# LIBERTY TREE ACADEMY – PHASE II IMPROVEMENTS

# FINAL DRAINAGE REPORT

**Prepared for:**

Liberty Tree Academy
8579 Eastonville Road
Peyton, CO 80831

**Prepared by:**

Matrix Design Group, Inc.
1601 Blake Street, Suite 200
Denver, CO 80202

Project Number: 18.995.001

**Submitted:**

December 11, 2020

**PPR2018**

Engineer's Statement

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the County for drainage reports and said report is in conformity with the 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.


Andrew Beck, PE
Matrix Design Group
1601 Blake Street, Suite 300
Denver, CO 80202

Developers Statement:

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

Michael E. Peterson, Board President
Liberty Tree Academy Building Corporation
PO Box 64614
Colorado Springs, CO 80962

EL PASO COUNTY ONLY:

Filed in accordance with the requirements of the Drainage Criteria Manual, Volumes 1 and 2, El Paso County Engineering Criteria Manual and Land Development Code as amended.

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

Conditions:

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## I. GENERAL LOCATION AND DESCRIPTION

This Final Drainage Report is for the Phase II improvements for the Liberty Tree Academy. This is an update to the Liberty Tree Academy Final Drainage Report submitted by Matrix Design Group in August 2018. The update includes a 5,705 sf building addition and a 0.9 acres of added parking area. The "historic" in this report refers to the condition previous to any development, the "existing condition" refers to Phase I improvements outlined in the August 2018 Drainage Report, and the "proposed condition" includes the new proposed addition and parking area within the Phase II development.

### A. Location

The proposed Liberty Tree Academy is within Township 12S, and Range 64W, Section 32, SW Quarter, of El Paso County, Colorado as shown in **Figure 1**. The approximate latitude and longitude are 38°57'35"N and 104°35'11 W. The lot is situated along the east side of Eastonville Road from Tex Tan Road to approximately 250 ft north of Snaffle Bit Road. The project area is located east, south, and north of residential parcels. Unplatted agricultural land exists to the east of the site. The project is situated in Woodmen Hills Filing No. 10 (Plat Number 10942). To the north is Woodmen Hills Filing No. 11 (Plat Number 11258). The current El Paso Assessor map is provided in **Appendix F**.

The project is located within the Bennett Ranch (CHWS1200) drainage basin. The project drains to the southeast to the Bennett Ranch drainageway.

### B. Description of Property

The overall 12-acre lot consists of the 4.38-acre project area (school, parking, and detention), undeveloped land, and a 240-ft wide drainage easement. This drainage report considers the entire 12-acre lot and the half of the adjacent Eastonville Road. The undeveloped land, except for the drainage easement, will be developed by future projects; this area is covered by the drainage report, but the future impervious values and detention requirements are not considered.

Phase I improvements included the construction of the school, parking lot, landscaping, and extended detention basin (EDB). Of the remaining area, the ground cover currently consists of native grasses, including Blue Grama with a few dispersed alders and other plant species consistent with pasture land in the Colorado Semi-arid plains environment. Willows line the drainageway on the east side of the site. Photos of existing site vegetation are included in **Appendix A**.

Slopes across the property typically range from 1-5%, with some local slopes around small mounds up to 20%. The slope from the access road to the drainageway is approximately 8:1. According to National Resources Conservation Service (NRCS) soil datasets, the predominant soil type is Columbine gravelly sandy loam, 0 to 3 percent slopes. This soil type is generally consistent with a Type A hydrologic soil group (HSG). NRCS soil data was obtained from the Soil Survey Geographic (SSURGO) Database for Arapahoe County, Colorado. The spatial dataset was last updated September 23, 2016 (version 7) and the tabular dataset was last updated October 10, 2017 (version 14). The soils map and a breakdown of HSG group by basin is provided in **Figure 2**.

The site includes a 240-ft wide drainage easement along the eastern boundary associated with Bennett Ranch drainageway, which flows from north to south along the property boundary. The boundary of the drainage easement is marked by an existing access road, which overlays a parallel water line. Existing sewer, gas, fiber optic, underground electric, underground telephone, water, and storm utilities are located within the Eastonville Road right-of-way. General locations of existing utilities are presented in **Figure B-1**. An irrigation system for the newly landscaped areas was installed with Phase I.

