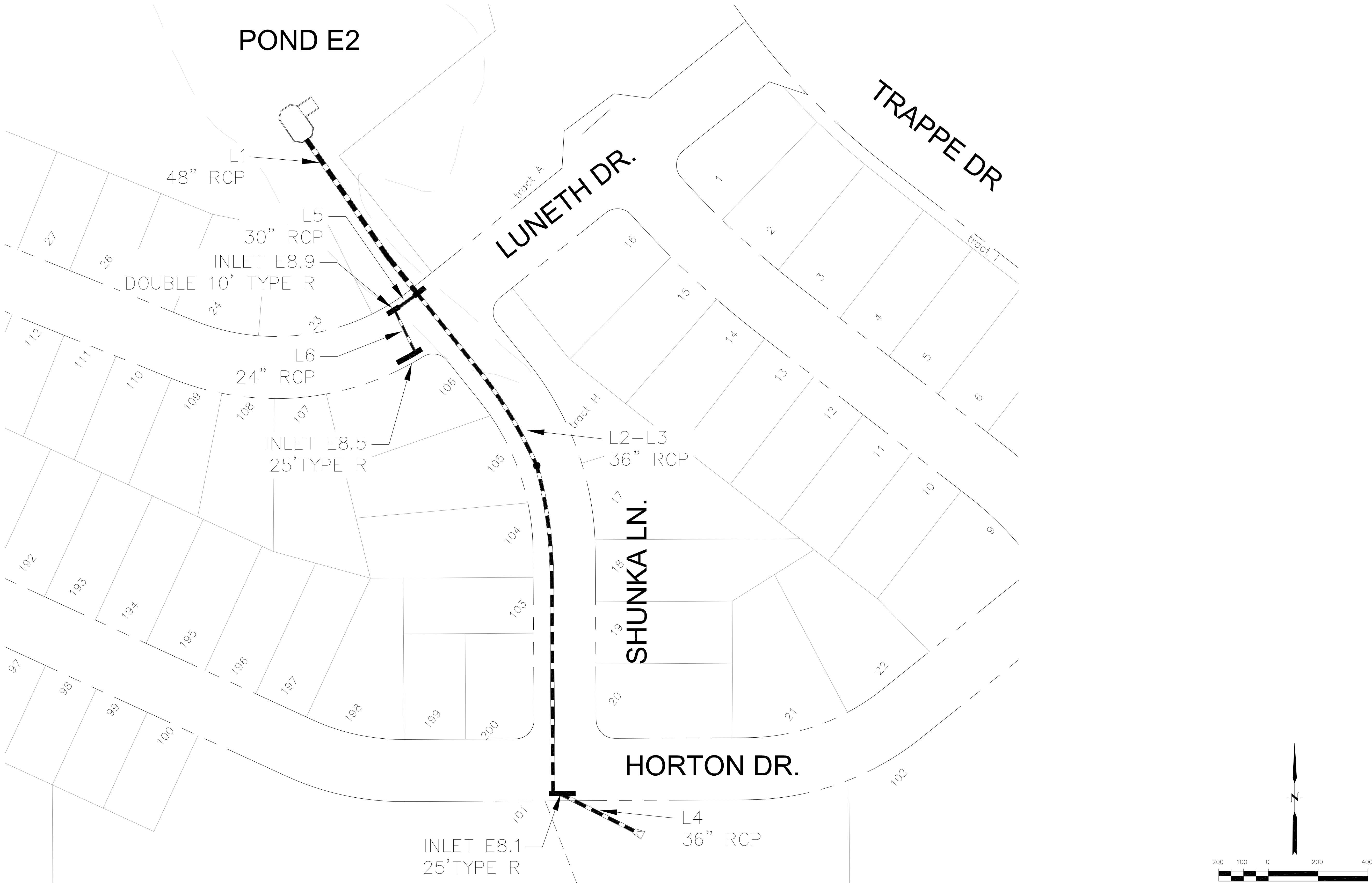
12. Access

A) Describe Sediment Removal Procedures

Notes:

APPENDIX E– STORM SEWER SCHEMATIC AND HYDRAFLOW STORM SEWER CALCS

BASIN 'E' STORM SCHEMATIC



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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 SOUTH
TRAPPE DR – HORTON DR
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

STORM SEWER SCHEMATIC
BASIN E
CREEKSIDE SOUTH AT LORSON RANCH

DATE
MAY 12, 2020

PROJECT NO.
100.051

SHEET NUMBER
1

TOTAL SHEETS: 1

Storm Sewer Summary Report

Line No.	Line ID	Flow rate (cfs)	Line size (in)	Line length (ft)	Invert EL Dn (ft)	Invert EL Up (ft)	Line slope (%)	HGL down (ft)	HGL up (ft)	Minor loss (ft)	HGL Junct (ft)	Dns line No.
1	L1-48"	43.30	48 c	151.0	5695.35	5699.40	2.682	5698.69	5701.35	n/a	5701.35 j	End
2	L2-36"	25.20	36 c	166.0	5700.40	5702.90	1.506	5701.94	5704.50	0.38	5704.50	1
3	L3-36"	25.20	36 c	264.0	5703.20	5709.07	2.223	5704.97	5710.67	n/a	5710.67 j	2
4	L4-36"	15.80	36 c	73.0	5709.40	5711.51	2.890	5711.26	5712.78	n/a	5712.78 j	3
5	L5-24"	18.10	30 c	17.0	5700.91	5701.76	4.998	5701.93	5703.18	n/a	5703.18	1
6	L6-24"	15.00	24 c	36.0	5703.26	5703.64	1.057	5704.43	5705.11	0.14	5705.25	5

Project File: 100.051 Basins E, 5yr flow.stm | Number of lines: 6 | Run Date: 01-22-2020

NOTES: c = cir; e = ellip; b = box; Return period = 5 Yrs. ; j - Line contains hyd. jump.

