FINAL DRAINAGE PLAN

LORSON RANCH EAST FILING NO. 4

FEBRUARY 28, 2019

SF-19-0XX / EGP 18-002

SF-19-008

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. 100.048

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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 Date
For and on Behalf of Core Engineering Group, LLC

OWNER'S STATEMENT

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

Lorson, LLC Date

By
Jeff Mark

Title
Manager

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

FLOODPLAIN STATEMENT

To the best of my knowledge and belief, this development is not located within a designated floodplain as shown on Flood Insurance Rate Map Panel No. 08041C0957 G and 08041C0976 G, dated December 7, 2018 (See Appendix A, FEMA FIRM Exhibit)

Richard L. Schindler, #33997 Date

EL PASO COUNTY

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

Jennifer Irvine P.E. Date
County Engineer/ECM Administrator

Conditions: ______________________________

1.0 LOCATION and DESCRIPTION

Lorson Ranch East Filing No. 4 is located east of the East Tributary of Jimmy Camp Creek and south of Fontaine Boulevard. The site is located on approximately 58.471 acres of vacant land. This project will develop this site into single-family residential developments. The land for the residential lots is currently owned by Lorson LLC or its nominees for Lorson Ranch.

The site is located in Section 13. 23, and 24, Township 15 South and Range 65 West of the 6th Principal Meridian. The property is bounded on the south by vacant land in Lorson Ranch, on the east by unplatted land in Lorson Ranch, on the west by The East Tributary of Jimmy Camp Creek, and the north by Fontaine Blvd. For reference, a vicinity map is included in Appendix A of this report.

Conformance with applicable Drainage Basin Planning Studies

There is an existing (unapproved) DBPS for Jimmy Camp Creek prepared by Wilson & Company in 1987, and is referenced in this report. The only major drainage improvements for this study area according to the 1987 Wilson study was the reconstruction of the East Tributary of Jimmy Camp Creek (East Tributary). In 2014 a portion of the East Tributary was reconstructed from Fontaine Boulevard south 2,800 feet in accordance with the 1987 study. 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 includes the East Tributary armoring concept and the 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. Channel improvements in the East Tributary north of Fontaine Boulevard were designed by Kiowa Engineering and are completed. Channel improvements south of Fontaine Boulevard were constructed in 2014. There are no channel improvements to construct as part of this plat.

Conformance with Lorson East MDDP and PDR by Core Engineering Group

Core Engineering Group has an approved MDDP for Lorson East and PDR for Lorson Ranch East which covers this final plat area and the East Tributary. This FDR conforms to the MDDP and PDR for Lorson East and is referenced in this report. Detention Pond C1, E1, and E2 will be required to be constructed as part of this plat. Detention Ponds C1 and E1 are just detention facilities with a pipe outlet. Detention Pond E2 is an interim pond sized for water quality. See report for more details on the ponds. The adjacent East Tributary Channel has also been reconstructed as part of Lorson Ranch East Filing No. 1. There are also two bridges over the East Tributary and one bridge over Jimmy Camp Creek at Lorson Boulevard that are required to be built for this plat. The East Tributary bridges are located at Fontaine Boulevard and Lorson Boulevard and are completed. Construction of all bridges must be complete prior to recordation of this plat.

Lorson Ranch East 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. No deviations from these published criteria are requested for this site.

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.

3.0 EXISTING HYDROLOGICAL CONDITIONS

Prior to the early grading of Lorson Ranch East the site was undeveloped with native vegetation (grass with no shrubs) and moderate to steep slopes in a westerly direction the East Tributary of Jimmy Camp Creek.

The Soil Conservation Service (SCS) classifies the soils within the Lorson Ranch East property as Manzanola clay loam; Midway Clay Loam, Nelson-Tassel fine Sandy loam; Razor clay loam; and Wiley silt loam [3]. The sandy and silty loams are considered hydrologic soil group B soils with moderate to moderately rapid permeability. The Midway and Razor clay loams are considered hydrologic soil group C soils with slow permeability. All of these soils are susceptible to erosion by wind and water, have low bearing strength, moderate shrink-swell potential, and high frost heave potential (see table 3.1 below). The clay loams are difficult to vegetate and comprise of a small portion of the study area. These soils can be mitigated easily by limiting their use as topsoil since they comprise of a small portion of the study area. Weathered bedrock may be encountered beneath some of the site but it can be excavated using conventional techniques.

Table 3.1: SCS Soils Survey

Soil	Hydro. Group	Shrink/Swell Potential	Permeability	Surface Runoff Potential	Erosion Hazard
52-Manzanola Clay Loam (14%)	C	High	Slow	Medium	Moderate
54-Midway Clay Loam (17%)	C	High	Slow	Medium to Rapid	Moderate to High
56-Nelson – Tassel Fine Sandy Loam (41%)	B	Moderate	Moderately Rapid	Slow	Moderate
75-Razor Clay Loam (3%)	C	High	Slow	Medium	Moderate
108-Wiley Silt Loam (26%)	B	Moderate	Moderate	Medium	Moderate

Excerpts from the SCS “Soil Survey of El Paso County Area, Colorado” [2] are provided in ***Appendix A*** for further reference.

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

An existing electrical easement, with existing transmission towers, is located east side of this site and will be set aside as open space in the future.

There are no portions of the site located within the delineated 100-year floodplain of the East Tributary of Jimmy Camp Creek per the Federal Emergency Management Agency (FEMA) Flood Rate Insurance Map (FIRM) number 08041C0957 F & 08041C1000 F, effective 17 March 1997 [2]. Floodplain along the East Tributary was modified per LOMR Case No. 14-08-0534P (see appendix). A portion of this map is provided in ***Appendix A*** for reference.

Call out latest FIRM December 7 2018

The existing basins for this large site were taken from the Lorson Ranch East MDDP East of the East Tributary and depict conditions prior to any grading in Lorson Ranch East. A map from the MDDP has been included in the appendix.

4.0 DEVELOPED AND INTERIM HYDROLOGICAL CONDITIONS

Hydrology for the **Lorson Ranch East Filing No. 4** final 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.

Soil type C/D has been assumed for the hydrologic conditions because most of the site requires fill and the majority of the fill will be from the school site which is Razor Clay Loam (75), Hydrologic Group C. This approach will provide a more conservative approach to designing the storm sewer infrastructure. Offsite Basin E3-ex does have a portion of the runoff as Type A soils. See Appendix A for SCS Soils Map.

The time of concentration for each basin and sub-basin was developed using an overland, ditch, street and pipe flow components. The maximum overland flow length for developed conditions was limited to 100 feet. Travel time velocities ranged from 2 to 6 feet per second. The travel time calculations are included in the back of this report.

Runoff coefficients for the various land uses were obtained from Table 6-6 dated May, 2014 from the updated City of Colorado Springs/El Paso County Drainage Criteria Manual. See Appendix B.

Interim hydrological conditions have been calculated based on grading that has been completed in accordance with Phase 1 of the Early Grading for Lorson Ranch East (PUDSP 16-003), Lorson Ranch East Filing No. 1 (SF18-008) and Fontaine Boulevard/Lamprey Drive construction (CDR 183). Interim condition existing flows have been calculated to determine interim drainage impacts to this final plat which is located downstream and to make sure runoff is accommodated by the street/storm sewer system constructed as part of this plat, LRE 1, and CDR 183. These interim condition calculations are also used to perform hydraulic modeling of Pond E1 to see what effect the existing upstream flows have on the downstream pond outflow rates when compared to pre-developed conditions at the East Tributary. The interim conditions hydraulic modeling will be done by a computer program called Hydraflow (Intellisolve) and is discussed in Interim Conditions (Section 6.0) of this report.

Interim conditions consist of Fontaine Boulevard construction from Old Glory Drive eastward 3,500 feet to the existing electric transmission lines, Lamprey Drive construction from Fontaine Boulevard south to Lorson Boulevard per SF18-008. CDR 183 and Lorson Ranch East Filing No. 1 (SF18-008) which includes street, storm sewer, sanitary sewer, and watermain construction which provides access to this plat.

<u>Basin C15-ex</u>
Basin C15-ex consists of runoff from vacant land in Lorson Ranch and areas under the electric easement. Runoff will be directed west to Detention Pond C1. The existing flow from this basin is 24cfs and 134cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D1-ex
Basin D1-ex consists of runoff from vacant land in Lorson Ranch and areas under the electric easement. Runoff will be directed west to Lorson Boulevard where an existing 48” standpipe will collect the flow in Lorson Boulevard. The existing flow from this basin is 8cfs and 47cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1-ex
Basin E1-ex consists of runoff from vacant land in Lorson Ranch and areas under the electric easement. Runoff will be directed west to Detention Pond E1. The existing flow from this basin is 25cfs and 142cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E2-ex
Basin E2-ex consists of runoff from vacant land in Lorson Ranch, areas under the electric easement, and the existing large lot subdivision in Peaceful Valley Estates.. Runoff will be directed northwest to Trappe Drive at Design Point 67a. The existing flow from this basin is 26cfs and 91cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E3-ex
Basin E3-ex consists of runoff from vacant land in Lorson Ranch, areas under the electric easement, and the existing large lot subdivision in Peaceful Valley Estates.. Runoff will be directed north to Detention Pond E2 and the East Tributary of Jimmy Camp Creek. The existing flow from this basin is 43cfs and 158cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.1
Basin C15.1 consists of runoff from areas under the electric easement and residential development. Runoff will be directed west to Design Point 21 in a swale where it will be collected by a storm sewer. The developed flow from this basin is 6.9cfs and 22.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.2
Basin C15.2 consists of runoff from areas under the electric easement, MVEA substation, and residential development. Runoff will be directed west to Design Point 21 in a swale where it will be collected by a storm sewer. The developed flow from this basin is 7.6cfs and 19.2cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.3
This basin consists of runoff from residential development. Runoff will be directed north in Tillamook Drive to Design Point 23 in curb/gutter where it will be collected by a Type R inlet on Tillamook Drive. The developed flow from this basin is 6.4cfs and 14.3cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.4
This basin consists of runoff from residential development. Runoff will be directed north to Design Point 23 in curb/gutter where it will be collected by a Type R inlet on Tillamook Drive. The developed flow from this basin is 2.6cfs and 5.8cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.5
This basin consists of runoff from residential development. Runoff will be directed north to Design Point 24 in curb/gutter. The developed flow from this basin is 5.9cfs and 13.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.6

This basin consists of runoff from residential development and Rockcastle Drive. Runoff will be directed west in Rockcastle Drive. The developed flow from this basin is 3.3cfs and 7.4cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.7

This basin consists of runoff from residential development and Rockcastle Drive. Runoff will be directed west in Rockcastle Drive. The developed flow from thosbasins is 3.9cfs and 8.8cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.8

Basin C15.8 consists of runoff from Fontaine Boulevard on the south side, residential lots, Rockcastle Drive, and open space under the existing electric lines. Runoff will be directed north in the curb/gutter to Design Point 20 in Fontaine Boulevard where it will be collected by an existing 15' Type R inlet. See CDR 183. The developed flow from this basin is 5.2cfs and 13.4cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.9

Basin C15.9 consists of runoff from Fontaine Boulevard on the south side and residential lots. Runoff will be directed north and west in the curb/gutter. The developed flow from this basin is 4.9cfs and 11.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.10

Basin C15.10 consists of runoff from Fontaine Boulevard on the south side, and residential lots. Runoff will be directed west in the curb/gutter to Design Point 29 at the SE corner of the Fontaine Boulevard/Lamprey Drive intersection where it will be collected by an existing 10' Type R inlet. See CDR 183. The developed flow from this basin is 1.2cfs and 2.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Show design point 29 on the drainage plan.

Basin C15.11

This basin consists of runoff from residential development and Vedder/Rockcastle Drive. Runoff will be directed north to Design Point 25 in curb/gutter where it will be collected by a Type R inlet on Rockcastle Drive. The developed flow from this basin is 6.1cfs and 13.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.12

This basin consists of runoff from residential development and Rockcastle Drive. Runoff will be directed west in Rockcastle Drive to Design Point 25 where it will be collected by a Type R inlet. The developed flow from this basin is 1.2cfs and 2.6cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.13

Basin C15.13 consists of runoff from residential development and Vedder/Rockcastle Drive. Runoff will be directed north to Design Point 26 in curb/gutter where it will be collected by a Type R inlet on Rockcastle Drive. The developed flow from this basin is 4.5cfs and 10.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin C15.14

This basin consists of runoff from residential development and Lamprey Drive. Runoff will be directed north to Design Point 29 in curb/gutter where it will be collected by an existing 10' Type R inlet on Lamprey Drive. See CDR 183. The developed flow from this basin is 2.9cfs and 6.4cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.1 & D2.3

Basin D2.1 & D2.3 consists of residential development, open space under the electric easement, Vedder Drive, Lamprey Drive, and Lorson Boulevard. The peak developed flow from Basin D2.1 is 5.4cfs and 12.1cfs for the 5/100-year storm event. The peak developed flow from Basin D2.3 is 2.7cfs and 9.7cfs for the 5/100-year storm event. Runoff is directed south and west in curb/gutter to an existing 15' Type R inlet at Design Point 59d in Lamprey Drive. See SF18-008. See the appendix for detailed calculations. Interim conveyance for Basin 2.1 will be overland to the curb/gutter in Lamprey Drive and then to Design Point 59d. Interim conveyance for Basin D2.3 flows overland to an existing 48" riser east of Design Point 59e on the 30" storm sewer in Lorson Boulevard. See SF18-008.

Basin D2.2

Basin D2.2 consists of residential development and Tillamook Drive. Runoff is directed south in curb/gutter to Design Point 59a. See the appendix for detailed calculations. The peak developed flow from this basin is 2.1cfs and 4.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.4

Basin D2.4 consists of residential development, Lorson Boulevard, and open space area under the electric easement. Runoff is directed west in curb/gutter in Lorson Boulevard an existing 10' Type R inlet at Design Point 59f. See SF18-008. The peak developed flow from this basin is 3.6cfs and 11.9cfs for the 5/100-year storm event.

Basin D2.5

Basin D2.5 consists of residential development, Skuna Drive, and Witcher Drive. Runoff is directed north in curb/gutter to Lorson Boulevard to an existing 10' Type R inlet at Design Point 59f. See SF18-008. The peak developed flow from this basin is 8.8cfs and 19.6cfs for the 5/100-year storm event.

Basin D2.6 & D2.7

Basin D2.6 & D2.7 consists of residential development, Skuna Drive, Abita Drive, Witcher Drive, and Yocona Drive. Runoff is directed west in curb/gutter to Design Point 61 in Witcher Drive. See the appendix for detailed calculations. The peak developed flow from Basin D2.6 is 4.3cfs and 9.5cfs for the 5/100-year storm event. The peak developed flow from Basin D2.7 is 6.7cfs and 15.0cfs for the 5/100-year storm event.

Basin D2.8

Basin D2.8 consists of residential development, Volga Drive, and Witcher Drive. Runoff is directed west and south in curb/gutter to Design Point 62 in Volga Drive. The peak developed flow from this basin is 7.7cfs and 17.2cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.9

Basin D2.9 consists of residential development, Volga Drive, Trappe Drive, and Witcher Drive. Runoff is directed west and north in curb/gutter to Design Point 60 in Trappe Drive. The peak developed flow from this basin is 5.5cfs and 12.2cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.10

Basin D2.10 consists of Trappe Drive and future adjacent areas to the west. Runoff is directed north in curb/gutter in Trappe Drive to Design Point 64. See the appendix for detailed calculations. The peak developed flow from this basin is 1.4cfs and 3.0cfs for the 5/100-year storm event.

Basin D2.10a

Basin D2.10a consists of future adjacent residential areas west of Trappe Drive. In the interim conditions runoff is directed northeast overland to Design Point 64. Future conditions consist of a Type R inlet and an 18" storm sewer connected to the inlet at Design Point 64. As part of this construction an 18" storm sewer will be stubbed out to the west to Design Point 64a. See the appendix for detailed calculations. The peak developed flow from this basin is 2.1cfs and 4.6cfs for the 5/100-year storm event.

Basin D2.11

Basin D2.11 consists of runoff from Lorson Boulevard on the south side. Runoff is directed west in curb/gutter to Design Point 65a in Lorson Boulevard. See SF 18-008. The peak developed flow from this basin is 2.0cfs and 3.6cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.12

Basin D2.12 consists of runoff from residential development and Lorson Boulevard on the south side. Runoff is directed west in curb/gutter to Design Point 60 in Trappe Drive. The peak developed flow from this basin is 5.4cfs and 12.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin D2.13

Basin D2.13 consists of runoff from existing residential development and existing Lorson Boulevard on the north side. Runoff is directed west in curb/gutter to an existing 10’ Type R inlet at Design Point 65b in Lorson Boulevard on the north side. See SF18-008. The peak developed flow from this basin is 4.0cfs and 9.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.1

Basin E1.1 consists of residential development and Skuna Drive. Runoff is directed south in curb/gutter in Skuna Drive to Design Point 66a. See the appendix for detailed calculations. The peak developed flow from this basin is 3.2cfs and 7.0cfs for the 5/100-year storm event.

Basin E1.2

Basin E1.2 consists of residential development, open space under the electric easement, Horton Drive, and Yocona Drive. Runoff is directed south in curb/gutter to Design Point 66d in Horton Drive. The peak developed flow from this basin is 7.3cfs and 16.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.3

Basin E1.3 consists of residential development and open space under the electric easement. Runoff is directed south in a swale to Design Point 67b next to Trappe Drive. The peak developed flow from this basin is 4.7cfs and 21.7cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.4

Basin E1.4 consists of residential development, Horton Drive, and Trappe Drive. Runoff is directed southwest in curb/gutter to Design Point 68 in Trappe Drive. The peak developed flow from this basin is 2.2 cfs and 5.0cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.5

Basin E1.5 consists of residential development, Horton Drive, Volga Drive, and Trappe Drive. Runoff is directed southwest in curb/gutter to Design Point 68 in Trappe Drive. The peak developed flow from this basin is 4.1cfs and 9.2cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.6

Basin E1.6 consists of residential development and Trappe Drive. Runoff is directed north in curb/gutter to Design Point 69 in Trappe Drive. The peak developed flow from this basin is 4.5cfs and 10.1cfs for the 5/100-year storm event. See the appendix for detailed calculations.

Basin E1.7
Basin E1.7 consists of residential development and Trappe Drive. Runoff is directed north in curb/gutter to Design Point 70 in Trappe Drive. See the appendix for detailed calculations. The peak developed flow from this basin is 5.4cfs and 15.3cfs for the 5/100-year storm event.

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

5.0 HYDRAULIC SUMMARY

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

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

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

	Residential Local		Residential Collector		Principal Arterial	
Street Slope	5-year	100-year	5-year	100-year	5-year	100-year
0.5%	6.3	26.4	9.7	29.3	9.5	28.5
0.6%	6.9	28.9	10.6	32.1	10.4	31.2
0.7%	7.5	31.2	11.5	34.6	11.2	33.7
0.8%	8.0	33.4	12.3	37.0	12.0	36.0
0.9%	8.5	35.4	13.0	39.3	12.7	38.2
1.0%	9.0	37.3	13.7	41.4	13.4	40.2
1.4%	10.5	44.1	16.2	49.0	15.9	47.6
1.8%	12.0	45.4	18.4	50.4	18.0	50.4
2.2%	13.3	42.8	19.4	47.5	19.5	47.5
2.6%	14.4	40.7	18.5	45.1	18.5	45.1
3.0%	15.5	39.0	17.7	43.2	17.8	43.2
3.5%	16.7	37.2	16.9	41.3	17.0	41.3
4.0%	17.9	35.7	16.2	39.7	16.3	39.7
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 3f (from CDR183)
Design Point 3f is located on the north side of Fontaine Boulevard at Rockcastle Drive and is the outflow pipe for Ponds C2.2, Pond C2.3, and Pond C1. The total allowed pipe flow is 14.0cfs in the 5-year and 131.0cfs in the 100-year which conforms to the outflow rates in the Lorson Ranch East MDDP for the ponds. This section of storm sewer has been oversized to accept 200cfs in a 54” RCP to account for emergency overflow conveyances from the future ponds as detailed in the MDDP.

Design Point 20a
Design Point 20a is located on the south side of Fontaine Boulevard south of Rockcastle Drive and is the outflow pipe for pond C1 located under the electric line easement. This 18” RCP outflow pipe will also function as the outflow pipe for interim Pond C1. The total allowed pipe flow is 4.0cfs in the 5-year and 18.0cfs in the 100-year which conforms to the outflow rates in the Lorson Ranch East MDDP for Pond C1. The interim pond outflow is 2.2cfs in the 5-year and 9.0cfs in the 100-year flow storm events as calculated by the hydraulic pond modeling program.

Design Point 20 (from CDR183)
Design Point 20 is located south side of Fontaine Boulevard north of the electric substation.

(5-year storm)

Tributary Basins: C15.8	**Inlet/MH Number:** Existing Inlet DP20
Upstream flowby:	**Total Street Flow:** 5.2cfs
Flow Intercepted: 5.2cfs	**Flow Bypassed:**
Inlet Size: 15’ type R, on-grade	

Street Capacity: Street slope = 1.0%, capacity = 13.0cfs, okay

(100-year storm)

Tributary Basins: C15.8	**Inlet/MH Number:** Existing Inlet DP20
Upstream flowby:	**Total Street Flow:** 13.4cfs
Flow Intercepted: 11.3cfs	**Flow Bypassed:** 2.1cfs to Inlet DP29
Inlet Size: 15’ type R, on-grade	

Street Capacity: Street slope = 1.0%, capacity = 40cfs (half street) is okay

Design Point 21
Design Point 21 is located west of the electric substation and is the surface runoff collected at a 30” end section (Line 22). The total flow in the storm sewer is from Basin C15.1+Basin C15.2 for a total flow of 13.2cfs/37.90cfs in the 5/100 year storm events in the storm sewer. The trapezoidal overflow swale between the lots is 1.0’ deep, 5:1 side slopes, 10’ wide bottom, 1% slope, velocity of 3.88cfs, and has a flow depth of 0.69 feet.

Design Point 23

Design Point 23 is located on Tillamook Drive north of Rockcastle Drive

(5-year storm)

Tributary Basins: C15.3&C15.4 **Inlet/MH Number:** Inlet DP23
Upstream flowby: **Total Street Flow:** 8.70cfs

Flow Intercepted: 8.43cfs **Flow Bypassed:** 0.3cfs to Inlet DP25
Inlet Size: 15' type R, on-grade

Street Capacity: Street slope = 1.1%, capacity = 9.2cfs, okay

(100-year storm)

Tributary Basins: C15.3&C15.4 **Inlet/MH Number:** Inlet DP23
Upstream flowby: **Total Street Flow:** 19.30cfs

Flow Intercepted: 13.93cfs **Flow Bypassed:** 5.4cfs to Inlet DP25
Inlet Size: 15' type R, on-grade

Street Capacity: Street slope = 1.1%, capacity = 38cfs (half street) is okay

Design Point 24

Design Point 24 is located in the south of Rockcastle Drive on Tillamook Drive and is the flow in the storm sewer per hydraflow. The total flow in the storm sewer is 20.30cfs/51.80cfs in the 5/100 year storm events.

Design Point 25

Design Point 25 is located on the south side of Rockcastle Drive east of Vedder Drive

(5-year storm)

Tributary Basins: C15.5,C15.6,C15.11, C15.12 **Inlet/MH Number:** Inlet DP25
Upstream flowby: 0.3cfs **Total Street Flow:** 15.9cfs

Flow Intercepted: 15.9cfs **Flow Bypassed:**
Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, okay since half flow from each side

(100-year storm)

Tributary Basins: C15.5,C15.6,C15.11, C15.12 **Inlet/MH Number:** Inlet DP25
Upstream flowby: 5.4cfs **Total Street Flow:** 38.8cfs

Flow Intercepted: 31.7cfs **Flow Bypassed:** 7.1cfs to Inlet DP26
Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 37.3cfs (half street) is okay since half flow from each side

Design Point 26

Design Point 26 is located on the north side of Rockcastle Drive east of Vedder Drive.

(5-year storm)

Tributary Basins: C15.7, C15.13 **Inlet/MH Number:** Inlet DP26

Upstream flowby: **Total Street Flow:** 8.4cfs

Flow Intercepted: 8.4cfs **Flow Bypassed:**

Inlet Size: 20' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, okay since half of flow is from each side.

(100-year storm)

Tributary Basins: C15.7, C15.13 **Inlet/MH Number:** Inlet DP26

Upstream flowby: 7.1cfs **Total Street Flow:** 26.0 cfs

Flow Intercepted: 26.0cfs **Flow Bypassed:**

Inlet Size: 20' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 37.3cfs (half street) is okay

Design Point 27

Design Point 27 is located in the north of Design Point 26 and is the flow in the storm sewer. The total flow in the storm sewer is 38.1cfs/92.6cfs in the 5/100 year storm events. The trapezoidal overflow swale between the lots is 1.0' deep, 4:1 side slopes, 15' wide bottom, 1% slope, velocity of 5.41cfs, and has a flow depth of 1.0 feet.

Design Point 59a

Design Point 59a is located at the south end of Tillamook Drive in a cul-de-sac

(5-year storm)

Tributary Basins: D2.2 **Inlet/MH Number:** Inlet DP59a

Upstream flowby: **Total Street Flow:** 2.1cfs

Flow Intercepted: 2.1cfs **Flow Bypassed:**

Inlet Size: 5' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, okay

(100-year storm)

Tributary Basins: D2.2 **Inlet/MH Number:** Inlet DP59a

Upstream flowby: **Total Street Flow:** 4.7cfs

Flow Intercepted: 4.7cfs **Flow Bypassed:**

Inlet Size: 5' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 37.3cfs (half street) is okay

Design Point 59b (from SF18-008 FDR)
Design Point 59b is located south of Lorson Boulevard under the electric easement and is the flow in the pipe from future Basin D1. The total future pipe flow (Line 27) is 23cfs/60cfs in the 5/100-year storm events.

Design Point 59c (from SF18-008 FDR)
Design Point 59c is located east of Lorson Boulevard and Lamprey Drive and is the flow in the pipe to Design Point 59e. The total pipe flow is 25.7cfs/75.4cfs in the 5/100 year storm events.

Design Point 59d (from SF18-008 FDR)
Design Point 59d is located in the northeast corner of Lorson Boulevard and Lamprey Drive.

(5-year storm)
Tributary Basins: D2.1 & D2.3 **Inlet/MH Number:** Inlet DP59d
Upstream flowby: **Total Street Flow:** 10.7cfs

Flow Intercepted: 10.7cfs **Flow Bypassed:**
Inlet Size: 15' type R, sump

Street Capacity: Street slope = 0.7%, capacity = 11.5cfs, okay

(100-year storm)
Tributary Basins: D2.1 & D2.3 **Inlet/MH Number:** Inlet DP59d
Upstream flowby: **Total Street Flow:** 23.7cfs

Flow Intercepted: 20.3cfs **Flow Bypassed:** 3.7cfs to Inlet DP65b
Inlet Size: 15' type R, sump

Street Capacity: Street slope = 0.7%, capacity = 34.6cfs (half street) is okay

Design Point 59e (from SF18-008 FDR)
Design Point 59e is located west of Lorson Boulevard and Lamprey Drive and is the flow in the pipe (Line 24) in Lorson Boulevard flowing west to Trappe Drive. The total pipe flow is 36.4cfs/93.2cfs in the 5/100 year storm events.

Design Point 59f (from SF18-008 FDR)
Design Point 59f is located at the SW corner of Lorson Boulevard and Skuna Drive.

(5-year storm)
Tributary Basins: D2.4 & D2.5 **Inlet/MH Number:** Inlet DP59f
Upstream flowby: **Total Street Flow:** 13.68cfs

Flow Intercepted: 8.58cfs **Flow Bypassed:** 5.1cfs to Inlet DP60
Inlet Size: 10' type R, on-grade

Street Capacity: Street slope = 1.9%, capacity Lorson Blvd.= 18.4cfs, okay

(100-year storm)
Tributary Basins: D2.4 & D2.5 **Inlet/MH Number:** Inlet DP59f
Upstream flowby: **Total Street Flow:** 30.47cfs

Flow Intercepted: 12.37cfs **Flow Bypassed:** 18.1cfs to Inlet DP60
Inlet Size: 10' type R, on-grade

Street Capacity: Street slope = 1.9%, capacity Lorson Blvd. = 50.4cfs (half street) is okay

Design Point 59g (from SF18-008 FDR)
Design Point 59g is located on Lorson Boulevard west of Skuna Drive and is the flow in the pipe (Line 23) in Lorson Boulevard flowing west to Trappe Drive. The total pipe flow is 45.0cfs/104.2cfs in the 5/100 year storm events.

Design Point 60 (from SF18-008 FDR)
Design Point 60 is located in the SE corner of Lorson Boulevard and Trappe Drive

(5-year storm)
Tributary Basins: D2.9, D2.12 **Inlet/MH Number:** Inlet DP60
Upstream flowby: 5.5cfs **Total Street Flow:** 16.2cfs

Flow Intercepted: 16.2cfs **Flow Bypassed:**
Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.8%, capacity = 18.4cfs, okay

(100-year storm)
Tributary Basins: D2.9, D2.12 **Inlet/MH Number:** Inlet DP60
Upstream flowby: 32.1cfs **Total Street Flow:** 55.9cfs

Flow Intercepted: 31.7cfs **Flow Bypassed:** 24.2cfs to Design Point 64
Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.8%, capacity = 50.4cfs (half street) is okay since half is from Lorson Blvd and half is from Trappe Drive.

Design Point 61
Design Point 61 is located on Witcher Drive just west of Yocona Drive.

(5-year storm)
Tributary Basins: D2.6 & D2.7 **Inlet/MH Number:** Inlet DP61
Upstream flowby: **Total Street Flow:** 10.20cfs

Flow Intercepted: 7.41cfs **Flow Bypassed:** 2.79cfs to Design Point 62
Inlet Size: 10' type R, on-grade

Street Capacity: Street slope = 3.1%, capacity = 15.5cfs, okay

(100-year storm)
Tributary Basins: D2.6 & D2.7 **Inlet/MH Number:** Inlet DP61
Upstream flowby: **Total Street Flow:** 22.70cfs

Flow Intercepted: 10.88cfs **Flow Bypassed:** 11.82cfs to Design Point 62
Inlet Size: 10' type R, on-grade

Street Capacity: Street slope = 3.1%, capacity = 39.0cfs (half street) is okay

Design Point 62

Design Point 62 is located on the east side of Volga Drive at Magothy Drive.

(5-year storm)
Tributary Basins: D2.8 **Inlet/MH Number:** Inlet DP62
Upstream flowby: 2.79cfs **Total Street Flow:** 10.5cfs

Flow Intercepted: 10.1cfs **Flow Bypassed:** 0.4cfs to Inlet DP60
Inlet Size: 10' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 9.0cfs, okay

(100-year storm)
Tributary Basins: D2.8 **Inlet/MH Number:** Inlet DP62
Upstream flowby: 11.82cfs **Total Street Flow:** 30.3cfs

Flow Intercepted: 16.3cfs **Flow Bypassed:** 14.0cfs to Design Point 60
Inlet Size: 10' type R, sump

Street Capacity: Street slope = 1.0%, capacity = 37.3cfs (half street) is okay

Design Point 63

Design Point 63 is located in the SE corner of Magothy Drive and Volga Drive and is the flow in the pipe (Line 34& 35) in Magothy Drive flowing west to Trappe Drive. The total pipe flow is 17.5cfs/27.30cfs in the 5/100 year storm events.

Design Point 64

Design Point 64 is located in the SW corner of Lorson Boulevard and Trappe Drive

(5-year storm)
Tributary Basins: D2.10+D2.10a **Inlet/MH Number:** Inlet DP64
Upstream flowby: **Total Street Flow:** 3.5cfs

Flow Intercepted: 3.5cfs **Flow Bypassed:**
Inlet Size: 25' type R, sump

Street Capacity: Street slope = 1.8%, capacity = 18.4cfs, okay

(100-year storm)
Tributary Basins: D2.10+D2.10a **Inlet/MH Number:** Inlet DP64
Upstream flowby: 24.2cfs **Total Street Flow:** 31.8cfs

Flow Intercepted: 31.8cfs **Flow Bypassed:**
Inlet Size: 25' type R, sump (okay since Basin D2.10a is in 18" pipe)

Street Capacity: Street slope = 1.8%, capacity = 50.4cfs, okay

Design Point 65 (from SF18-008 FDR)
Design Point 65 is located at the SW corner of Lorson Boulevard and Trappe Drive and is the flow in the pipe north (Line 30) to Design Point 65c. The total pipe flow is 36.60cfs/88.3cfs in the 5/100 year storm events.

Design Point 65a (from SF18-008 FDR)
Design Point 65a is located on the south side of Lorson Boulevard west of Trappe Drive

(5-year storm)
Tributary Basins: D2.11 **Inlet/MH Number:** Inlet DP65a
Upstream flowby: **Total Street Flow:** 2.0cfs

Flow Intercepted: 2.0cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 0.66%, capacity = 10.6 cfs, okay

(100-year storm)
Tributary Basins: D2.11 **Inlet/MH Number:** Inlet DP65a
Upstream flowby: **Total Street Flow:** 3.6cfs

Flow Intercepted: 3.6cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 0.66%, capacity = 32.1cfs (half street) is okay

Design Point 65b (from SF18-008 FDR)
Design Point 65b is located on the north side of Lorson Boulevard west of Trappe Drive

(5-year storm)
Tributary Basins: D2.13 **Inlet/MH Number:** Inlet DP65b
Upstream flowby: **Total Street Flow:** 4.2cfs

Flow Intercepted: 4.2cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 0.66%, capacity = 10.6cfs, okay

(100-year storm)
Tributary Basins: D2.13 **Inlet/MH Number:** Inlet DP65b
Upstream flowby: 3.7cfs **Total Street Flow:** 12.7cfs

Flow Intercepted: 12.7cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 0.66%, capacity = 32.1cfs (half street) is okay

Design Point 65c (from SF18-008 FDR)
Design Point 65c is located west of Lorson Boulevard and Trappe Drive and is the flow in the pipe (Line 17) north to Pond D2. The total pipe flow is 88.3cfs/174.2cfs in the 5/100 year storm events.

Design Point 66a
Design Point 66a is located at the south end of Skuna Drive in the cul-de-sac

(5-year storm)
Tributary Basins: E1.1 **Inlet/MH Number:** Inlet DP66a
Upstream flowby: **Total Street Flow:** 3.2cfs

Flow Intercepted: 3.2cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 2.5%, capacity = 14cfs, okay

(100-year storm)
Tributary Basins: E1.1 **Inlet/MH Number:** Inlet DP66a
Upstream flowby: **Total Street Flow:** 7.0cfs

Flow Intercepted: 7.0cfs **Flow Bypassed:**
Inlet Size: 5' type R, sump

Street Capacity: Street slope = 2.5%, capacity = 40cfs (half street) is okay

Design Point 66b
Design Point 66b is located east of Horton Drive/Yocona Drive knuckle and is the flow in the pipe from Pond E1. The total future pipe flow (Line 15) is allowed to be 12.8cfs in the 5-year and 36.3cfs in the 100-year storm events per Lorson Ranch East MDDP. Pond E1 interim release rate (from hydraflow) is 9.0cfs in the 5-year and 20cfs in the 100-year storm events so the pipes will be sized adequately for any possible future flows. The future flows will be used in the pipe sizing downstream.

Design Point 66c
Design Point 66c is located east of the Horton Drive/Yocona Drive knuckle and is the flow in the pipe as it discharges into a swale flowing west to Trappe Drive. The total pipe flow (Line 14) is 16.0cfs/43.3cfs in the 5/100-year storm events.

Design Point 66d
Design Point 66d is located on the southeast side of the Horton Drive/Volga Drive intersection.

(5-year storm)
Tributary Basins: E1.2
Inlet/MH Number: Inlet DP66d
Upstream flowby:
Total Street Flow: 7.30cfs

Flow Intercepted: 6.1cfs
Flow Bypassed: 1.2cfs to Design Pt. 68
Inlet Size: 10’ type R, on-grade

Street Capacity: Street slope = 2.5%, capacity = 14.0cfs, okay

(100-year storm)
Tributary Basins: E1.2
Inlet/MH Number: Inlet DP66d
Upstream flowby:
Total Street Flow: 16.10cfs

Flow Intercepted: 9.30cfs
Flow Bypassed: 6.8cfs to Design Pt. 68
Inlet Size: 10’ type R, on-grade

Street Capacity: Street slope = 2.5%, capacity = 40.7cfs (half street) is okay

Design Point 67a
Design Point 67a is located at the east end of Trappe Drive on the south side at the electric easement. Flow at this design point is from Basin E2-ex which includes offsite flows. A 30” storm sewer (Line 13) will extend to the ROW where the flow will be collected in double Type D inlets and swale. The total flow is 26.0cfs/91cfs in the 5/100-year storm events to the end section. The storm sewer system will collect 26.0cfs and 70.0cfs in the 5/100-year storm events and 21.0cfs in the 100-year storm event will flow to Trappe Drive and then west to Design Point 70 in the street. The diversion swale at this design point is 3.0’ deep, 4:1 side slopes, 0’ wide bottom, 2.0% slope, velocity of 7.43cfs, and has a flow depth of 1.75 feet. This swale will be lined with coconut ECB.

Design Point 67b
Design Point 67b is located on the east end of Trappe Drive on the north side. Flow at this design point is from Basin E1.3 and Design Point 66c. A 30” storm (Line 8) will collect this area and convey it to Trappe Drive. The total flow at the end section is 20.0cfs/64.10cfs in the 5/100 year storm events. The storm sewer system will collect 20.0cfs and 42.0cfs in the 5/100-year storm events and 22.1cfs in the 100-year storm event will flow to Trappe Drive and then west to Design Point 68 in the street. The Basin E1.3 swale is sized for 210cfs which is the future emergency overflow from Pond E1. The swale is 3.0’ deep, 4:1 side slopes, 0’ wide bottom, up to a 2.96% slope, velocity of 10.65cfs, and has a flow depth of 2.22 feet.

Design Point 68

Design Point 68 is located in the NE corner of Trappe Drive and Horton Drive.

(5-year storm)

Tributary Basins: E1.4 & E1.5 **Inlet/MH Number:** Inlet DP68

Upstream flowby: 1.2cfs **Total Street Flow:** 7.5cfs

Flow Intercepted: 7.5cfs **Flow Bypassed:**

Inlet Size: 15' type R, on-grade

Street Capacity: Street slope = 1.15%, capacity = 14.0cfs, okay

(100-year storm)

Tributary Basins: E1.4 & E1.5 **Inlet/MH Number:** Inlet DP68

Upstream flowby: 28.9cfs **Total Street Flow:** 43.1cfs

Flow Intercepted: 20.17cfs **Flow Bypassed:** 22.93cfs to Design Pt. 69

Inlet Size: 15' type R, on-grade

Street Capacity: Street slope = 1.15%, capacity = 43cfs (half street) is okay

Design Point 69

Design Point 69 is located on the east side of Trappe Drive south of Magothy Drive at a low point.