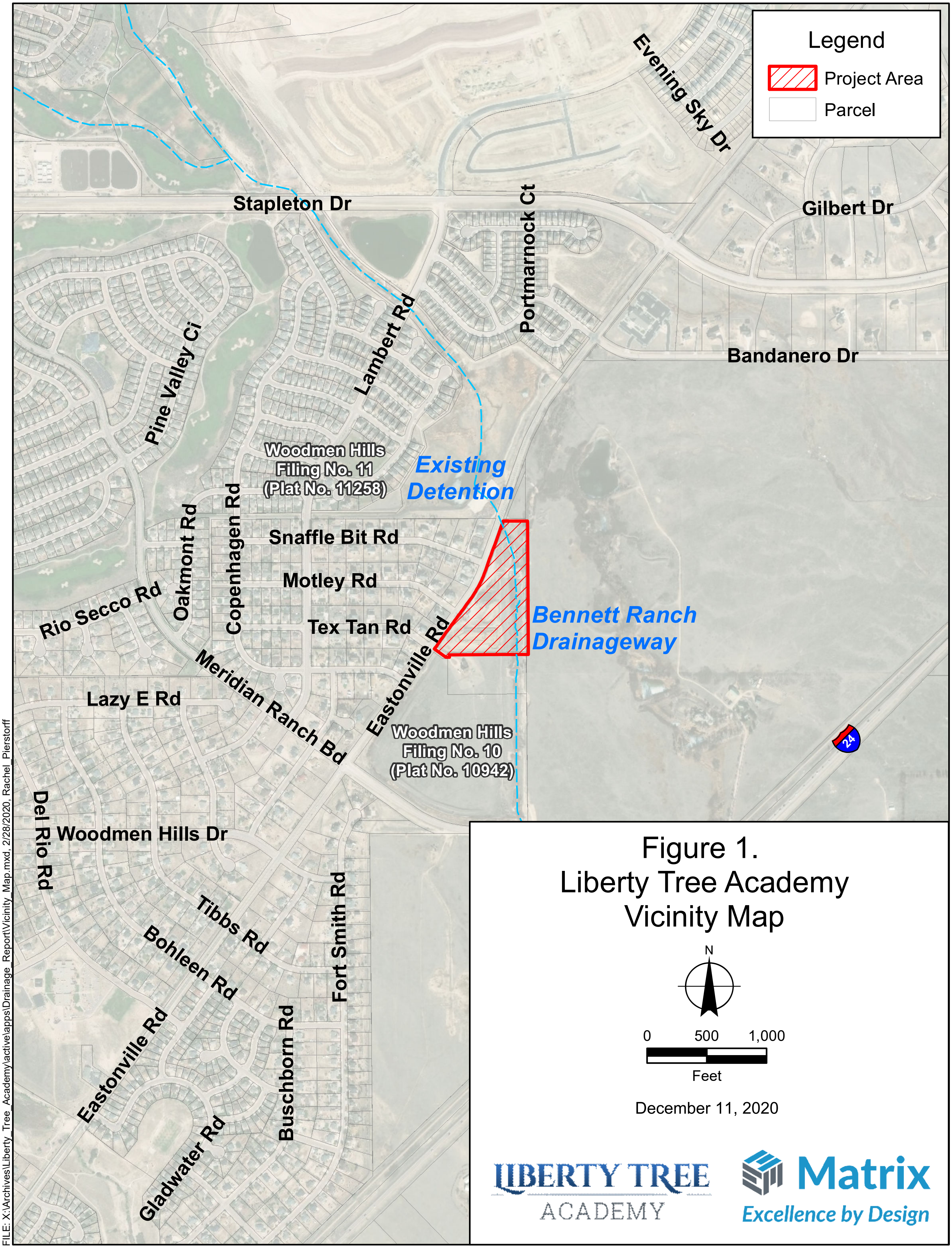


Figure 1.
Liberty Tree Academy
Vicinity Map

December 11, 2020

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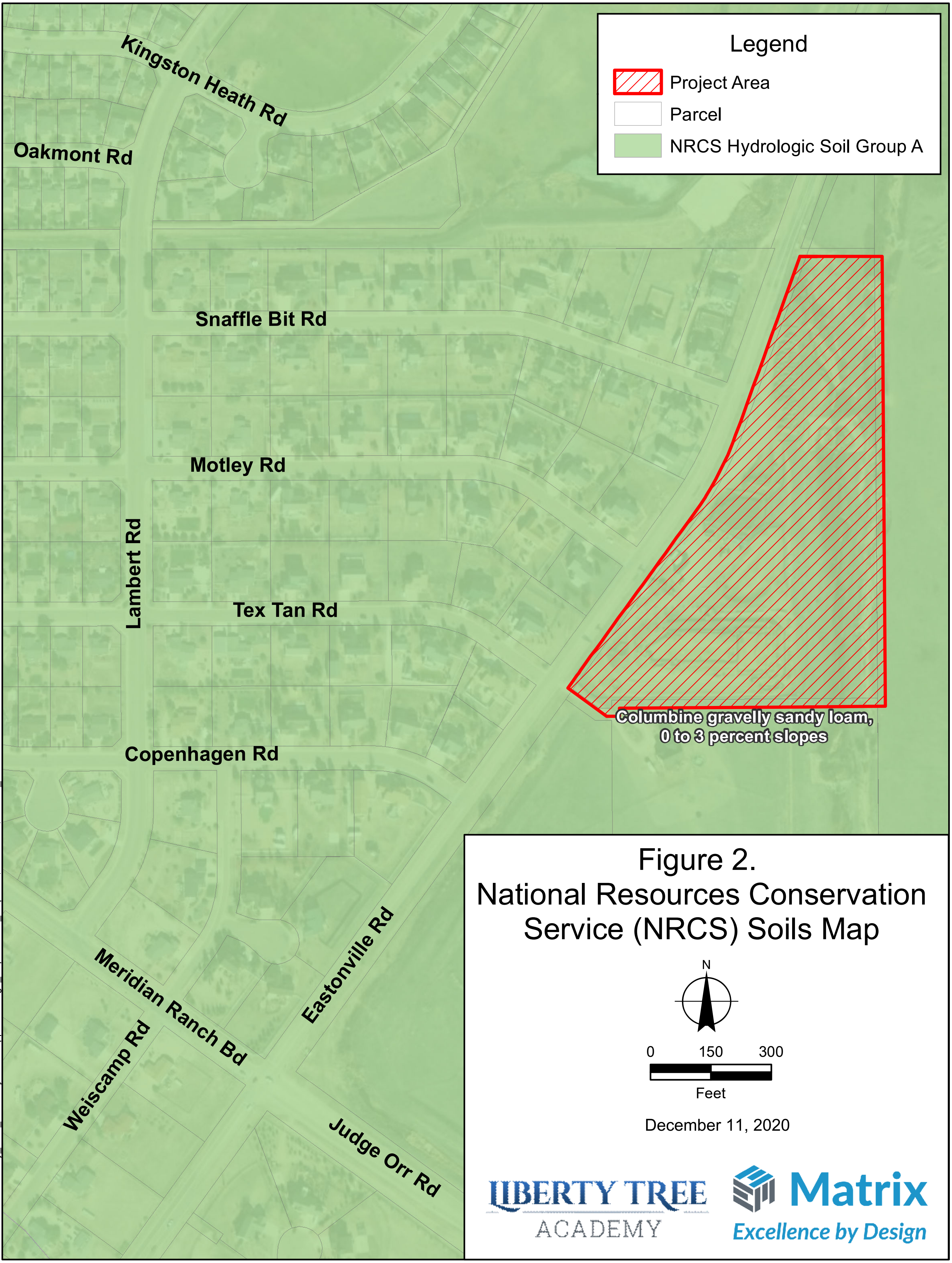