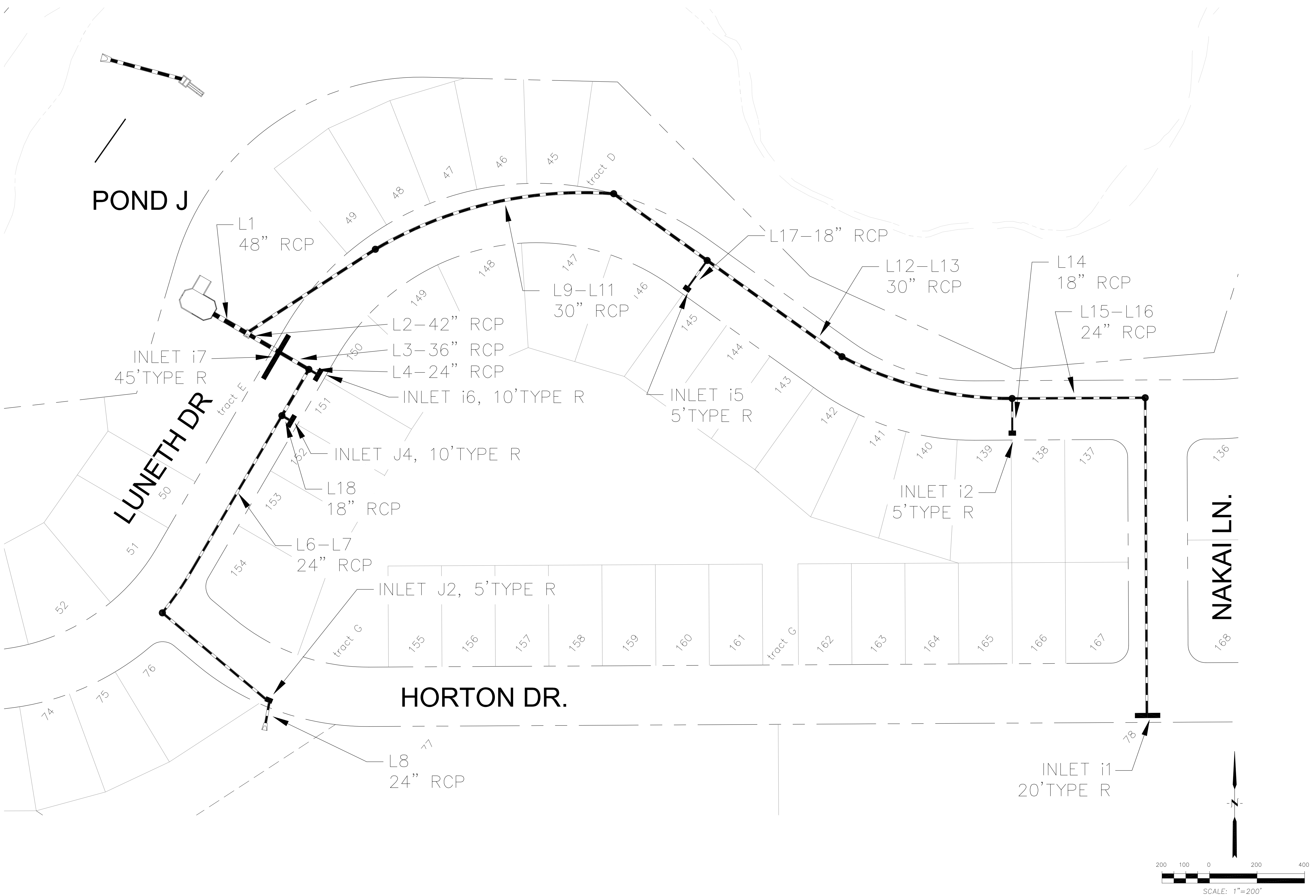
Storm Sewer Summary Report

Line No.	Line ID	Flow rate (cfs)	Line size (in)	Line length (ft)	Invert EL Dn (ft)	Invert EL Up (ft)	Line slope (%)	HGL down (ft)	HGL up (ft)	Minor loss (ft)	HGL Junct (ft)	Dns line No.
1	L1-48"	139.5	48 c	151.0	5695.35	5699.40	2.683	5698.87	5702.92	n/a	5702.92	End
2	L2-36"	85.50	36 c	166.0	5700.40	5702.90	1.506	5703.00	5705.73	1.33	5707.06	1
3	L3-36"	85.50	36 c	264.0	5703.20	5709.07	2.223	5707.17	5711.89	3.30	5711.89	2
4	L4-36"	70.00	36 c	73.0	5709.40	5711.51	2.890	5712.76	5714.18	1.73	5714.18	3
5	L5-24"	54.00	30 c	17.0	5700.91	5701.76	4.998	5703.24	5704.11	0.99	5704.11	1
6	L6-24"	32.00	24 c	36.0	5703.26	5703.64	1.057	5705.26*	5705.98*	0.40	5706.38	5

Project File: 100.051 Basins E, 100yr flow.stm	Number of lines: 6	Run Date: 01-22-2020

NOTES: c = cir; e = ellip; b = box; Return period = 100 Yrs. ; *Surcharged (HGL above crown).