(5-year storm)

Tributary Basins: E1.6 **Inlet/MH Number:** Inlet DP69

Upstream flowby: **Total Street Flow:** 4.5cfs

Flow Intercepted: 4.5cfs **Flow Bypassed:**

Inlet Size: 30' type R, sump

Street Capacity: Street slope = 1.15%, capacity = 14.0cfs, okay

(100-year storm)

Tributary Basins: E1.6 **Inlet/MH Number:** Inlet DP69

Upstream flowby: 22.93cfs **Total Street Flow:** 33.03cfs

Flow Intercepted: 33.03cfs **Flow Bypassed:**

Inlet Size: 30' type R, sump

Street Capacity: Street slope = 1.15%, capacity = 43cfs (half street) is okay

Design Point 70

Design Point 70 is located on the west side of Trappe Drive south of Magothy Drive at a low point.

(5-year storm)

Tributary Basins: E1.7 **Inlet/MH Number:** Inlet DP70
Upstream flowby: **Total Street Flow:** 5.4cfs

Flow Intercepted: 5.4cfs **Flow Bypassed:**
Inlet Size: 30' type R, sump

Street Capacity: Street slope = 1.15%, capacity = 14.0cfs, okay

(100-year storm)

Tributary Basins: E1.7 **Inlet/MH Number:** Inlet DP70
Upstream flowby: 21.0cfs **Total Street Flow:** 36.3cfs

Flow Intercepted: 36.3cfs **Flow Bypassed:**
Inlet Size: 30' type R, sump

Street Capacity: Street slope = 1.15%, capacity = 43.0cfs (half street)

Design Point 71

Design Point 71 is located East of Trappe Drive and is the flow into Interim Pond E2. The total pipe flow (Line1) is 69.7cfs/211.50cfs in the 5/100 year storm events. Interim Pond E2 will need to be updated in the future as additional tributary areas are developed. This pond is only to treat developed runoff for water quality because Pond E1 reduces upstream flows. See Pond discussions. Flows that exceed the water quality outlet capacity will flow over a trapezoid spillway to the south and enter existing swale that drain to the East Tributary. See Section 6.1 for interim flow rates at the East Tributary for downstream flows entering the East Tributary at Design Point 73.

Design Point 72

Design Point 72 has been added so the ultimate storm sewer outfall for Future Pond E2 can be referenced. The size of the storm sewer is 48" and corresponds to Design Pt 14a in the MDDP

Design Point 73

Design Point 73 is located downstream of Interim Pond E2 next to the East Tributary on an existing natural swale draining to the East Tributary. The future ultimate developed flows at this design point are 97.0cfs/266.0cfs in the 5/100-year storm events (Design Pt 14a in MDDP) when all upstream areas are developed and future Pond E2 is built. However, we are in an interim condition since we are not constructing future Pond E2 yet and the interim flows are 95cfs/280cfs in the 5/100-year storm events based on upstream development in this filing, existing offsite basins, and Pond E1. See Appendix for additional calculations. The interim flows are near pre-development flows of (100cfs/280cfs) as calculated in the MDDP. There are negligible negative impacts downstream due to the interim ponds in the "E" basins.

6.0 DETENTION AND WATER QUALITY PONDS & INTERIM DETENTION POND CALCULATIONS

Detention and Storm Water Quality for Lorson Ranch East Filing No. 4 is required per El Paso County criteria. We have implemented the Full Spectrum approach for detention per the Denver Urban Drainage Districts specifications. Runoff from Lorson Ranch East Filing No. 4 is broken into three separate areas. The northern areas are the “C15” basins which flow north to Fontaine Boulevard where existing storm sewer conveys the runoff to existing Pond C5. The middle areas are the “D” basins which flow to Lorson Boulevard and then west in existing storm sewer to existing Pond D2. The southern areas are the “E” basins which flow west to Trappe Drive in proposed storm sewer to proposed Pond E2. Pond E2 is a full spectrum/WQ interim pond that will be expanded in the future when additional tributary areas are developed.

<u>Detention Pond C5 (Full Spectrum Design, See SF08-008 FDR)</u>
This is an existing permanent full spectrum detention pond constructed in 2018 as part of the Lorson Ranch East Filing No. 1 subdivision that includes water quality and discharges directly into the East Tributary. Pond C5 was designed in the UDCF Full Spectrum spreadsheets and include Water Quality and EURV volumes. The outlet structure is a five cell CDOT type D outlet in parallel and the overflow spillway is a wier set slightly above the outlet structure so it releases the 5yr/100yr storm events quickly to match pre-developed rates.

- Watershed Ares: 171 acres (Ultimate Area)
- Watershed Imperviousness: 63%
- Hydrologic Soils Group C/D
- Forebay: 3.51ac-ft (see spreadsheet in appendix) divided between two forebays
- Zone 1 WQCV: 3.298ac-ft, WSEL: 5709.92
- Zone 2 EURV: 9.524ac-ft, WSEL: 5712.27, Top outlet structure set at 5712.60, 3’x30’ five CDOT Type D outlets in parallel.
- (5-yr): 13.06ac-ft, WSEL: 5713.49, 126.3cfs (hydraflow)
- Zone 3 (100-yr): 15.86ac-ft, WSEL: 5714.42, 453.2cfs (hydraflow)
- Pipe Outlet: 48” RCP at 0.5%
- Overflow Spillway: 52’ wide bottom, elevation=5713, 4:1 side slopes, flow depth=2.0’ at 519cfs inflow, 1’ freeboard
- Pond Bottom Elevation: 5706.00

<u>Detention Pond D2 (Full Spectrum Design, Ultimate Conditions, See SF18-008 FDR)</u>
This is an existing permanent full spectrum detention pond constructed in 2018 that includes water quality and discharges directly into the East Tributary. Pond D2 is a typical full spectrum pond was designed using the UDCF Full Spectrum spreadsheets. The outlet structure is a standard 4’x20’ full spectrum sloped outlet structure and the overflow spillway is a weir set above the outlet structure designed by the full spectrum spreadsheets to match pre-developed rates.

- Watershed Ares: 89 acres (Ultimate Area)
- Watershed Imperviousness: 55%
- Hydrologic Soils Group B
- Forebay: 1.635ac-ft (see spreadsheet in appendix) divided between two forebays
- Zone 1 WQCV: 1.52ac-ft, WSEL: 5697.72
- Zone 2 EURV: 5.02ac-ft, WSEL: 5699.74, Top EURV set at 5700.00, 4’x20’ outlet with 20:1 slope, 2.0cfs
- (5-yr): 5.53ac-ft, WSEL: 5700.02, 2.1cfs
- Zone 3 (100-yr): 9.05ac-ft, WSEL: 5701.84, 145cfs
- Pipe Outlet: 54” RCP at 0.5% with restrictor plate up 35”
- Overflow Spillway: 30’ wide bottom, elevation=5702.00, 4:1 side slopes, flow depth=1.64’ at 277.1cfs

- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5695.00

Pond C1

Pond C1 is located east of the powerlines and substation, south of Fontaine Boulevard and was designed as part of the Lorson Ranch East PDR and Early Grading Plans. This pond will be constructed to reduce existing upstream runoff east of powerlines so the capacity of the downstream runoff conveyance (streets/storm sewer) in Lorson Ranch East Filing No. 4 is not exceeded. Detention Pond C1 consists of an 18" pipe outlet, 3:1 pond slopes, maintenance access road, and an emergency overflow. The pond outfall flows north in an 18" storm sewer to Fontaine Boulevard where a storm sewer was constructed as part of the Fontaine Boulevard Improvements (CDR183). Pond C1 will be upgraded in the future to a full spectrum pond with an outlet structure, concrete low flow channel, and forebays when upstream development occurs.

- Inflow is 24cfs and 134cfs in the 5/100-year storm events for existing conditions from Basin C15-ex.
- The proposed pond outfall storm sewer is designed to convey 4cfs and 18cfs in the 5/100-year storm events for future conditions to an existing 18" storm sewer in Fontaine Boulevard. Interim flows are 2cfs and 9cfs in the 5/100-year storm events which are less than the future conditions.
- 5-year Interim WSEL=5746.67 and 0.62 ac-ft storage
- 100-year Interim WSEL=5749.25 and 3.94 ac-ft storage
- Emergency Overflow Weir sized for future developed flow = 175cfs, Inv=5753.00, 28' wide, 3' deep, flow depth=1.44' deep The emergency overflow is designed for future conditions per the Lorson Ranch East MDDP.
- Spillway swale to Fontaine: 175cfs, 50' btm, 0.3% slope, 2' deep, 4:1 sides, velocity=3.14cfs, flow depth=1.04' and a section with a 20' btm, 0.3% slope, 3' deep, 4:1 sides, velocity=3.90cfs, flow depth=1.68'

Pond E1

Pond E1 is located east of the powerlines, south of Lorson Boulevard and was designed as part of the Lorson Ranch East PDR and Early Grading Plans. This pond will be constructed to reduce existing upstream runoff east of powerlines so the capacity of the downstream runoff conveyance (streets/storm sewer) in Lorson Ranch East Filing No. 4 is not exceeded. Detention Pond E1 consists of an 18" pipe outlet, 3:1 pond slopes, and a maintenance access road. The pond outfall flows west in a 24" storm sewer where a storm sewer in Trappe Drive will convey the flow west to the East Tributary. Pond E1 will be upgraded in the future to a full spectrum pond with an outlet structure, concrete low flow channel, and forebays when upstream development occurs.

- Inflow is 25cfs and 142cfs in the 5/100-year storm events for existing conditions from Basin E1-ex.
- The proposed pond outfall storm sewer is designed to convey 12.8cfs and 36.3cfs in the 5/100-year storm events for future conditions. Interim flows are 9cfs and 20cfs in the 5/100-year storm events which are less than the future conditions.
- 5-year Interim WSEL=5730.42 and 0.54 ac-ft storage
- 100-year Interim WSEL=5732.89 and 2.45 ac-ft storage

Detention Pond E2 (Full Spectrum Design, Interim Conditions)

This is an off-site full spectrum detention pond that includes water quality and discharges directly into the East Tributary. Pond E2 is a typical full spectrum pond is designed using the UDCF Full Spectrum spreadsheets. Pond E2 will be designed for interim conditions of 21.2 acres of developed upstream areas. The outlet structure is a standard 4'x6' full spectrum flat topped outlet structure and the overflow spillway is a weir set above the outlet structure designed by the full spectrum spreadsheets to match pre-developed rates. The ultimate tributary area for Pond E2 is 125 acres and will require the pond size to be increased as well as the outlet structure. We have designed the outlet structure to be easily modified by adding additional width to the structure to be a 4'x25' flat topped outlet structure while keeping the WQ/EURV portion of the structure the same. The only change will be to the water quality/EURV plate which can be easily modified. Both the interim and ultimate conditions have been modeled in the excel spreadsheets. The outlet structure construction plans are also included in the appendix of this report which include modifications necessary to the structure to change it to the ultimate conditions.

The full spectrum print outs are in the appendix of this report. See map in appendix for watershed areas.

<u>Pond E2 Interim Conditions:</u>

- Watershed Ares: 21.2 acres (Interim Area)
- Watershed Imperviousness: 50%
- Hydrologic Soils Group B and C
- Forebay: 0.0256ac-ft (see spreadsheet in appendix). Second forebay same size to be built when additional development occurs in basin.
- Zone 1 WQCV: 0.338ac-ft, WSEL: 5695.45
- Zone 2 EURV: 0.971ac-ft, WSEL: 5697.09, Top EURV set at 5698.60, 4'x6' outlet with flat top, 0.8cfs
- (5-yr): 1.221ac-ft, WSEL: 5697.67, 1.0cfs
- Zone 3 (100-yr): 2.055ac-ft, WSEL: 5699.40, 31.2cfs
- Pipe Outlet: 48" RCP at 0.5% with no restrictor plate
- Interim Overflow Spillway: 40' wide bottom, elevation=5699.50, 4:1 side slopes, soil rip rap
- Pre-development release rate into creek compliance from full spectrum pond spreadsheets
- Pond Bottom Elevation: 5693.00

<u>Pond E2 Ultimate Conditions:</u>

- Watershed Ares: 125 acres (Basin E3-ex+ Basin E2-ex+21.2ac)
- Watershed Imperviousness: 35%
- Hydrologic Soils Group B and C
- Forebay: 0.0256ac-ft (see spreadsheet in appendix). Need two forebays of this size.
- Zone 1 WQCV: 1.618ac-ft, WSEL: 5696.05
- Zone 2 EURV: 3.869ac-ft, WSEL: 5697.57, Top EURV set at 5698.60, 4'x25' outlet with flat top, 3.5cfs
- (5-yr): 5.072ac-ft, WSEL: 5698.33, 4.5cfs
- Zone 3 (100-yr): 9.675ac-ft, WSEL: 5701.06, 150.1cfs
- 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

<u>Water Quality Design</u>

Water quality for this final plat will be provided by existing Pond C5 for the 'C15' basins, existing Pond D2 for the 'D' Basins, and Pond E2 for Basins E1.1-E1.7. The two existing ponds have been designed in the Lorson Ranch East Filing No. 1 FDR and account for development of this phase of Lorson Ranch in their water quality features. Pond E2 includes water quality design for the remaining portions of this plat.

7.0 DRAINAGE AND BRIDGE FEES

Lorson Ranch East Filing No. 4 is located within the Jimmy Camp Creek drainage basin which is currently a fee basin in El Paso County.

Lorson Ranch East Filing No. 4 contains 58.471acres. This project consists of 6.935 acres of open space (2% impervious), and the remaining 51.536 acres is residential (50% impervious). The 20~~18~~ drainage fees are $~~17,197~~, bridge fees are $~~804~~ and Drainage Surety fees are $7,285 per impervious acre per Resolution 17-348. The drainage and bridge fees are calculated when the final plat is submitted. The fees are due at plat recordation. The following table details the drainage fees for the platted area. 2019 fees $18,350 Drainage & $858 Bridge.

Table 1: Drainage/Bridge Fees

Type of Land Use	Total Area (ac)	Imperviousness	Drainage Fee	Bridge Fee	Surety Fee
Residential Area	51.536	50%	$443,132	$20,717	$187,719
Open Space, Landscape Tracts,	6.935	2%	$2,385	$111	$1,010
		Total	$445,517	$20,828	$188,729

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

Item	Quantity	Unit	Unit Cost	Item Total
Rip Rap	100	CY	$50/CY	$5,000
Inlets/Manholes	27	EA	$3000/EA	$81,000
18” Storm	1180	LF	$35	$41,300
24” Storm	580	LF	$40	$23,200
30” Storm	625	LF	$45	$28,125
36” Storm	400	LF	$55	$22,000
42” Storm	200	LF	$85	$17,000
48” Storm	820	LF	$95	77,900
54” Storm	100	LF	$115	$11,500
			Subtotal	$307,025
			Eng/Cont (15%)	$46,053
			Total Est. Cost	$353,078

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

Item	Quantity	Unit	Unit Cost	Item Total
Interim Pond and Outlet	1	LS	$50,000	$50,000
			Subtotal	$50,000
			Eng/Cont (15%)	$7,500
			Total Est. Cost	$57,500

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
Lorson Ranch East Filing No. 4 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 has been added along the SDS watermain easement which reduces impervious areas
- Lorson Ranch Metro District requires homeowners to maintain landscaping on lots
- Full Spectrum Detention Pond E2 ~~has been~~ 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. Lorson Ranch East Filing No. 4 will utilize Pond C5, D2, 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 2018 the East Tributary of JCC was 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

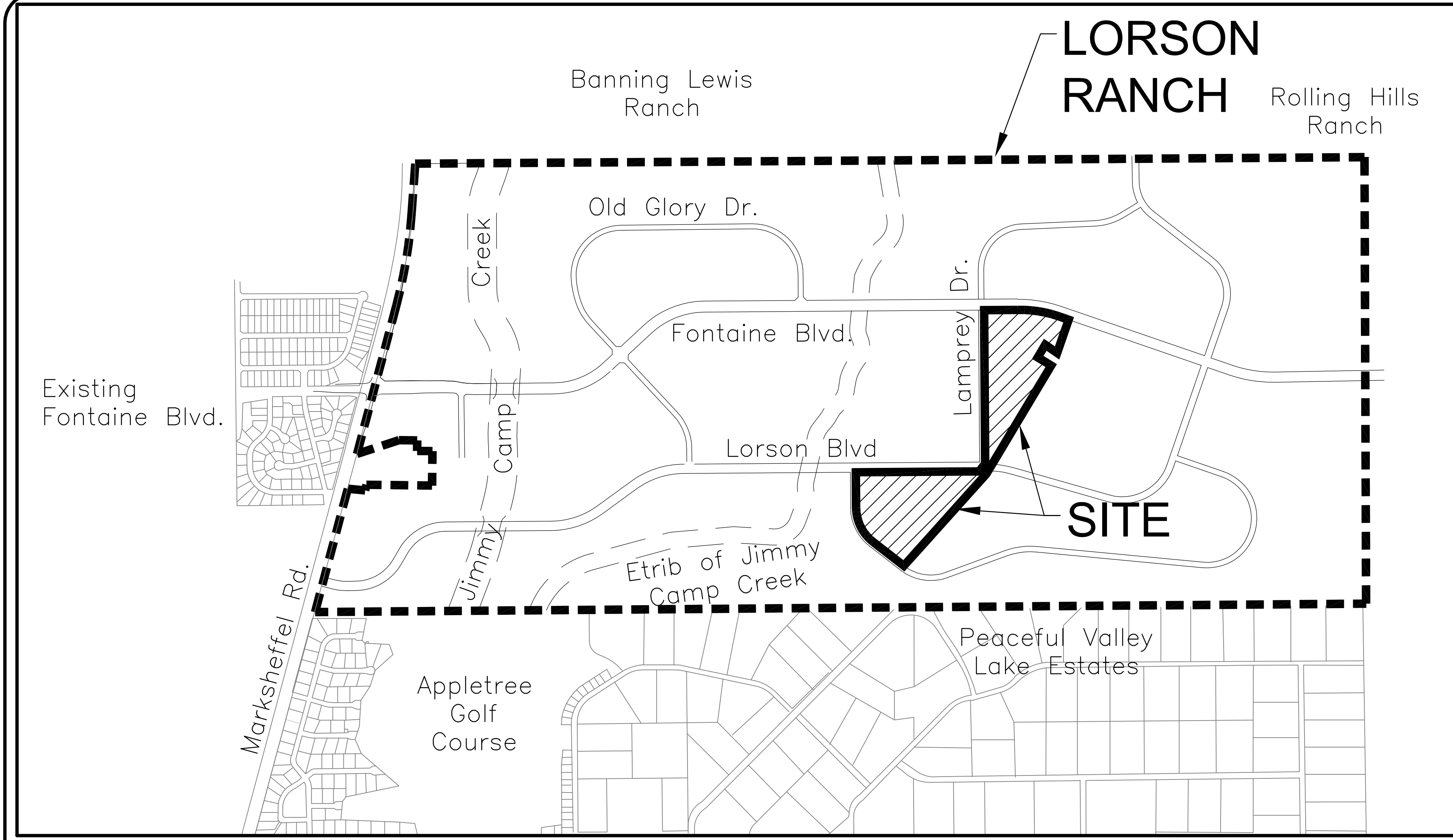
This drainage report has been prepared in accordance with the City of Colorado Springs/El Paso County Drainage Criteria Manual. The proposed development and drainage infrastructure will not cause adverse impacts to adjacent properties or properties located downstream. Several key aspects of the development 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 has been reconstructed west of this study area
- Bridges over the East Tributary are constructed at Lorson Boulevard and Fontaine Boulevard providing access to this site.
- The bridge over Jimmy Camp Creek at Lorson Boulevard is required for this plat
- Detention and water quality for this site area will be provided in existing pond C5, existing pond D2, and proposed Pond E2 which are maintained by the Lorson Ranch Metro District.
- Access to existing maintenance trails for the East Tributary of Jimmy Camp Creek will be provided offsite west of this site.
- Lorson Ranch Metro District will maintain all 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 Drainage Report for Fontaine Boulevard, Old Glory Drive, and Marksheffel Road Phase 1 Improvements, Dated February 6, 2006, Revised September 7, 2006, by Pentacor Engineering.
8. Final construction plans “Fontaine Boulevard and East Fork Jimmy Camp Creek Channel Design”, Dated March 10, 2017, by Kiowa Engineering Corporation
9. 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.
10. Kiowa Engineering Corporation “Final Bridge and Channel Design Report, CDR 16-009” revised August 24, 2017
11. Lorson Ranch East Filing No. 1 FDR prepared by Core Engineering Group, dated July 2, 2018
12. Lorson Ranch East PDR prepared by Core Engineering Group, dated December 18, 2017
13. Final Drainage Report for Fontaine Boulevard prepared by Core Engineering Group, Reference CDR183, dated December 20, 2017

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

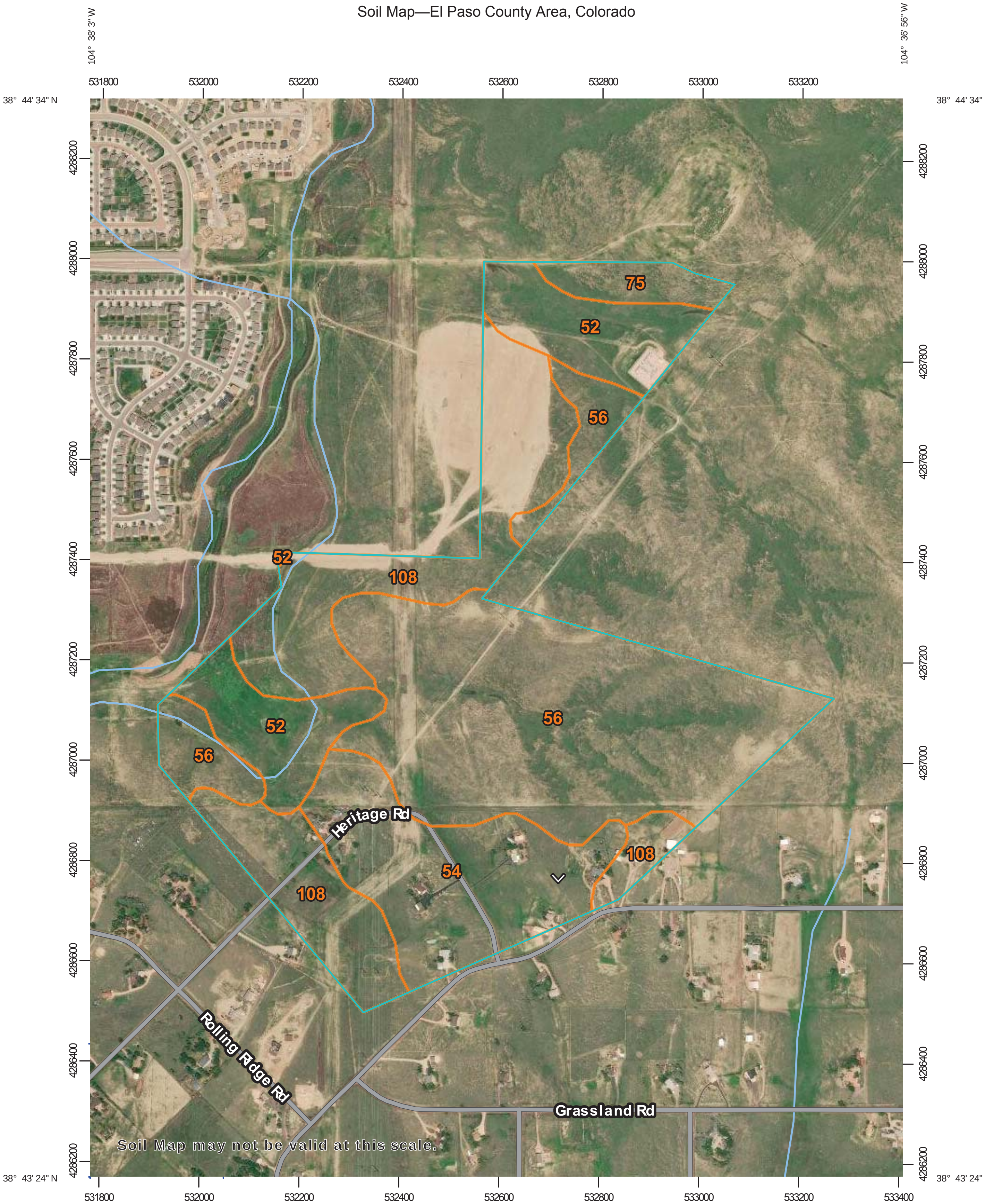
CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

LORSON RANCH EAST FILING NO. 4
VICINITY MAP

SCALE:	DATE:	FIGURE NO.
NTS	JANUARY 31, 2019	--


Zone AE
G
G
F
F
E
EL PASO COUNTY
080059
site
08041C0976G
eff. 12/7/2018
AREA OF MINIMAL FLOOD HAZARD
Zone X

Soil Map—El Paso County Area, Colorado



Map Scale: 1:10,500 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0 150 300 600 900

Feet: 0 500 1000 2000 3000

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 16, Sep 10, 2018

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

Date(s) aerial images were photographed: Apr 12, 2017—Nov 17, 2017

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
52	Manzanst clay loam, 0 to 3 percent slopes	30.1	13.6%
54	Midway clay loam, 3 to 25 percent slopes	36.8	16.6%
56	Nelson-Tassel fine sandy loams, 3 to 18 percent slopes	91.6	41.4%
75	Razor-Midway complex	6.5	2.9%
108	Wiley silt loam, 3 to 9 percent slopes	56.5	25.5%
Totals for Area of Interest		**221.5**	**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:February 28, 2019
Checked By: Leonard Beasley

Job No: 100.048
Project: Lorson Ranch East 4 Final Drainage
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
C15.1			7.10	0.30	18.04	2.13	3.24	6.9													
C15.2			4.63	0.42	11.51	1.94	3.92	7.6													
	21								18.04	4.07	3.24	13.2									
C15.3			3.60	0.49	13.83	1.76	3.64	6.4													
C15.4			1.25	0.49	9.05	0.61	4.28	2.6													
	23								13.83	2.38	3.64	8.7									
C15.5			2.90	0.49	9.86	1.42	4.15	5.9													
C15.6			1.80	0.49	12.88	0.88	3.75	3.3													
C15.7			2.07	0.49	11.73	1.01	3.89	3.9													
C15.8			3.76	0.40	15.51	1.50	3.47	5.2													
C15.9			2.27	0.49	8.22	1.11	4.42	4.9													
C15.10			0.60	0.49	9.85	0.29	4.15	1.2													
C15.11			3.20	0.49	11.58	1.57	3.91	6.1													
C15.12			0.61	0.49	11.47	0.30	3.92	1.2													
	23								12.88	4.17	3.75	15.6									
C15.13			2.35	0.49	11.49	1.15	3.92	4.5													
C15.14			1.32	0.49	8.11	0.65	4.44	2.9													
D2.1			3.14	0.49	14.87	1.54	3.53	5.4													
D2.2			1.11	0.49	11.93	0.54	3.86	2.1													
D2.3			2.80	0.27	14.09	0.76	3.61	2.7													
D2.4			3.33	0.29	13.48	0.97	3.68	3.6													
D2.5			3.93	0.49	7.40	1.93	4.58	8.8													
D2.6			2.13	0.49	10.37	1.04	4.07	4.3													
D2.7			2.98	0.49	7.22	1.46	4.62	6.7													
	61								10.37	2.50	4.07	10.2									
D2.8			3.70	0.49	9.24	1.81	4.25	7.7													
D2.9			3.15	0.49	14.83	1.54	3.54	5.5													
D2.10			0.57	0.49	6.24	0.28	4.84	1.4													
D2.10a			1.10	0.49	12.20	0.54	3.83	2.1													
D2.11			0.40	0.90	3.68	0.36	5.63	2.0													
D2.12			2.78	0.49	11.27	1.36	3.95	5.4													
D2.13			2.51	0.49	17.67	1.23	3.28	4.0													

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

CORE ENGINEERING GROUP

Calculated By: Leonard Beasley
Date:February 28, 2019
Checked By: Leonard Beasley

Job No: 100.048
Project: Lorson Ranch East 4 Final Drainage
Design Storm: **5 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	Σ (CA)	i	Q	Slope	Street Flow	Design Flow	Slope	Pipe Size	Length	Velocity	tt	
			ac.		min.		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	
E1.1			1.41	0.49	7.40	0.69	4.58	3.2													
E1.2			3.61	0.49	10.20	1.77	4.10	7.3													
E1.3			6.81	0.20	15.70	1.36	3.45	4.7		0.25											
E1.4			1.10	0.49	9.92	0.54	4.14	2.2													
E1.5			1.95	0.49	8.86	0.96	4.31	4.1													
E1.6			2.32	0.49	10.94	1.14	3.99	4.5													
E1.7			4.00	0.38	14.72	1.52	3.55	5.4													
C15-ex			55	0.15	22.61	8.25	2.91	24													
D1-ex			17	0.15	17.78	2.55	3.27	8													
E1-ex			57	0.15	21.72	8.55	2.97	25													
E2-ex			30	0.26	16.78	7.67	3.35	26													
E3-ex			72.5	0.20	22.00	14.50	2.95	43													

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

CORE ENGINEERING GROUP

Calculated By: Leonard Beasley
Date: February 28, 2019
Checked By: Leonard Beasley

Job No: 100.048
Project: Lorson Ranch East 4 Final Drainage
Design Storm: **100 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff: Area Design	Direct Runoff: Area (A) ac.	Direct Runoff: Runoff Coeff. (C)	Direct Runoff: tc min.	Direct Runoff: CA	Direct Runoff: i in/hr	Direct Runoff: Q cfs	Total Runoff: tc min	Total Runoff: Σ (CA)	Total Runoff: i in/hr	Total Runoff: Q cfs	Street: Slope %	Street: Street Flow cfs	Pipe: Design Flow cfs	Pipe: Slope %	Pipe: Pipe Size in	Travel Time: Length ft	Travel Time: Velocity ft/sec	Travel Time: tt min	Remarks
C15.1			7.10	0.57	18.04	4.05	5.45	22.0													
C15.2			4.63	0.63	11.51	2.92	6.58	19.2													
	21								18.04	6.96	5.45	37.9									
C15.3			3.60	0.65	13.83	2.34	6.12	14.3													
C15.4			1.25	0.65	9.05	0.81	7.18	5.8													
	23								13.83	3.15	6.12	19.3									
C15.5			2.90	0.65	9.86	1.89	6.97	13.1													
C15.6			1.80	0.65	12.88	1.17	6.29	7.4													
C15.7			2.07	0.65	11.73	1.35	6.53	8.8													
C15.8			3.76	0.61	15.51	2.29	5.83	13.4													
C15.9			2.27	0.65	8.22	1.48	7.43	11.0													
C15.10			0.60	0.65	9.85	0.39	6.97	2.7													
C15.11			3.20	0.65	11.58	2.08	6.56	13.7													
C15.12			0.61	0.65	11.47	0.40	6.59	2.6													
	25	**(no upstream flowby)**							14.30	5.53	6.03	33.4									
C15.13			2.35	0.65	11.49	1.53	6.58	10.1													
C15.14			1.32	0.65	8.11	0.86	7.46	6.4													
D2.1			3.14	0.65	14.87	2.04	5.93	12.1													
D2.2			1.11	0.65	11.93	0.72	6.49	4.7													
D2.3			2.80	0.57	14.09	1.60	6.07	9.7													
D2.4			3.33	0.58	13.48	1.93	6.18	11.9													
D2.5			3.93	0.65	7.40	2.55	7.69	19.6													
D2.6			2.13	0.65	10.37	1.38	6.84	9.5													
D2.7			2.98	0.65	7.22	1.94	7.75	15.0													
	61								10.37	3.32	6.84	22.7									
D2.8			3.70	0.65	9.24	2.41	7.13	17.2													
D2.9			3.15	0.65	14.83	2.05	5.94	12.2													
D2.10			0.57	0.65	6.24	0.37	8.12	3.0													
D2.10a			1.10	0.65	12.20	0.72	6.43	4.6													
D2.11			0.40	0.96	3.68	0.38	9.45	3.6													
D2.12			2.78	0.65	11.27	1.81	6.63	12.0													

CORE ENGINEERING GROUP

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

Calculated By: Leonard Beasley
Date: February 28, 2019
Checked By: Leonard Beasley

Job No: 100.048
Project: Lorson Ranch East 4 Final Drainage
Design Storm: **100 - Year Event, Proposed Conditions**

Street or Basin	Design Point	Direct Runoff							Total Runoff				Street		Pipe			Travel Time			Remarks
		Area Design	Area (A)	Runoff Coeff. (C)	tc	CA	i	Q	tc	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	
D2.13			2.51	0.65	17.67	1.63	5.50	9.0													
E1.1			1.41	0.65	7.40	0.92	7.69	7.0													
E1.2			3.61	0.65	10.20	2.35	6.88	16.1													
E1.3			6.81	0.55	15.70	3.75	5.80	21.7		0.57											
E1.4			1.10	0.65	9.92	0.72	6.95	5.0													
E1.5			1.95	0.65	8.86	1.27	7.24	9.2													
E1.6			2.32	0.65	10.94	1.51	6.71	10.1													
E1.7			4.00	0.64	14.72	2.56	5.96	15.3													
C15-ex			55	0.50	22.61	27.50	4.88	134													
D1-ex			17	0.50	17.78	8.50	5.48	47													
E1-ex			57	0.50	21.72	28.50	4.98	142													
E2-ex			30	0.55	16.78	16.23	5.63	91													
E3-ex			72.50	0.44	22.00	31.90	4.95	157.8													

Standard Form SF-1. Time of Concentration-Proposed

CORE ENGINEERING GROUP

Calculated By: Leonard Beasley
Date: February 28, 2019
Checked By: Leonard Beasley

Job No: 100.048
Project: Lorson Ranch East 4 Final Drainage

Sub-Basin Data				Initial Overland Time (t_i)				Travel Time (t_t)					t_c Check (urbanized Basins)		Final t_c
BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc tc=(L/180)+10 minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
C15.1	0.30	7.10	15.0	100.00	4.50%	0.19	8.79	747.0	3.41%	1.41	8.83				
			15.0					600.0	1.92%	2.08	4.81	22.43	1447.00	18.04	18.04
C15.2	0.42	4.63	15.0	100.00	6.20%	0.25	6.72	604.0	1.97%	2.11	4.78	11.51	704.00	13.91	11.51
C15.3	0.49	3.60	15.0	100.00	2.05%	0.19	8.69	161.0	3.35%	1.41	1.90				
			20.0					658.0	2.87%	3.39	3.24	13.83	919.00	15.11	13.83
C15.4	0.49	1.25	15.0	91.00	7.14%	0.28	5.49	100.0	2.60%	1.41	1.18				
			20.0					406.0	2.02%	2.84	2.38	9.05	597.00	13.32	9.05
C15.5	0.49	2.90	20.0	35.00	2.00%	0.11	5.18	979.0	3.04%	3.49	4.68	9.86	1014.00	15.63	9.86
C15.6	0.49	1.80	15.0	59.00	1.36%	0.13	7.64	100.0	2.00%	2.12	0.79				
			20.0					731.0	1.87%	2.73	4.45	12.88	890.00	14.94	12.88
C15.7	0.49	2.07	20.0	39.00	2.05%	0.12	5.43	966.0	1.63%	2.55	6.31	11.73	1005.00	15.58	11.73
C15.8	0.40	3.76	15.0	100.00	7.00%	0.25	6.65	89.0	11.35%	5.05	0.29				
			15.0					463.0	0.60%	1.16	6.64				
			20.0					240.0	1.08%	2.08	1.92	15.51	892.00	14.96	15.51
C15.9	0.49	2.27	15.0	53.00	1.20%	0.12	7.55	96.0	3.02%	2.61	0.61				
			20.0					8.6	1.61%	2.54	0.06	8.22	157.55	10.88	8.22
C15.10	0.49	0.60	15.0	100.00	2.20%	0.20	8.49	37.0	2.20%	2.22	0.28				
			20.0					160.0	1.51%	2.46	1.09	9.85	297.00	11.65	9.85
C15.11	0.49	3.20	20.0	74.00	4.19%	0.21	5.90	1105.0	2.63%	3.24	5.68	11.58	1179.00	16.55	11.58
C15.12	0.49	0.61	15.0	100.00	2.16%	0.20	8.54	34.0	2.16%	2.20	0.26				
			20.0					321.0	1.00%	2.00	2.68	11.47	455.00	12.53	11.47
C15.13	0.49	2.35	20.0	52.00	2.12%	0.14	6.20	967.0	2.32%	3.05	5.29	11.49	1019.00	15.66	11.49
C15.14	0.49	1.32	20.0	33.00	1.82%	0.11	5.19	595.0	2.89%	3.40	2.92	8.11	628.00	13.49	8.11
D2.1	0.49	3.14	15.0	100.00	2.32%	0.20	8.34	90.0	2.32%	2.28	0.66				
			20.0					897.0	1.62%	2.55	5.87	14.87	1087.00	16.04	14.87
D2.2	0.49	1.11	15.0	100.00	1.70%	0.18	9.24	167.0	3.47%	2.79	1.00				
			20.0					218.0	1.15%	2.14	1.69	11.93	485.00	12.69	11.93
D2.3	0.27	2.80	15.0	100.00	2.10%	0.14	11.73	344.0	4.77%	3.28	1.75				
			20.0					292.0	3.20%	3.58	1.36	14.84	736.00	14.09	14.09
D2.4	0.29	3.33	15.0	100.00	4.50%	0.19	8.90	386.0	6.30%	3.76	1.71				
			20.0					487.0	2.00%	2.83	2.87	13.48	973.00	15.41	13.48
D2.5	0.49	3.93	15.0	61.00	14.75%	0.29	3.54	219.0	2.19%	2.22	1.64				
			20.0					447.0	2.82%	3.36	2.22	7.40	727.00	14.04	7.40