# Figure 2.
# National Resources Conservation Service (NRCS) Soils Map

December 11, 2020

LIBERTY TREE ACADEMY

Matrix
Excellence by Design

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## II. DRAINAGE BASINS AND SUB-BASINS

### A. Major Basin Descriptions

The project falls between Design Points (DP) D and C in the *Bennett Ranch Drainage Basin Planning Study (DBPS)* (El Paso County 2001). Between these design points, surface runoff flows to the southeast and the drainageway flows from north to south. The selected plan in the DBPS proposes a 50 acre-ft detention pond upstream of the project area at DP D and 9,500 linear-ft of new channel at 0.25% with thirteen 3-ft drop structures between Eastonville Road (DP D) and Drake Pond (between DP C and B). Based on the 2016 aerial, it appears that these proposed improvements are constructed. Selected pages from the DBPS are provided in **Appendix D**.

A Flood Insurance Study exists for El Paso County, Colorado and Incorporated Areas (FEMA 1999). The property is not located within a FEMA defined Floodplain, as identified on Flood Insurance Rate Map, Unincorporated El Paso County Community Panel Number 08041C0554 G, Effective December 7, 2018 (see **Appendix D**).

### B. Sub-basin Description

The overall project area in the historic, Phase I, and Phase II proposed condition drain southeast to the Bennett Ranch Drainageway. Historic topography routes surface flows southeast across the lot to the drainage easement access road. East of the access road, an approximately 8:1 embankment slopes to the east to the invert of the Bennett Ranch drainageway. The lot encompasses both banks of the drainageway within the 240-ft wide drainage easement. In the Phase I and Phase II condition, developed flows are routed the EDB before being discharged to Bennett Ranch drainageway

The site has been sub-divided into sub-basins. A description of these sub-basins follows:

<u>Eastonville Road</u>

Proposed Basins ER1 and ER2 are 0.94 and 0.45 acres in size, respectively, and consist of existing roadway, lawn, proposed roadway widening and sidewalk. As in the historic condition, runoff generated in these basins will drain into the existing curb and gutter in Eastonville Road and continue southwest at Design Point (DP) ER2. The total impervious area added from the proposed roadway widening and sidewalk will be less than an acre and the increase in peak runoff is 0.5 cfs and 1.2 cfs in the 5yr and 100yr event, respectively. There is sufficient capacity in Eastonville Road, see Curb and Gutter/Street Capacity calculations in **Appendix C**.

Southern Boundary

In the historic condition, Basin OS1 drains to the property to the south before reaching the Bennett Ranch Drainageway. With the Phase I improvements, Basin OS1 was reduced in size from 2.39 acres to 0.30 acres and will remain vegetated. Runoff generated in basin OS-1 will continue to sheet flow to the property to the south before reaching the Bennet Ranch Drainageway. Because of the decrease in tributary area and peak flows, no adverse impact to the property to the south is anticipated.

Extended Detention Basin

Proposed basins A-C drain to the proposed EDB. The proposed Basin A is 2.57 acres in size and contains the Phase I building, the Phase II building expansion, playground, and Phase I parking areas. The majority of runoff from basin A will be routed via curb and gutter to a curb cut at DP A, where it will be routed into the EDB. A separate piped system conveys roof runoff to the EDB; an underdrain draining directly to the Bennett Ranch Drainageway serves the playground area. The runoff from the roof and playground are included within Basin A and not calculated separately. The storm infrastructure for Basin A was constructed with Phase I.

The proposed Basins B1 and B2 are 0.44 and 0.45 acres, respectively, and consist of new parking and landscaped area proposed with Phase II. Runoff from these basins is routed via curb and gutter to Type-R Inlets in the low spot of each basin. The storm system serving Basins B1 and B2 discharges into the EDB.