BASIN I & J STORM SCHEMATIC



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NO.	DESCRIPTION	DATE

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(719) 635-3200
CONTACT: JEFF MARK

PROJECT:
CREEKSIDE SOUTH
TRAPPE DR – HORTON DR.
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

STORM SEWER SCHEMATIC
BASIN I & J
CREEKSIDE SOUTH AT LORSON RANCH

DATE
MAY 12, 2020

PROJECT NO.
100.051

SHEET NUMBER
1

TOTAL SHEETS: 1

Storm Sewer Summary Report

Line No.	Line ID	Flow rate (cfs)	Line size (in)	Line length (ft)	Invert EL Dn (ft)	Invert EL Up (ft)	Line slope (%)	HGL down (ft)	HGL up (ft)	Minor loss (ft)	HGL Junct (ft)	Dns line No.
1	L1	45.60	48 c	50.8	5684.60	5685.37	1.515	5688.06	5687.92	0.25	5688.17	End
2	L2	27.40	42 c	28.6	5687.29	5693.00	19.979	5688.50	5694.60	0.32	5694.60	1
3	L3	22.40	36 c	27.5	5695.20	5695.50	1.090	5696.37	5697.39	0.35	5697.75	2
4	L4	8.30	24 c	7.5	5696.50	5696.57	0.931	5697.86	5697.84	0.24	5698.08	3
5	L5	14.10	30 c	45.0	5696.00	5696.74	1.646	5697.97	5698.00	n/a	5698.00 j	3
6	L6	7.50	24 c	204.7	5697.24	5701.60	2.130	5698.41	5702.57	n/a	5702.57 j	5
7	L7	7.50	24 c	112.9	5701.71	5703.74	1.799	5702.86	5704.71	n/a	5704.71 j	6
8	L8	6.40	24 c	23.0	5703.90	5704.53	2.739	5705.03	5705.43	n/a	5705.43 j	7
9	L9	18.20	30 c	129.0	5690.55	5696.73	4.791	5691.31	5698.16	n/a	5698.16	1
10	L10	18.20	30 c	209.0	5696.83	5698.92	1.000	5698.56	5700.35	n/a	5700.35 j	9
11	L11	18.20	30 c	97.0	5699.02	5699.99	1.000	5700.75	5701.42	n/a	5701.42	10
12	L12	14.70	30 c	139.0	5700.39	5701.51	0.805	5701.89	5702.79	n/a	5702.79 j	11
13	L13	14.70	30 c	150.0	5701.88	5703.07	0.793	5703.18	5704.35	n/a	5704.35 j	12
14	L14	3.60	18 c	27.0	5703.98	5704.14	0.593	5704.81	5704.87	0.28	5705.15	13
15	L15	11.10	24 c	112.0	5703.58	5704.48	0.803	5704.68	5705.66	0.26	5705.66	13
16	L16	11.10	24 c	265.3	5704.65	5708.21	1.342	5705.98	5709.39	n/a	5709.39 j	15
17	L17	3.50	18 c	27.0	5700.92	5701.21	1.074	5701.97	5701.93	n/a	5702.20 j	11
18	L18	6.60	18 c	35.3	5697.72	5698.27	1.558	5698.47	5699.32	0.20	5699.51	5

Project File: 100.051 Basins I & J, 5yr flow.stm	Number of lines: 18	Run Date: 01-23-2020

NOTES: c = cir; e = ellip; b = box; Return period = 5 Yrs. ; j - Line contains hyd. jump.