CORE ENGINEERING GROUP

Standard Form SF-1. Time of Concentration-Proposed

Calculated By: Leonard Beasley — Job No: 100.048

Date: February 28, 2019 — Project: Lorson Ranch East 4 Final Drainage

Checked By: Leonard Beasley

Sub-Basin Data				Initial Overland Time (t_i)				Travel Time (t_t)					t_c Check (urbanized Basins)		Final t_c
BASIN or DESIGN	C_5	AREA (A) acres	NRCS Convey.	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_i minutes	LENGTH (L) feet	SLOPE (S) %	VELOCITY (V) ft/sec	t_t minutes	Computed t_c Minutes	TOTAL LENGTH (L) feet	Regional tc $t_c=(L/180)+10$ minutes	USDCM Recommended $t_c=t_i+t_t$ (min)
D2.6	0.49	2.13	15.0	100.00	3.00%	0.22	7.66	20.0	2.50%	2.37	0.14				
			20.0					528.0	2.94%	3.43	2.57	10.37	648.00	13.60	10.37
D2.7	0.49	2.98	20.0	25.00	2.00%	0.10	4.38	631.0	3.44%	3.71	2.84	7.22	656.00	13.64	7.22
D2.8	0.49	3.70	15.0	35.00	15.71%	0.22	2.63	162.0	2.34%	2.29	1.18				
			20.0					665.0	1.04%	2.04	5.43	9.24	862.00	14.79	9.24
D2.9	0.49	3.15	20.0	75.00	1.87%	0.16	7.76	1342.0	2.50%	3.16	7.07	14.83	1417.00	17.87	14.83
D2.10	0.49	0.80	20.0	17.00	2.00%	0.08	3.61	392.0	1.54%	2.48	2.63	6.24	409.00	12.27	6.24
D2.10a	0.49	1.10	20.0	100.00	1.00%	0.15	11.01	200.0	1.90%	2.76	1.21	12.22	300.00	11.67	12.22
D2.11	0.90	0.40	20.0	10.00	2.00%	0.18	0.91	278.0	0.70%	1.67	2.77	3.68	288.00	11.60	3.68
D2.12	0.49	2.78	20.0	100.00	5.20%	0.26	6.39	1009.0	2.97%	3.45	4.88	11.27	1109.00	16.16	11.27
D2.13	0.49	2.51	20.0	20.00	2.00%	0.09	3.92	2334.0	2.00%	2.83	13.75	17.67	2354.00	23.08	17.67
E1.1	0.49	1.41	15.0	92.00	9.24%	0.30	5.07	145.0	2.75%	2.49	0.97				
			20.0					296.0	3.31%	3.64	1.36	7.40	533.00	12.96	7.40
E1.2	0.49	3.61	15.0	100.00	6.60%	0.28	5.91	203.0	5.22%	3.43	0.99				
			20.0					563.0	2.01%	2.84	3.31	10.20	866.00	14.81	10.20
E1.3	0.20	6.81	15.0	100.00	4.80%	0.17	9.68	763.0	5.22%	3.43	3.71				
			20.0					415.0	2.24%	2.99	2.31	15.70	1278.00	17.10	15.70
E1.4	0.49	1.10	15.0	100.00	2.00%	0.19	8.76	20.0	2.00%	2.12	0.16				
			20.0					165.0	1.87%	2.73	1.01	9.92	285.00	11.58	9.92
E1.5	0.49	1.95	20.0	30.00	2.00%	0.10	4.80	729.0	2.24%	2.99	4.06	8.86	759.00	14.22	8.86
E1.6	0.49	2.32	20.0	100.00	5.12%	0.26	6.42	566.0	1.09%	2.09	4.52	10.94	666.00	13.70	10.94
E1.7	0.38	4.00	15.0	100.00	4.50%	0.21	7.91	155.0	7.95%	4.23	0.61				
			20.0					769.0	1.07%	2.07	6.20	14.72	1024.00	15.69	14.72
C15-ex	0.15	55	7.0	300.00	3.83%	0.26	19.07	1970.0	2.61%	1.13	29.03	48.11	2270.00	22.61	22.61
D1-ex	0.15	17	7.0	300.00	2.67%	0.23	21.48	1100.0	4.55%	1.49	12.28	33.76	1400.00	17.78	17.78
E1-ex	0.15	57	7.0	300.00	4.67%	0.28	17.87	1810.0	3.73%	1.35	22.31	40.18	2110.00	21.72	21.72
E2-ex	0.26	29.5	15.0	100.00	2.70%	0.15	10.93	200.00	2.70%	2.46	1.35				
			20.0					920.00	1.70%	2.61	5.88	18.16	1220.00	16.78	16.78
E3-ex	0.20	72.5	15.0	220.00	3.00%	0.22	16.77	200.00	2.00%	2.12	1.57				
			20.0					1800.00	1.70%	2.61	11.50	29.85	2220.00	22.33	22.33

APPENDIX C – HYDRAULIC CALCULATIONS

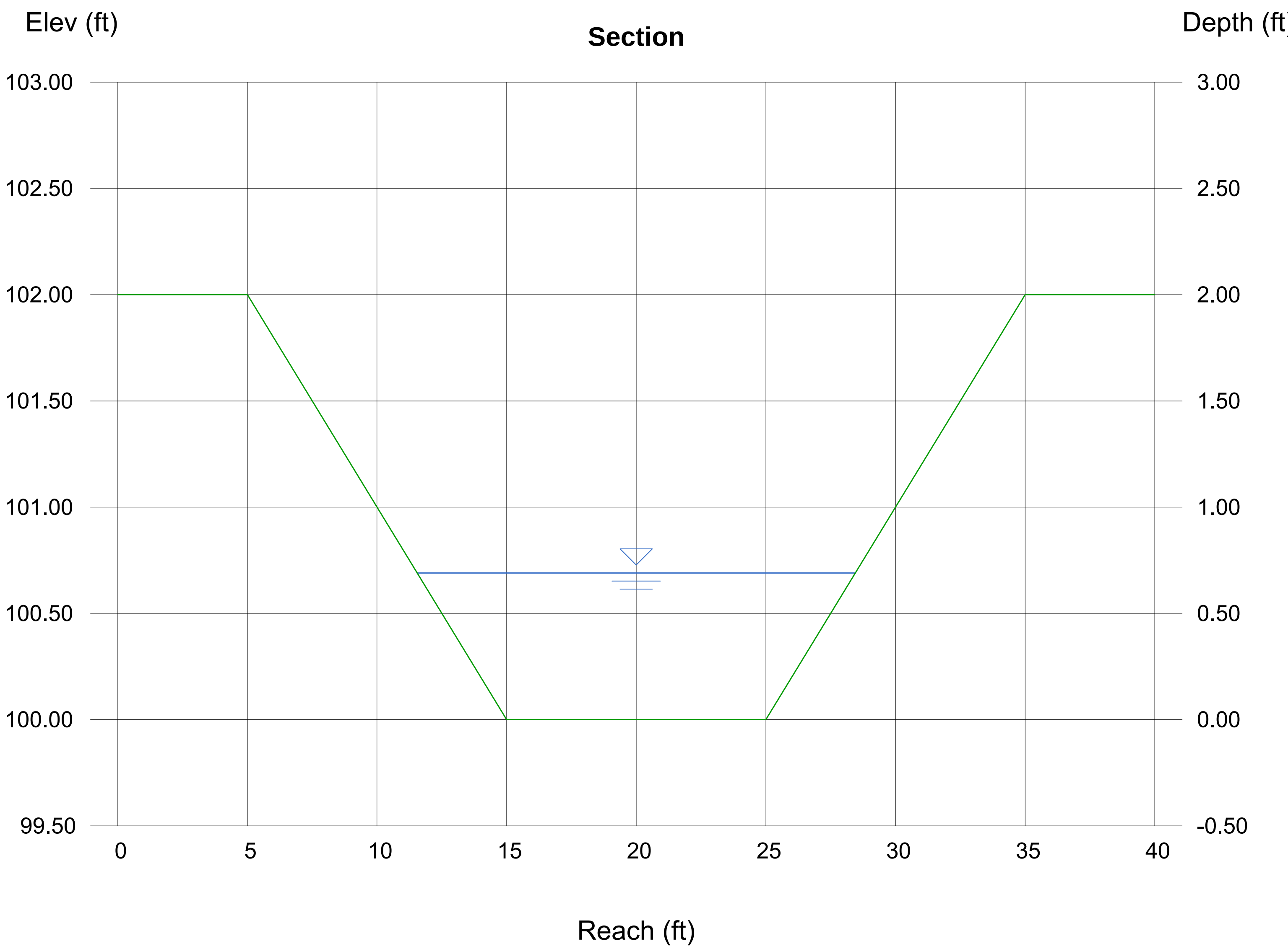
Channel Report

Overflow Swale at DP-21

Trapezoidal	
Botom Width (ft)	= 10.00
Side Slope (z:1)	= 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.00
N-Value	= 0.025

Calculations	
Compute by:	Known Q
Known Q (cfs)	= 36.00

Highlighted	
Depth (ft)	= 0.69
Q (cfs)	= 36.00
Area (sqft)	= 9.28
Velocity (ft/s)	= 3.88
Wetted Perim (ft)	= 17.04
Crit Depth, Yc (ft)	= 0.66
Top Width (ft)	= 16.90
EGL (ft)	= 0.92



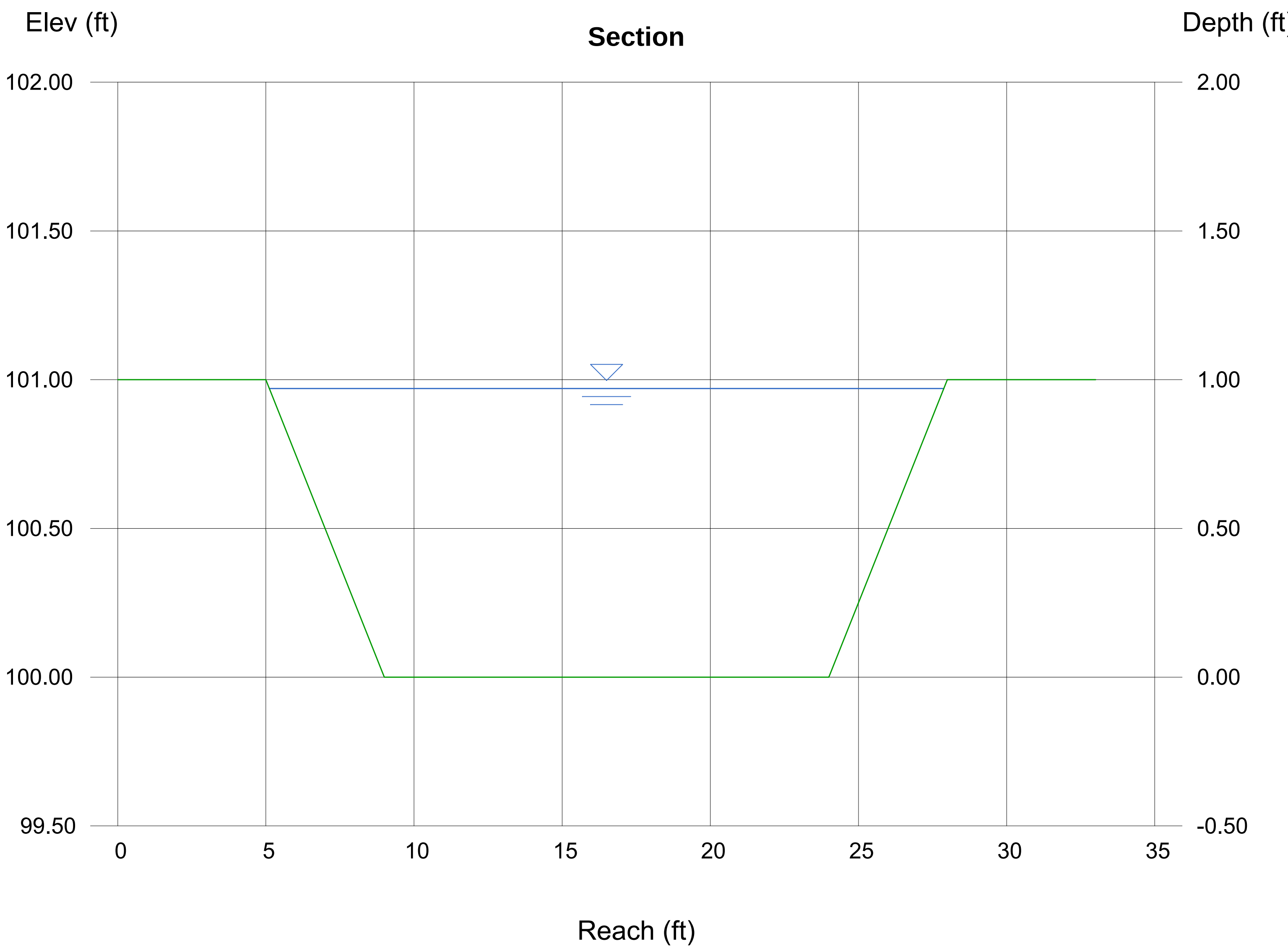
Channel Report

Overflow Swale-DP-27

Trapezoidal
Botom Width (ft) = 15.00
Side Slope (z:1) = 4.00
Total Depth (ft) = 1.00
Invert Elev (ft) = 100.00
Slope (%) = 1.00
N-Value = 0.025

Calculations
Compute by: Known Q
Known Q (cfs) = 92.60

Highlighted
Depth (ft) = 0.97
Q (cfs) = 92.60
Area (sqft) = 18.31
Velocity (ft/s) = 5.06
Wetted Perim (ft) = 23.00
Crit Depth, Yc (ft) = 0.97
Top Width (ft) = 22.76
EGL (ft) = 1.37



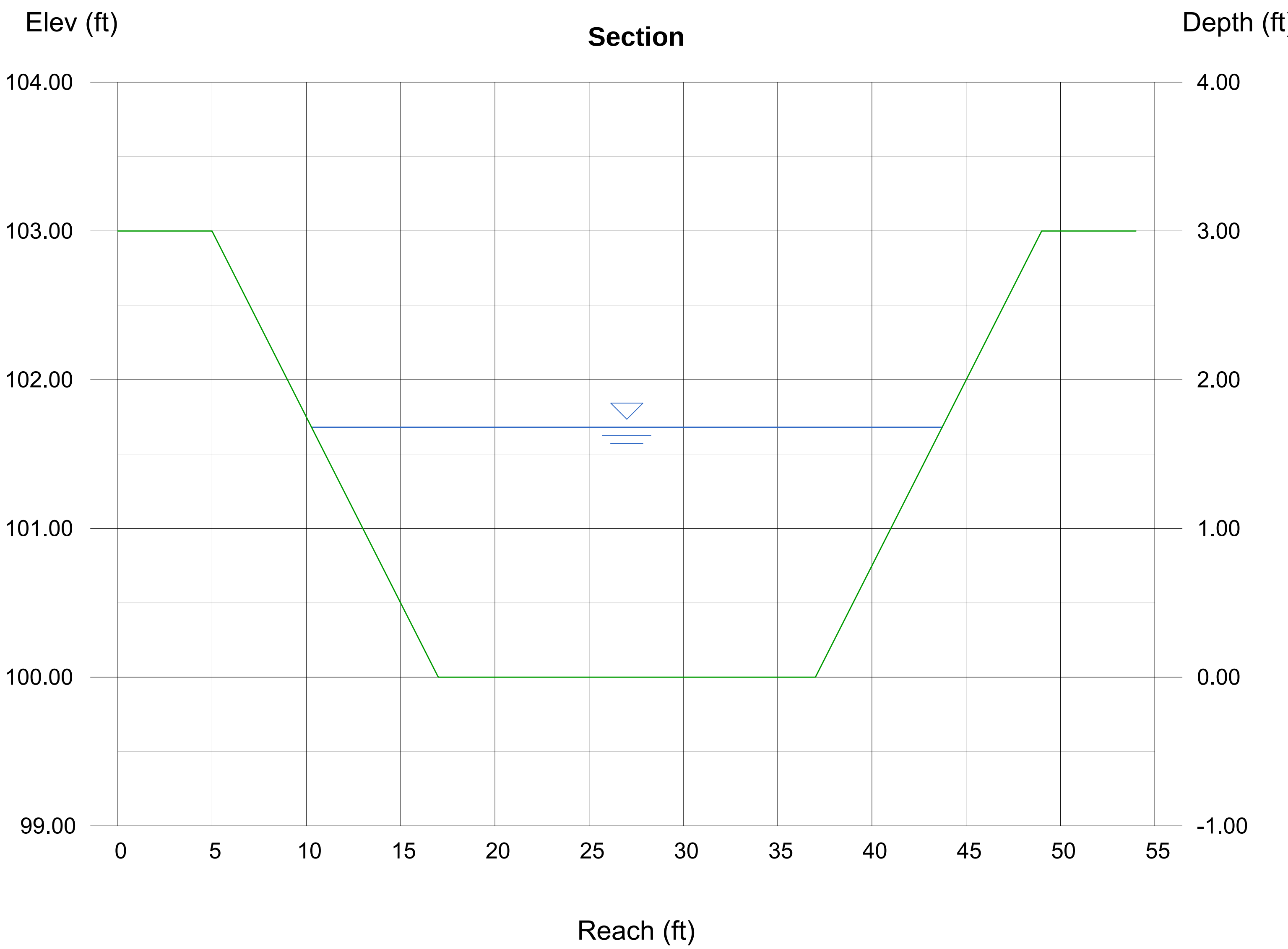
Channel Report

Pond C1 Overflow-20'btm

Trapezoidal	
Botom Width (ft)	= 20.00
Side Slope (z:1)	= 4.00
Total Depth (ft)	= 3.00
Invert Elev (ft)	= 100.00
Slope (%)	= 0.30
N-Value	= 0.025

Calculations	
Compute by:	Known Q
Known Q (cfs)	= 175.00

Highlighted	
Depth (ft)	= 1.68
Q (cfs)	= 175.00
Area (sqft)	= 44.89
Velocity (ft/s)	= 3.90
Wetted Perim (ft)	= 33.85
Crit Depth, Yc (ft)	= 1.23
Top Width (ft)	= 33.44
EGL (ft)	= 1.92



Channel Report

Hydraflow Express by Intelisolve — Wednesday, Feb 13 2019, 2:23 PM

Pond C1 Overflow-50'btm

Trapezoidal

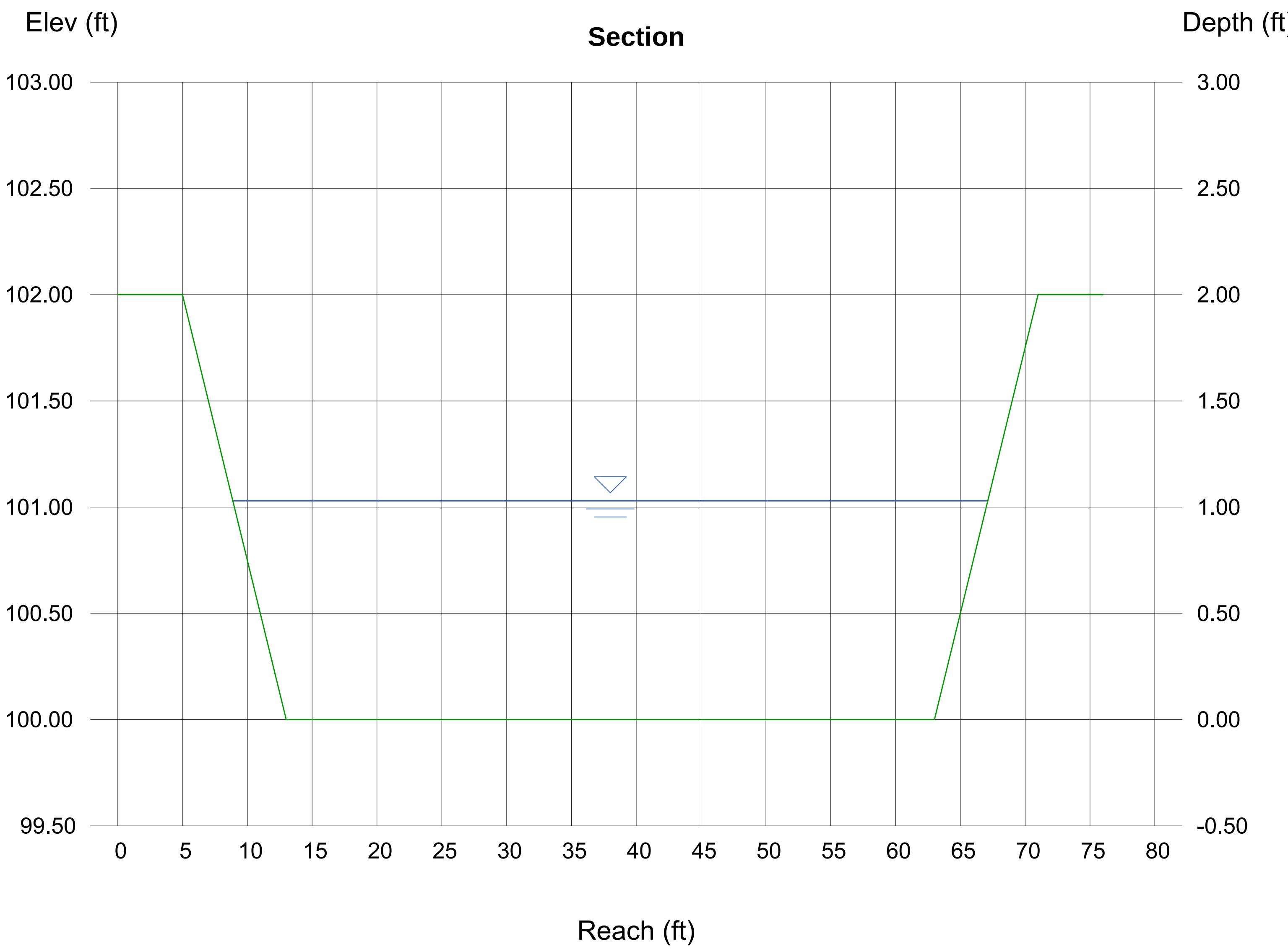
Botom Width (ft)	=	50.00
Side Slope (z:1)	=	4.00
Total Depth (ft)	=	2.00
Invert Elev (ft)	=	100.00
Slope (%)	=	0.30
N-Value	=	0.025

Calculations

Compute by:		Known Q
Known Q (cfs)	=	175.00

Highlighted

Depth (ft)	=	1.03
Q (cfs)	=	175.00
Area (sqft)	=	55.74
Velocity (ft/s)	=	3.14
Wetted Perim (ft)	=	58.49
Crit Depth, Yc (ft)	=	0.72
Top Width (ft)	=	58.24
EGL (ft)	=	1.18

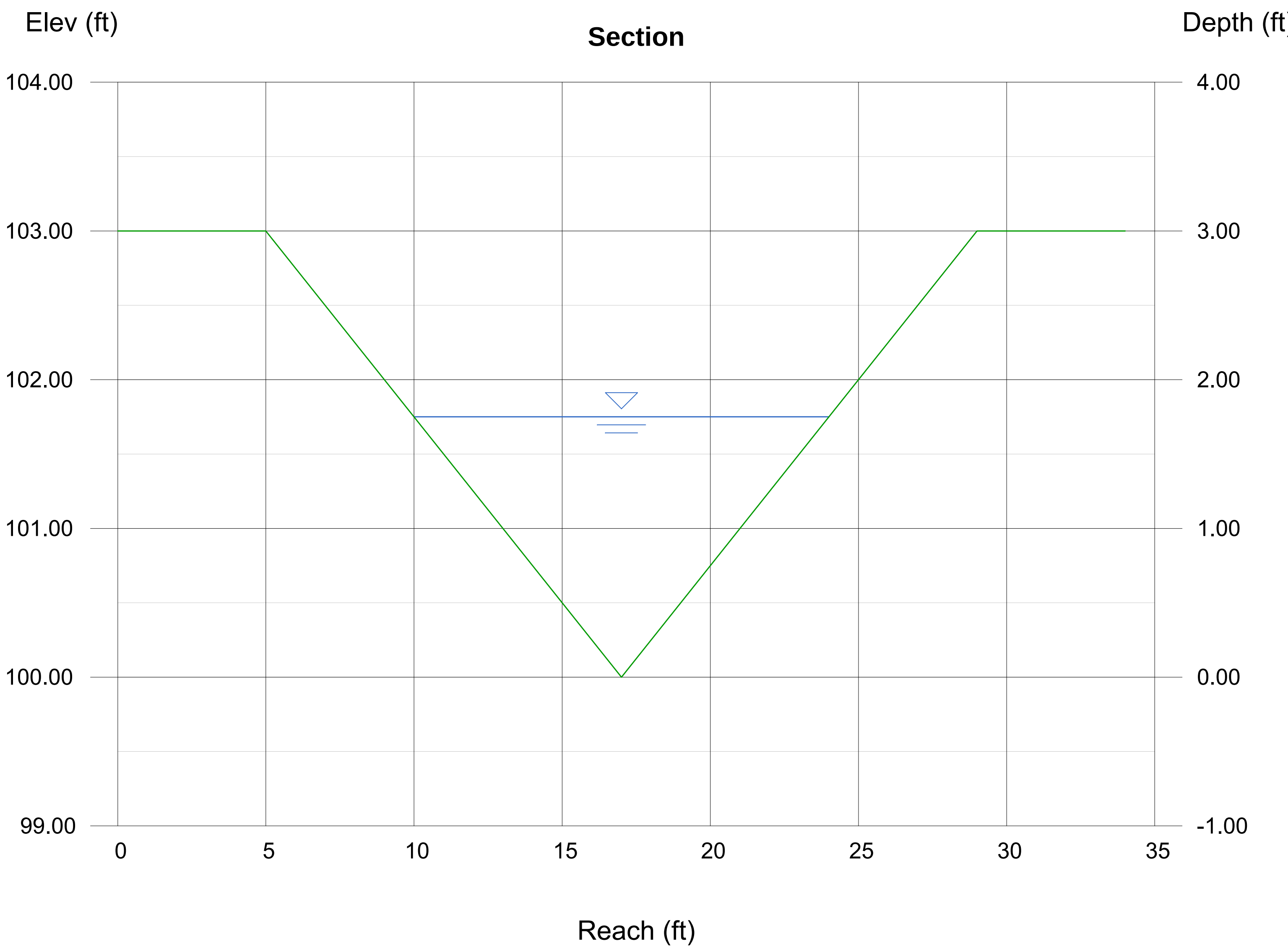


Channel Report

Hydraflow Express by Intelisolve — Wednesday, Feb 13 2019, 9:37 PM

Des. Pt 67a Swale

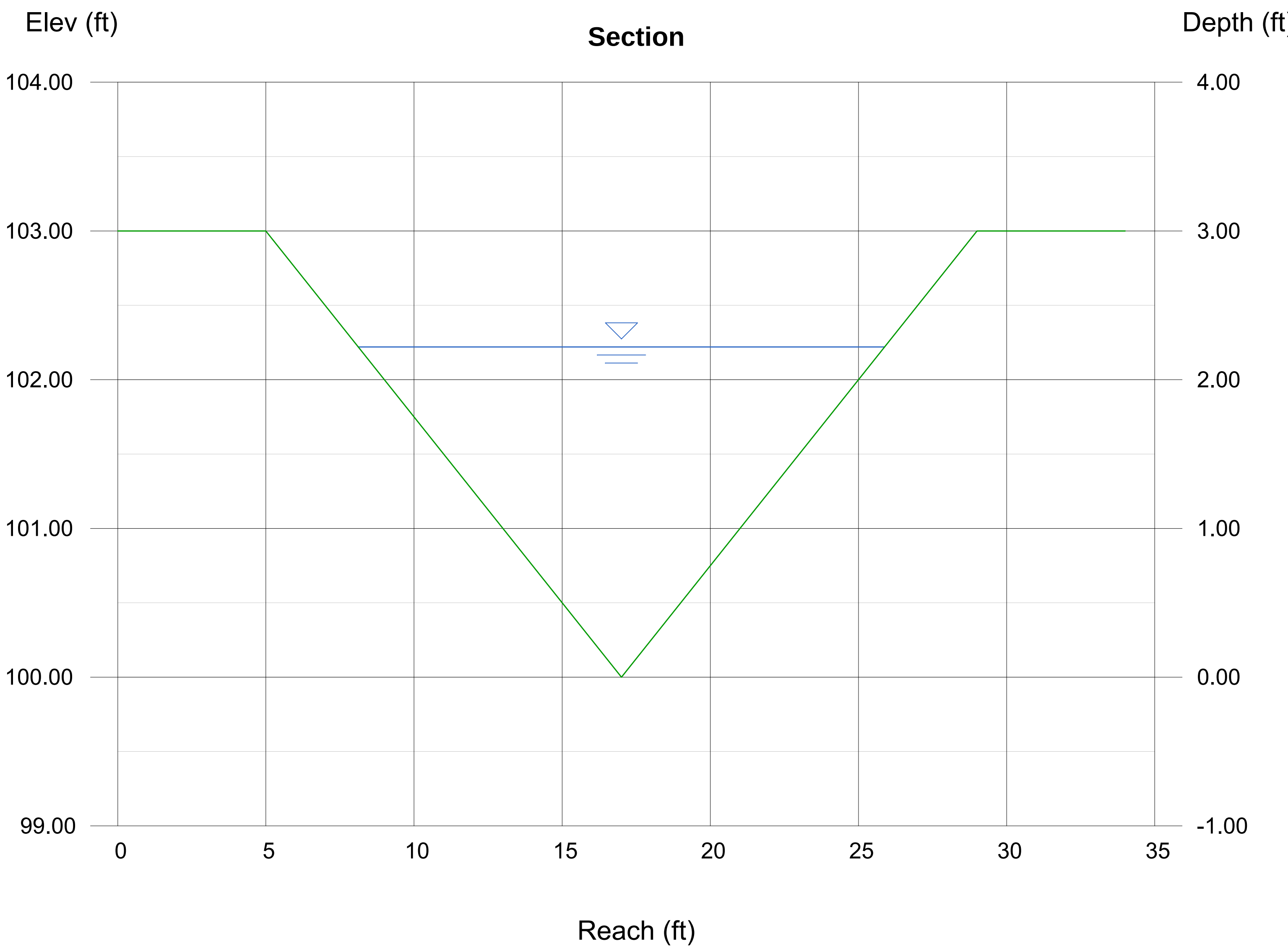
Triangular		**Highlighted**	
Side Slope (z:1)	= 4.00	Depth (ft)	= 1.75
Total Depth (ft)	= 3.00	Q (cfs)	= 91.00
		Area (sqft)	= 12.25
Invert Elev (ft)	= 100.00	Velocity (ft/s)	= 7.43
Slope (%)	= 2.00	Wetted Perim (ft)	= 14.43
N-Value	= 0.025	Crit Depth, Yc (ft)	= 2.01
		Top Width (ft)	= 14.00
Calculations		EGL (ft)	= 2.61
Compute by:	Known Q		
Known Q (cfs)	= 91.00		



Channel Report

Des. Pt 67b Swale

Triangular		**Highlighted**	
Side Slope (z:1)	= 4.00	Depth (ft)	= 2.22
Total Depth (ft)	= 3.00	Q (cfs)	= 210.00
		Area (sqft)	= 19.71
Invert Elev (ft)	= 100.00	Velocity (ft/s)	= 10.65
Slope (%)	= 2.96	Wetted Perim (ft)	= 18.31
N-Value	= 0.025	Crit Depth, Yc (ft)	= 2.80
		Top Width (ft)	= 17.76
Calculations		EGL (ft)	= 3.98
Compute by:	Known Q		
Known Q (cfs)	= 210.00		



Culvert Report

Des. Pt 67b(end section)

Invert Elev Dn (ft)	=	5714.37
Pipe Length (ft)	=	58.00
Slope (%)	=	4.50
Invert Elev Up (ft)	=	5716.98
Rise (in)	=	30.0
Shape	=	Cir
Span (in)	=	30.0
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

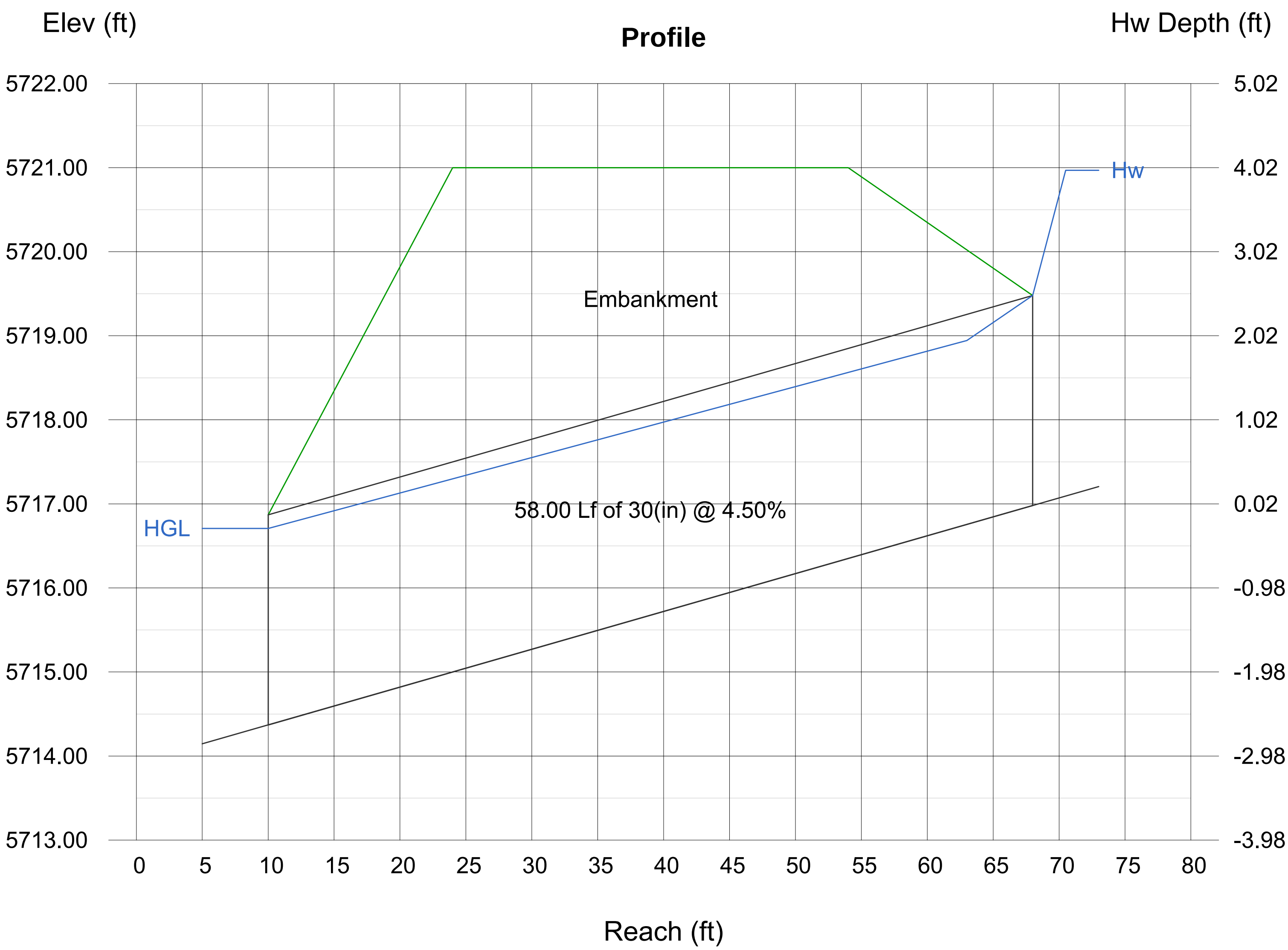
Top Elevation (ft)	=	5721.00
Top Width (ft)	=	30.00
Crest Width (ft)	=	100.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	42.00
Tailwater Elev (ft)	=	(dc+D)/2

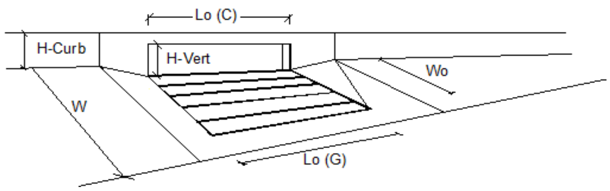
Highlighted

Qtotal (cfs)	=	42.00
Qpipe (cfs)	=	42.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	8.80
Veloc Up (ft/s)	=	9.26
HGL Dn (ft)	=	5716.71
HGL Up (ft)	=	5719.16
Hw Elev (ft)	=	5720.97
Hw/D (ft)	=	1.60
Flow Regime	=	Inlet Control



INLET ON A CONTINUOUS GRADE

Project: Lorson East 4 FDR
Inlet ID: Inlet DP-23 (Basins C15.3+C15.4)

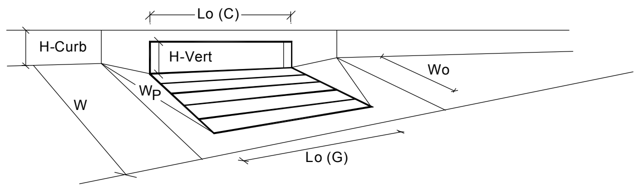


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	15.00	15.00	ft
Width of a Unit Grate (cannot be greater than W from Q-Allow)	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 < maximum allowable from sheet 'Q-Allow'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Q-Peak*)	Q_o =	8.7	19.3	cfs
Water Spread Width	T =	16.0	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	5.4	6.8	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	1.2	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.373	0.273	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	5.5	13.9	cfs
Discharge within the Gutter Section W	Q_w =	3.2	5.2	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.1	cfs
Flow Area within the Gutter Section W	A_W =	2.68	4.74	sq ft
Velocity within the Gutter Section W	V_W =	3.2	4.0	fps
Water Depth for Design Condition	d_{LOCAL} =	8.4	9.8	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	0.071	ft/ft
Required Length L_T to Have 100% Interception	L_T =	16.94	28.21	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	15.00	15.00	ft
Interception Capacity	Q_i =	8.5	14.3	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.31	1.31	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.04	0.04	
Effective (Unclogged) Length	L_e =	13.03	13.03	ft
Actual Interception Capacity	Q_a =	8.4	13.9	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	0.3	5.4	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	**8.43**	**13.93**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q_b =	**0.3**	**5.4**	**cfs**
Capture Percentage = Q_a/Q_o =	C% =	**97**	**72**	**%**

INLET IN A SUMP OR SAG LOCATION

Project = Lorson East 4 FDR

Inlet ID = Inlet DP-25 (Basin C15.5+C15.6+C15.11+C12+bypass from Inlet DP-23)

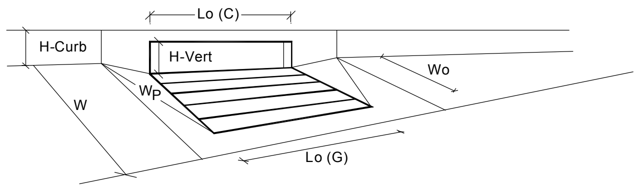


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
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	
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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	19.14	32.57	cfs
Interception with Clogging	Q_{wa} =	18.63	31.70	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	50.55	55.95	cfs
Interception with Clogging	Q_{oa} =	49.20	54.47	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	28.92	39.70	cfs
Interception with Clogging	Q_{ma} =	28.16	38.64	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**18.63**	**31.70**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	25.00	25.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	39.3	52.1	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	2.7	4.2	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**18.6**	**31.7**	**cfs**
WARNING: Inlet Capacity less than Q Peak for MAJOR Storm	$Q_{PEAK\ REQUIRED}$ =	15.9	38.8	cfs

INLET IN A SUMP OR SAG LOCATION

Project = Lorson East 4 FDR

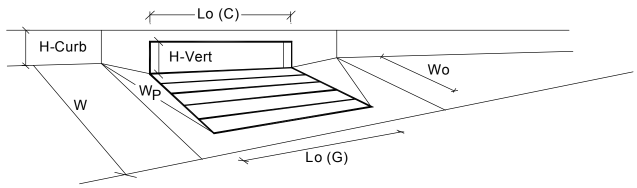
Inlet ID = Inlet DP-26 (Basin C15.7+C15.13+bypass from Inlet DP-25)



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	15.79	26.87	cfs
Interception with Clogging	Q_{wa} =	15.27	25.98	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	40.44	44.76	cfs
Interception with Clogging	Q_{oa} =	39.09	43.28	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	23.50	32.26	cfs
Interception with Clogging	Q_{ma} =	22.72	31.18	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**15.27**	**25.98**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	20.00	20.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	39.3	52.1	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	2.7	4.2	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**15.3**	**26.0**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	8.4	26.0	cfs

INLEET IN A SUMP OR SAG LOCATION

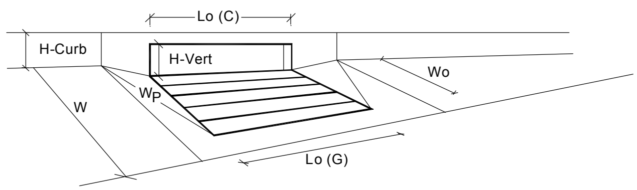
Project = Lorson East Prelim Plan #100.040
Inlet ID = Inlet DP-59a (Basin D2.2)



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	7.06	10.97	cfs
Interception with Clogging	Q_{wa} =	6.35	9.87	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	10.11	11.19	cfs
Interception with Clogging	Q_{oa} =	9.10	10.07	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	7.86	10.30	cfs
Interception with Clogging	Q_{ma} =	7.07	9.27	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**6.35**	**9.27**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**6.4**	**9.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.2	4.8	cfs