The proposed Basin C is 0.69 acres in size and contains undeveloped area and the EDB. Runoff from Basin A reaches the EDB at a rundown at DP A, and runoff from Basins B1 and B2 reaches the EDB via a piped storm system. The EDB was designed and constructed with Phase I improvements and will serve the Phase II improvements as well. Modifications from the original EDB design are discussed in Section IV.C.2. If the undeveloped area of Basin C develops in the future, the EDB will likely need to be re-designed.

Bennett Ranch Drainageway

The EDB outfall, Basin OS1, and Basin OS2 eventually drain to the Bennett Ranch Drainageway. Basin OS2 is reduced in size from 8.37 acres to 6.01 acres from the historic to Phase II condition and will continue to sheet flow into the drainageway. Basin OS2 is largely undisturbed except some grading to meet historic grade and the installation of a riprap rundown with Phase I. Disturbed areas will be revegetated. The tributary runoff from Basin OS2 will be less in the proposed condition than in the historic condition. Basin OS2 will continue to serve as a drainage easement.

### C. Conveyance of Offsite Runoff

Basins ER1 and ER2 contain half of offsite Eastonville Road and the proposed roadway widening, sidewalk and landscaped area within the ROW. These basins will continue to drain into Eastonville Road, which can adequately convey the proposed runoff. The total area added by the proposed roadway widening, and sidewalk is less than an acre, and total peak runoff released from the entire site will be less than in the existing condition. See Section IV.C.1 and Appendix C for more details on the runoff quantifications.

Some undeveloped area within Basin C will sheet flow into a proposed Extended Detention Basin (EDB). The extended detention basin is designed to accommodate this extra undeveloped area; should this area develop in the future, the extended detention basin may have to be redesigned to accommodate the increased impervious area. Basin OS-2 will remain undeveloped and will continue to drain east to Bennett Ranch Drainageway.

A normal depth flow analysis was performed to ensure the existing off-site Bennett Ranch Drainageway could sufficiently pass 100-year peak offsite runoff without resulting in adverse site impacts. A detailed description of this analysis can be found in Section IV.C.3. 100-yr offsite runoff in the channel is lower than the emergency overflow weir elevation of the onsite extended detention basin.

## III. DRAINAGE DESIGN CRITERIA

### A. Development Criteria Reference

#### 1. Design Criteria

This report is prepared in accordance with the following criterion:

- Chapter Six, Section 6.3.2 – Drainage and Section 6.3.5 - Grading and Erosion Control of the El Paso County Land Development Code.
- El Paso County Drainage Criteria Manual Volume 1 and Volume 1 Update (DCM-V1, DCM-V1-Update)
- El Paso County Drainage Criteria Manual Volume 2 (DCM-V2)
- Engineering Criteria Manual for El Paso County, revised July 2019.

In addition, the Mile High Flood District (MHFD) criteria manuals and spreadsheet tools were used to guide design assumptions. El Paso County adopts the use of MHFD's MHFD-Rational and MHFD-Detention within the listed references above.

### 2. Previous Drainage Studies

There are several existing drainage reports and studies used in the development of this report. They are:

- Liberty Tree Academy Final Drainage Report by Matrix Design Group, August 2018. (Phase I Drainage Report).
- El Paso County. 2001. Bennett Ranch Pilot Project Drainage Basin Planning Study. El Paso County. November 2001.
- Federal Emergency Management Agency (FEMA). 2018. Flood Insurance Rate Map Number 08041C0554 G. El Paso County, Colorado and Unincorporated Areas. Effective December 7, 2018.

The site is not within a FEMA regulatory floodplain (See **Appendix D**). The Bennett Ranch Pilot Project Drainage Basin Planning Study (Bennet Ranch DBPS) outlines the improvements to the adjacent drainage channel and upstream detention basin. Anticipated runoff in the adjacent channel during the 100-yr event may cause backwater events into the proposed extended detention basin but will be below the crest elevation of the emergency overflow weir (see Section IV.C.4).

The site will utilize an extended detention basin and will therefore not cause significant increases in runoff rates due to development which would negatively impact downstream properties (see Section IV.C.2).

### B. Hydrologic Criteria

Based on Figures 6-6 through 6-17 of the DCM-V1, the NOAA Atlas 2 rainfall depths presented in Table 6-2 of the DCM-V1 Update applies. The basin size is less than 2 square-miles; therefore, Depth Area Reduction Factors are not required. The one-hour rainfall depths used in this analysis are presented in **Table 1**.

**Table 1 – One-Hour Rainfall Depths**

| $D_2$ (in) | $D_5$ (in) | $D_{10}$ (in) | $D_{25}$ (in) | $D_{50}$ (in) | $D_{100}$ (in) |
|---|---|---|---|---|---|
| 1.19 | 1.50 | 1.75 | 2.00 | 2.25 | 2.52 |

The rational method was used to calculate the runoff, as outlined in Section 6.2 of the MHFD Volume 1, with the exception of the impervious values and runoff coefficients which were taken from the DMC-V1-Update. For street and gutter capacity, the minor design storm was the 5-yr event. The major design storm is the 100-yr event.