Storm Sewer Summary Report

Line No.	Line ID	Flow rate (cfs)	Line size (in)	Line length (ft)	Invert EL Dn (ft)	Invert EL Up (ft)	Line slope (%)	HGL down (ft)	HGL up (ft)	Minor loss (ft)	HGL Junct (ft)	Dns line No.
1	L1	133.6	48 c	50.8	5684.60	5685.37	1.515	5688.06	5688.83	1.17	5688.83	End
2	L2	104.0	42 c	28.6	5687.29	5693.00	19.979	5689.09	5696.12	1.03	5696.12	1
3	L3	53.30	36 c	27.5	5695.20	5695.50	1.090	5697.29	5697.95	1.15	5699.11	2
4	L4	15.50	24 c	7.5	5696.50	5696.57	0.931	5699.88*	5699.92*	0.38	5700.30	3
5	L5	37.80	30 c	45.0	5696.00	5696.74	1.646	5699.34*	5699.72*	0.74	5700.46	3
6	L6	27.90	24 c	204.7	5697.24	5701.60	2.130	5700.46	5703.60	1.23	5704.83	5
7	L7	27.90	24 c	112.9	5701.71	5703.74	1.799	5704.83*	5706.54*	1.79	5708.33	6
8	L8	26.00	24 c	23.0	5703.90	5704.53	2.739	5708.50*	5708.80*	1.06	5709.86	7
9	L9	29.60	30 c	129.0	5690.55	5696.73	4.791	5691.54	5698.55	0.14	5698.55	1
10	L10	29.60	30 c	209.0	5696.83	5698.92	1.000	5698.91	5700.74	n/a	5700.74 j	9
11	L11	29.60	30 c	97.0	5699.02	5699.99	1.000	5701.10	5701.81	0.93	5701.81	10
12	L12	22.60	30 c	139.0	5700.39	5701.51	0.805	5702.41	5703.10	n/a	5703.10 j	11
13	L13	22.60	30 c	150.0	5701.88	5703.07	0.793	5703.50	5704.66	n/a	5704.66 j	12
14	L14	7.10	18 c	27.0	5703.98	5704.14	0.593	5705.14	5705.25	0.40	5705.65	13
15	L15	15.50	24 c	112.0	5703.58	5704.48	0.803	5705.01	5705.88	n/a	5705.88 j	13
16	L16	15.50	24 c	265.3	5704.65	5708.21	1.342	5706.18	5709.60	n/a	5709.60 j	15
17	L17	7.00	18 c	27.0	5700.92	5701.21	1.074	5702.39	5702.42	0.33	5702.75	11
18	L18	9.90	18 c	35.3	5697.72	5698.27	1.558	5700.89*	5701.21*	0.24	5701.45	5

Project File: 100.051 Basins I & J, 100yr flow.stm | Number of lines: 18 | Run Date: 05-11-2020

NOTES: c = cir; e = ellip; b = box; Return period = 100 Yrs. ; *Surcharged (HGL above crown). ; j - Line contains hyd. jump.