INLET IN A SUMP OR SAG LOCATION

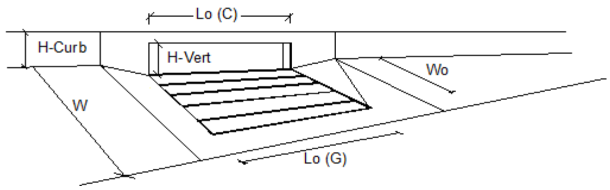
Project = **Lorson East Prelim Plan #100.040**
Inlet ID = **Inlet DP-59d (Basin D2.1+D2.3)**



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	15.00	15.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.31	1.31	
Clogging Factor for Multiple Units	Clog =	0.04	0.04	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	12.45	21.18	cfs
Interception with Clogging	Q_{wa} =	11.90	20.25	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	30.33	33.57	cfs
Interception with Clogging	Q_{oa} =	29.00	32.11	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	18.07	24.80	cfs
Interception with Clogging	Q_{ma} =	17.28	23.72	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**11.90**	**20.25**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	15.00	15.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**11.9**	**20.3**	**cfs**
WARNING: Inlet Capacity less than Q Peak for MAJOR Storm	$Q_{PEAK\ REQUIRED}$ =	10.7	23.7	cfs

INLET ON A CONTINUOUS GRADE

Project: **Lorson East Prelim Plan #100.040**
Inlet ID: **Inlet DP-59f (Basins D2.4+D2.5)**

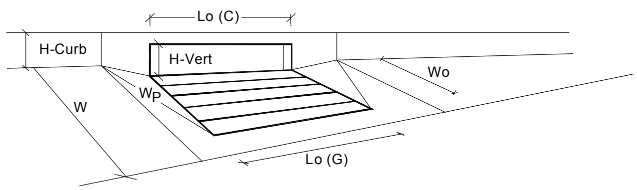


Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	10.00	10.00	ft
Width of a Unit Grate (cannot be greater than W from Q-Allow)	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 < maximum allowable from sheet 'Q-Allow'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Q-Peak*)	Q_o =	13.6	30.5	cfs
Water Spread Width	T =	15.8	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	5.3	6.8	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	1.2	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.378	0.275	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	8.5	22.0	cfs
Discharge within the Gutter Section W	Q_w =	5.2	8.3	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.2	cfs
Flow Area within the Gutter Section W	A_W =	2.61	4.66	sq ft
Velocity within the Gutter Section W	V_W =	5.2	6.5	fps
Water Depth for Design Condition	d_{LOCAL} =	8.3	9.8	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 =	0.091	0.072	ft/ft
Required Length L_T to Have 100% Interception	L_T =	22.42	37.58	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 =	8.9	12.9	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 =	8.6	12.4	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	5.1	18.1	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	**8.58**	**12.37**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q_b =	**5.1**	**18.1**	**cfs**
Capture Percentage = Q_a/Q_o =	C% =	**63**	**41**	**%**

INLET IN A SUMP OR SAG LOCATION

Project = **Lorson East Prelim Plan #100.040**

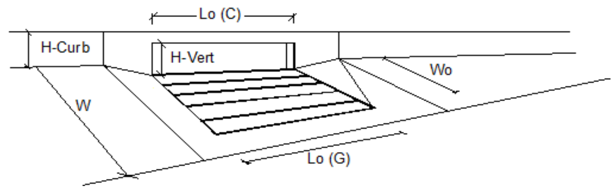
Inlet ID = **Inlet DP-60 (Basin D2.9+D2.12+bypass from Inlet DP59f)+bypass from Inlet DP62 in 100year**



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				Override Depths
Grate Information		MINOR	MAJOR	
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	
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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	19.14	32.57	cfs
Interception with Clogging	Q_{wa} =	18.63	31.70	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	50.55	55.95	cfs
Interception with Clogging	Q_{oa} =	49.20	54.47	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	28.92	39.70	cfs
Interception with Clogging	Q_{ma} =	28.16	38.64	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**18.63**	**31.70**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	25.00	25.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**18.6**	**31.7**	**cfs**
WARNING: Inlet Capacity less than Q Peak for MAJOR Storm	$Q_{PEAK\ REQUIRED}$ =	15.8	55.9	cfs

INLET ON A CONTINUOUS GRADE

Project: Lorson East 4 FDR
Inlet ID: Inlet DP-61 (Basins D2.6+D2.7)

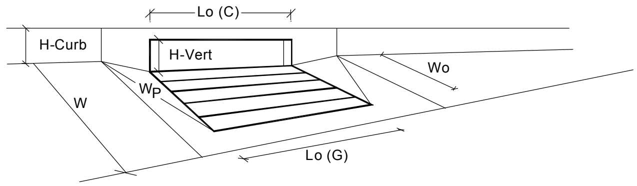


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	10.00	10.00	ft
Width of a Unit Grate (cannot be greater than W from Q-Allow)	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 < maximum allowable from sheet 'Q-Allow'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Q-Peak*)	Q_o =	10.2	22.7	cfs
Water Spread Width	T =	13.8	17.0	ft
Water Depth at Flowline (outside of local depression)	d =	4.8	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 =	0.431	0.312	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	5.8	15.6	cfs
Discharge within the Gutter Section W	Q_w =	4.4	7.1	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	2.03	3.74	sq ft
Velocity within the Gutter Section W	V_W =	5.0	6.1	fps
Water Depth for Design Condition	d_{LOCAL} =	7.8	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 =	0.101	0.079	ft/ft
Required Length L_T to Have 100% Interception	L_T =	18.50	31.23	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 =	7.7	11.4	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 =	7.4	10.9	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	2.8	11.8	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**7.41**	**10.88**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**2.8**	**11.8**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**73**	**48**	**%**

INLET IN A SUMP OR SAG LOCATION

Project = **Lorson East 4 FDR**

Inlet ID = **Inlet DP-62 (Basin D2.8+bypass from Inlet DP61)**

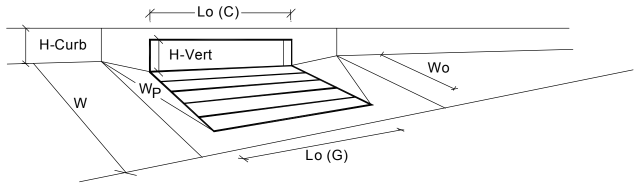


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.25	1.25	
Clogging Factor for Multiple Units	Clog =	0.06	0.06	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	10.72	17.34	cfs
Interception with Clogging	Q_{wa} =	10.05	16.26	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	20.22	22.38	cfs
Interception with Clogging	Q_{oa} =	18.96	20.98	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	13.69	18.32	cfs
Interception with Clogging	Q_{ma} =	12.84	17.18	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**10.05**	**16.26**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	10.00	10.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**10.1**	**16.3**	**cfs**
WARNING: Inlet Capacity less than Q Peak for Minor and Major Storms	$Q_{PEAK\ REQUIRED}$ =	10.5	29.0	cfs

INLET IN A SUMP OR SAG LOCATION

Project = Lorson East Prelim Plan #100.040

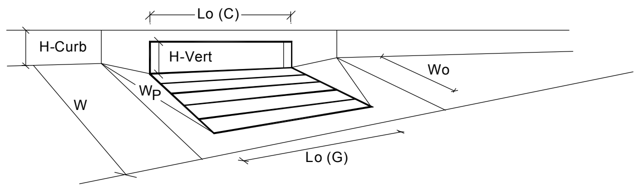
Inlet ID = Inlet DP-64 (Basin D2.10+bypass from Inlet DP60 in 100 year)



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
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	
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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	19.14	32.57	cfs
Interception with Clogging	Q_{wa} =	18.63	31.70	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	50.55	55.95	cfs
Interception with Clogging	Q_{oa} =	49.20	54.47	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	28.92	39.70	cfs
Interception with Clogging	Q_{ma} =	28.16	38.64	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**18.63**	**31.70**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	25.00	25.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**18.6**	**31.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	3.2	29.2	cfs

INLET IN A SUMP OR SAG LOCATION

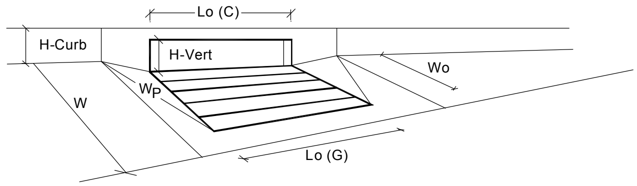
Project = **Lorson East Prelim Plan #100.040**
Inlet ID = **Inlet DP-65a (Basin D2.11)**



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
			☑ Override Depths	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	5.00	5.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	7.06	10.97	cfs
Interception with Clogging	Q_{wa} =	6.35	9.87	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	10.11	11.19	cfs
Interception with Clogging	Q_{oa} =	9.10	10.07	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	7.86	10.30	cfs
Interception with Clogging	Q_{ma} =	7.07	9.27	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**6.35**	**9.27**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**6.4**	**9.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.0	4.0	cfs

INLET IN A SUMP OR SAG LOCATION

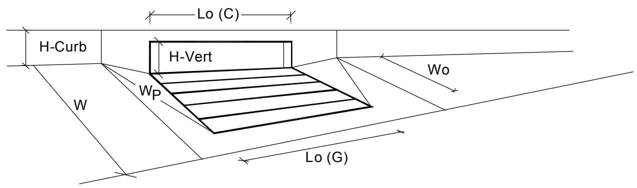
Project = **Lorson East Prelim Plan #100.040**
Inlet ID = **Inlet DP-65b (Basin D2.13)+bypass from Inlet DP59d in 100 year**



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
				Override Depths
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.25	1.25	
Clogging Factor for Multiple Units	Clog =	0.06	0.06	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	10.72	17.34	cfs
Interception with Clogging	Q_{wa} =	10.05	16.26	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	20.22	22.38	cfs
Interception with Clogging	Q_{oa} =	18.96	20.98	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	13.69	18.32	cfs
Interception with Clogging	Q_{ma} =	12.84	17.18	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**10.05**	**16.26**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	10.00	10.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**10.1**	**16.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	4.2	12.7	cfs

INLET IN A SUMP OR SAG LOCATION

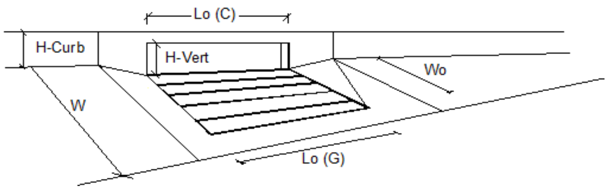
Project = Lorson East 4 FDR
Inlet ID = Inlet DP-66a (Basin E1.1)



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	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) =	5.00	5.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	7.06	10.97	cfs
Interception with Clogging	Q_{wa} =	6.35	9.87	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	10.11	11.19	cfs
Interception with Clogging	Q_{oa} =	9.10	10.07	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	7.86	10.30	cfs
Interception with Clogging	Q_{ma} =	7.07	9.27	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**6.35**	**9.27**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**6.4**	**9.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	3.2	7.0	cfs

INLET ON A CONTINUOUS GRADE

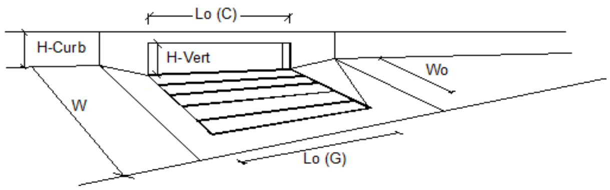
Project: Lorson East4 FDR
Inlet ID: Inlet DP-66d (Basin E1.2)



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 from Q-Allow)	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 < maximum allowable from sheet 'Q-Allow'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Q-Peak*)	Q_o =	7.3	16.1	cfs
Water Spread Width	T =	12.1	16.9	ft
Water Depth at Flowline (outside of local depression)	d =	4.4	5.6	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 =	0.487	0.353	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	3.8	10.4	cfs
Discharge within the Gutter Section W	Q_w =	3.6	5.7	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	1.60	2.97	sq ft
Velocity within the Gutter Section W	V_W =	4.6	5.4	fps
Water Depth for Design Condition	d_{LOCAL} =	7.4	8.6	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 =	0.111	0.086	ft/ft
Required Length L_T to Have 100% Interception	L_T =	14.85	25.00	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 =	6.3	9.7	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 =	6.1	9.3	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	1.2	6.8	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	**6.14**	**9.29**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q_b =	**1.2**	**6.8**	**cfs**
Capture Percentage = Q_a/Q_o =	C% =	**84**	**58**	**%**

INLET ON A CONTINUOUS GRADE

Project: Lorson East 4 FDR
Inlet ID: Inlet DP-68 (Basin E1.4+E1.5+pypass from Inlet DP66d) + bypass from end section for Basin E1.3 in 100yr storm

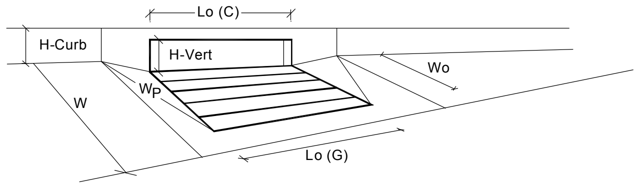


Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	15.00	15.00	ft
Width of a Unit Grate (cannot be greater than W from Q-Allow)	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 *Q-Peak*)	Q_o =	7.5	43.1	cfs
Water Spread Width	T =	14.8	20.0	ft
Water Depth at Flowline (outside of local depression)	d =	5.1	8.5	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	2.2	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.403	0.209	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	4.5	31.7	cfs
Discharge within the Gutter Section W	Q_w =	3.0	8.4	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	3.1	cfs
Flow Area within the Gutter Section W	A_W =	2.31	7.81	sq ft
Velocity within the Gutter Section W	V_W =	3.2	5.1	fps
Water Depth for Design Condition	d_{LOCAL} =	8.1	11.5	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 =	0.096	0.059	ft/ft
Required Length L_T to Have 100% Interception	L_T =	15.35	44.92	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	15.00	15.00	ft
Interception Capacity	Q_i =	7.5	20.8	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.31	1.31	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.04	0.04	
Effective (Unclogged) Length	L_e =	13.03	13.03	ft
Actual Interception Capacity	Q_a =	7.5	20.2	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	Q_b =	0.0	22.9	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**7.46**	**20.17**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**0.0**	**22.9**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**99**	**47**	**%**

INLET IN A SUMP OR SAG LOCATION

Project = **Lorson East 4 FDR**

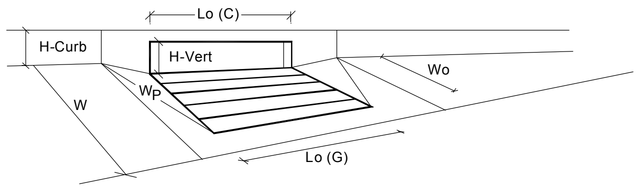
Inlet ID = **Inlet DP-69 (Basin E1.6+bypass from Inlet DP68)**



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	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 =	6.5	8.0	inches
			☑ Override Depths	
Grate Information		MINOR	MAJOR	
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) =	30.00	30.00	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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 =	0.02	0.02	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	22.48	38.26	cfs
Interception with Clogging	Q_{wa} =	21.98	37.41	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	60.66	67.15	cfs
Interception with Clogging	Q_{oa} =	59.31	65.66	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	34.34	47.14	cfs
Interception with Clogging	Q_{ma} =	33.58	46.09	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**21.98**	**37.41**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	30.00	30.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**22.0**	**37.4**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	5.7	33.0	cfs

INLET IN A SUMP OR SAG LOCATION

Project = **Lorson East 4 FDR**
Inlet ID = **Inlet DP-70 (Basin E1.7+bypass from Basin E2-ex Type D inlet)**

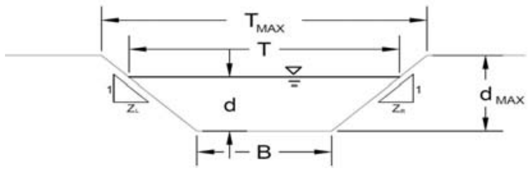


		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.5	8.0	inches
				☑ Override Depths
Grate Information		MINOR	MAJOR	
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) =	30.00	30.00	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 UDFCD - CSU 2010 Study)		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 UDFCD - CSU 2010 Study)		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 =	0.02	0.02	
Curb Opening as a Weir (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	22.48	38.26	cfs
Interception with Clogging	Q_{wa} =	21.98	37.41	cfs
Curb Opening as an Orifice (based on UDFCD - CSU 2010 Study)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	60.66	67.15	cfs
Interception with Clogging	Q_{oa} =	59.31	65.66	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	34.34	47.14	cfs
Interception with Clogging	Q_{ma} =	33.58	46.09	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**21.98**	**37.41**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	30.00	30.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	20.7	27.0	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.9	2.4	inches
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**22.0**	**37.4**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	5.4	36.3	cfs

AREA INLET IN A TRAPEZOIDAL GRASS-LINED CHANNEL

Lorson East 4 FDR

Flow from Basin E2-ex into Type D inlet



Grass Type	Limiting Manning's n
A	0.06
B	0.04
C	0.033
D	0.03
E	0.024

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

	NRCS Vegetal Retardance (A, B, C, D, or E)	A, B, C, D or E	B	
	Manning's n (Leave cell D16 blank to manually enter an n value)	n =	see details below	
	Channel Invert Slope	S_O =	0.0280	ft/ft
	Bottom Width	B =	4.00	ft
Warning 01	Left Side Slope	Z1 =	0.33	ft/ft
Warning 01	Right Side Slope	Z2 =	0.33	ft/ft

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Sandy	5.0 fps	0.50
Non-Sandy	7.0 fps	0.80

Choose One:
- Sandy
- ◉ Non-Sandy

		Minor Storm	Major Storm	
Max. Allowable Top Width of Channel for Minor & Major Storm	T_{MAX} =	18.00	18.00	feet
Max. Allowable Water Depth in Channel for Minor & Major Storm	d_{MAX} =	3.00	3.00	feet

Allowable Channel Capacity Based On Channel Geometry

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	**97.37**	**97.37**	**cfs**
MAJOR STORM Allowable Capacity is based on Depth Criterion	d_{allow} =	**3.00**	**3.00**	**ft**

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow	Q_o =	**26.00**	**91.00**	**cfs**
Water Depth	d =	**1.69**	**2.91**	**feet**

Minor storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'

Major storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'

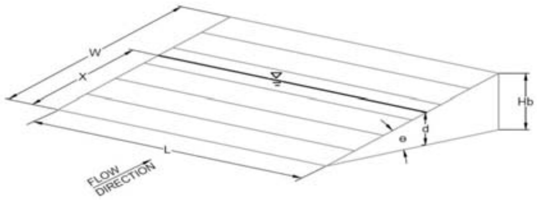
AREA INLET IN A TRAPEZOIDAL GRASS-LINED CHANNEL

Lorson East 4 FDR

Flow from Basin E2-ex into Type D inlet

Inlet Design Information (Input)

Type of Inlet	Inlet Type =	CDOT Type D (In Series & Depressed)	
Angle of Inclined Grate (must be <= 30 degrees)	θ =	0.00	degrees
Width of Grate	W =	3.00	feet
Length of Grate	L =	6.00	feet
Open Area Ratio	A_{RATIO} =	0.70	
Height of Inclined Grate	H_B =	0.00	feet
Clogging Factor	C_f =	0.38	
Grate Discharge Coefficient	C_d =	0.72	
Orifice Coefficient	C_o =	0.48	
Weir Coefficient	C_w =	1.53	



		MINOR	MAJOR	
Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)	d =	2.69	3.91	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**49.43**	**59.59**	**cfs**
WARNING: Inlet Capacity is less than Q Peak for Major Storm	**Bypassed Flow, Q_b =**	**0.00**	**31.41**	**cfs**
	Capture Percentage = Q_a/Q_o = C%	**100**	**65**	**%**

Warning 01: Sideslope steepness exceeds USDCM Volume I recommendation.
Warning 02: Depth (d) exceeds USDCM Volume I recommendation.

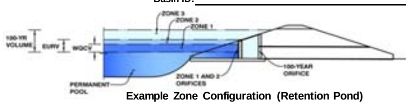
APPENDIX D – POND AND ROUTING CALCULATIONS

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Lorson Ranch East 4 FDR

Basin ID: Interim Pond E2



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	21.20	acres		
Watershed Length =	1,500	ft		
Watershed Slope =	0.035	ft/ft		
Watershed Imperviousness =	50.00%	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) =	0.364	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	1.055	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.932	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	1.339	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	1.755	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	2.417	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	2.900	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	3.515	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	0.873	acre-feet		
Approximate 5-yr Detention Volume =	1.260	acre-feet		
Approximate 10-yr Detention Volume =	1.523	acre-feet		
Approximate 25-yr Detention Volume =	1.656	acre-feet		
Approximate 50-yr Detention Volume =	1.722	acre-feet		
Approximate 100-yr Detention Volume =	1.944	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.364	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.691	acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) =	1.071	acre-feet
Total Detention Basin Volume =	2.126	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	--	--	--	100	0.002	19	0.000
5694	--	1.00	--	--	--	1,171	0.027	434	0.010
5695	--	2.00	--	--	--	14,395	0.330	8,085	0.186
5696	--	3.00	--	--	--	16,191	0.372	23,521	0.540
5697	--	4.00	--	--	--	18,055	0.414	40,644	0.933
5698	--	5.00	--	--	--	19,986	0.459	59,665	1.370
5699	--	6.00	--	--	--	21,986	0.505	80,651	1.851
5700	--	7.00	--	--	--	23,000	0.528	103,144	2.368
5701	--	8.00	--	--	--	25,000	0.574	127,144	2.919
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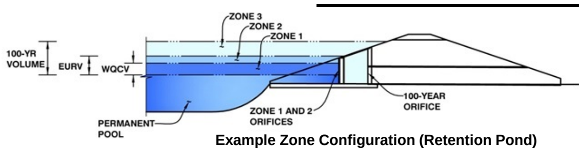
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Lorson Ranch East 4 FDR

Basin ID: Interim Pond E2



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.52	0.364	Orifice Plate
Zone 2 (EURV)	4.29	0.691	Rectangular Orifice
(100+1/2WQCV)	6.54	1.071	Weir&Pipe (Restrict)
		2.126	Total

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

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

Calculated Parameters for Underdrain

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

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

Invert of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	2.52	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	10.00	inches
Orifice Plate: Orifice Area per Row =	1.21	sq. inches (diameter = 1-1/4 inches)

Calculated Parameters for Plate

WQ Orifice Area per Row =	8.403E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	0.80	1.60	2.40				
Orifice Area (sq. inches)	1.21	1.21	1.21	1.21				

	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 =	4.29	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Height =	2.00	N/A	inches
Vertical Orifice Width =	10.73		inches

Calculated Parameters for Vertical Orifice

	Zone 2 Rectangular	Not Selected	
Vertical Orifice Area =	0.15	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 =	5.60	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	6.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	52%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	5.60	N/A	feet
Over Flow Weir Slope Length =	4.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	1.34	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	16.80	N/A	ft^2
Overflow Grate Open Area w/ Debris =	8.01	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=	7.00	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	20.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =		feet

Calculated Parameters for Spillway

Spillway Design Flow Depth=	1.13	feet
Stage at Top of Freeboard =	8.13	feet
Basin Area at Top of Freeboard =	0.57	acres

Routed Hydrograph Results

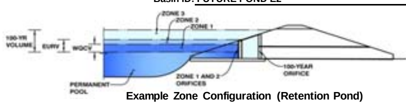
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.364	1.055	0.932	1.339	1.755	2.417	2.900	3.515	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.364	1.055	0.931	1.338	1.754	2.416	2.900	3.514	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.08	0.29	0.76	1.01	1.34	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	0.3	1.8	6.2	16.0	21.5	28.4	0.0
Peak Inflow Q (cfs) =	6.2	17.8	15.8	22.5	29.4	40.4	48.3	58.4	#N/A
Peak Outflow Q (cfs) =	0.2	0.8	0.6	1.0	1.3	12.3	21.0	31.2	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.6	0.2	0.8	1.0	1.1	#N/A
Structure Controlling Flow =	Plate	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	#N/A
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	N/A	0.6	1.2	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	63	62	65	67	65	63	61	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**40**	67	66	70	73	73	72	71	#N/A
Maximum Ponding Depth (ft) =	2.45	4.09	3.84	4.67	5.49	6.01	6.21	6.40	#N/A
Area at Maximum Ponding Depth (acres) =	0.35	0.42	0.41	0.44	0.48	0.50	0.51	0.51	#N/A
Maximum Volume Stored (acre-ft) =	0.338	0.971	0.863	1.221	1.595	1.857	1.953	2.055	#N/A

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: LORSON RANCH EAST 4 FDR

Basin ID: FUTURE 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	--	--	--	100	0.002	19	0.000
5694	--	1.00	--	--	--	2,250	0.052	785	0.018
5695	--	2.00	--	--	--	35,024	0.804	19,095	0.438
5696	--	3.00	--	--	--	62,057	1.425	67,984	1.561
5697	--	4.00	--	--	--	65,120	1.495	131,573	3.020
5698	--	5.00	--	--	--	68,248	1.567	198,257	4.551
5699	--	6.00	--	--	--	71,443	1.640	268,102	6.155
5700	--	7.00	--	--	--	74,705	1.715	341,176	7.832
5701	--	8.00	--	--	--	78,040	1.792	417,549	9.586
5702	--	9.00	--	--	--	81,442	1.870	497,290	11.416
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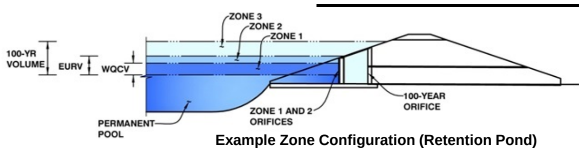
Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: LORSON RANCH EAST 4 FDR
Basin ID: FUTURE POND E2



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	3.12	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 =	5.60	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	25.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	5.60	N/A	feet
Over Flow Weir Slope Length =	4.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	5.57	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	70.00	N/A	ft^2
Overflow Grate Open Area w/ Debris =	35.00	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	38.9	117.0	137.2	150.1	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.4	0.9	1.1	0.9	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	0.5	1.6	1.9	2.0	#N/A
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	#N/A
Time to Drain 97% of Inflow Volume (hours) =	38	54	53	56	55	52	49	46	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**40**	57	56	60	61	59	58	57	#N/A
Maximum Ponding Depth (ft) =	3.05	4.57	4.23	5.33	6.02	6.53	7.04	8.06	#N/A
Area at Maximum Ponding Depth (acres) =	1.43	1.54	1.51	1.59	1.64	1.68	1.72	1.80	#N/A
Maximum Volume Stored (acre-ft) =	1.618	3.869	3.351	5.072	6.171	7.018	7.884	9.675	#N/A

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)

Designer: Richard Schindler
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

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:

Channel Report

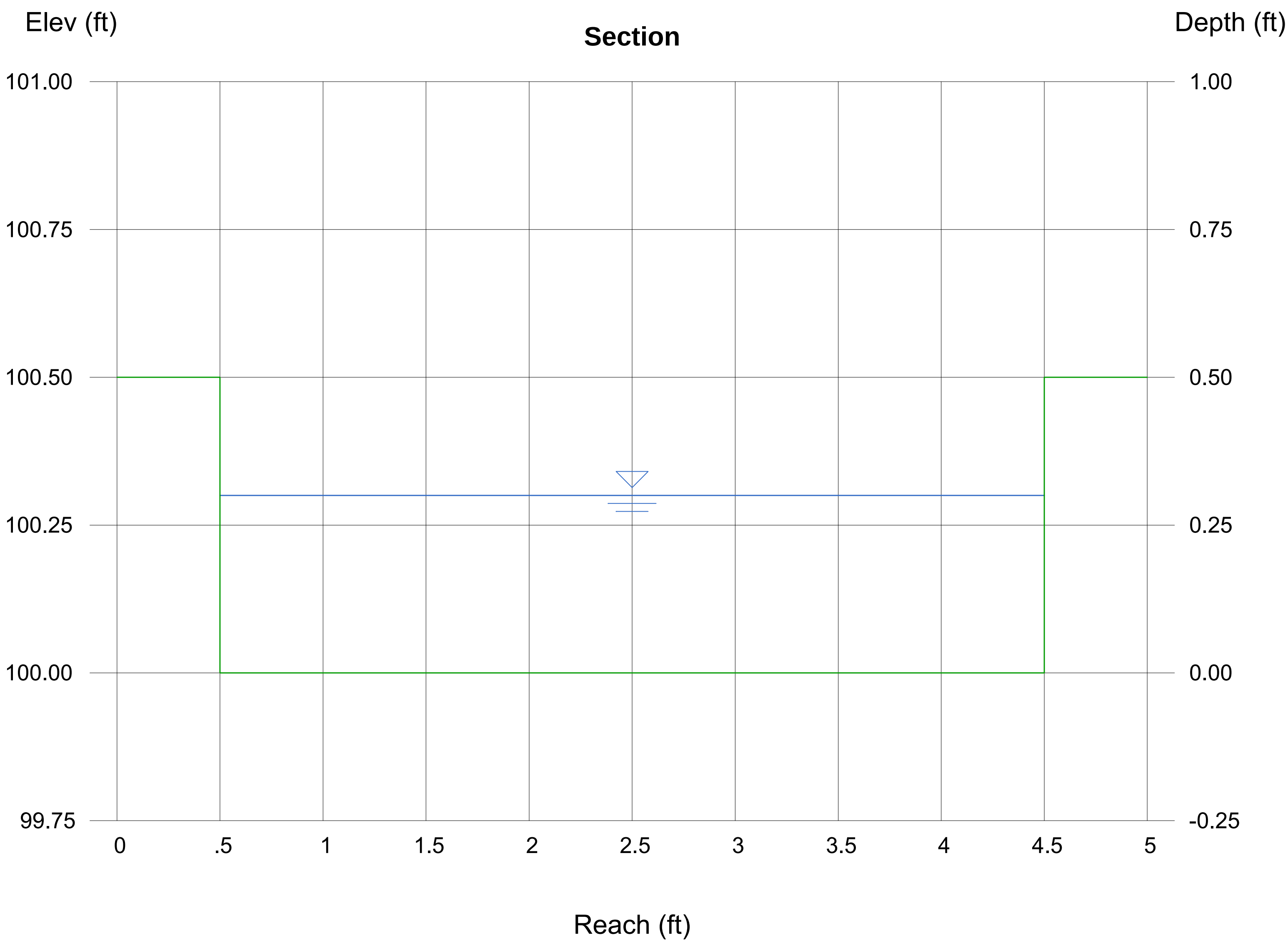
low flow channel

Rectangular
Botom Width (ft) = 4.00
Total Depth (ft) = 0.50

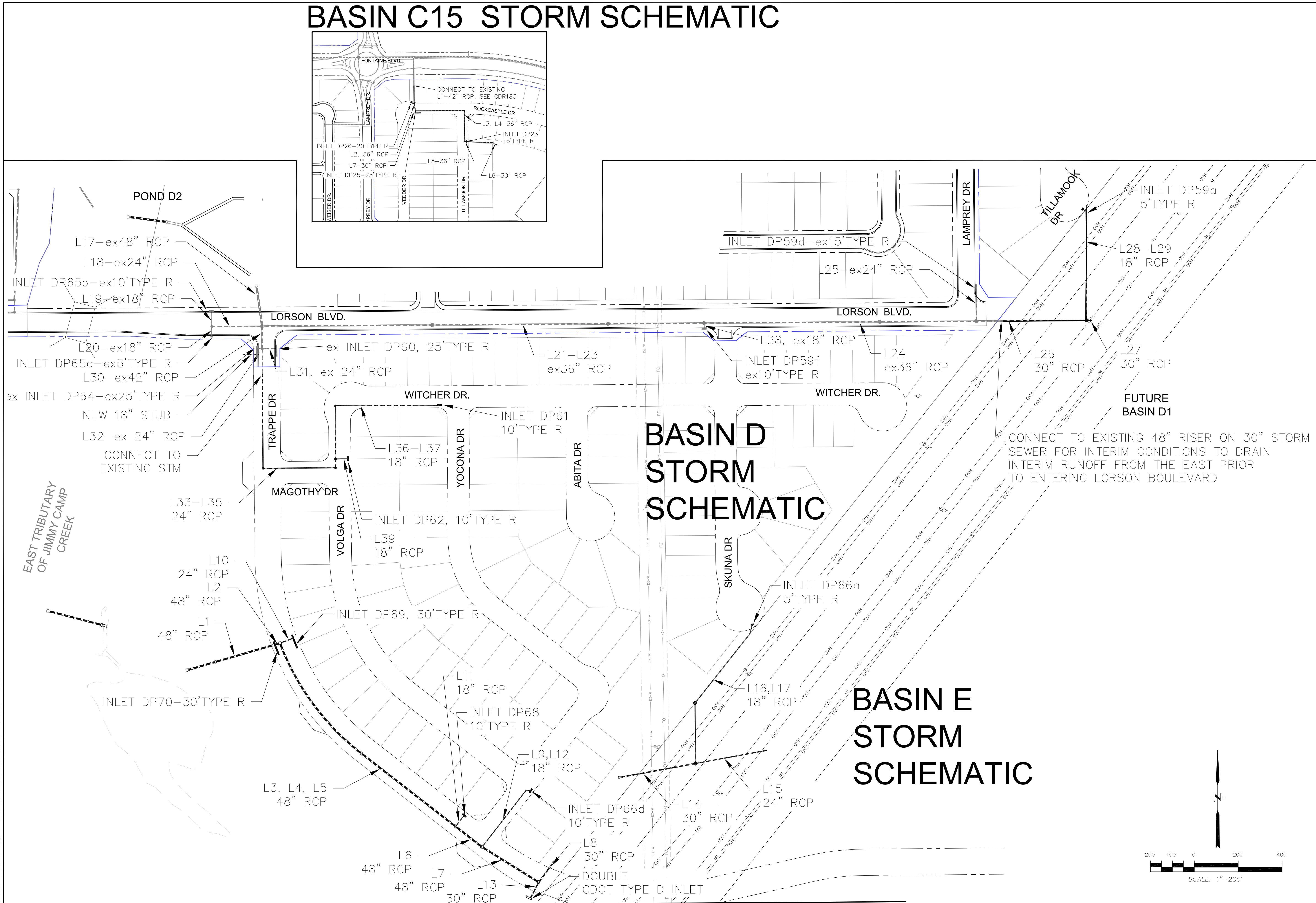
Invert Elev (ft) = 100.00
Slope (%) = 0.50
N-Value = 0.013

Calculations
Compute by: Q vs Depth
No. Increments = 10

Highlighted
Depth (ft) = 0.30
Q (cfs) = 3.958
Area (sqft) = 1.20
Velocity (ft/s) = 3.30
Wetted Perim (ft) = 4.60
Crit Depth, Yc (ft) = 0.26
Top Width (ft) = 4.00
EGL (ft) = 0.47



APPENDIX E– STORM SEWER SCHEMATIC AND HYDRAFLOW STORM SEWER CALCS



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BURNSVILLE, MN 55306
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CONTACT: RICHARD L. SCHINDLER, P.E.
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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:
LORSON RANCH EAST 4
EAST OF EAST TRIBUTARY
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

STORM SEWER SCHEMATIC

LORSON RANCH EAST FIL. 4

DATE
FEBRUARY 28, 2018

PROJECT NO.
100.048

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	38.10	42 c	202.3	5723.10	5727.36	2.106	5724.99	5729.25	n/a	5729.25	End
2	L2	31.63	36 c	30.7	5728.15	5728.46	1.011	5729.74	5730.25	0.00	5730.25	1
3	L3	19.82	36 c	223.4	5728.50	5730.75	1.007	5730.93	5732.17	n/a	5732.17 j	2
4	L4	20.28	36 c	141.8	5730.95	5732.40	1.022	5732.60	5733.84	n/a	5733.84 j	3
5	L5	20.31	36 c	11.2	5732.70	5732.79	0.805	5734.28	5734.23	0.00	5734.23	4
6	L6	13.20	30 c	139.3	5733.40	5735.50	1.508	5734.69	5736.71	n/a	5736.71 j	5
7	L7	15.85	30 c	10.8	5729.21	5729.48	2.506	5730.89	5730.81	n/a	5730.81 j	2

Lorson East 4 FDR - C15-5yr | Number of lines: 7 | Run Date: 02-14-2019

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.
17	L17	87.57	48 c	100.0	5697.00	5699.50	2.500	5699.78	5702.27	0.55	5702.27	End
18	L18	6.17	18 c	101.3	5701.90	5702.93	1.017	5703.46	5703.88	n/a	5703.88 j	17
19	L19	4.16	18 c	30.6	5703.13	5703.45	1.048	5704.22	5704.23	n/a	5704.23 j	18
20	L20	2.00	18 c	20.0	5703.33	5703.73	2.000	5704.29	5704.27	n/a	5704.27 j	18
21	L21	44.98	36 c	403.4	5700.75	5715.00	3.533	5703.02	5717.14	n/a	5717.14 j	17
22	L22	44.98	36 c	400.0	5715.30	5725.70	2.600	5717.59	5727.84	n/a	5727.84 j	21
23	L23	44.98	36 c	217.3	5726.00	5732.00	2.762	5728.29	5734.14	n/a	5734.14 j	22
24	L24	36.40	36 c	621.3	5732.00	5743.26	1.812	5734.81	5745.18	n/a	5745.18 j	23
25	L25	10.66	24 c	67.0	5745.16	5745.96	1.192	5746.08	5747.12	n/a	5747.12	24
26	L26	25.74	36 c	248.8	5743.86	5748.50	1.865	5745.88	5750.12	n/a	5750.12 j	24
27	L27	23.56	30 c	19.8	5749.50	5749.69	0.962	5750.88	5751.60	0.00	5751.60	26
28	L28	2.19	18 c	249.0	5752.72	5762.38	3.879	5753.05	5762.94	n/a	5762.94	26
29	L29	2.19	18 c	10.0	5762.38	5762.68	3.003	5763.12	5763.25	n/a	5763.25 j	28
30	L30	36.42	42 c	53.0	5700.23	5701.27	1.962	5703.43	5703.12	0.31	5703.12	17
31	L31	15.76	24 c	28.1	5702.57	5703.37	2.852	5703.50	5705.37	0.00	5705.37	30
32	L32	3.15	24 c	12.5	5702.97	5703.53	4.493	5703.88	5704.16	n/a	5704.16 j	30
33	L33	17.51	24 c	273.9	5702.93	5707.47	1.658	5704.05	5708.95	n/a	5708.95	30
34	L34	17.51	24 c	145.5	5707.81	5710.70	1.987	5709.23	5712.18	n/a	5712.18	33
35	L35	17.51	24 c	19.4	5711.00	5711.30	1.548	5712.46	5712.78	n/a	5712.78	34
36	L36	7.41	18 c	120.7	5711.80	5713.30	1.242	5713.27	5714.34	n/a	5714.34 j	35
37	L37	7.41	18 c	219.8	5713.60	5719.23	2.562	5714.57	5720.27	0.00	5720.27	36
38	L38	8.58	18 c	13.6	5733.60	5733.93	2.430	5734.85	5735.05	0.00	5735.05	23
39	L39	10.10	18 c	28.3	5711.80	5712.09	1.023	5713.04	5713.31	0.00	5713.31	35