Composite percent imperviousness, assuming Type A soils (see **Figure 2**), for each historic and proposed basin were determined using the land use categories in Table 6-6 of the DCM-V1-Update. These values are presented in **Table 2** and **Appendix B**.

**Table 2 – Percent Imperviousness from Table 6-6 of DCM-V1-Update**

| Land Use or Surface Characteristic | Percent Imperviousness | $C_{5yr}$ | $C_{100yr}$ |
|---|---|---|---|
| Pasture/ Meadow, Lawn | 0 | 0.08 | 0.35 |
| Playground | 13 | 0.16 | 0.41 |
| Roofs | 90 | 0.73 | 0.81 |
| Paved, Drive and Walks | 100 | 0.90 | 0.96 |

Water quality and stormwater detention will be provided by the onsite extended detention basin. Total detention volumes and discharges were determined using MHFD's MHFD-Detention_v4.02 (See Section IV).

## IV. DRAINAGE FACILITY DESIGN

### A. General Concept

Generally, existing site flows are to the southeast. The proposed development will maintain the overall drainage patterns. As in the historic condition, the area within the ROW of Eastonville Road will continue to drain west to the curb and gutter. A portion of the site along the southern border graded and vegetated with Phase I will continue drain to the property to the south; the proposed tributary area will remain vegetated and total area and runoff reaching the property to the south will be less than in the historic condition.East of the access road, there is a surface break and the site slopes approximately 8:1 towards the offsite drainageway. This section of the property is not included in planned development in order to maintain existing drainage patterns and avoid changes to the drainage easement. Any minor grading changes to this area will be re-vegetated.

With the exception of the sidewalk along Eastonville Road, all runoff from the developed area will be routed to the extended detention basin (EDB) in the eastern side of the project area constructed with Phase I. The extended detention basin will maintain historic outflow to the existing Bennett Ranch Drainageway. The EDB will serve the Phase II improvements with minor changes in grading for the access road, see section IV.C.2.

sidewalk and
roadway widening

### B. BMP Selection Process

Per section I.7.2 of El Paso ECM, a four-step process is used to select structural BMPs for the site. Discussion of these four steps and decision matrix is found below.

#### 1. Employ Runoff Reduction Practices

Opportunities to minimize directly connected impervious areas were limited for this site; Most of the disturbed site is utilized for the building footprint or parking area or will be in future improvements. The sidewalk along Eastonville Road is disconnected from the road. Developed runoff from the site is routed to an extended detention basin.

#### 2. Stabilize Drainageways

All channelized runoff on the site is conveyed via curb and gutter to curb cuts and inlets at DPs A and B, respectively. Runoff from Basin A is routed to the EDB via a riprap rundown and runoff from Basin B1 and B2 is routed to the EDB via a storm system. The energy for inflow to the EDB is dissipated in a concrete forebay. A trickle channel conveys channelized runoff within the extended detention basin to its outlet. The extended detention basin outfalls to Bennett Ranch Drainageway, which was previously stabilized with drop structures, in accordance with the Bennett Ranch DBPS (See Section II.A and **Appendix D**).