MAP POCKET

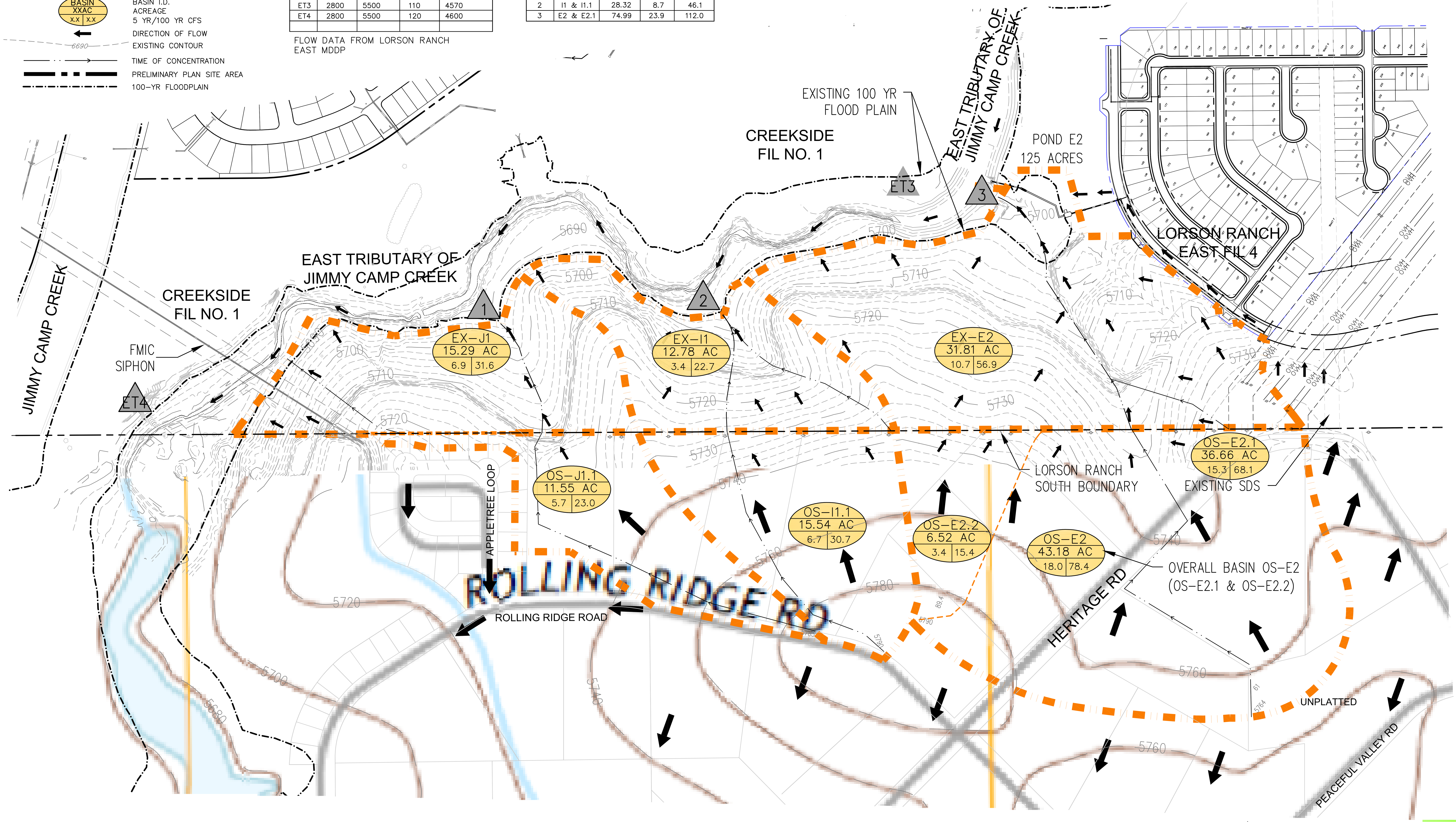
LEGEND

- BASIN BOUNDARY
- BASIN DESIGN POINT
- BASIN I.D. / ACREAGE / 5 YR/100 YR CFS (BASIN XXAC X.X X.X)
- DIRECTION OF FLOW
- EXISTING CONTOUR (6690)
- TIME OF CONCENTRATION
- PRELIMINARY PLAN SITE AREA
- 100-YR FLOODPLAIN

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

DESIGN POINT SUMMARY TABLE				
DESIGN POINT	BASIN (EX & OS)	DRAINAGE AREA (AC)	RUNOFF 5 YR (CFS)	RUNOFF 100 YR (CFS)
1	J1 & J1.1	26.84	10.8	46.6
2	I1 & I1.1	28.32	8.7	46.1
3	E2 & E2.1	74.99	23.9	112.0



N

200 100 0 200 400
SCALE: 1"=200'

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PH: 719.570.1100
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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 SOUTH
TRAPPE DRIVE - HORTON DR
EL PASO COUNTY, COLORADO