Lorson East fdr-D Basins-5yr	Number of lines: 23	Run Date: 02-14-2019

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	69.67	48 c	76.0	5700.38	5701.90	2.000	5702.85	5704.37	n/a	5704.37	End
2	L2	64.11	48 c	15.0	5702.20	5702.58	2.533	5705.10	5704.95	n/a	5704.95	1
3	L3	59.60	48 c	169.5	5702.68	5704.38	1.003	5705.66	5706.66	n/a	5706.66 j	2
4	L4	59.60	48 c	169.5	5704.58	5706.30	1.015	5707.32	5708.58	n/a	5708.58 j	3
5	L5	59.60	48 c	269.4	5706.50	5709.20	1.002	5709.24	5711.48	n/a	5711.48 j	4
6	L6	52.10	48 c	76.4	5709.30	5710.10	1.047	5712.22	5712.24	n/a	5712.24 j	5
7	L7	46.00	48 c	152.3	5710.30	5711.83	1.005	5712.93	5713.84	n/a	5713.84 j	6
8	L8	20.00	30 c	52.5	5713.89	5716.03	4.072	5714.73*	5718.59*	0.00	5718.59	7
9	L9	6.10	18 c	149.4	5712.40	5716.67	2.859	5713.01	5717.61	0.08	5717.61	6
10	L10	4.50	24 c	29.3	5705.76	5706.34	1.981	5706.27	5707.21	0.00	5707.21	2
11	L11	7.50	18 c	31.3	5712.00	5713.31	4.180	5712.61*	5714.97*	0.00	5714.97	5
12	L12	6.10	18 c	21.4	5716.75	5717.46	3.315	5717.85	5718.40	n/a	5718.40 j	9
13	L13	26.00	30 c	35.2	5713.30	5713.66	1.025	5714.73	5715.60	0.00	5715.60	7
14	L14	16.00	30 c	189.0	5724.00	5726.29	1.212	5725.42	5727.63	n/a	5727.63 j	End
15	L15	12.80	24 c	166.0	5726.90	5727.90	0.602	5728.17	5729.17	0.17	5729.34	14
16	L16	3.20	18 c	142.8	5727.30	5728.90	1.120	5728.13	5729.58	n/a	5729.58 j	14
17	L17	3.20	18 c	220.3	5728.90	5739.48	4.802	5729.79	5740.16	n/a	5740.16 j	16

Lorson East4FDR - E Basins-5yr | Number of lines: 17 | Run Date: 02-14-2019

NOTES: c = cir; e = ellip; b = box; Return period = 5 Yrs. ; *Surcharged (HGL above crown). ; 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	92.62	42 c	202.3	5724.40	5727.36	1.463	5727.35	5730.31	0.00	5730.31	End
2	L2	78.33	36 c	30.7	5728.15	5728.46	1.011	5731.15*	5731.57*	0.00	5731.57	1
3	L3	51.31	36 c	223.4	5728.50	5730.75	1.007	5732.66*	5733.99*	0.00	5733.99	2
4	L4	51.79	36 c	141.8	5730.95	5732.40	1.022	5733.99	5734.69	0.00	5734.69	3
5	L5	51.83	36 c	11.2	5732.70	5732.79	0.805	5735.10	5735.11	0.00	5735.11	4
6	L6	37.90	30 c	139.3	5733.40	5735.50	1.508	5735.40	5737.56	0.00	5737.56	5
7	L7	31.70	30 c	10.8	5729.21	5729.48	2.506	5732.83*	5732.90*	0.00	5732.90	2

Lorson East 4 FDR - C15-100yr	Number of lines: 7	Run Date: 02-14-2019

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

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.
17	L17	173.6	48 c	100.0	5697.00	5699.50	2.500	5700.75	5703.25	1.25	5703.25	End
18	L18	15.99	24 c	101.3	5701.50	5702.63	1.115	5705.97*	5706.48*	0.16	5706.64	17
19	L19	12.70	18 c	30.6	5702.93	5703.25	1.046	5706.64*	5707.09*	0.00	5707.09	18
20	L20	3.59	18 c	20.0	5702.93	5703.33	2.000	5706.98*	5707.00*	0.00	5707.00	18
21	L21	103.3	36 c	400.0	5700.75	5715.00	3.563	5703.25	5717.91	n/a	5717.91	17
22	L22	103.9	36 c	400.0	5715.30	5725.70	2.600	5717.93	5728.61	0.00	5728.61	21
23	L23	104.2	36 c	217.3	5726.00	5732.00	2.762	5728.65	5734.91	1.03	5734.91	22
24	L24	93.04	36 c	621.3	5732.00	5743.26	1.812	5735.66*	5747.75*	1.08	5748.83	23
25	L25	20.30	24 c	67.0	5745.06	5745.96	1.343	5750.87*	5751.41*	0.00	5751.41	24
26	L26	75.38	36 c	251.2	5744.56	5749.20	1.847	5749.75*	5752.96*	0.53	5753.49	24
27	L27	60.85	30 c	27.3	5750.00	5750.49	1.793	5753.49*	5754.10*	0.00	5754.10	26
28	L28	4.84	18 c	249.0	5752.72	5762.38	3.879	5755.15	5763.22	n/a	5763.22 j	26
29	L29	4.85	18 c	10.0	5762.38	5762.68	3.003	5763.45	5763.52	n/a	5763.52 j	28
30	L30	88.30	42 c	53.7	5700.23	5701.27	1.935	5705.07*	5705.48*	0.52	5706.01	17
31	L31	31.70	24 c	28.1	5702.97	5703.77	2.843	5706.01*	5706.56*	0.00	5706.56	30
32	L32	29.20	24 c	13.0	5702.97	5703.48	3.934	5706.01*	5706.22*	0.40	5706.63	30
33	L33	27.30	24 c	272.5	5702.98	5707.49	1.655	5706.14*	5710.11*	0.35	5710.47	30
34	L34	27.25	24 c	144.3	5707.81	5710.40	1.795	5710.47*	5712.56*	0.35	5712.92	33
35	L35	27.26	24 c	19.4	5710.70	5711.20	2.581	5712.92	5713.18	0.47	5713.65	34
36	L36	10.87	18 c	120.7	5711.70	5714.00	1.905	5714.24	5715.42	0.18	5715.61	35
37	L37	10.88	18 c	219.8	5714.20	5719.23	2.289	5715.63	5720.49	n/a	5720.49 j	36
38	L38	12.37	18 c	13.6	5733.60	5733.93	2.430	5737.59*	5737.78*	0.00	5737.78	23
39	L39	16.30	18 c	28.3	5711.80	5712.09	1.023	5713.65*	5714.34*	0.00	5714.34	35

Lorson East fdr- D Basins-100y | Number of lines: 23 | Run Date: 02-14-2019

NOTES: c = cir; e = ellip; b = box; Return period = 100 Yrs. ; *Surcharged (HGL above crown). ; 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	211.5	48 c	76.0	5700.38	5701.90	1.999	5704.26	5705.90	0.00	5705.90	End
2	L2	174.5	48 c	15.0	5702.20	5702.58	2.533	5707.31*	5707.53*	0.00	5707.53	1
3	L3	141.5	48 c	169.5	5702.78	5704.48	1.002	5708.56*	5710.20*	0.00	5710.20	2
4	L4	141.5	48 c	169.5	5704.48	5706.20	1.015	5710.20*	5711.85*	0.00	5711.85	3
5	L5	141.5	48 c	269.4	5706.40	5709.10	1.002	5711.85*	5714.46*	0.79	5715.25	4
6	L6	121.3	48 c	76.4	5709.30	5710.07	1.009	5715.77*	5716.31*	0.58	5716.89	5
7	L7	112.0	48 c	152.3	5710.30	5711.83	1.005	5717.11*	5718.03*	0.49	5718.53	6
8	L8	42.00	30 c	52.5	5714.93	5715.46	1.008	5718.62*	5719.18*	0.00	5719.18	7
9	L9	9.30	18 c	149.4	5712.40	5716.67	2.859	5717.91*	5719.08*	0.09	5719.17	6
10	L10	33.03	24 c	29.3	5705.76	5706.34	1.981	5708.81*	5709.43*	0.00	5709.43	2
11	L11	20.17	18 c	31.3	5712.00	5713.31	4.180	5715.25*	5716.40*	0.00	5716.40	5
12	L12	9.30	18 c	21.4	5716.75	5717.46	3.315	5719.17*	5719.34*	0.00	5719.34	9
13	L13	70.00	30 c	35.2	5713.33	5713.69	1.015	5718.53*	5719.55*	0.00	5719.55	7
14	L14	43.30	30 c	189.0	5724.00	5726.30	1.216	5726.21	5728.50	n/a	5728.50	End
15	L15	36.30	24 c	165.6	5726.90	5727.90	0.604	5728.90*	5733.17*	0.62	5733.79	14
16	L16	7.00	18 c	142.8	5727.30	5728.90	1.120	5729.65	5730.21	0.00	5730.21	14
17	L17	7.00	18 c	220.3	5728.90	5739.48	4.802	5730.25	5740.49	n/a	5740.49 j	16

LRE 4 FDR - E Basins-100yr | Number of lines: 17 | Run Date: 02-14-2019

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

APPENDIX F –INTERIM POND CALCULATIONS

Legend

Hyd.	Origin	Description
1	Rational	Basins C1 & C2- Pond C1 inflow
2	Reservoir	Pond C1 Interim flow

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	Rational	22.14	1	22	29,228	----	------	------	Basins C1 & C2- Pond C1 inflow
2	Reservoir	2.156	1	42	28,324	1	5746.67	26,964	Pond C1 Interim flow

interim pond-C1-5yr.gpw	Return Period: 5 Year	Thursday, Feb 14 2019, 9:56 AM

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	Rational	135.24	1	23	186,632	----	------	------	Basin C15-ex
2	Reservoir	8.670	1	45	185,404	1	5749.25	171,675	Pond C1 Interim

interim pond-C1-100yr.gpw	Return Period: 100 Year	Thursday, Feb 14 2019, 9:57 AM

Hydrograph Plot

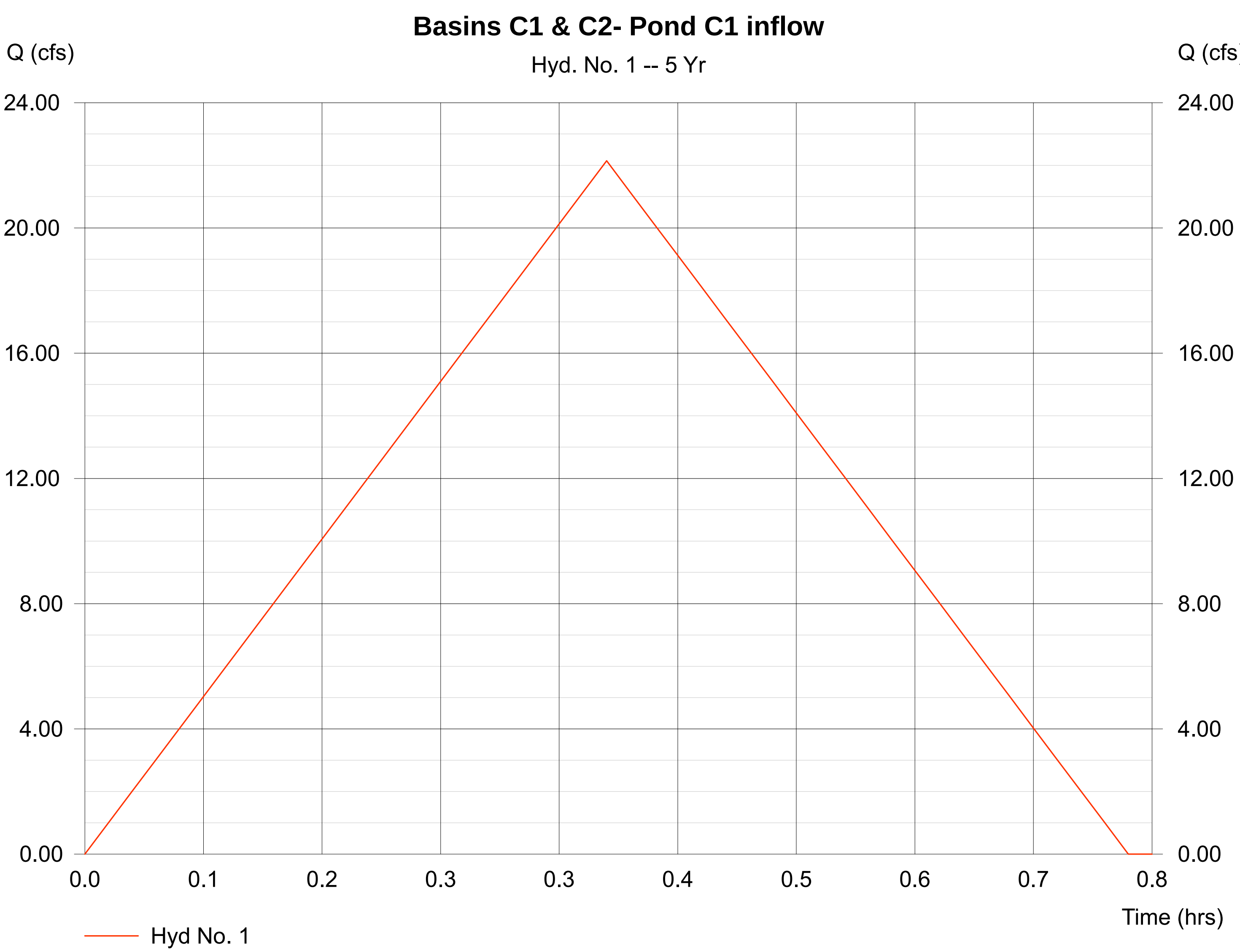
Hydraflow Hydrographs by Intelisolve | Thursday, Feb 14 2019, 9:56 AM

Hyd. No. 1

Basins C1 & C2- Pond C1 inflow

Hydrograph type	= Rational	Peak discharge	= 22.14 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Drainage area	= 55.000 ac	Runoff coeff.	= 0.15
Intensity	= 2.684 in/hr	Tc by User	= 22.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 29,228 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

Thursday, Feb 14 2019, 9:56 AM

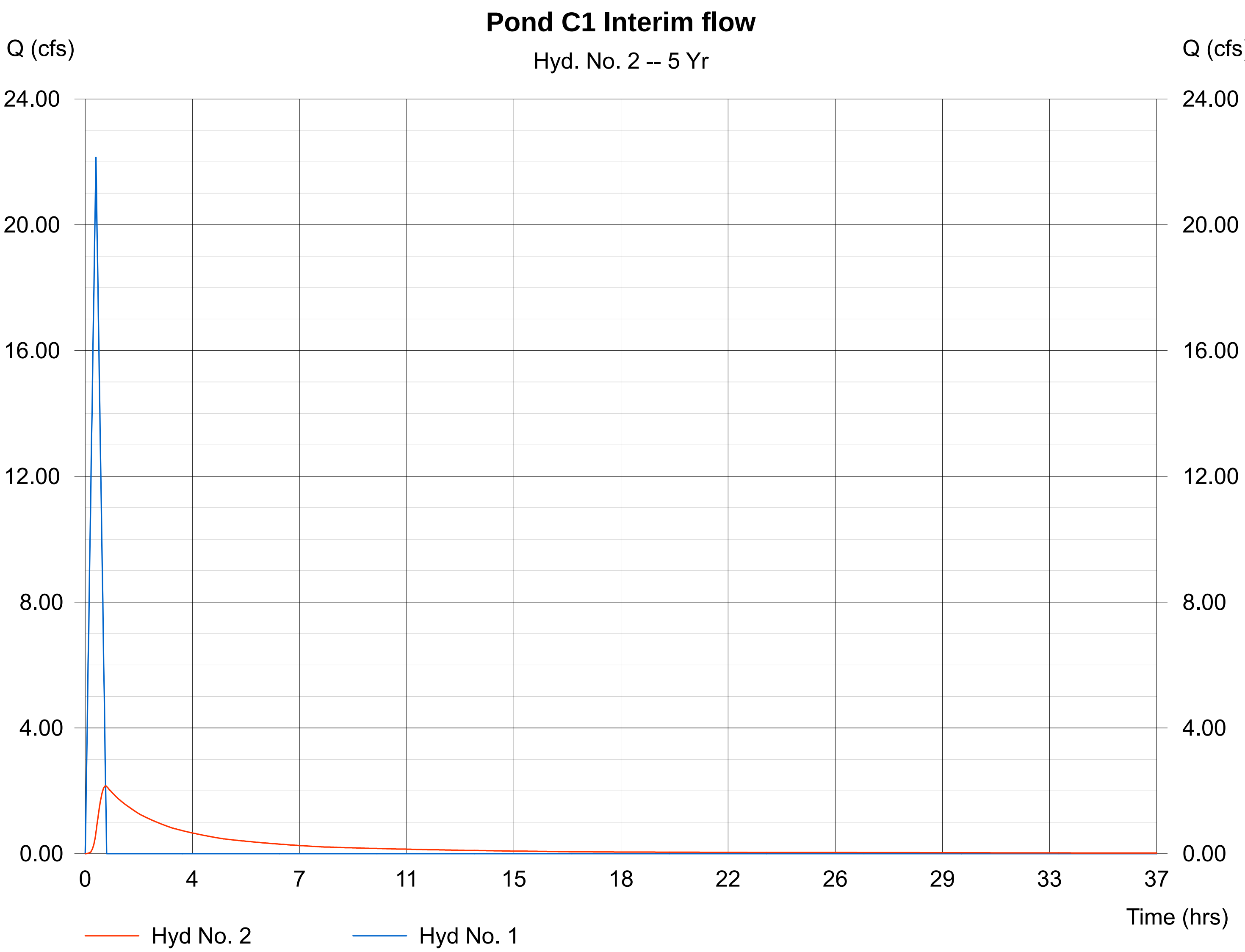
Hyd. No. 2

Pond C1 Interim flow

Hydrograph type	= Reservoir	Peak discharge	= 2.156 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Inflow hyd. No.	= 1	Max. Elevation	= 5746.67 ft
Reservoir name	= Pond C1	Max. Storage	= 26,964 cuft

Storage Indication method used.

Hydrograph Volume = 28,324 cuft



Hydrograph Plot

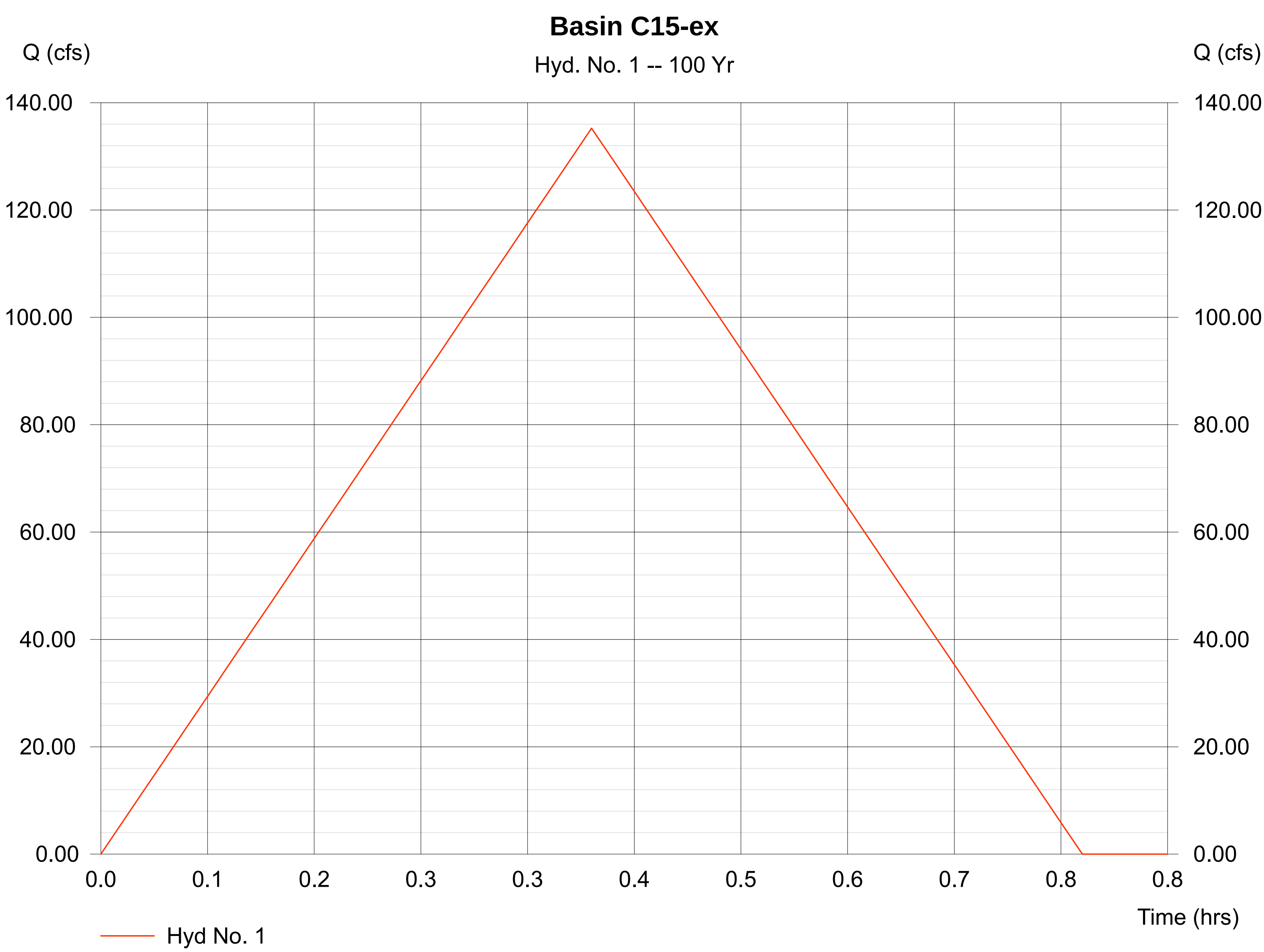
Hydraflow Hydrographs by Intelisolve — Thursday, Feb 14 2019, 9:57 AM

Hyd. No. 1

Basin C15-ex

Hydrograph type	= Rational	Peak discharge	= 135.24 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Drainage area	= 55.000 ac	Runoff coeff.	= 0.5
Intensity	= 4.918 in/hr	Tc by User	= 23.00 min
IDF Curve	= El Paso County-Table.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 186,632 cuft



Hydrograph Plot

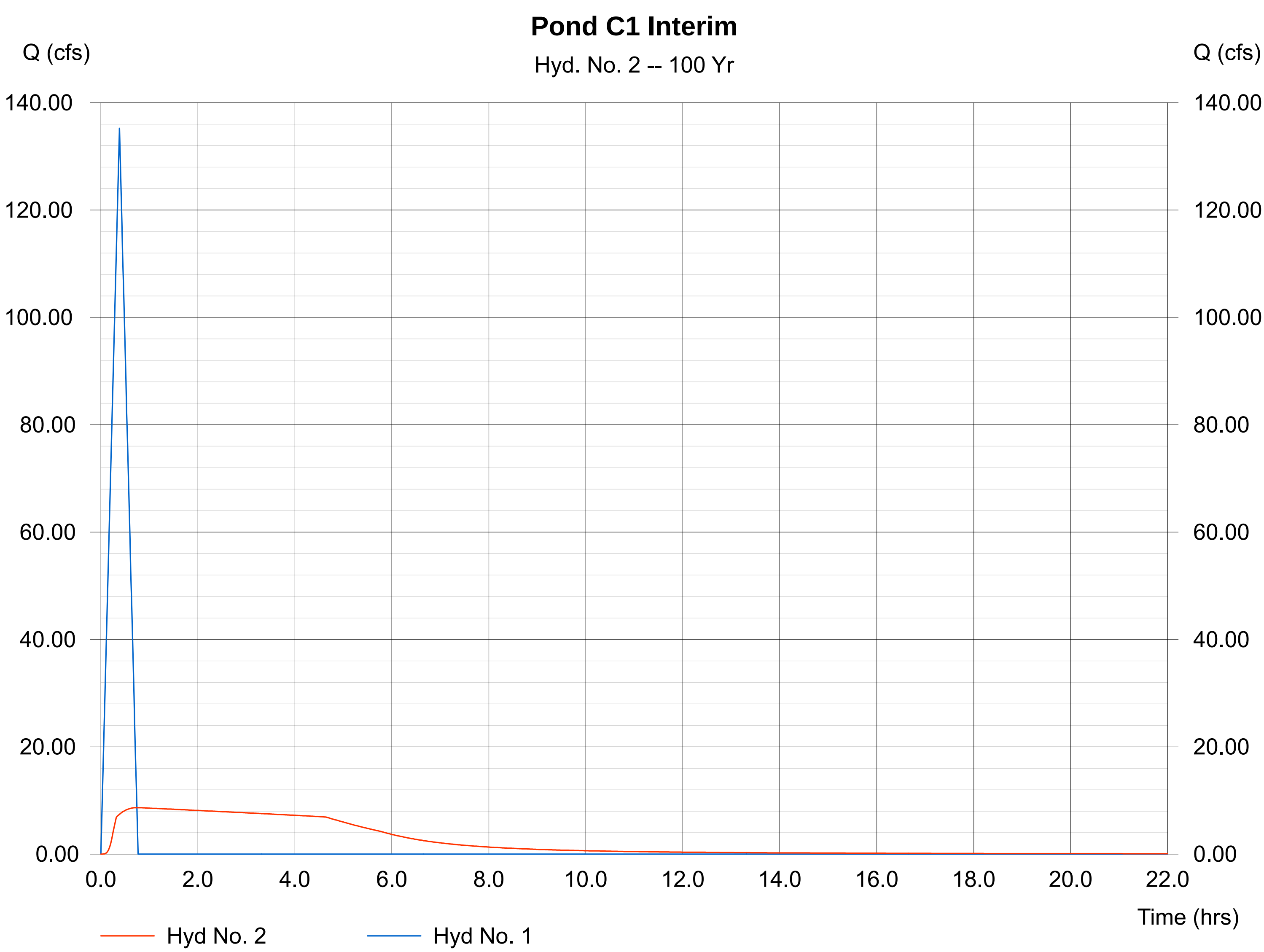
Hydraflow Hydrographs by Intelisolve | Thursday, Feb 14 2019, 9:57 AM

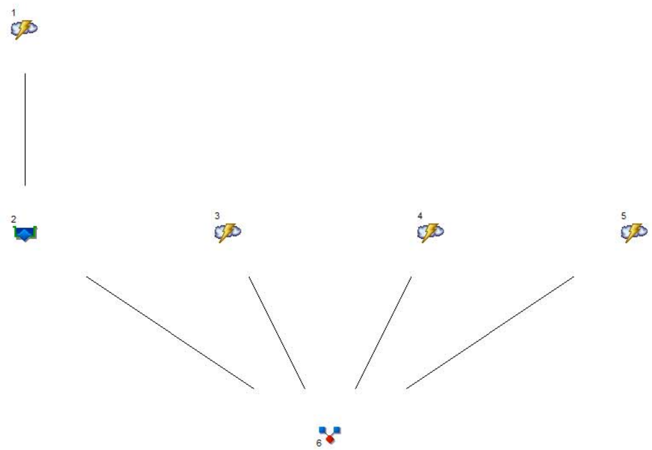
Hyd. No. 2

Pond C1 Interim

Hydrograph type	= Reservoir	Peak discharge	= 8.670 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Inflow hyd. No.	= 1	Max. Elevation	= 5749.25 ft
Reservoir name	= Pond C1	Max. Storage	= 171,675 cuft

Storage Indication method used. | Hydrograph Volume = 185,404 cuft





Legend

Hyd.	Origin	Description
1	Rational	Pond E1 Inflow
2	Reservoir	Pond E1 Outflow
3	Rational	Basin E2-ex
4	Rational	Basin E3-ex
5	Rational	Basin E-developed
6	Combine	Interim Flow Des.Pt.73

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	Rational	25.72	1	22	33,951	----	------	------	Pond E1 Inflow
2	Reservoir	8.656	1	37	33,929	1	5730.32	23,484	Pond E1 Outflow
3	Rational	28.85	1	19	32,892	----	------	------	Basin E2-ex
4	Rational	47.82	1	29	83,204	----	------	------	Basin E3-ex
5	Rational	30.26	1	20	36,315	----	------	------	Basin E-developed
6	Combine	95.02	1	20	186,341	2, 3, 4, 5	------	------	Interim Flow Des.Pt.73

interim pond-E-BASINS-5yr.gpw	Return Period: 5 Year	Thursday, Feb 14 2019, 9:54 AM

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	Rational	142.36	1	15	128,125	----	------	------	Pond E1 Inflow
2	Reservoir	19.62	1	28	128,103	1	5732.89	106,828	Pond E1 Outflow
3	Rational	92.88	1	18	100,308	----	------	------	Basin E2-ex
4	Rational	155.33	1	29	270,273	----	------	------	Basin E3-ex
5	Rational	69.58	1	22	91,845	----	------	------	Basin E-developed
6	Combine	278.53	1	22	590,528	2, 3, 4, 5	------	------	Interim Flow Des.Pt.73

interim pond-E-BASINS-100yr.gpw	Return Period: 100 Year	Thursday, Feb 14 2019, 9:46 AM

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

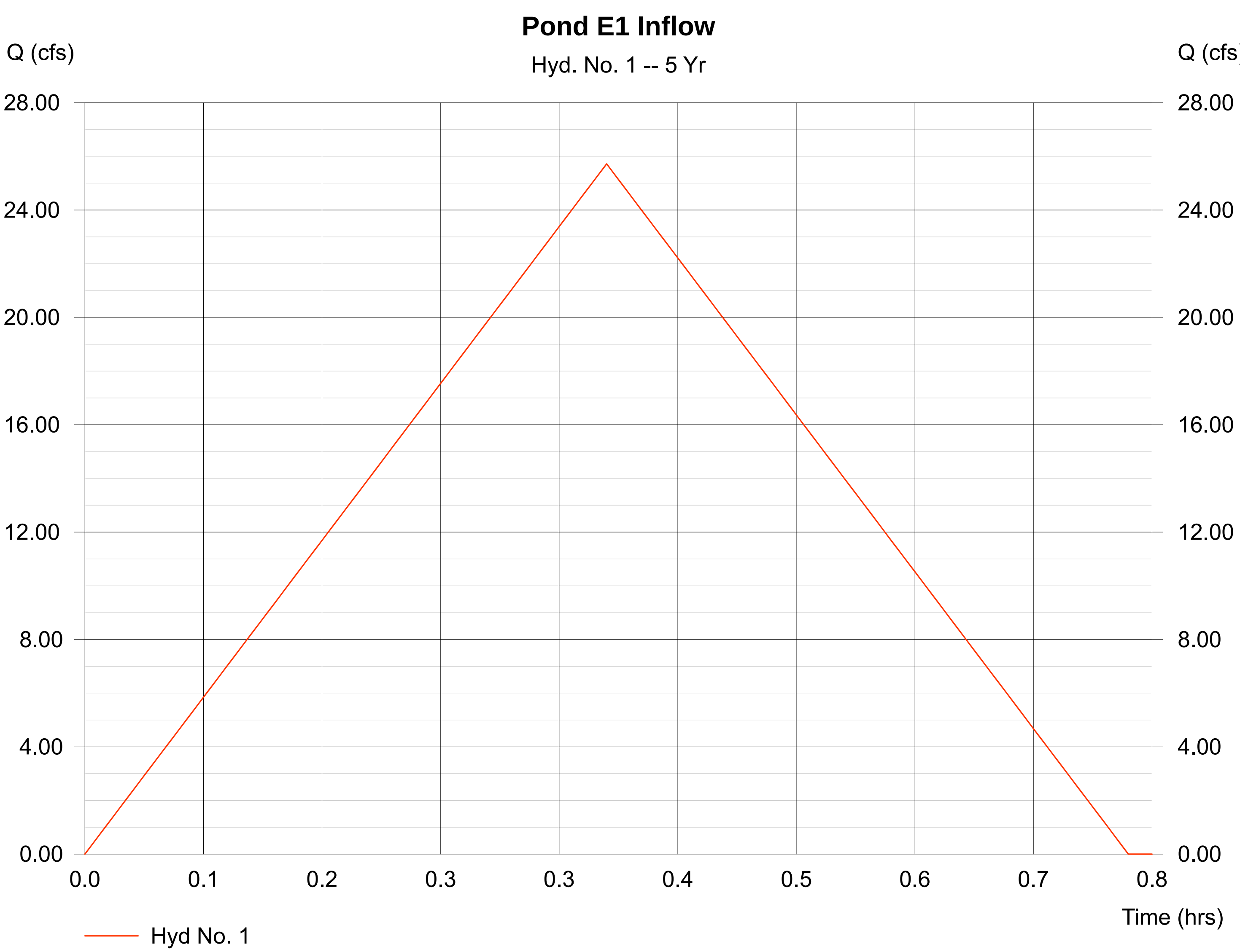
Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 1

Pond E1 Inflow

Hydrograph type	= Rational	Peak discharge	= 25.72 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Drainage area	= 57.000 ac	Runoff coeff.	= 0.15
Intensity	= 3.008 in/hr	Tc by User	= 22.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 33,951 cuft



Hydrograph Plot

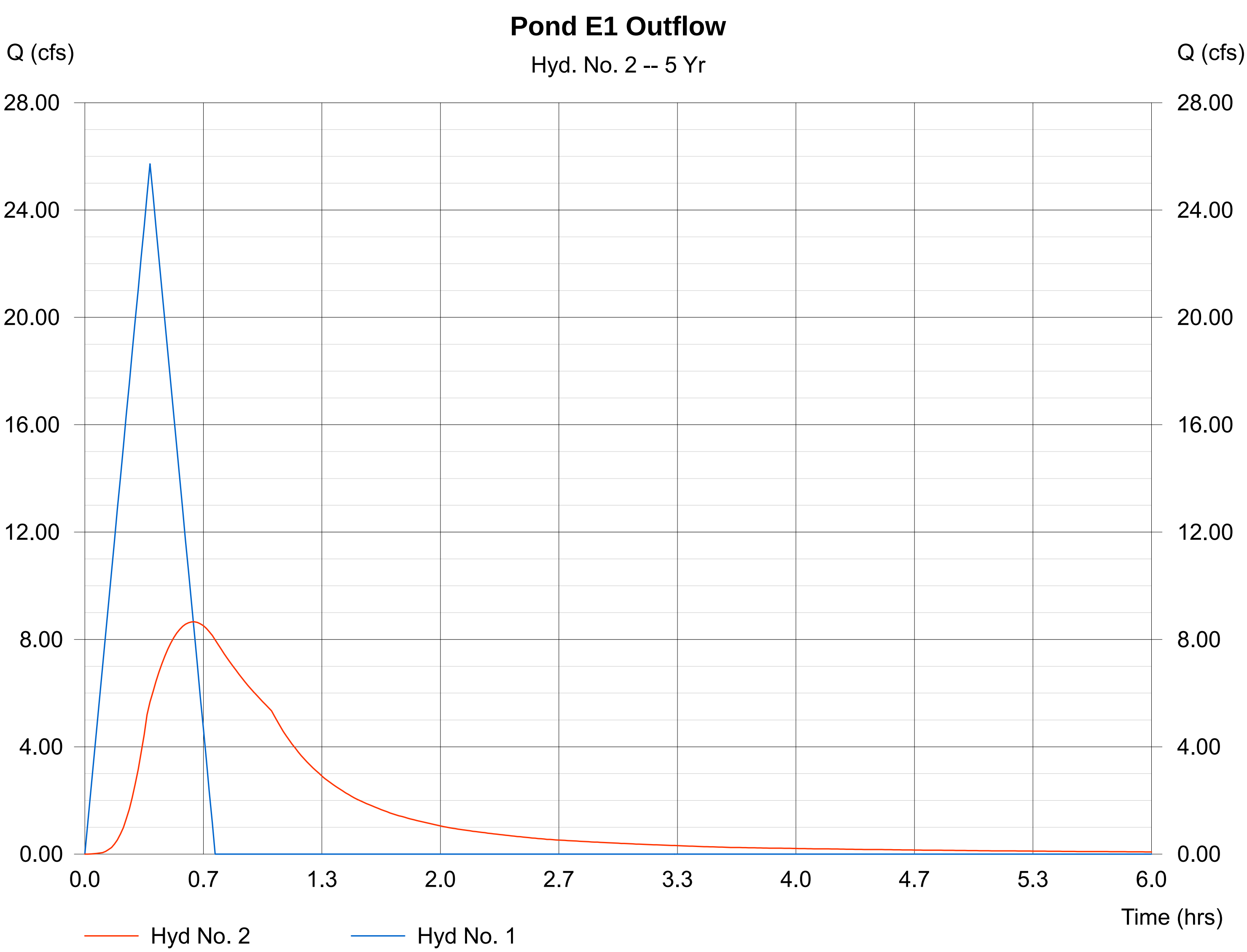
Hydraflow Hydrographs by Intelisolve — Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 2

Pond E1 Outflow

Hydrograph type	= Reservoir	Peak discharge	= 8.656 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Inflow hyd. No.	= 1	Max. Elevation	= 5730.32 ft
Reservoir name	= Pond E1	Max. Storage	= 23,484 cuft

Storage Indication method used. — Hydrograph Volume = 33,929 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

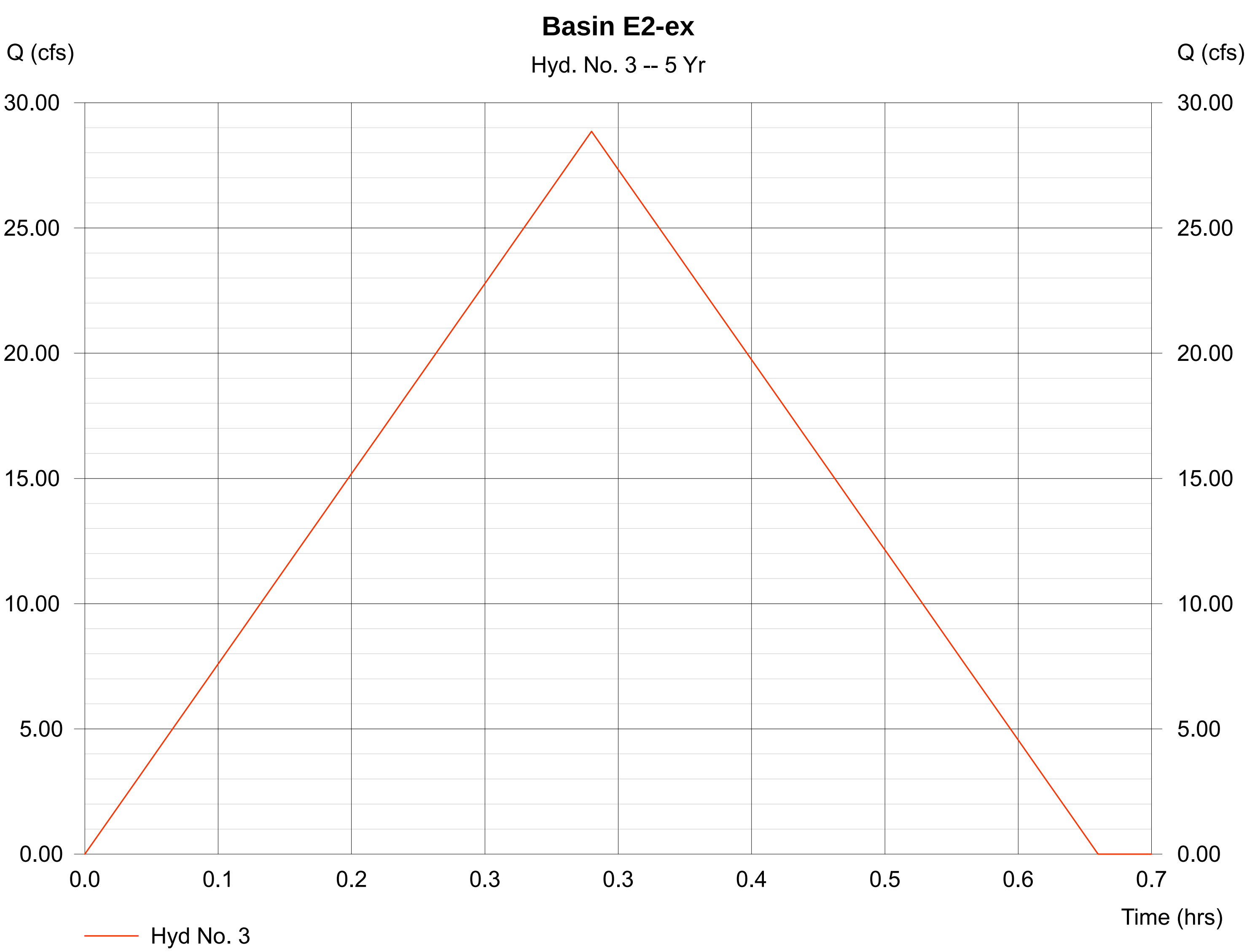
Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 3