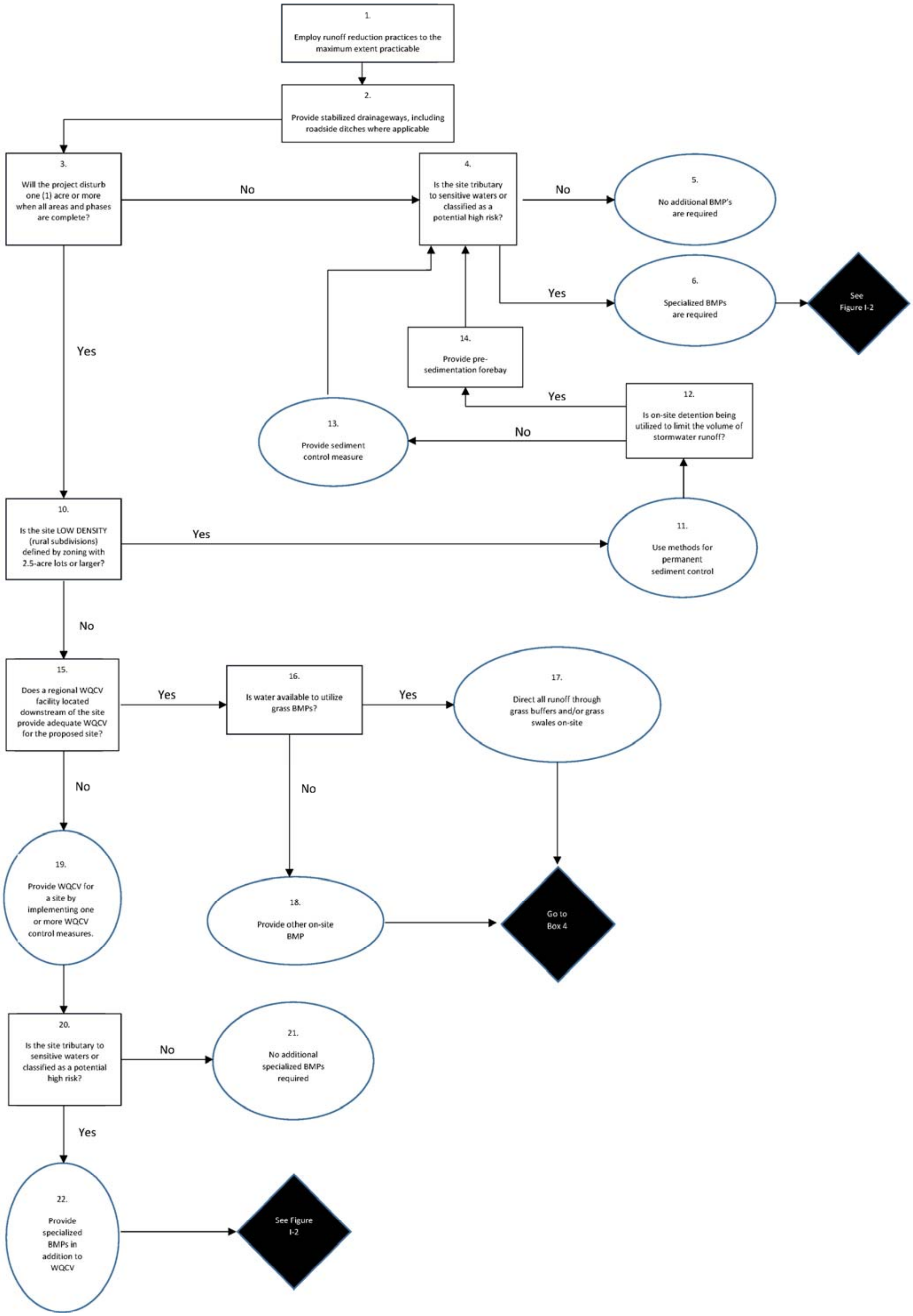
#### 3. Provide WQCV

Water Quality Capture Volume (WQCV) is provided within the Excess Urban Runoff Volume (EURV) in the onsite extended detention basin. See **Appendix C** for extended detention basin design. The only areas within the site not routed to the EDB are the landscaped areas, sidewalks, and a portion of the driveway adjacent to Eastonville Road. Because the sidewalks are graded towards Eastonville Road, it would be impractical to route this runoff towards the site. The total impervious area from the sidewalk and driveway draining towards Eastonville Road is less than 0.14 acres for both the Phase I and Phase II development, and is excluded from Water Quality treatment requirements per Section I.7.1.C.1 of the revised ECM. The landscaped portion which drains to Eastonville Road is exempted from Water Quality Treatment by section I.7.1.B.7. of the revised ECM.

#### 4. Consider Need for Industrial and Commercial BMPs

The proposed use for this site, a school, does not warrant Covering of Storage/Handling Areas or Spill Containment and Control.

Please revise to include the road widening area that will not be captured in the EDB


1.
Employ runoff reduction practices to the maximum extent practicable
2.
Provide stabilized drainageways, including roadside ditches where applicable
3.
Will the project disturb one (1) acre or more when all areas and phases are complete?
No
4.
Is the site tributary to sensitive waters or classified as a potential high risk?
No
5.
No additional BMP's are required
Yes
6.
Specialized BMPs are required
See Figure I-2
14.
Provide pre-sedimentation forebay
Yes
12.
Is on-site detention being utilized to limit the volume of stormwater runoff?
No
13.
Provide sediment control measure
Yes
10.
Is the site LOW DENSITY (rural subdivisions) defined by zoning with 2.5-acre lots or larger?
Yes
11.
Use methods for permanent sediment control
No
15.
Does a regional WQCV facility located downstream of the site provide adequate WQCV for the proposed site?
Yes
16.
Is water available to utilize grass BMPs?
Yes
17.
Direct all runoff through grass buffers and/or grass swales on-site
No
No
19.
Provide WQCV for a site by implementing one or more WQCV control measures.
18.
Provide other on-site BMP
Go to Box 4
20.
Is the site tributary to sensitive waters or classified as a potential high risk?
No
21.
No additional specialized BMPs required
Yes
22.
Provide specialized BMPs in addition to WQCV
See Figure I-2

## C. Specific Details

### 1. Proposed Runoff Patterns and Quantities

The proposed development will maintain the overall drainage patterns. As in the existing condition, the area within the ROW of Eastonville Road (Basins ER1 and ER2) will continue to drain west to the curb and gutter. A portion of the site along the southern border (Basin OS1) will drain to the property to the south; the proposed tributary area will remain vegetated and total area and runoff reaching the property to the south will be less than in the historic condition. Basin OS2 will remain undeveloped and drain to the Bennett Ranch Drainageway; any regraded area in basin OS2 will be revegetated.

Basins A, B1, B2, and C contain all the developed areas apart from the sidewalk along Eastonville Road. Basins A-C are tributary to the EDB. Runoff from the parking lot and fire lane of Basin A is routed via curb gutter to a curb cut at DP A and is conveyed to a forebay in the EDB by a riprap rundown. A separate piped system will convey roof runoff to the EDB. Runoff from Basins B1 and B2 will be routed via curb and gutter to inlets at DP B1 and DP B2, and will then be routed via a piped system to the EDB. Basin C contains the EDB; runoff generated in this basin sheet flows into the EDB.

Historic and proposed runoff values, calculated using the rational method, are presented in **Tables 3** and **4**, respectively. See Appendix B for detailed hydrology calculations, see Appendix C for detailed hydraulic calculations.