DRAWN: LAB
DESIGNED: LAB
CHECKED: RLS

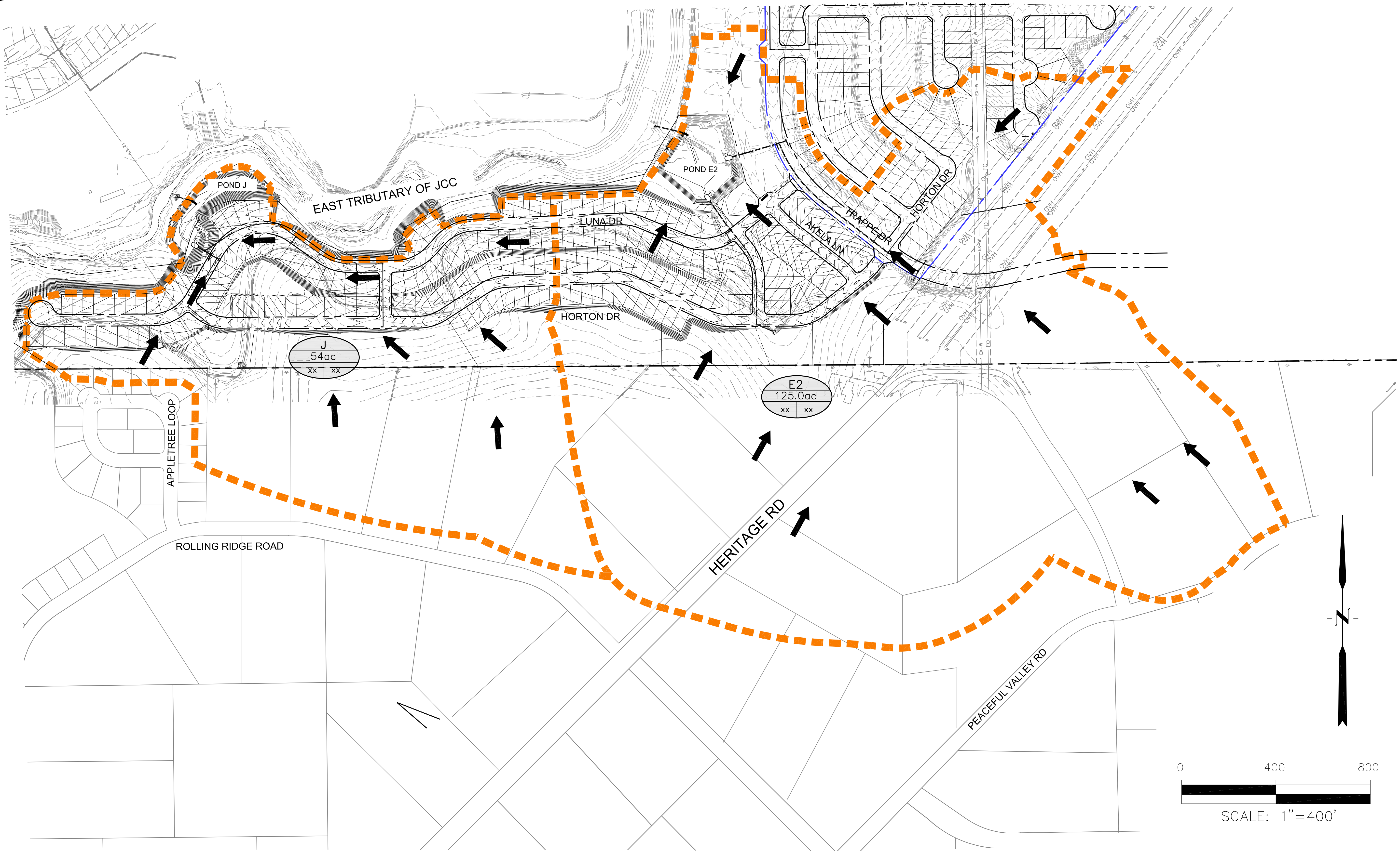
EXISTING CONDITIONS
CREEKSIDE SOUTH AT LORSON RANCH

DATE:
JANUARY 15, 2020

PROJECT NO.
100.051

SHEET NUMBER
1

TOTAL SHEETS: 1



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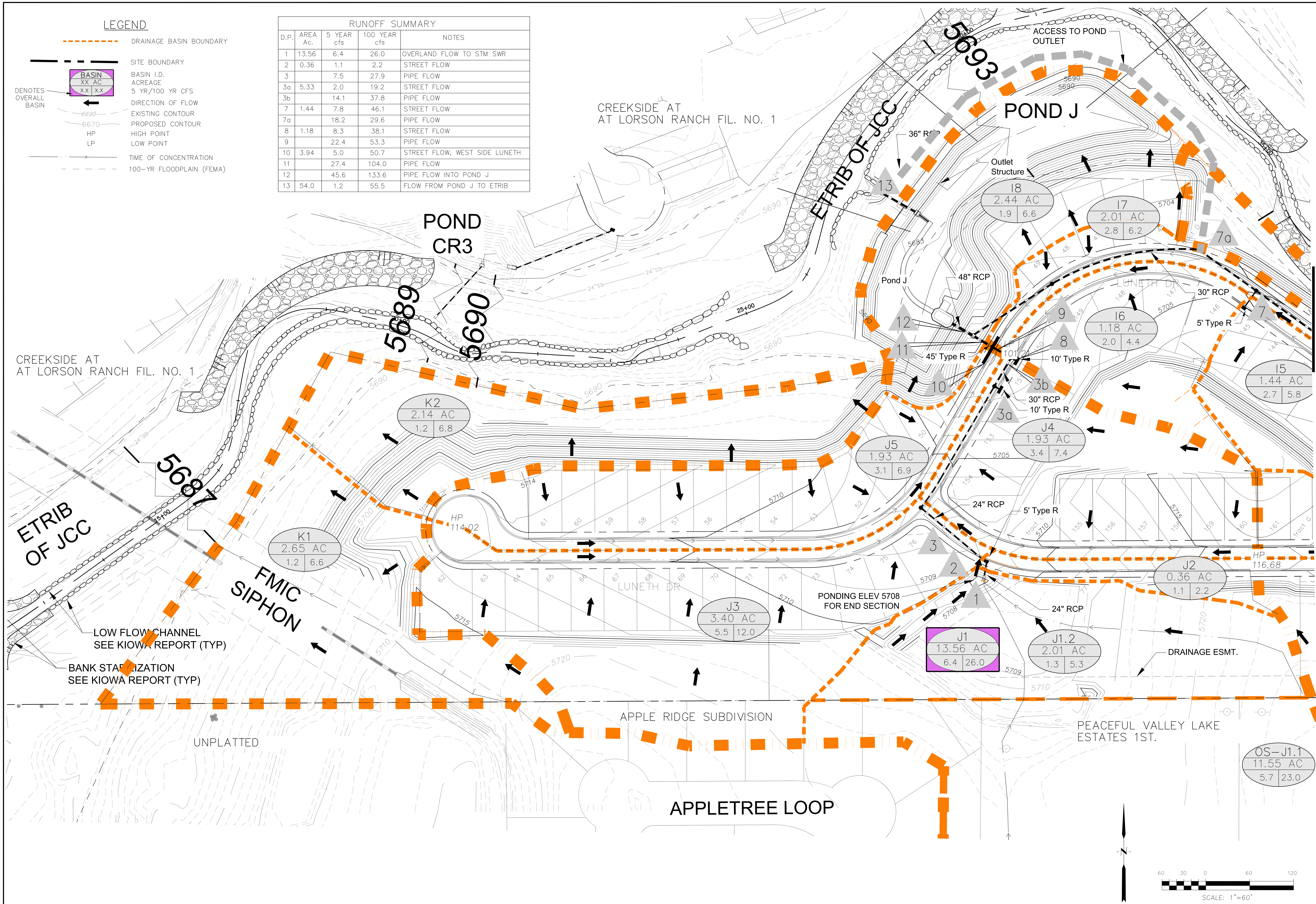
CREEKSIDE SOUTH AT LORSON RANCH
WATER QUALITY & POND TRIBUTARY AREAS

SCALE:	DATE:	FIGURE NO.
NTS	JANUARY 15, 2020	1

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)

RUNOFF SUMMARY				
D.P.	AREA Ac.	5 YEAR cfs	100 YEAR cfs	NOTES
1	13.56	6.4	26.0	OVERLAND FLOW TO STM SWR
2	0.36	1.1	2.2	STREET FLOW
3		7.5	27.9	PIPE FLOW
3a	5.33	2.0	19.2	STREET FLOW
3b		14.1	37.8	PIPE FLOW
7	1.44	7.8	46.1	STREET FLOW
7a		18.2	29.6	PIPE FLOW
8	1.18	8.3	38.1	STREET FLOW
9		22.4	53.3	PIPE FLOW
10	3.94	5.0	50.7	STREET FLOW, WEST SIDE LUNETH
11		27.4	104.0	PIPE FLOW
12		45.6	133.6	PIPE FLOW INTO POND J
13	54.0	1.2	55.5	FLOW FROM POND J TO ETRIB