Basin E2-ex

Hydrograph type	= Rational	Peak discharge	= 28.85 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Drainage area	= 29.500 ac	Runoff coeff.	= 0.3
Intensity	= 3.260 in/hr	Tc by User	= 19.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 32,892 cuft



Hydrograph Plot

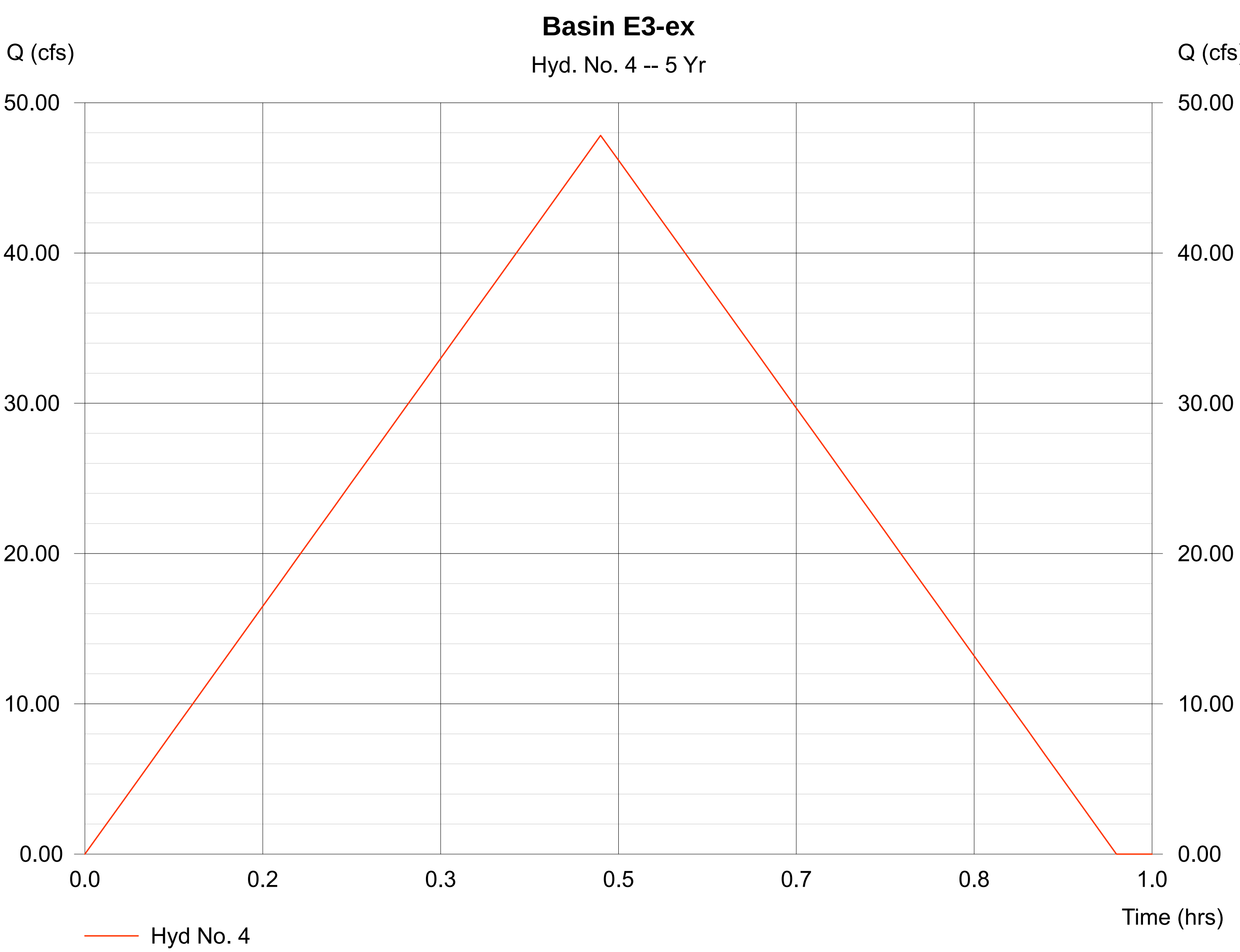
Hydraflow Hydrographs by Intelisolve | Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 4

Basin E3-ex

Hydrograph type	= Rational	Peak discharge	= 47.82 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Drainage area	= 72.500 ac	Runoff coeff.	= 0.26
Intensity	= 2.537 in/hr	Tc by User	= 29.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 83,204 cuft



Hydrograph Plot

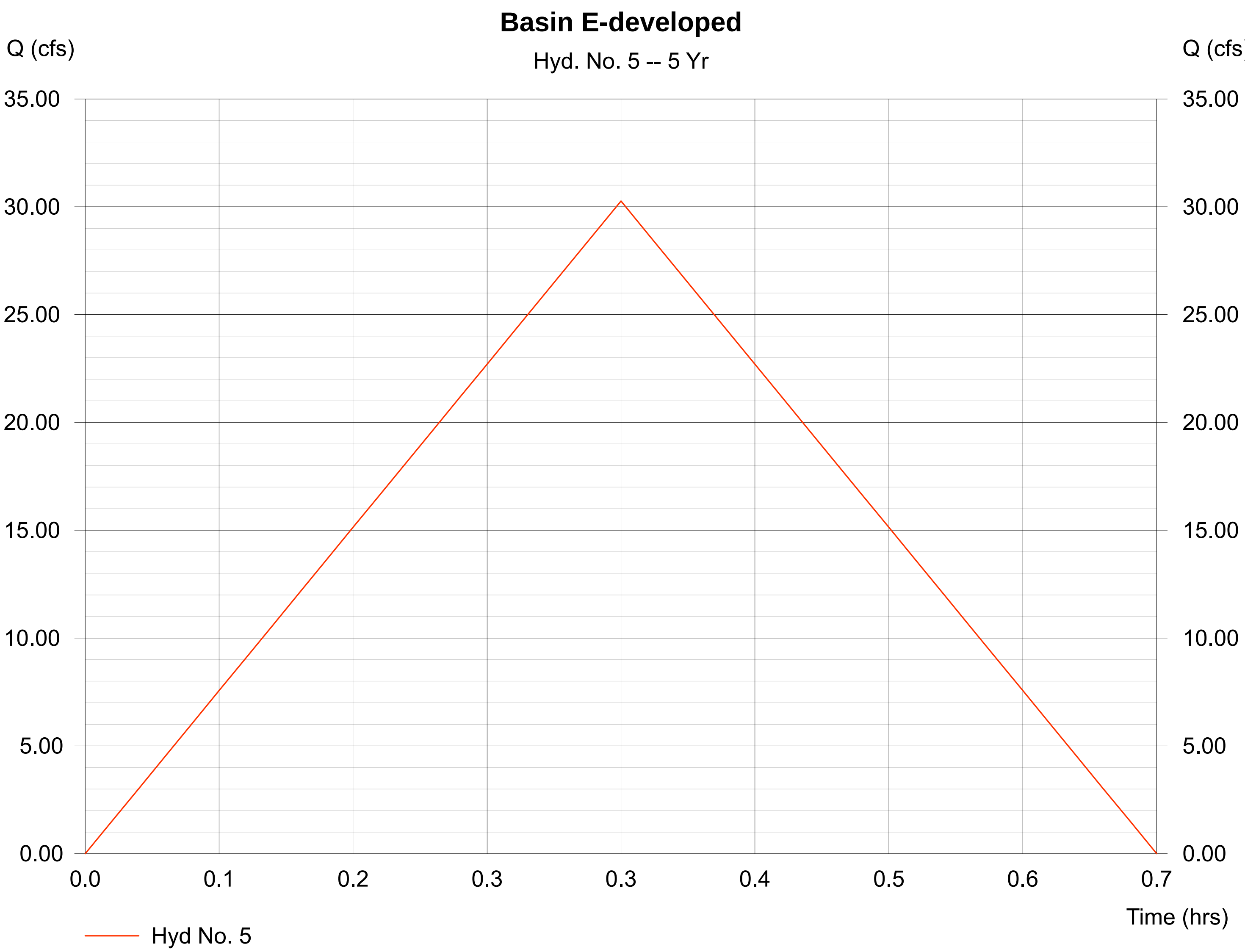
Hydraflow Hydrographs by Intelisolve — Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 5

Basin E-developed

Hydrograph type	= Rational	Peak discharge	= 30.26 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Drainage area	= 21.200 ac	Runoff coeff.	= 0.45
Intensity	= 3.172 in/hr	Tc by User	= 20.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 36,315 cuft



Hydrograph Plot

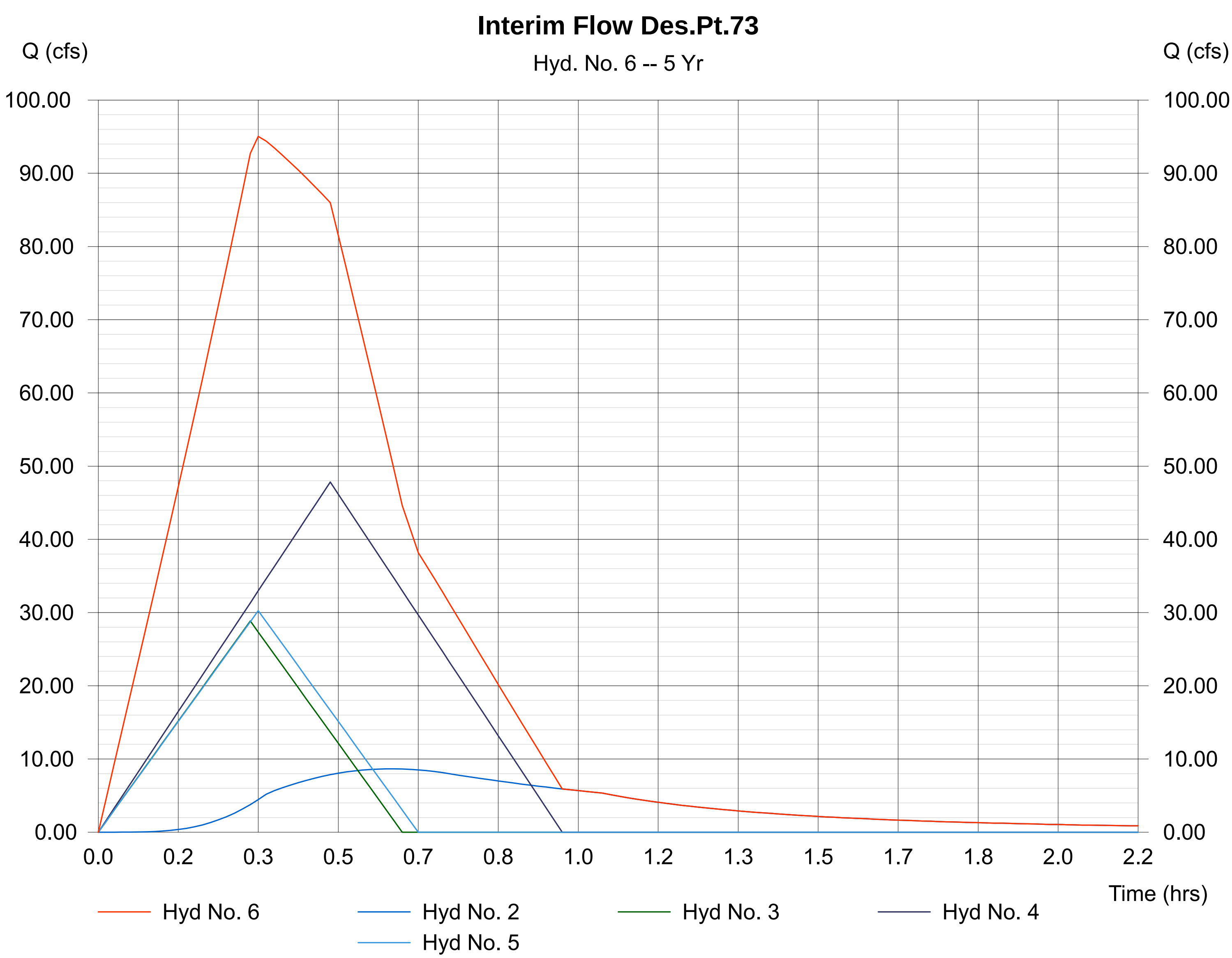
Hydraflow Hydrographs by Intelisolve Thursday, Feb 14 2019, 9:54 AM

Hyd. No. 6

Interim Flow Des.Pt.73

Hydrograph type	= Combine	Peak discharge	= 95.02 cfs
Storm frequency	= 5 yrs	Time interval	= 1 min
Inflow hyds.	= 2, 3, 4, 5		

Hydrograph Volume = 186,341 cuft



Hydrograph Plot

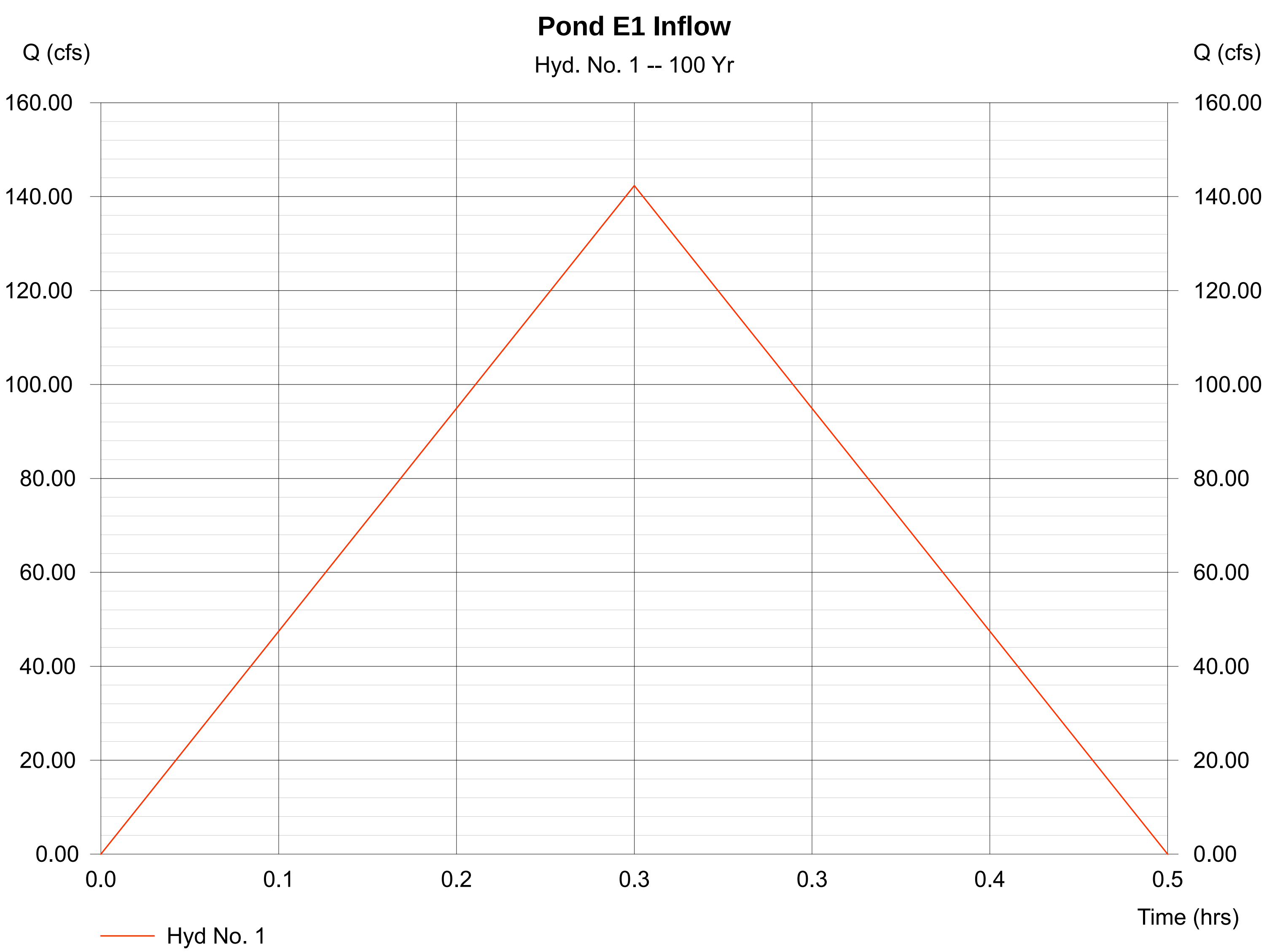
Hydraflow Hydrographs by Intelisolve Thursday, Feb 14 2019, 9:46 AM

Hyd. No. 1

Pond E1 Inflow

Hydrograph type	= Rational	Peak discharge	= 142.36 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Drainage area	= 56.500 ac	Runoff coeff.	= 0.41
Intensity	= 6.146 in/hr	Tc by User	= 15.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 128,125 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve — Thursday, Feb 14 2019, 9:46 AM

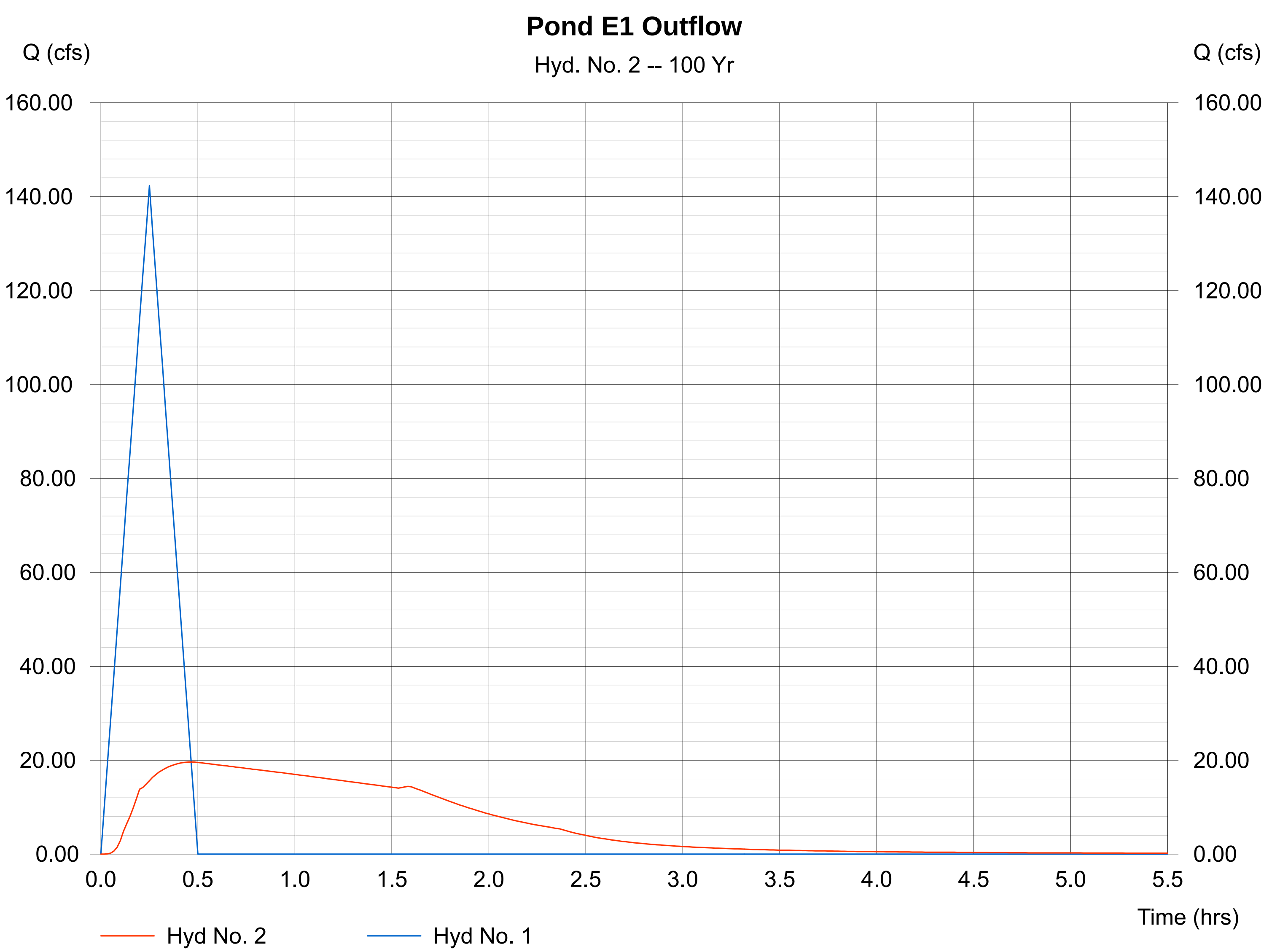
Hyd. No. 2

Pond E1 Outflow

Hydrograph type	= Reservoir	Peak discharge	= 19.62 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Inflow hyd. No.	= 1	Max. Elevation	= 5732.89 ft
Reservoir name	= Pond E1	Max. Storage	= 106,828 cuft

Storage Indication method used.

Hydrograph Volume = 128,103 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

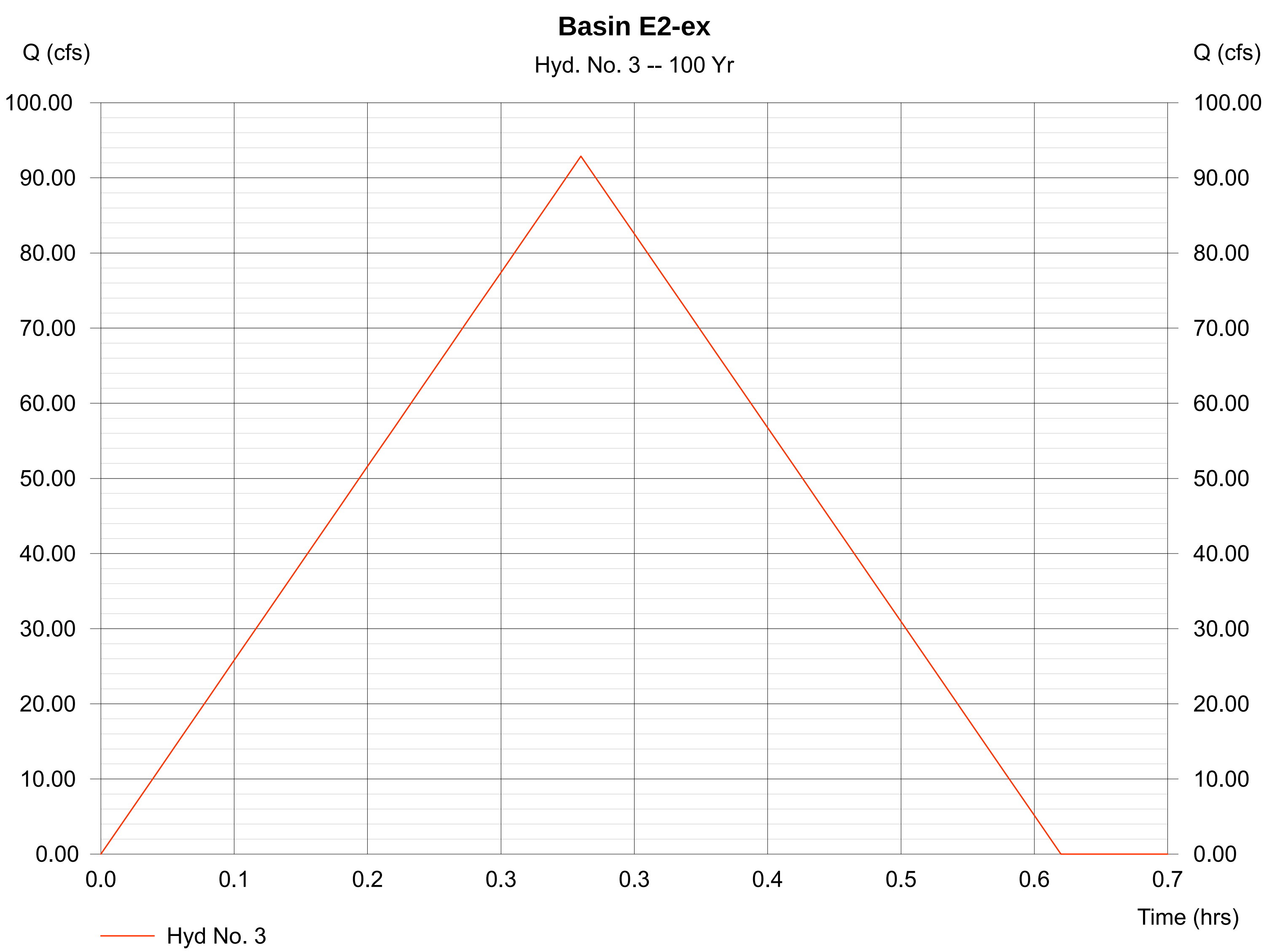
Thursday, Feb 14 2019, 9:46 AM

Hyd. No. 3

Basin E2-ex

Hydrograph type	= Rational	Peak discharge	= 92.88 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Drainage area	= 30.000 ac	Runoff coeff.	= 0.55
Intensity	= 5.629 in/hr	Tc by User	= 18.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 100,308 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

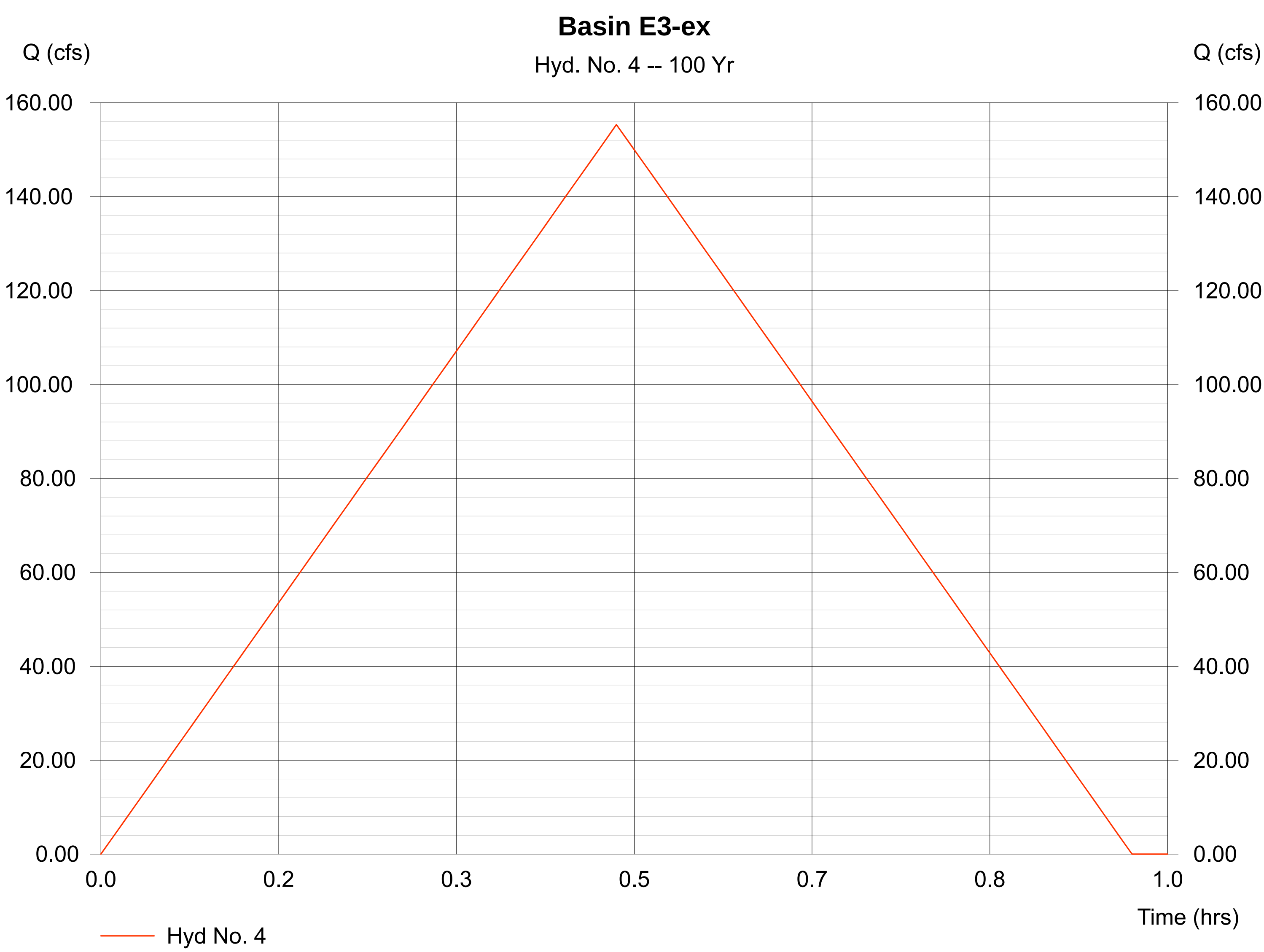
Thursday, Feb 14 2019, 9:46 AM

Hyd. No. 4

Basin E3-ex

Hydrograph type	= Rational	Peak discharge	= 155.33 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Drainage area	= 73.000 ac	Runoff coeff.	= 0.5
Intensity	= 4.256 in/hr	Tc by User	= 29.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 270,273 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

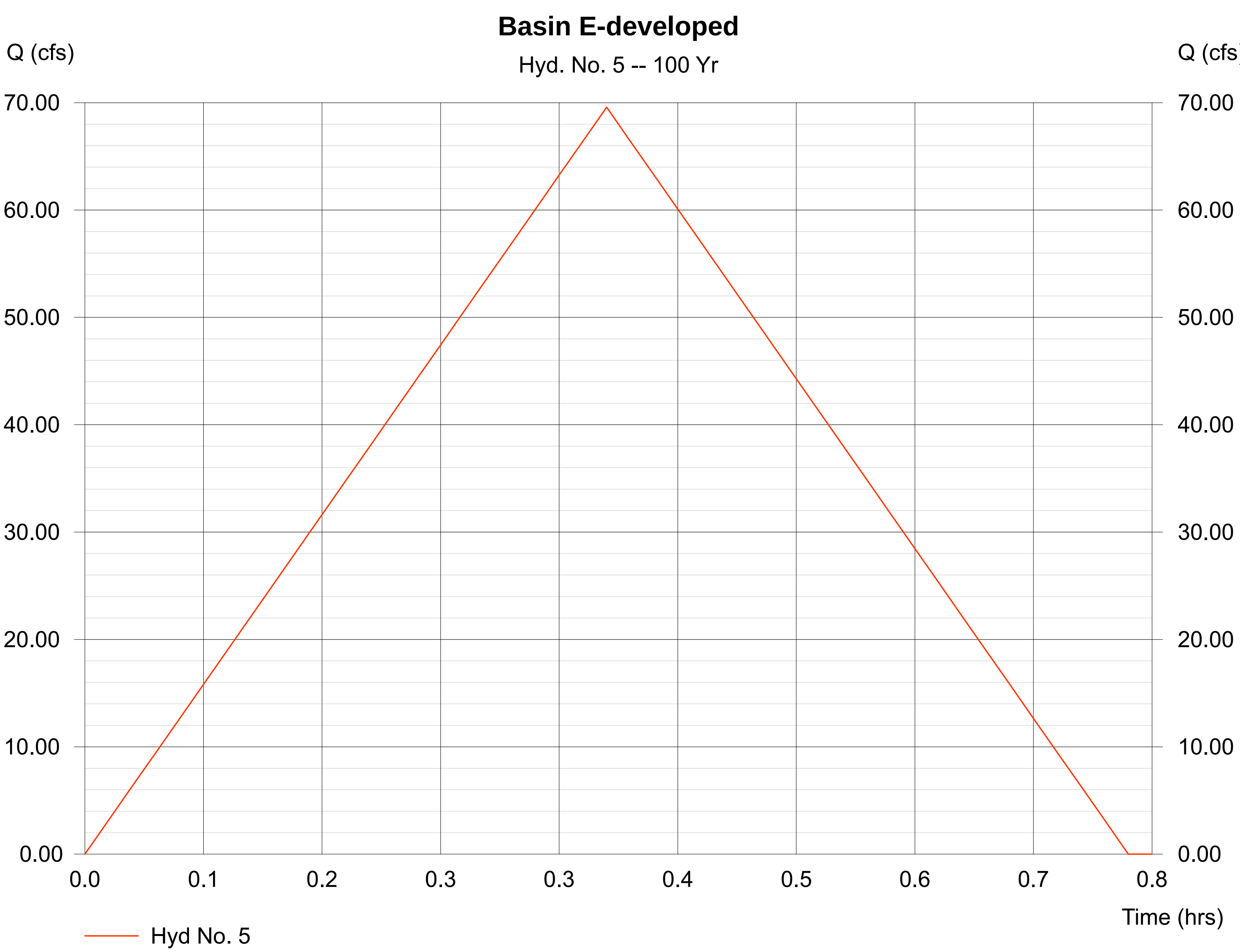
Thursday, Feb 14 2019, 9:46 AM

Hyd. No. 5

Basin E-developed

Hydrograph type	= Rational	Peak discharge	= 69.58 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Drainage area	= 21.200 ac	Runoff coeff.	= 0.65
Intensity	= 5.049 in/hr	Tc by User	= 22.00 min
IDF Curve	= 2016-idf curves-rls.IDF	Asc/Rec limb fact	= 1/1

Hydrograph Volume = 91,845 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

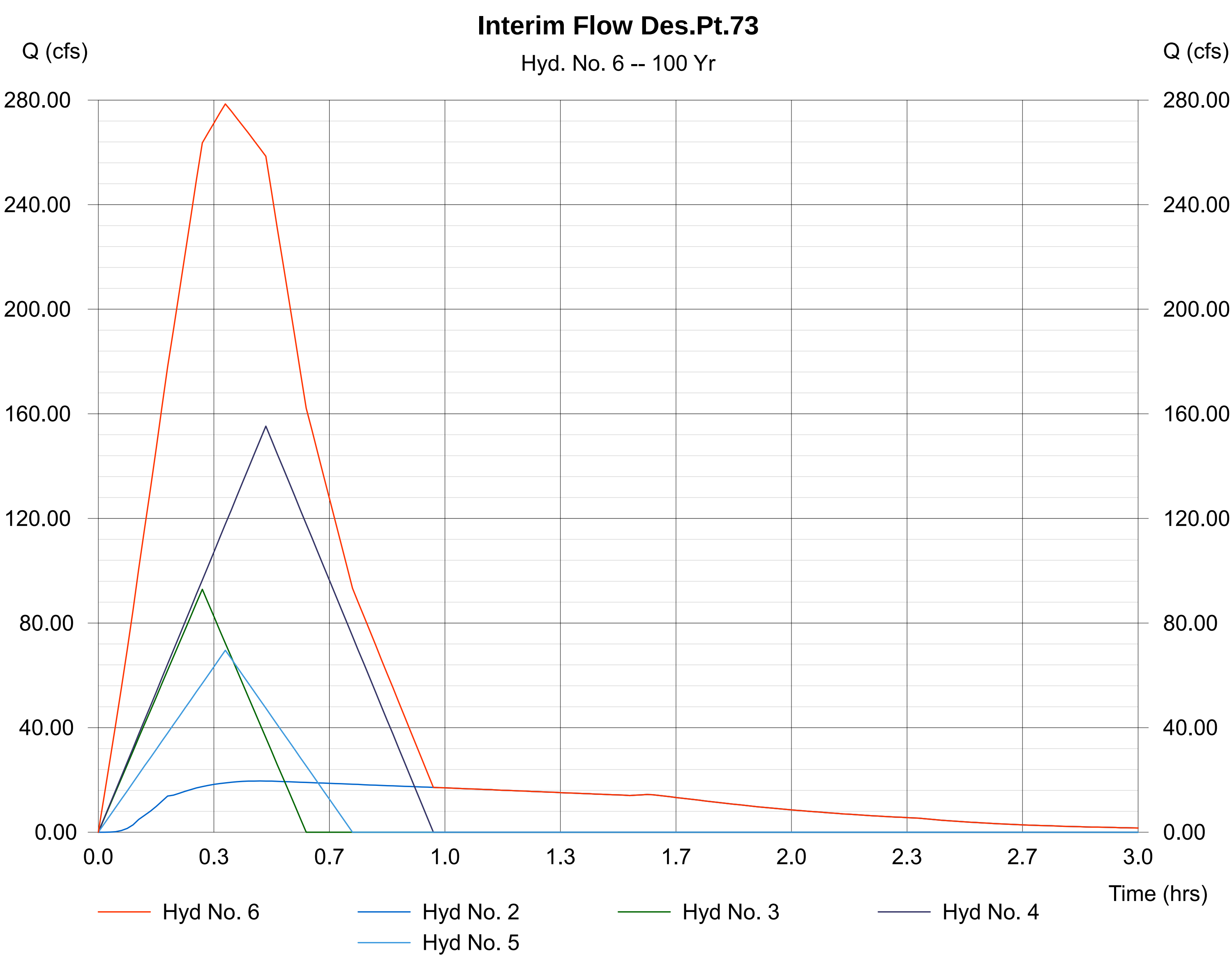
Thursday, Feb 14 2019, 9:46 AM

Hyd. No. 6

Interim Flow Des.Pt.73

Hydrograph type	= Combine	Peak discharge	= 278.53 cfs
Storm frequency	= 100 yrs	Time interval	= 1 min
Inflow hyds.	= 2, 3, 4, 5		