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NO.	DESCRIPTION	DATE

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(719) 635-3200
CONTACT: JEFF MARK

PROJECT:
CREEKSIDE SOUTH
TRAPPE DRIVE - HORTON DR
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

DEVELOPED CONDITIONS
CREEKSIDE SOUTH

DATE
DEC 25, 2020

PROJECT NO.
100.051

SHEET NUMBER
1

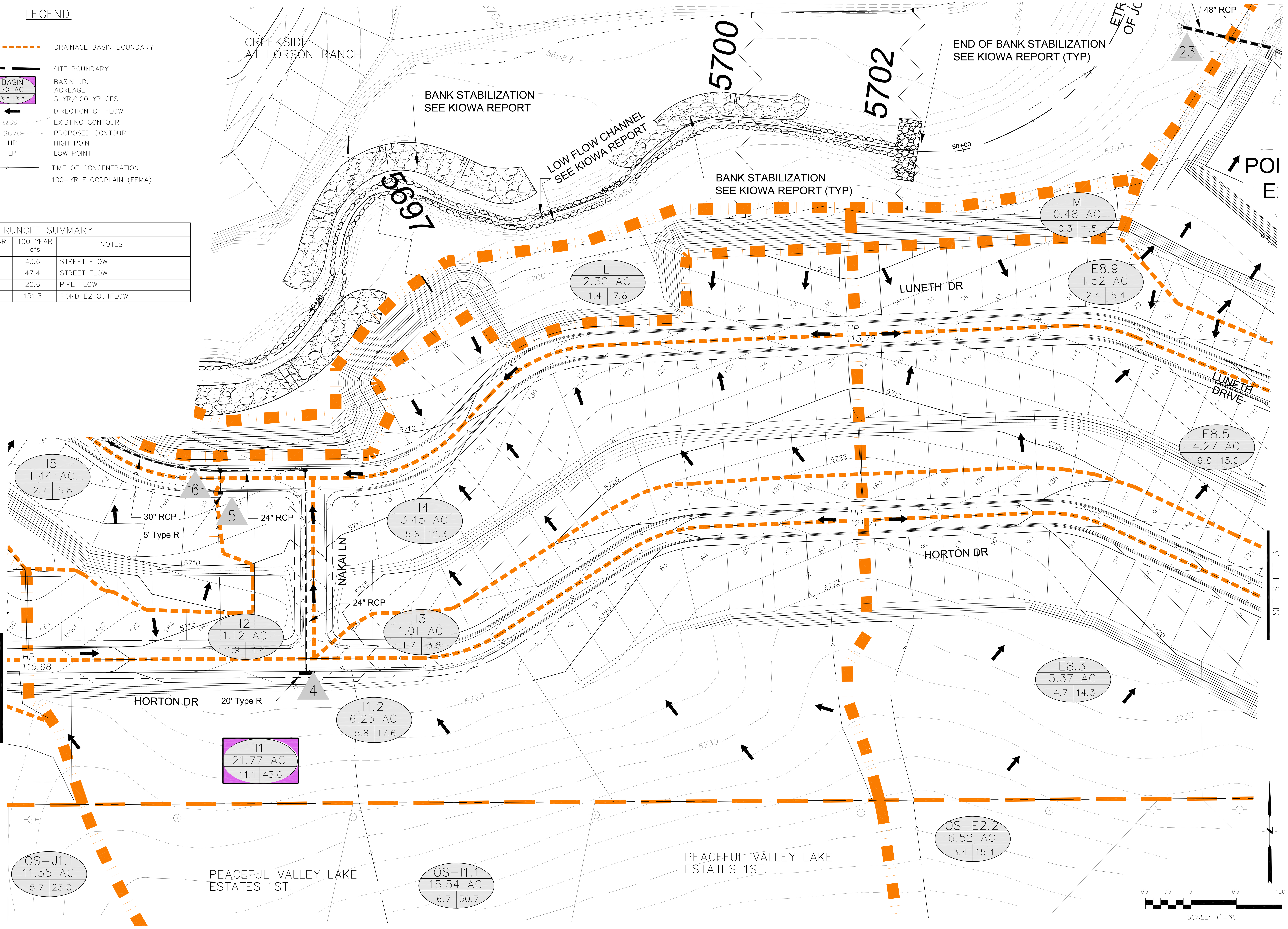
TOTAL SHEETS: 3

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)

RUNOFF SUMMARY

D.P.	AREA Ac.	5 YEAR cfs	100 YEAR cfs	NOTES
4	21.77	11.1	43.6	STREET FLOW
5	5.58	8.7	47.4	STREET FLOW
6		14.7	22.6	PIPE FLOW
14	125	4.5	151.3	POND E2 OUTFLOW