Hydrograph Volume = 590,528 cuft

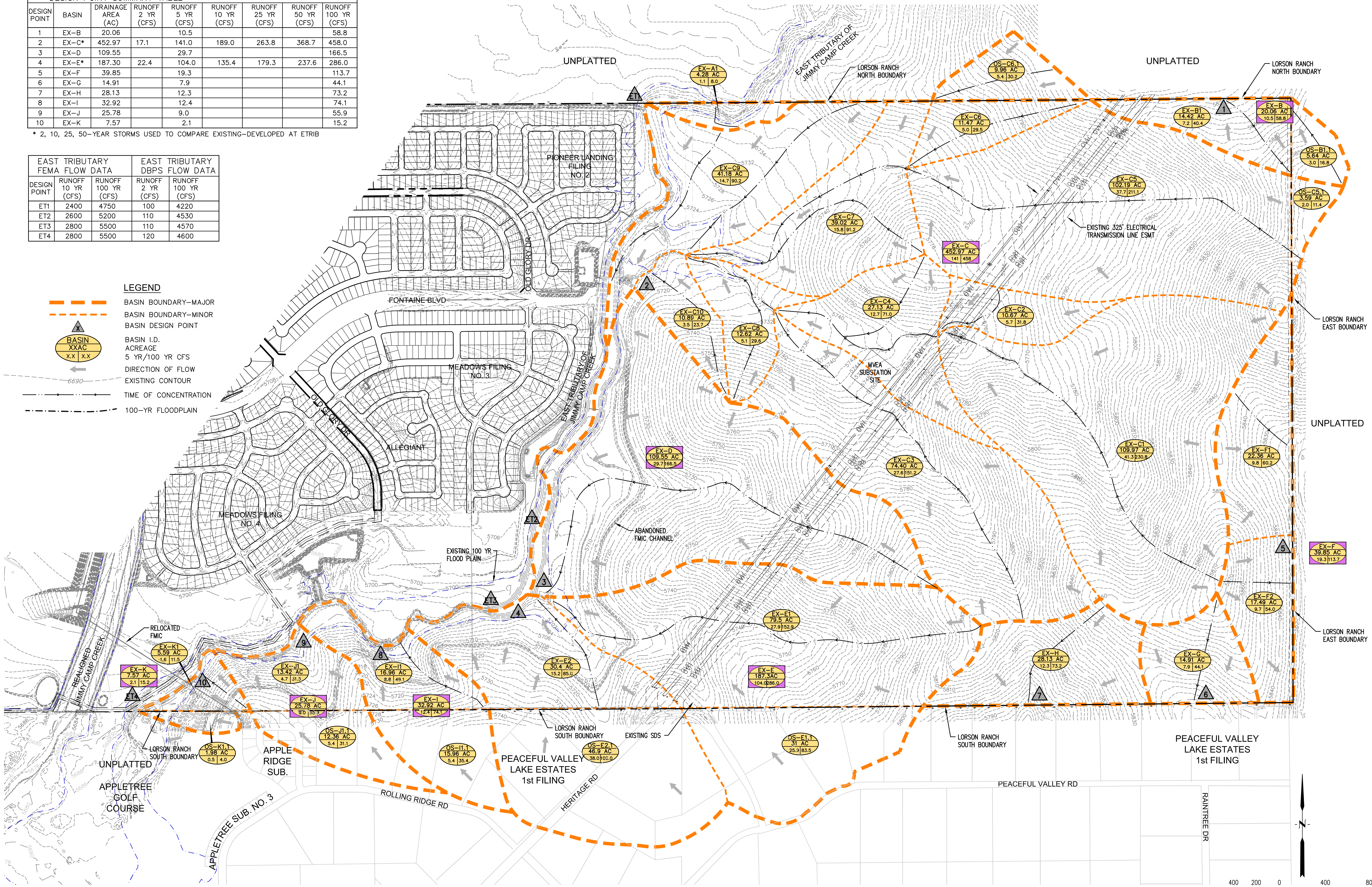


MAP POCKET

DESIGN POINT SUMMARY TABLE								
DESIGN POINT	BASIN	DRAINAGE AREA (AC)	RUNOFF 2 YR (CFS)	RUNOFF 5 YR (CFS)	RUNOFF 10 YR (CFS)	RUNOFF 25 YR (CFS)	RUNOFF 50 YR (CFS)	RUNOFF 100 YR (CFS)
1	EX–B	20.06		10.5				58.8
2	EX–C*	452.97	17.1	141.0	189.0	263.8	368.7	458.0
3	EX–D	109.55		29.7				166.5
4	EX–E*	187.30	22.4	104.0	135.4	179.3	237.6	286.0
5	EX–F	39.85		19.3				113.7
6	EX–G	14.91		7.9				44.1
7	EX–H	28.13		12.3				73.2
8	EX–I	32.92		12.4				74.1
9	EX–J	25.78		9.0				55.9
10	EX–K	7.57		2.1				15.2

* 2, 10, 25, 50–YEAR STORMS USED TO COMPARE EXISTING–DEVELOPED AT ETRIB

	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)
ET1	2400	4750	100	4220
ET2	2600	5200	110	4530
ET3	2800	5500	110	4570
ET4	2800	5500	120	4600



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BURNSVILLE, MN 55306
PH: 719.570.1100
CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

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

PROJECT:
LORSON RANCH EAST MDDP
EAST OF THE EAST TRIBUTARY
EL PASO COUNTY, COLORADO

DRAWN: LJA
DESIGNED: LAB
CHECKED: RLS

EXISTING CONDITIONS
LORSON RANCH EAST MDDP
EAST OF ETRIB OF JIMMY CAMP CREEK

DATE: NOV, 2017
PROJECT NO. 100.013
SHEET NUMBER 1
TOTAL SHEETS: 1

DESIGN POINT	DRAINAGE AREA	RUNOFF 5 YR (CFS)	RUNOFF 100 YR (CFS)	COMMENTS
DESIGN POINT SUMMARY TABLE				
1	9.54ac	8.5	28	
2	57.7ac	80.1	171.0	INFLOW POND C1
2a		4.0	18.0	POND C1 OUTFLOW
3a		11.0	63.3	POND C2.1 OUTFLOW
3b		31.0	111.0	INFLOW POND C2.3
3c		4.5	52.0	POND C2.3 OUTFLOW
3d		49.0	138.0	INFLOW POND C2.2
3e		6.7	61.0	POND C2.2 OUTFLOW
3f		14.0	127.0	FLOW IN PIPE
4a		12.4	40.5	POND C4 OUTFLOW
4	30	45.0	134.0	INFLOW POND C3
4b		5.0	18.0	POND C3 OUTFLOW
5		119.5	254.5	INFLOW POND C4
6a		68.1	151.8	
6b		75.7	192.3	
6c	93.2	90	222	INFLOW C16+C12+C13
7		88	273	
7a		114	337	INFLOW FROM FONTAINE
7b		167.5	519.1	INFLOW POND C5
7c		126.3	453.2	POND C5 OUTFLOW
8		45	95	
9a	23.0	38.0	84.5	
9b	36.4	61.7	137.5	
10a	89.0	118.2	277.1	INFLOW POND D2
10b		8.8	133.6	FROM POND D2
11		26.2	89.8	
12	56.5	77.6	194.8	INFLOW POND E1
12a		13.0	35.3	POND E1 OUTFLOW
13		78.0	205.0	
14		190.0	423.0	INFLOW POND E2
14a		97.0	266.0	FROM POND E2
15	33.0	48.0	102.9	INFLOW POND F
15a		3.5	40.9	FROM POND F
16	20.05	42.8	96.8	INFLOW POND G
16a		1.1	32.6	FROM POND G
17	22.3	41.2	93.2	INFLOW POND H
17a		1.7	35.3	FROM POND H
18		24.5	78.1	INFLOW POND I
18a		0.8	34.3	POND I OUTFLOW
19		20.4	65.0	INFLOW POND J
19a		1.8	31.5	FROM POND J
20		6.4	20.5	INFLOW POND K
20a		0.2	10.5	FROM POND K

DESIGN POINT	EAST TRIBUTARY FEMA FLOW DATA RUNOFF 10 YR (CFS)	EAST TRIBUTARY FEMA FLOW DATA RUNOFF 100 YR (CFS)	EXISTING FLOW DATA FROM JCC DBPS RUNOFF 2 YR (CFS)	EXISTING FLOW DATA FROM JCC DBPS RUNOFF 100 YR (CFS)
ET1	2400	4750	100	4220
ET2	2600	5200	110	4530
ET3	2800	5500	110	4570
ET4	2800	5500	120	4600

LEGEND

BASIN BOUNDARY-MAJOR

BASIN BOUNDARY-MINOR

PROPOSED CONTOURS

BASIN DESIGN POINT

BASIN I.D.
ACREAGE
5 YR/100 YR CFS

DIRECTION OF FLOW

EXISTING CONTOUR

100-YR FLOODPLAIN

HP HIGH POINT

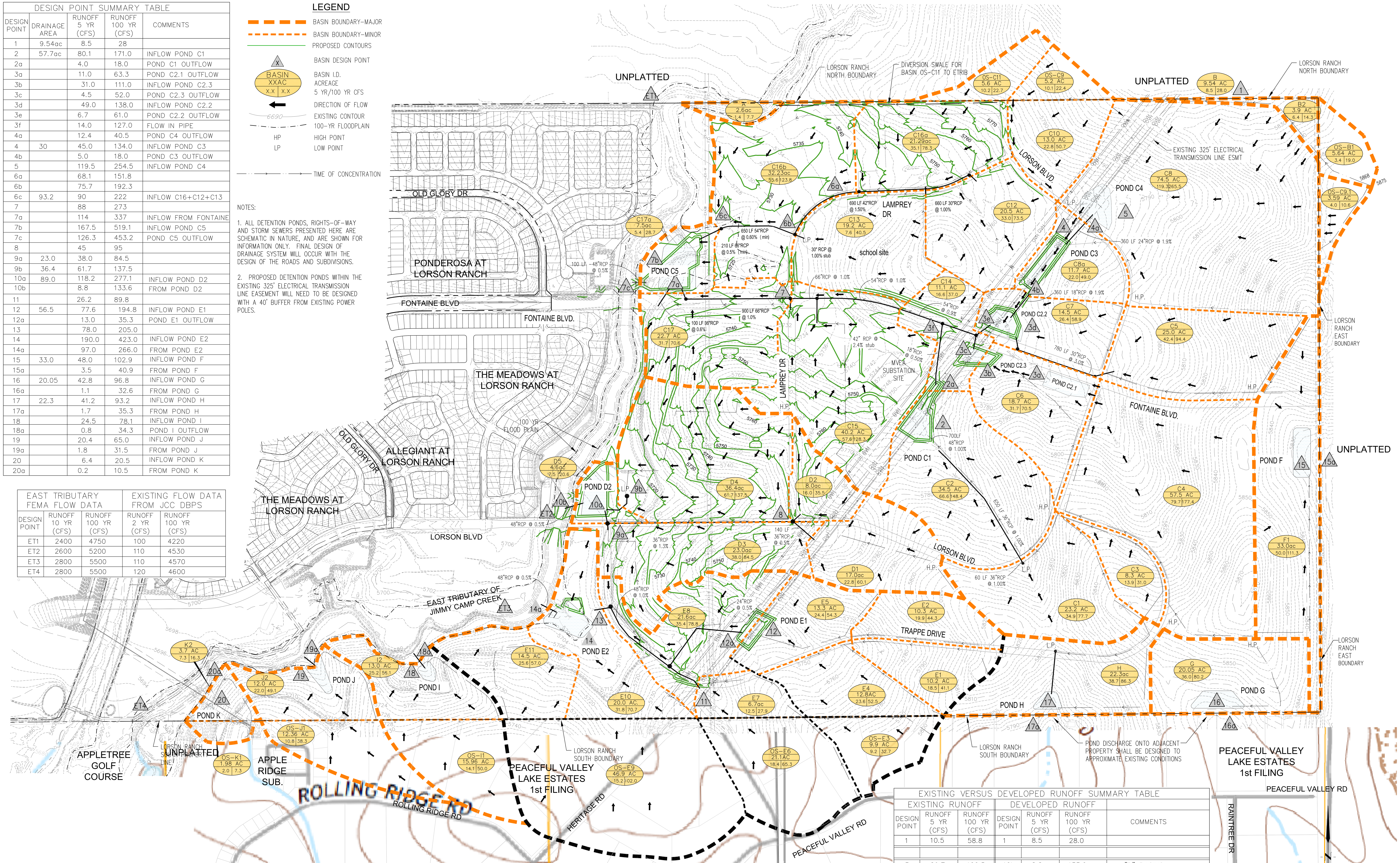
LP LOW POINT

TIME OF CONCENTRATION

NOTES:

1. ALL DETENTION PONDS, RIGHTS-OF-WAY AND STORM SEWERS PRESENTED HERE ARE SCHEMATIC IN NATURE, AND ARE SHOWN FOR INFORMATION ONLY. FINAL DESIGN OF DRAINAGE SYSTEM WILL OCCUR WITH THE DESIGN OF THE ROADS AND SUBDIVISIONS.

2. PROPOSED DETENTION PONDS WITHIN THE EXISTING 325' ELECTRICAL TRANSMISSION LINE EASEMENT WILL NEED TO BE DESIGNED WITH A 40' BUFFER FROM EXISTING POWER POLES.



DESIGN POINT	RUNOFF 2 YR (CFS)	RUNOFF 5 YR (CFS)	RUNOFF 10 YR (CFS)	RUNOFF 25 YR (CFS)	RUNOFF 50 YR (CFS)	RUNOFF 100 YR (CFS)	DESIGN POINT	RUNOFF 2 YR (CFS)	RUNOFF 5 YR (CFS)	RUNOFF 10 YR (CFS)	RUNOFF 25 YR (CFS)	RUNOFF 50 YR (CFS)	RUNOFF 100 YR (CFS)	COMMENTS
EXISTING VERSUS DEVELOPED RUNOFF SUMMARY TABLE WITH FULL SPECTRUM PONDS IN SERIES														
EXISTING RUNOFF							DEVELOPED RUNOFF							
2	17.1	141.0	189.0	263.8	368.7	458.0	7c	1.8	126.3	166.1	238.3	296.5	453.2	"C" BASINS
4	22.4	104.0	135.4	179.3	237.6	286.0	14a	22.3	97.0	114.1	152.4	212.8	266.0	"E" BASINS

DESIGN POINT	RUNOFF 5 YR (CFS)	RUNOFF 100 YR (CFS)	DESIGN POINT	RUNOFF 5 YR (CFS)	RUNOFF 100 YR (CFS)	COMMENTS
EXISTING VERSUS DEVELOPED RUNOFF SUMMARY TABLE						
EXISTING RUNOFF			DEVELOPED RUNOFF			
1	10.5	58.8	1	8.5	28.0	
3	29.7	166.5	10b	8.8	133.6	"D" BASINS
5	19.3	113.7	15a	3.5	40.9	BASIN F
6	7.9	44.1	16a	1.1	32.6	BASIN G
7	12.3	73.2	17a	1.7	35.3	BASIN H
8	12.4	74.1	18a	0.8	34.3	BASIN I
9	9.0	55.9	19a	1.8	31.5	BASIN J
10	2.1	15.2	20a	0.2	10.5	BASIN K

CORE ENGINEERING GROUP
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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: DAVE COCOLIN

PROJECT:
LORSON RANCH EAST MDDP
EAST OF THE EAST TRIBUTARY
EL PASO COUNTY, COLORADO

DRAWN: LJA
DESIGNED: LAB
CHECKED: RLS

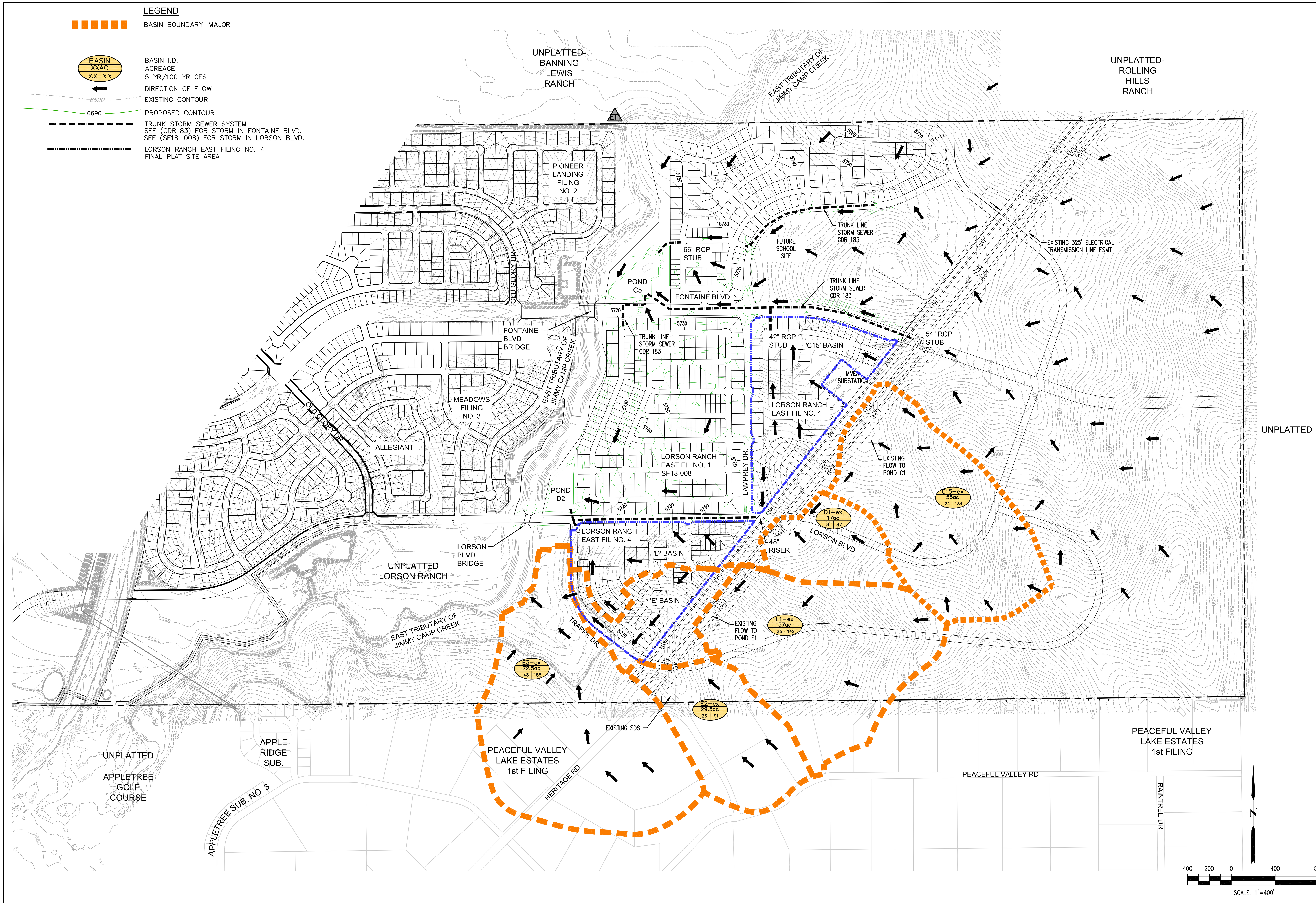
OVERALL DEVELOPED CONDITIONS
LORSON RANCH EAST MDDP
EAST OF EAST TRIBUTARY OF JCC

DATE: NOV, 2017

PROJECT NO. 100.013

SHEET NUMBER 1

TOTAL SHEETS: 5



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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:
LORSON RANCH EAST 4
EAST OF THE EAST TRIBUTARY
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: RLS
CHECKED: RLS

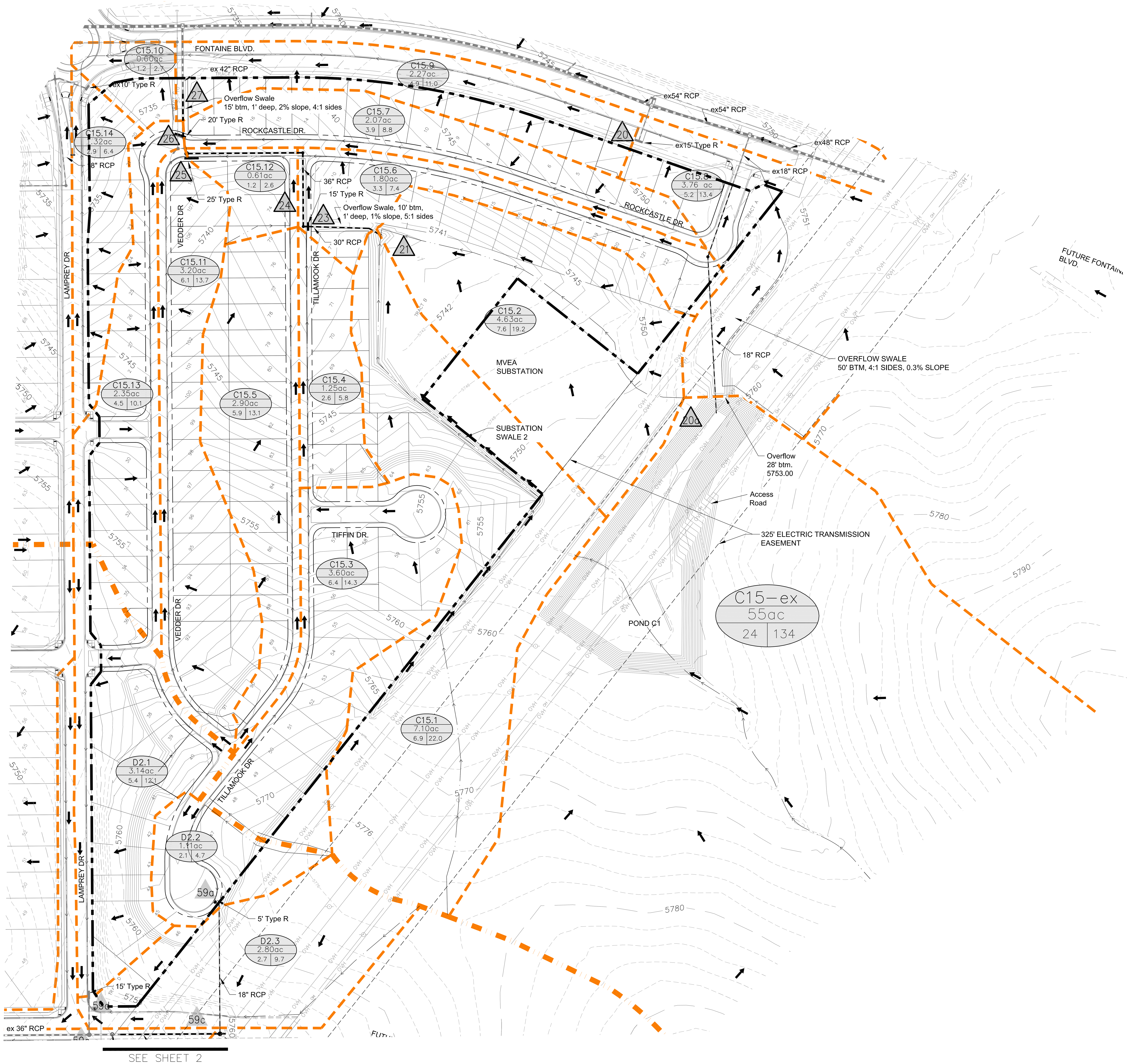
INTERIM HYDROLOGICAL CONDITIONS
LORSON RANCH EAST FILING NO. 4
OFFSITE DRAINAGE BASINS

DATE:
FEBRUARY 28, 2019

PROJECT NO.
100.048

SHEET NUMBER
1

TOTAL SHEETS: 1



LEGEND

DRAINAGE MAJOR BASIN BOUNDARY

DRAINAGE MINOR BASIN BOUNDARY (OFF–SITE)

SITE BOUNDARY

BASIN I.D. / ACREAGE / 5 YR/100 YR CFS (BASIN XX AC X.X X.X)

DIRECTION OF FLOW

EXISTING CONTOUR (6690)

PROPOSED CONTOUR (6670)

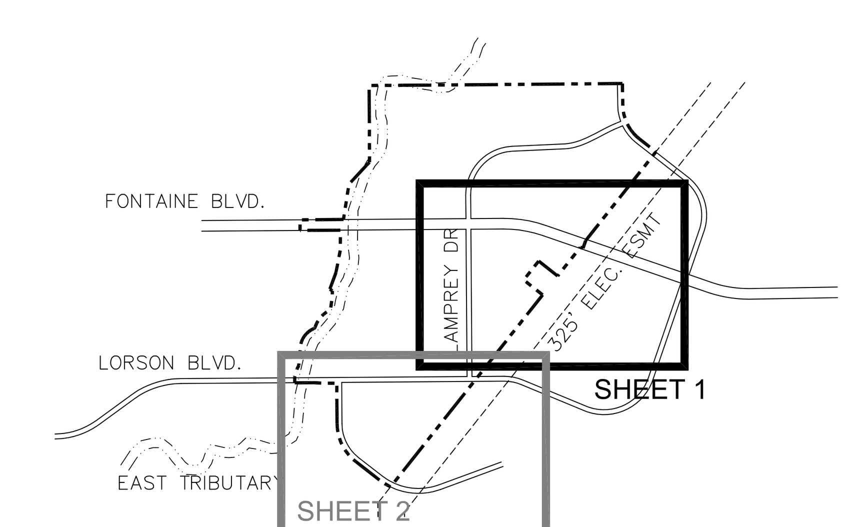
HP HIGH POINT

LP LOW POINT

TIME OF CONCENTRATION

100–YR FLOODPLAIN

RUNOFF SUMMARY			
DESIGN POINT	5 YEAR	100 YEAR	NOTES
20	5.20	13.40	STREET FLOW
20a	4.0	18.0	STM SWR FROM POND C1
21	13.20	37.90	FLOW IN STM SWR
23	8.70	19.30	STREET FLOW
24	20.30	51.80	FLOW IN STM SWR
25	15.90	38.80	STREET FLOW
26	8.40	26.00	STREET FLOW
27	38.10	92.60	FLOW IN STM SWR
32	23.58	163.4	FLOW IN STM SWR
59a	2.10	4.70	STREET FLOW
59c	25.70	75.40	FLOW IN STM SWR



KEY MAP
NO SCALE

100 50 0 100 200
SCALE: 1"=100'

CORE ENGINEERING GROUP
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NO.	DESCRIPTION	DATE

PREPARED FOR:
LORSON, LLC
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COLORADO SPRINGS, COLORADO 80903
(719) 635–3200
CONTACT: JEFF MARK

PROJECT:
LORSON RANCH EAST 4
FONTAINE BLVD. – EAST TRIBUTARY OF JCC
EL PASO COUNTY, COLORADO

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

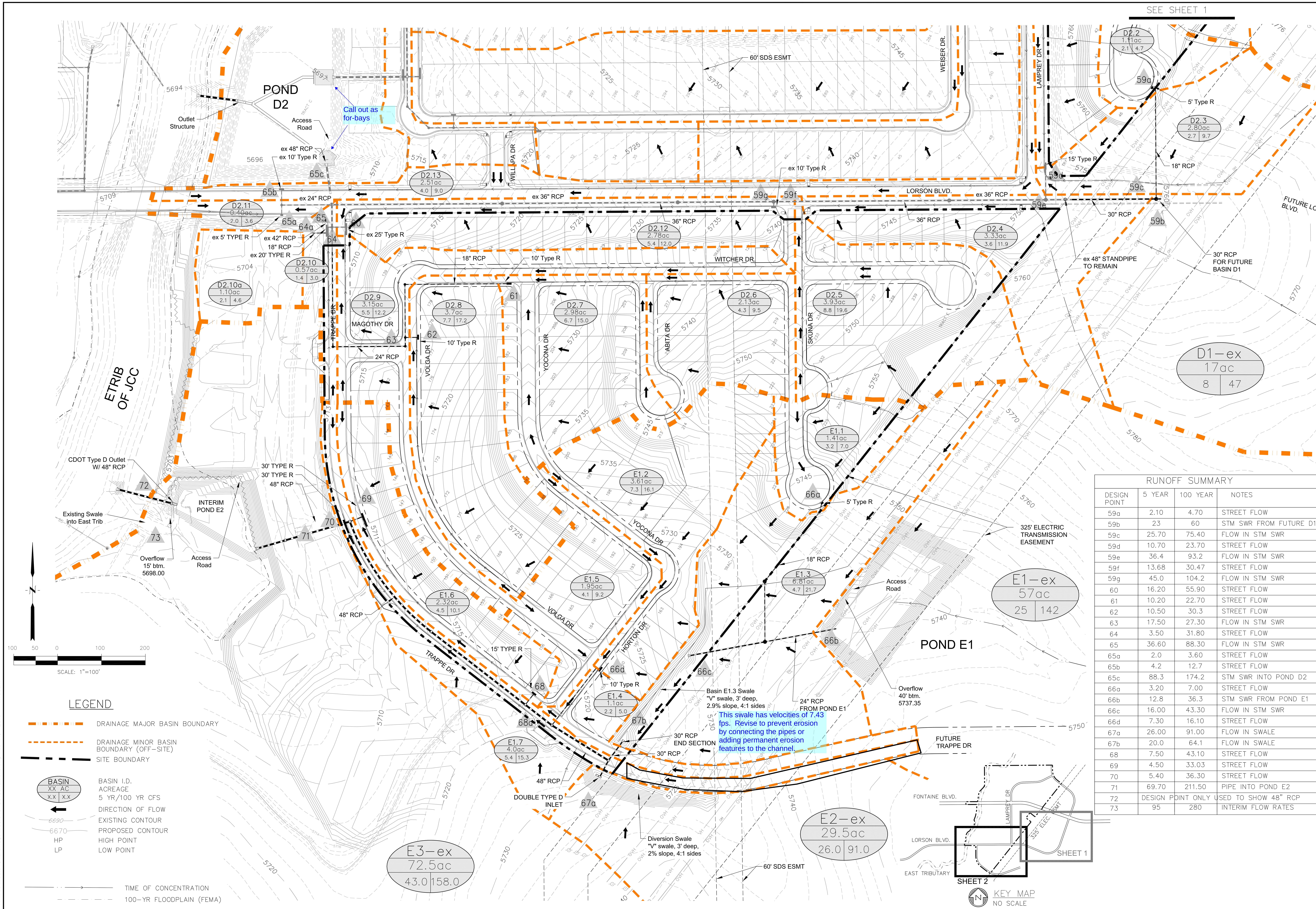
DEVELOPED CONDITIONS DRAINAGE PLAN - NORTH AREA LORSON RANCH EAST FILING NO. 4

DATE
FEBRUARY 28, 2019

PROJECT NO.
100.048

SHEET NUMBER
1

TOTAL SHEETS: 2



RUNOFF SUMMARY

DESIGN POINT	5 YEAR	100 YEAR	NOTES
59a	2.10	4.70	STREET FLOW
59b	23	60	STM SWR FROM FUTURE D1
59c	25.70	75.40	FLOW IN STM SWR
59d	10.70	23.70	STREET FLOW
59e	36.4	93.2	FLOW IN STM SWR
59f	13.68	30.47	STREET FLOW
59g	45.0	104.2	FLOW IN STM SWR
60	16.20	55.90	STREET FLOW
61	10.20	22.70	STREET FLOW
62	10.50	30.3	STREET FLOW
63	17.50	27.30	FLOW IN STM SWR
64	3.50	31.80	STREET FLOW
65	36.60	88.30	FLOW IN STM SWR
65a	2.0	3.60	STREET FLOW
65b	4.2	12.7	STREET FLOW
65c	88.3	174.2	STM SWR INTO POND D2
66a	3.20	7.00	STREET FLOW
66b	12.8	36.3	STM SWR FROM POND E1
66c	16.00	43.30	FLOW IN STM SWR
66d	7.30	16.10	STREET FLOW
67a	26.00	91.00	FLOW IN SWALE
67b	20.0	64.1	FLOW IN SWALE
68	7.50	43.10	STREET FLOW
69	4.50	33.03	STREET FLOW
70	5.40	36.30	STREET FLOW
71	69.70	211.50	PIPE INTO POND E2
72	DESIGN POINT ONLY USED TO SHOW 48" RCP		
73	95	280	INTERIM FLOW RATES

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PREPARED FOR: LORSON, LLC
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COLORADO SPRINGS, COLORADO 80903
(719) 635-3200
CONTACT: JEFF MARK

PROJECT: LORSON RANCH EAST 4
FONTAINE BLVD. - EAST TRIBUTARY OF JCC
EL PASO COUNTY, COLORADO

NO. | DATE | DESCRIPTION

DRAWN: RLS
DESIGNED: LAB
CHECKED: LAB

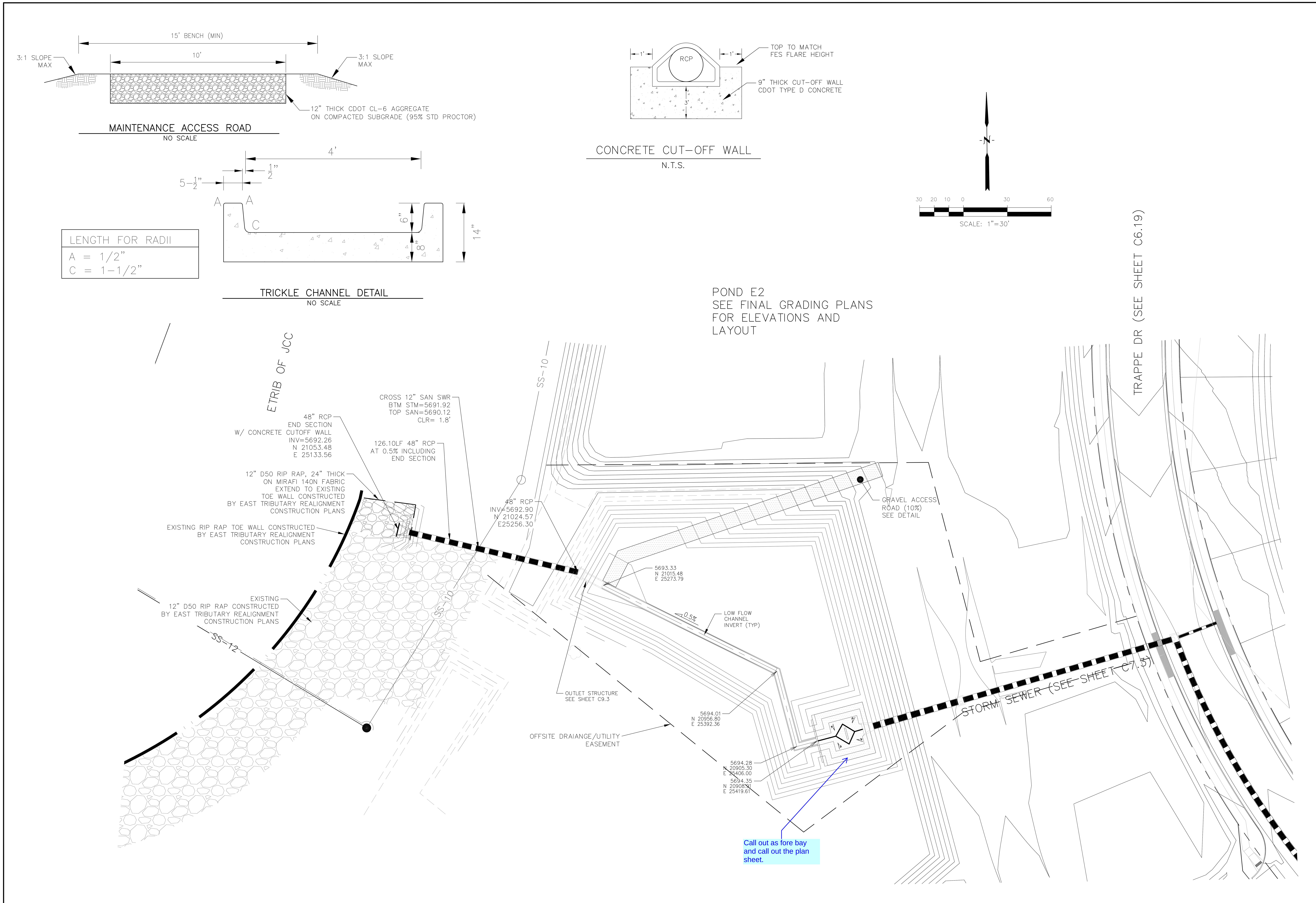
DEVELOPED CONDITIONS DRAINAGE PLAN - SOUTH AREA LORSON RANCH EAST FILING NO. 4

DATE: FEBRUARY 28, 2019

PROJECT NO. 100.048

SHEET NUMBER 2

TOTAL SHEETS: 2



CORE
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BURNSVILLE, MN 55306
PH: 719.570.1100
CONTACT: RICHARD L. SCHINDLER, P.E.
EMAIL: Rich@ceg1.com

NO.	DESCRIPTION	DATE

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

PROJECT:
LORSON RANCH EAST
FILING NO. 4
LORSON BLVD - LAMPREY DR
COLORADO SPRINGS, COLORADO

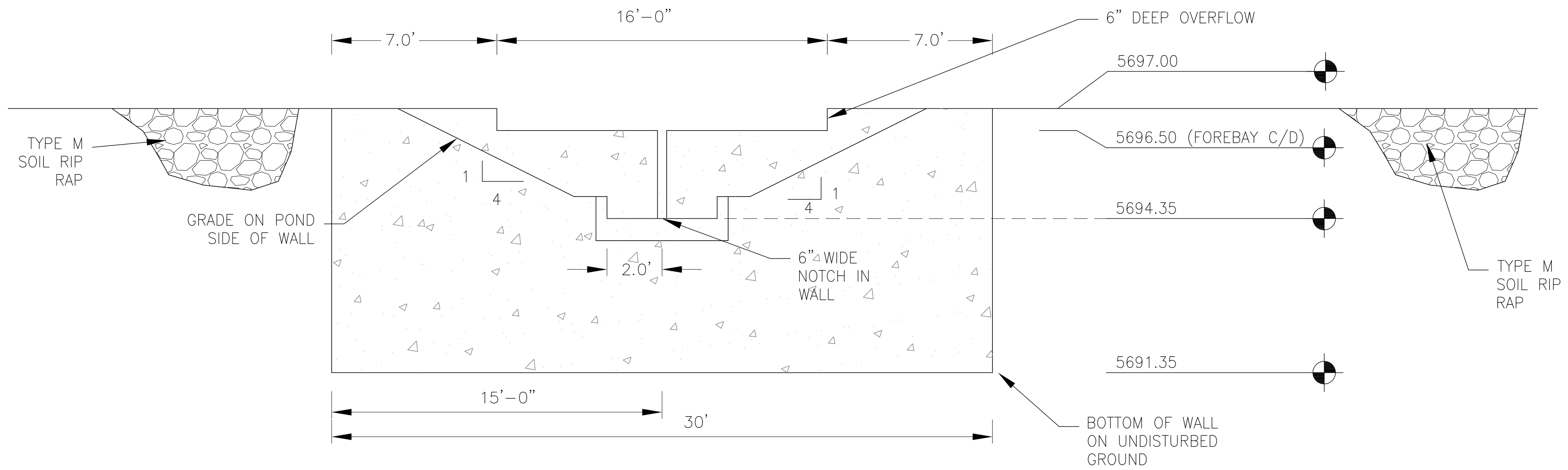
DRAWN: RLS
DESIGNED: RLS
CHECKED: RLS

POND E2
POND E2 OUTFALL AND TRICKLE CHANNEL

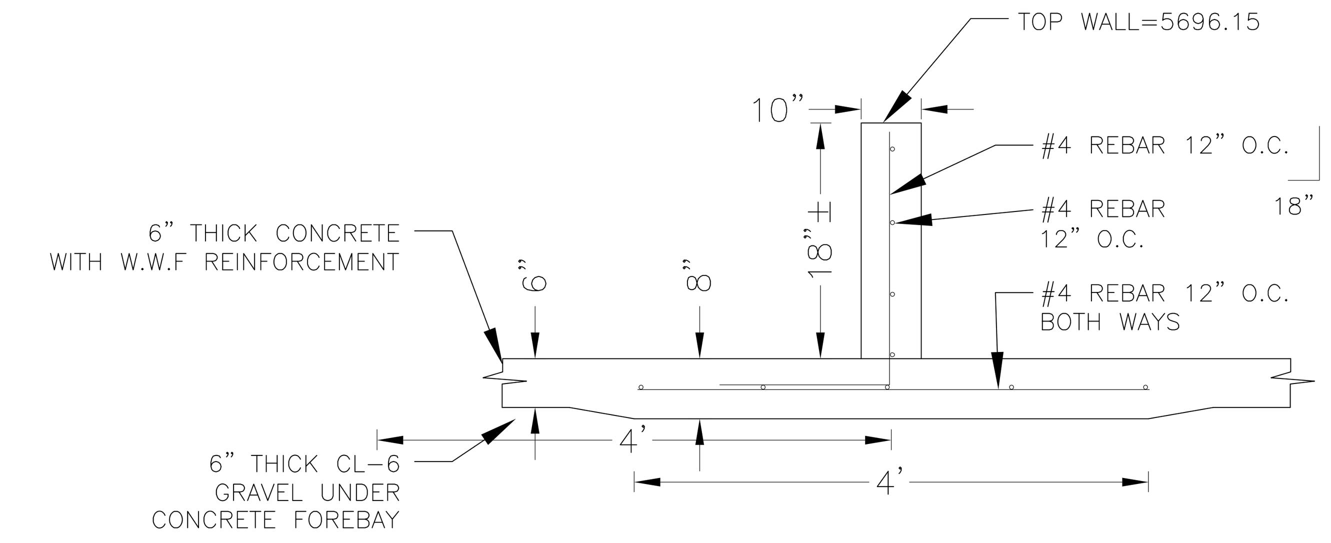
DATE:
FEBRUARY 28, 2019

PROJECT NO.
100.048

SHEET NUMBER
C9.1
TOTAL SHEETS: 40

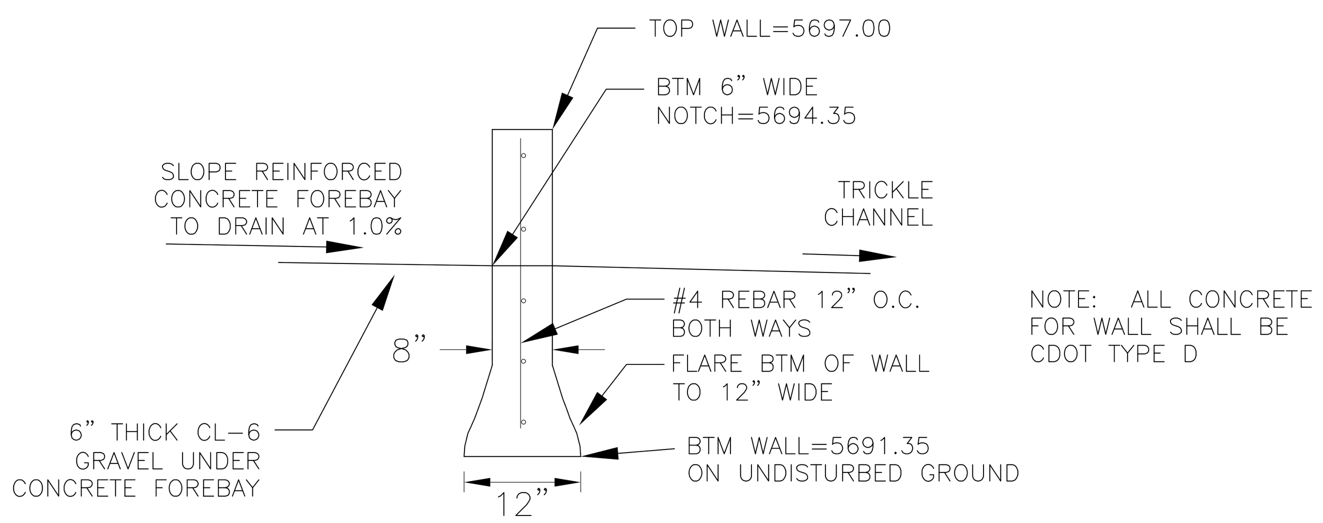


WALL SECTION A–A
1"=10'



ENERGY DISSIPATER WALL
NO SCALE

NOTE: ALL CONCRETE FOR WALL SHALL BE CDOT TYPE D

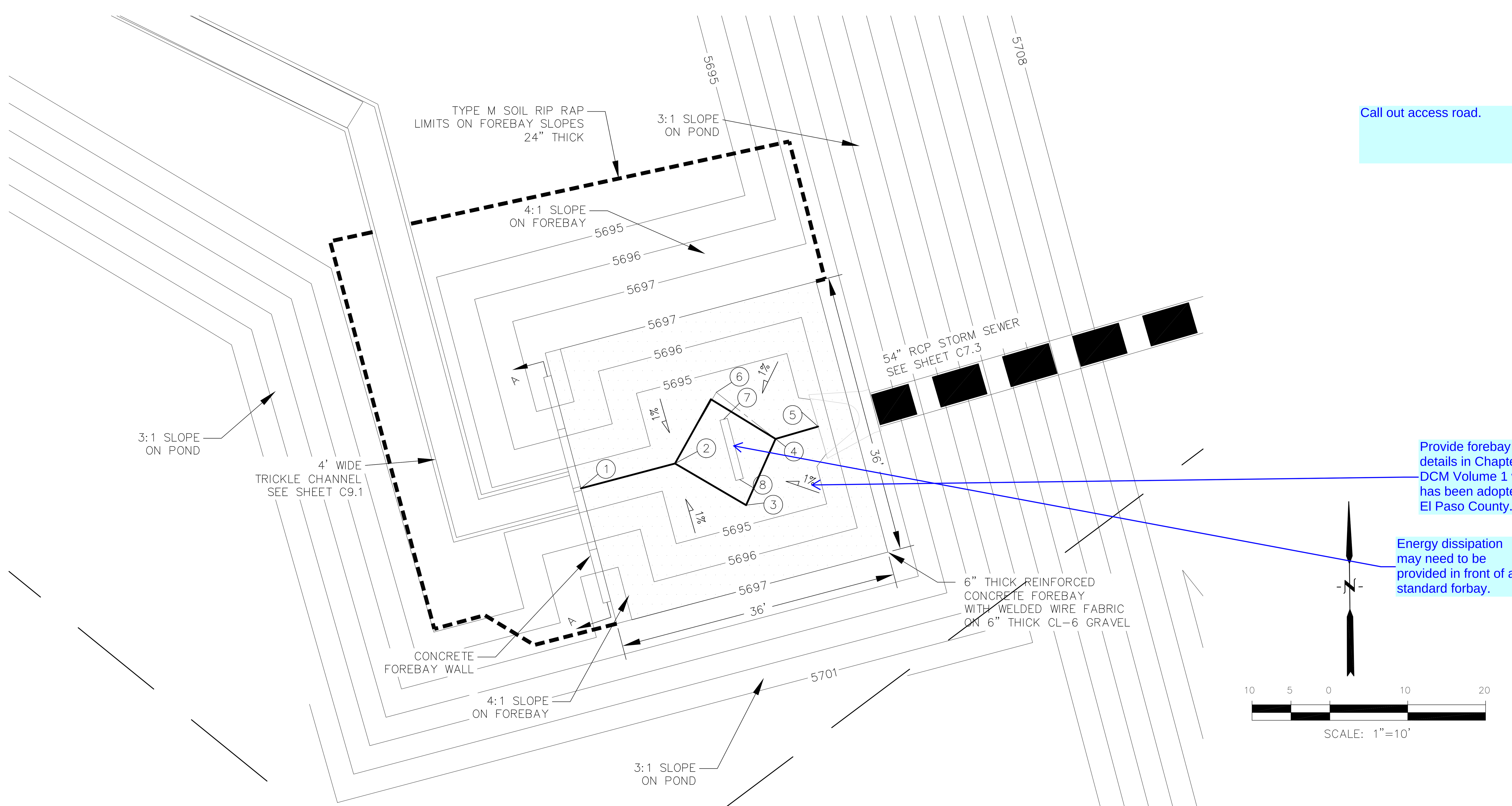


WALL SECTION B–B
NO SCALE

NOTE: ALL CONCRETE FOR WALL SHALL BE CDOT TYPE D

POINT TABLE (FOREBAY)

NUMBER	NORTHING	EASTING	ELEVATION	NOTES
1	20909.17	25420.57	5694.35	FOREBAY BOTTOM
2	20912.36	25432.66	5694.47	FOREBAY BOTTOM
3	20907.02	25441.78	5694.58	FOREBAY BOTTOM
4	20915.53	25445.57	5694.67	FOREBAY BOTTOM
5	20917.11	25451.04	5694.90	FOREBAY BOTTOM, INV 54"=5695.50
6	20921.52	25437.96	5694.58	FOREBAY BOTTOM
7	20918.05	25438.91	5694.65	ENERGY DISSIPATER WALL
8	20910.31	25440.95	5694.65	ENERGY DISSIPATER WALL



POND E2 – FOREBAY LAYOUT
SCALE: 1"=10'

Call out access road.

Provide forebay per details in Chapter 13 DCM Volume 1 which has been adopted by El Paso County.

Energy dissipation may need to be provided in front of a standard forbay.

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

NO.	DATE	DESCRIPTION

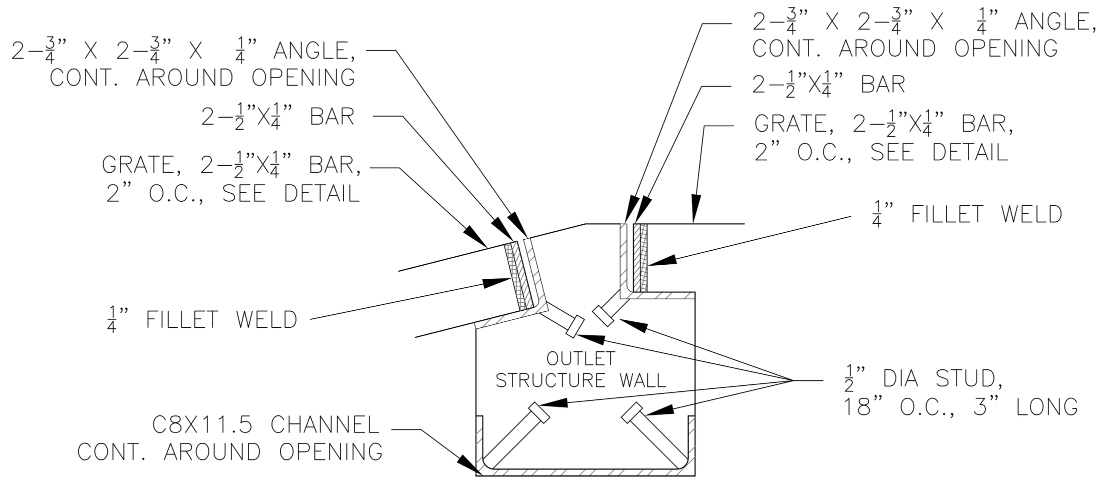
PREPARED FOR:
LORSON, LLC
212 N. WAHSATCH AVE, SUITE 301
COLORADO SPRINGS, COLORADO 80903
(719) 635-3200
CONTACT: JEFF MARK

PROJECT:
LORSON RANCH EAST FILING NO. 4
LORSON BLVD – LAMPREY DR
COLORADO SPRINGS, COLORADO

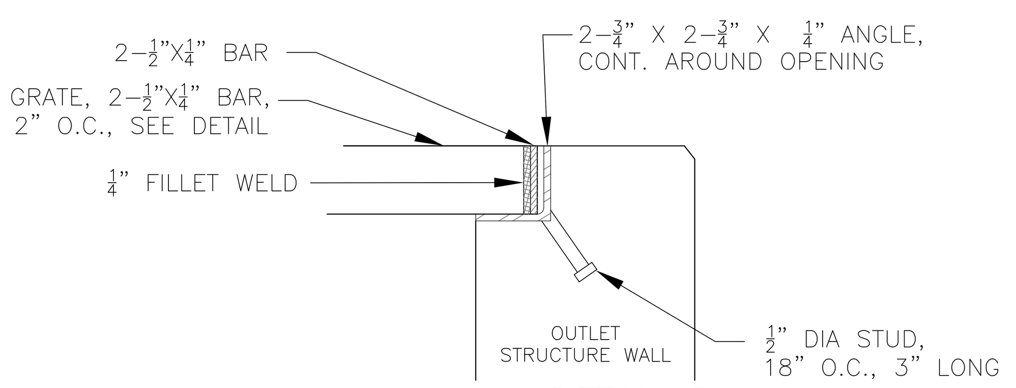
DRAWN: RLS
DESIGNED: RLS
CHECKED: RLS

POND E2
FOREBAY DETAILS

DATE: FEBRUARY 28, 2019
PROJECT NO. 100.048
SHEET NUMBER C9.2
TOTAL SHEETS: 40

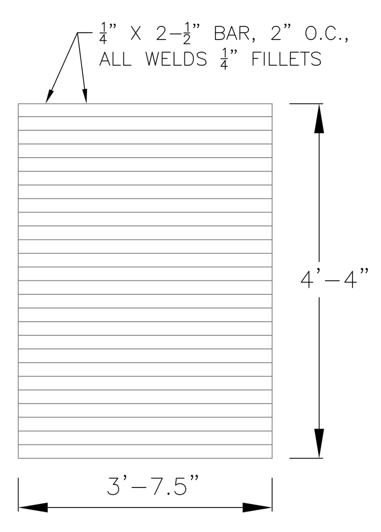


DETAIL A
NO SCALE

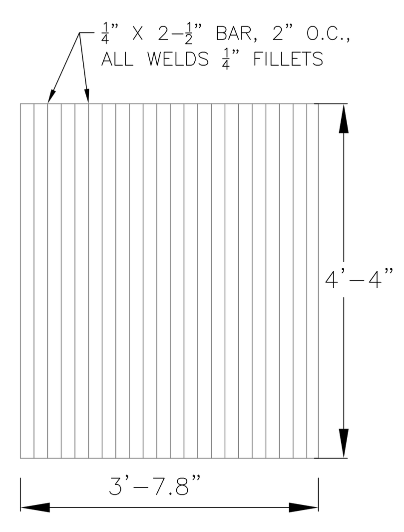


DETAIL B
NO SCALE

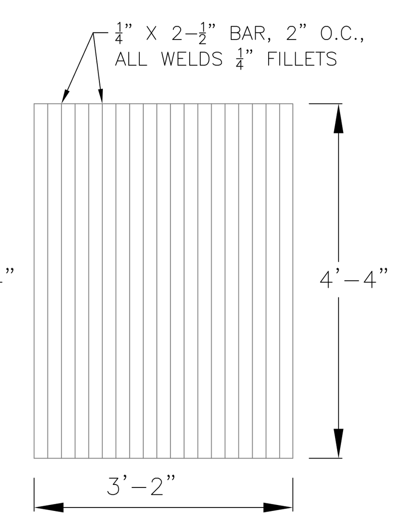
NOTE:
AFTER CONCRETE STRUCTURE HAS BEEN POURED ALL GRATE DIMENSIONS SHALL BE FIELD VERIFIED PRIOR TO GRATE CONSTRUCTION



GRATE 1
NO SCALE



GRATES 2–4
NO SCALE



GRATES 5–6
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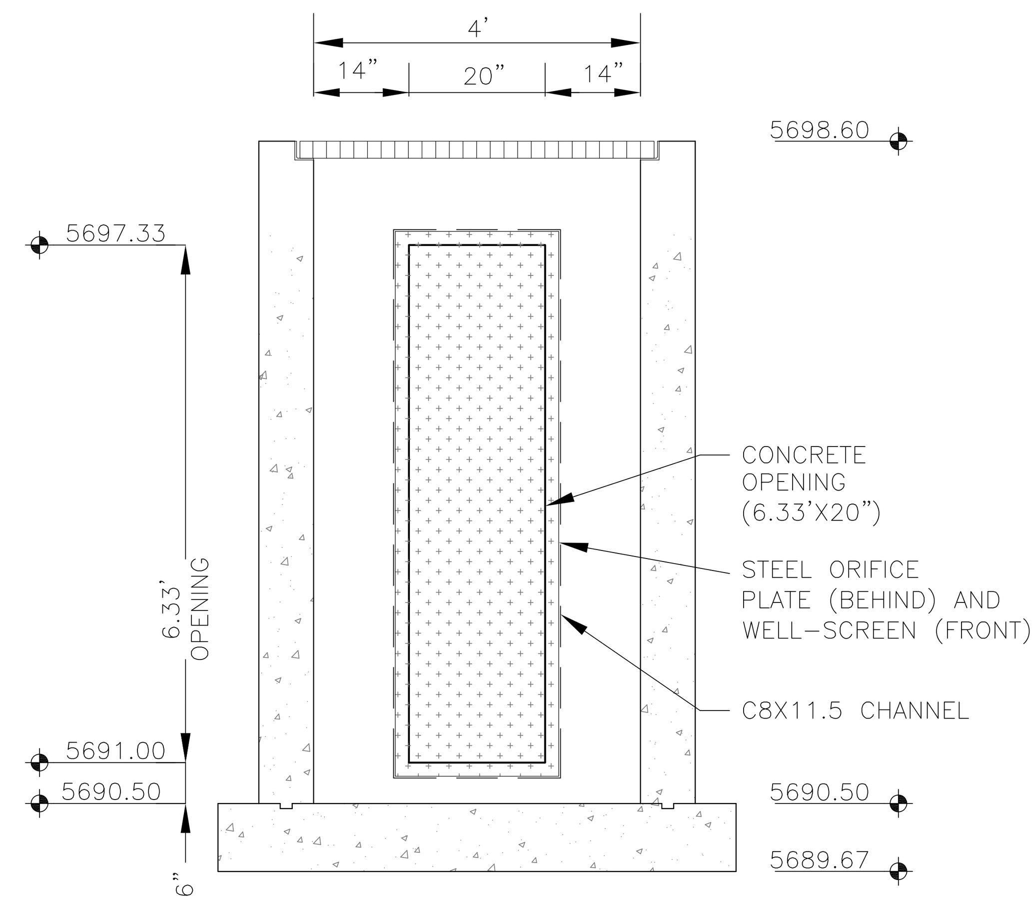
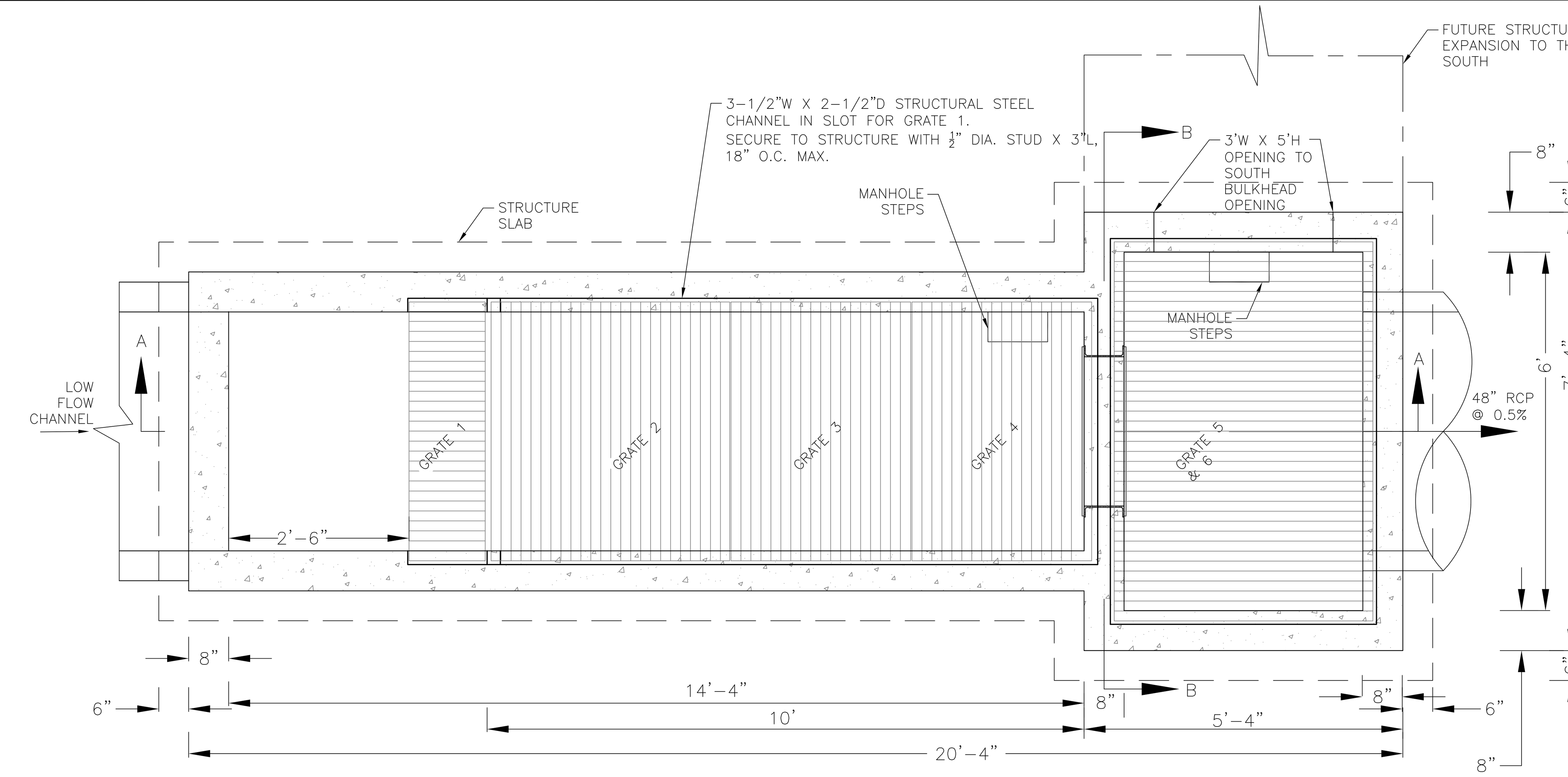
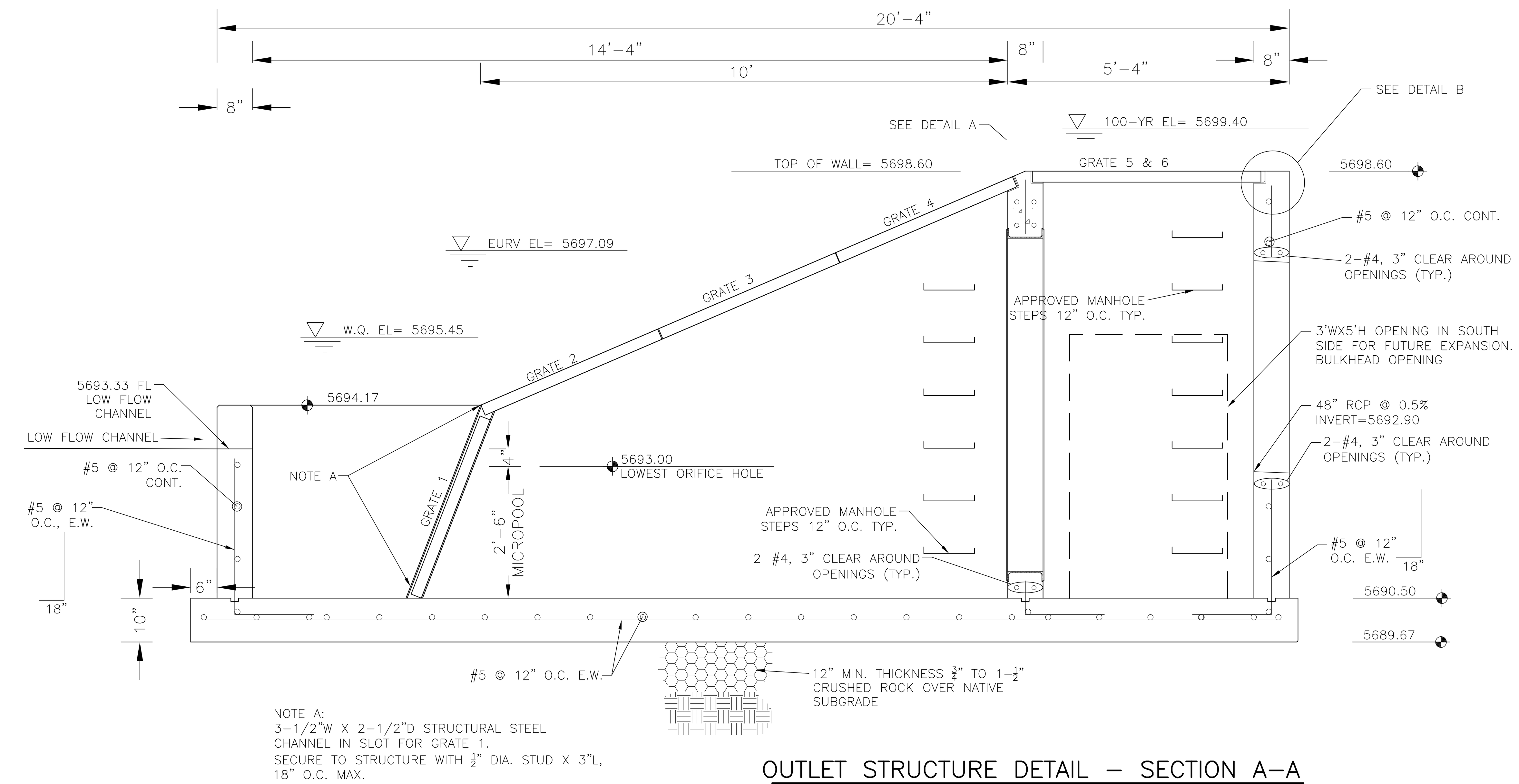


OUTLET STRUCTURE DETAIL – SECTION B–B
NO SCALE

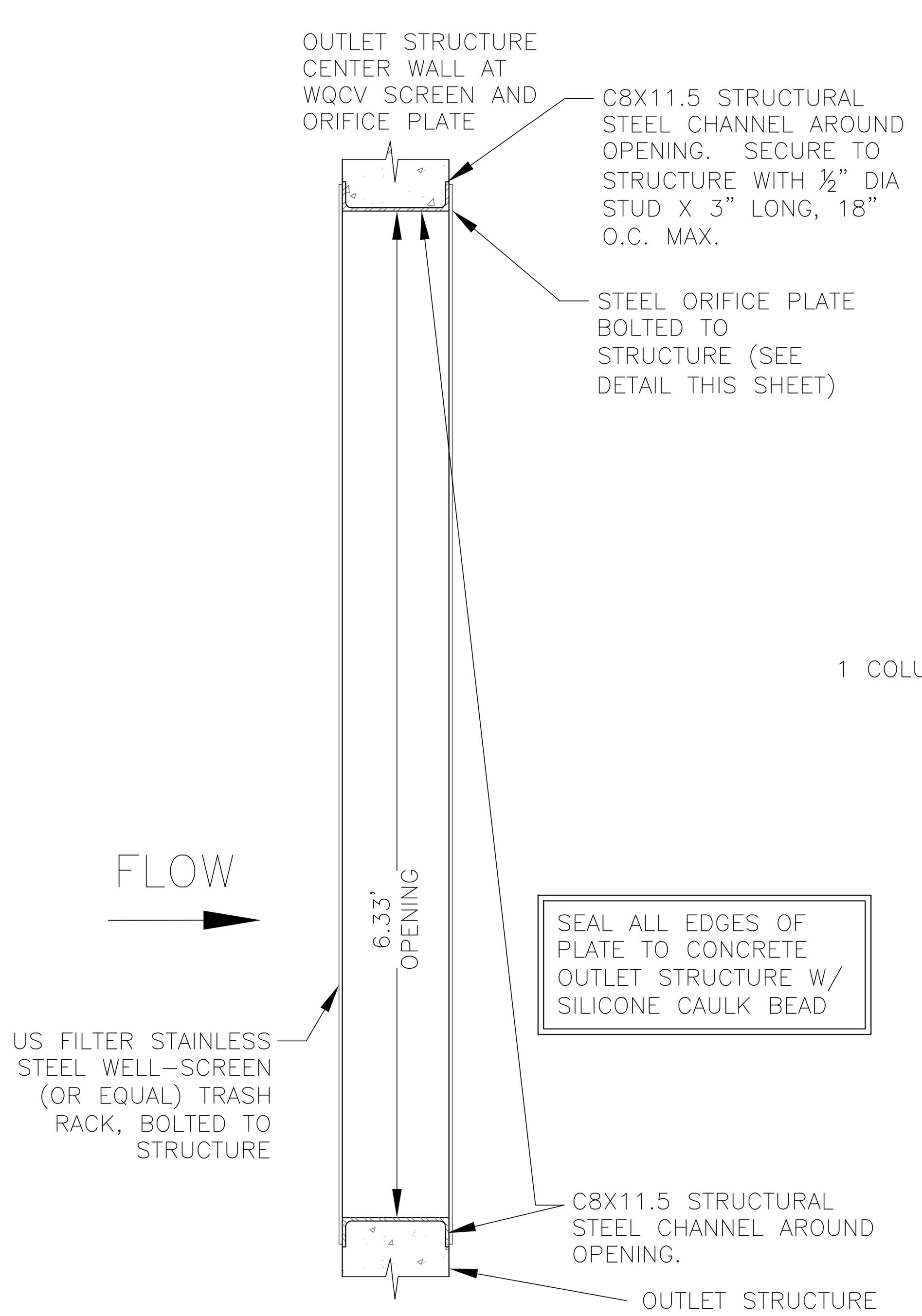


OUTLET STRUCTURE DETAIL – PLAN VIEW
NO SCALE

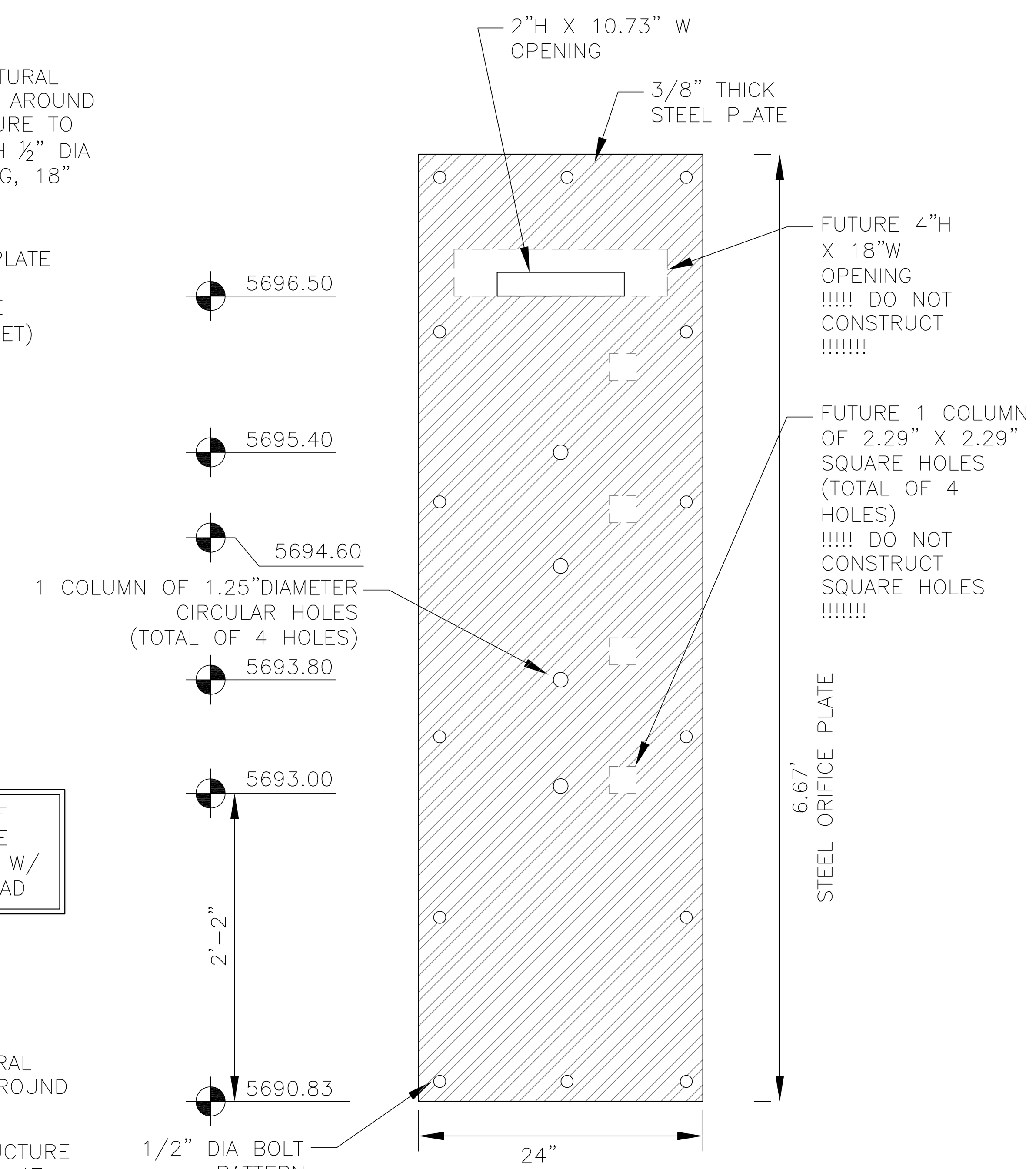


NOTE A:
3-1/2"W X 2-1/2"D STRUCTURAL STEEL CHANNEL IN SLOT FOR GRATE 1. SECURE TO STRUCTURE WITH 1/2" DIA. STUD X 3"L, 18" O.C. MAX.

OUTLET STRUCTURE DETAIL – SECTION A–A
NO SCALE



TRASH RACK DETAIL
NO SCALE



ORIFICE PLATE DETAIL
NO SCALE

OUTLET STRUCTURE, FOREBAY, AND DRAIN CHANNEL NOTES:

1. PRIOR TO CONSTRUCTION, CONTRACTOR SHALL PROVIDE SHOP DRAWINGS FOR ALL COMPONENTS OF THE OUTLET STRUCTURE.

2. GRADE 60 REINFORCING STEEL REQUIRED. SEE TABLE FOR THE MINIMUM LAP SPLICE LENGTH FOR REINFORCING BARS. ALL REINFORCING STEEL SHALL HAVE A TWO-INCH MINIMUM CLEARANCE FROM EDGE OF CONCRETE, UNLESS OTHERWISE NOTED.

BAR SIZE	#4	#5	#6
MIN. SPLICE LENGTH	1'-3"	1'-7"	2'-0"

3. CONCRETE FOR THE OUTLET STRUCTURE AND FOREBAY SHALL BE CDOT CLASS D CONCRETE.

4. CONCRETE FOR DRAIN CHANNELS SHALL BE CDOT CLASS B CONCRETE

5. EXPANSION JOINT MATERIAL SHALL MEET AASHTO SPECIFICATION M-213. EXPANSION JOINT MATERIAL SHALL BE 1/2" THICK, SHALL EXTEND THE FULL DEPTH OF CONTACT SURFACE AND THE JOINT SHALL BE SEALED, REFER TO DETAILS.
6. ALL EXPOSED CONCRETE CORNERS SHALL HAVE A 3/8" CHAMFER UNLESS OTHERWISE NOTED.

7. SUBGRADE TO BE 12" THICK CLEAN FILL COMPACTED TO 95% STANDARD PROCTOR DENSITY PER ASTM M698 UNDER STRUCTURE.

8. REFER TO SHEET C9.1 FOR PRESEDIMENTATION/FOREBAY DESIGN.

9. ENGINEER SHALL BE NOTIFIED PRIOR TO BEGINNING CONSTRUCITON OF OUTLET STRUCTURE TO SCHEDULE OBSERVATION VISITS FOR STRUCTURES.

10. SEE RIGHT FOR WELL-SCREEN DESIGN NOTES

WQCV WELL-SCREEN NOTES:

1. Well-Screen shall be stainless steel and attached by stainless steel bolts along edge of the mounting frame.

2. WQCV Well Screen
- Type of Screen: Stainless steel #93 Vee Wire (Johnson Vee Wire (tm) Stainless Steel Screen or equivalent with 60% open area)
- Screen slot opening dimension: 0.139" (Screen #93 Vee Wire Slot Opening)
- Type and Size of Support Rod: TE 0.074"X0.50"
- Spacing of Support Rod (O.C.): 1.0 Inch
- Total Screen Thickness: 0.655"
- Carbon Steel Holding Frame Type: 3/4" x 1.0" angle

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

NO.	DESCRIPTION	DATE

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

PROJECT:
LORSON RANCH EAST FILING NO. 4
LORSON BLVD – LAMPREY DR
COLORADO SPRINGS, COLORADO

DRAWN: RLS
DESIGNED: RLS
CHECKED: RLS

LORSON RANCH EAST FILING NO. 4
FULL SPECTRUM POND E2
OUTLET STRUCTURE

DATE:
FEBRUARY 28, 2019

PROJECT NO.
100.048

SHEET NUMBER
C9.3
TOTAL SHEETS: 40

Markup Summary

Steve Kuehster (18)

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Call out access road.

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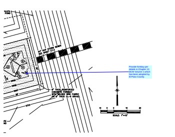
Call out access road.



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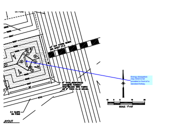
Provide forebay per details in Chapter 13 DCM Volume 1 which has been adopted by El Paso County.



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Page Label: 117
Author: Steve Kuehster
Date: 4/22/2019 1:37:02 PM
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Energy dissipation may need to be provided in front of a standard forbay.

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rom Fontaine Boulevard on the south side
curb/gutter to Design Point 29 at the
ection where it will be collected by an ex
rom this basin is 1.2cfs and 2.7cfs for the
tions. Show design point 29 on the drainage plan.
n residential development and Vedder/Roc
25 in curb/gutter where it will be colle
d flow from this basin is 6.1cfs and 13.7
ailed calculations.

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Author: Steve Kuehster
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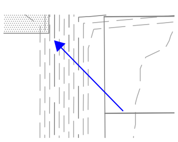
Show design point 29 on the drainage plan.



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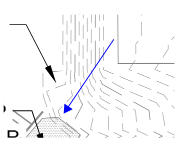

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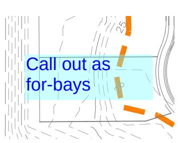

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Date: 4/22/2019 10:42:52 AM
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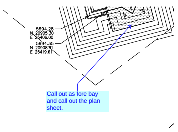
Call out as for-bays



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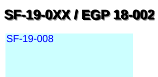
Call out as fore bay and call out the plan sheet.



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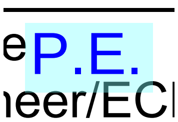
SF-19-008



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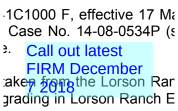
P.E.



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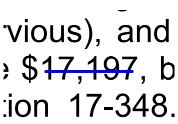
Call out latest FIRM December 7 2018



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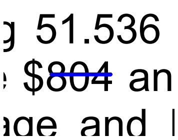

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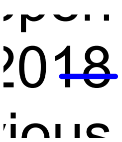

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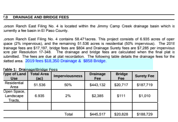

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Author: Steve Kuehster
Date: 4/23/2019 12:22:35 PM
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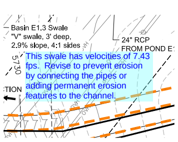
2019 fees $18,350 Drainage & $858 Bridge.



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Author: Steve Kuehster
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This swale has velocities of 7.43 fps. Revise to prevent erosion by connecting the pipes or adding permanent erosion features to the channel.