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CONTACT: JEFF MARK

PROJECT:
CREEKSIDE SOUTH
TRAPPE DRIVE – HORTON DRIVE
EL PASO COUNTY, COLORADO

NO.	DESCRIPTION	DATE

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

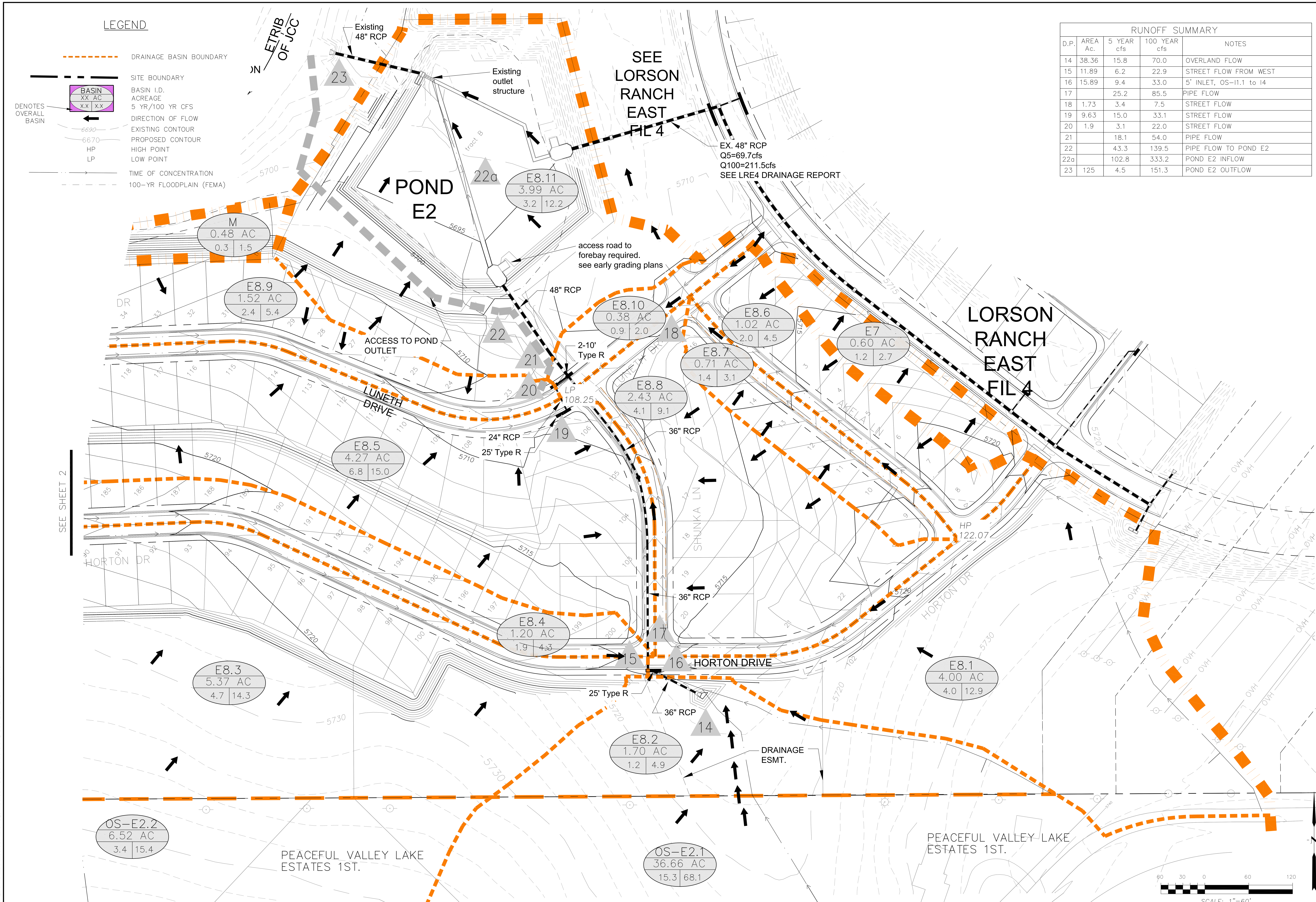
DEVELOPED CONDITIONS CREEKSIDE SOUTH

DATE
JANUARY 15, 2020

PROJECT NO.
100.051

SHEET NUMBER
2

TOTAL SHEETS: 3



RUNOFF SUMMARY

D.P.	AREA Ac.	5 YEAR cfs	100 YEAR cfs	NOTES
14	38.36	15.8	70.0	OVERLAND FLOW
15	11.89	6.2	22.9	STREET FLOW FROM WEST
16	15.89	9.4	33.0	5' INLET, OS-I1.1 to I4
17		25.2	85.5	PIPE FLOW
18	1.73	3.4	7.5	STREET FLOW
19	9.63	15.0	33.1	STREET FLOW
20	1.9	3.1	22.0	STREET FLOW
21		18.1	54.0	PIPE FLOW
22		43.3	139.5	PIPE FLOW TO POND E2
22a		102.8	333.2	POND E2 INFLOW
23	125	4.5	151.3	POND E2 OUTFLOW

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TRAPPE DRIVE - HORTON DRIVE
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

DEVELOPED CONDITIONS CREEKSIDE SOUTH

DATE
DEC 25, 2020

PROJECT NO.
100.051

SHEET NUMBER
3

TOTAL SHEETS: 3