**Table 3 – Pre-developed Hydrology (Rational Method)**

| Design Point | Tributary Basins | Drainage Area (ac) | Tc (min) | $Q_5$ (cfs) | $Q_{100}$ (cfs) |
|---|---|---|---|---|---|
| ER1 (Driveway) | ER1 | 0.76 | 9.8 | 1.74 | 3.70 |
| ER2 | ER2 | 0.33 | 5.0 | 0.92 | 1.97 |
| ER2 | ER1+ER2 | 1.10 | 12.2 | 2.26 | 4.83 |
| OS1 | OS1 | 2.39 | 23.1 | 0.52 | 3.83 |
| OS2 | OS1 | 8.37 | 23.1 | 2.43 | 14.19 |

**Table 4 – Post-developed Hydrology (Rational Method)**

| Design Point | Tributary Basin | Drainage Area (ac) | Tc (min) | $Q_5$ (cfs) | $Q_{100}$ (cfs) |
|---|---|---|---|---|---|
| ER1 (Driveway) | ER1 | 0.94 | 9.8 | 2.11 | 4.52 |
| ER2 | ER2 | 0.45 | 6.1 | 1.08 | 2.40 |
| ER2 | ER1+ER2 | 1.39 | 12.3 | 2.76 | 5.98 |
| OS1 | OS1 | 0.30 | 5.0 | 0.12 | 0.90 |
| OS2 | OS2 | 6.01 | 23.0 | 1.92 | 10.44 |
| A | A | 2.57 | 9.5 | 7.66 | 14.65 |
| B1 | B1 | 0.44 | 5.0 | 1.34 | 2.77 |
| B2 | B2 | 0.45 | 5.0 | 1.94 | 3.53 |
| C | C | 0.69 | 13.7 | 0.28 | 1.54 |
| C* (Pond Inflow) | A+B1+B2+C | 4.15 | 11.3 | 9.93* | 20.11* |

*Note:. *For DP C, flows represent inflow into the EDB, not the attenuated outflow.*

### 2. Extended Detention Basin Design

The extended detention basin was constructed with Phase I and is located at the eastern edge of the developed area. The EDB will intercept all developed runoff from the site and convey attenuated flows east to the Bennett Ranch Drainageway. The EDB will preserve historic flow rates to Bennett Ranch Drainageway and provide full spectrum detention (WQCV, EURV and 100-yr detention). Detailed design calculations, outlet configuration, and design drawings for the section IV.C can be found in **Appendix C**.

Modifications for Phase II Improvements

The existing EDB will serve Phase I and Phase II improvements. Since the approval to the Phase I plans, the UD-Detention v3.07 spreadsheet has been replaced by the MHFD-Detention v4.02. spreadsheet with differences in calculated hydrographs and WQCV/EURV storage calculations. The as-built outlet structure is modeled in the new MHFD-Detention v4.02 spreadsheet, including the increased impervious areas for Phase II to verify that drain times are still in compliance with CRS § 37-92-602(8).

Changes to pond grading are proposed with Phase II, to include the access road which was not constructed with Phase I and to adjust the bottom of the pond to more closely match the area outlined in the Phase I construction drawings.

The outlet structure will not need to be modified. With the proposed alterations to grading, the Phase II WQCV drains in 40 hours, the EURV in 59 hours, release 97% of all events in less than 72 hours, and releases 99% of all events in less than 120 hours. The discussion of volumes and release rate for the Phase II improvements are presented in the subsequent discussion within this section.

The emergency overflow weir was constructed at an elevation of 6951.52’ instead of 6951.00’, and at a width of 8.82’ instead of 10.00’. The emergency overflow will convey the 100-yr emergency flow with less than 12” of ponding in the parking lot and 3’ below the finished floor elevation of the building, see the subsequent discussion in this section.

Volumes and Release Rates

The basin and outlet structure were originally sized and built in Phase I using the UDFCD-Detention spreadsheet v3.07, which was the accepted criteria at the time. This report utilizes the MHFD-Detention spreadsheet, version 4.02 in accordance with DCM-V1-Update criteria to verify the structure is still in compliance. The outlet structure was designed utilizes an orifice plate to release the water quality capture volume (WQCV) over 40-hours and the extended urban runoff volume (EURV) in between 52 and 72-hours. Because of the change in tributary area and updated methodology between the detention workbooks, the EURV now overtops the weir of the outlet structure and is controlled by the restrictor plate instead. The drainage times are still in compliance with CRS § 37-92-602(8) and thus the outlet structure will not need to be modified. A 2.5’ micropool in front of the orifice plate provides settlement. A drop box and 18-inch pipe with a restrictor plate attenuates runoff events exceeding the EURV. Outflows will be conveyed under to the existing stabilized channel in Bennett Ranch Drainageway. The Phase II development will increase the impervious area routed to the EDB; these volumes are adequately contained in the existing EDB. Total proposed detention volumes and release rate summary are provided in the table below:

**Table 5 - EDB Volume and Flow Rates Summary**

**EDB Summary**

| Design Storm Return Period | WQCV | EURV | 2 Year | 5 Year | 10 Year | 25 Year | 50 Year | 100 Year |
|---|---|---|---|---|---|---|---|---|
| **Rational Calculations** | | | | | | | | |
| Predev. Peak Inflow (cfs)= | N/A | N/A | 0.24 | 1.20 | 2.61 | 4.98 | 6.72 | 8.78 |
| **MHFD Detention Spreadsheet** | | | | | | | | |
| Pre-dev. Peak Inflow (CUHP) (cfs) | N/A | N/A | 0.03 | 0.06 | 0.08 | 0.75 | 1.51 | 2.46 |
| Dev. Peak Inflow (CUHP) Q (cfs) | N/A | N/A | 4.11 | 5.33 | 6.22 | 7.87 | 9.30 | 11.22 |
| Dev. Peak Outflow Q (cfs)= | 0.05 | 1.23 | 0.27 | 0.96 | 1.18 | 1.22 | 1.25 | 1.29 |
| Calculated Runoff Volume (acre-ft) | N/A | N/A | 0.248 | 0.324 | 0.385 | 0.463 | 0.540 | 0.632 |
| Maximum Volume Stored (acre-ft) | 0.093 | 0.355 | 0.227 | 0.247 | 0.271 | 0.337 | 0.398 | 0.488 |
| Maximum Ponding Elevation (ft) | 6946.94 | 6948.87 | 6948.04 | 6948.19 | 6948.34 | 6948.77 | 6949.12 | 6949.61 |

Per the MHFD's Detention Workbook, the proposed 100-yr release rate from the detention basin will be 52% of the existing inflow rate. In the 2-yr through 25-yr events, the predevelopment runoff calculated using MHFD-Detention workbook was very low (<1 cfs) which is due to the small tributary area. As a result, the workbook calculated developed outflow exceeds the predeveloped inflow for these conditions. Restricting outflow for these conditions any further would cause retention in excess of 72 hours. A more appropriate method for calculating the 2-yr through 25-yr events is the rational method, the results of which are presented the table above. Proposed outflow from the extended detention basin at DP C is less than the historic runoff at DP C as calculated with the rational method. **See Appendix B** for detailed calculations.

The peak outflow from the EDB is also different than the peak timing of basin OS2. A more accurate quantification of the peak flows to the Bennet Ranch Drainageway generated within the site would be to add the peak outflows from the EDB and Basin OS2 at the time of concentration of Basin OS2. Those calculations are presented below.

**Table 6 – Peak Flow to Bennett Ranch Drainageway**

| | Q100 (cfs) | | |
|---|---|---|---|
| Event | OS2 | EDB Outflow | Total |
| 5-yr | 1.92 | 0.03 | 1.95 |
| 100-yr | 10.44 | 0.03 | 10.47 |
| Tc= | 23.0 | min | |

The peak outflow into Bennett Ranch is reduced by 0.48 cfs in the 5-yr event and 3.72 cfs in the major event.

Emergency Overflow and Freeboard

The calculated 100-yr WSEL is 6949.61'. Placing the emergency overflow weir crest at this elevation would have reduced the cover of the waterline parallel to the access road to less than 5'. In order to maintain 5' of cover over the waterline the emergency overflow weir elevation was designed to be at 6951.00', with as-built survey showing the crest elevation at 6951.52. In the condition that the outlet structure became completely clogged, the 100-yr event runoff volume, 0.632 ac-ft, would have a ponded elevation of 6950.13'. Total freeboard between the 100-yr WSEL (clogged condition) and weir crest is 1.23'. In the condition where the outlet structure is completely clogged in the 500-year event, the ponded elevation would be 6951.13'. There would be 0.39' of freeboard between the clogged 500-yr event and the spillway.

The constructed emergency spillway is 8.82-ft wide and is calculated to convey 100-year undetained flows (20.1 cfs) with 8.6" inches of flow depth and consists of soil riprap (Type VL riprap) in accordance with Figure 12-21 from MHFD Volume 2. In the event the emergency overflow weir is activated, some ponding in the parking lot built with Phase I would occur, but would be less than 12" and would be more than 3' below the finished floor elevation of the building.

Other Design Components

Runoff will enter the EDB via the concrete pans within the parking lot in drainage Basin A and flow down a riprap rundown to the forebay. From Basins B1 and B2, runoff will be captured by Type-R inlets and piped to a trickle channel, and then to the pond forebay. The 6" tall pond forebay is sized for 2% of the WQCV of Basins A-B2, as per DCM-V2. The target release rate (2% of undetained runoff into the EDB) is 0.40 cfs, which can be achieved with a 5.3-inch notch. This forebay has already been constructed with Phase I with a 4.1-inch notch, which should not affect the overall performance of the forebay.

The main trickle channel constructed with Phase I and the new trickle channel for the B Basin storm system will be concrete and 4-inch deep as per DCM-V2. Per criteria, the capacity of the channel is sized to convey the maximum possible forebay outlet capacity, at a minimum. The flat bottom longitudinal slope will be graded at 0.5% per MHFD Volume 3. The adjacent vegetated areas will slope towards the low flow channel at 3%, as per DCM-V2 and MHFD Volume 3.

Maintenance access was not built with Phase I, and will be built with Phase II along the west side of the EDB and include ramps with less than 10% slopes to the forebay, pond bottom, and outlet structure. The pond will be maintained by Liberty Tree Academy as part of grounds maintenance via a Stormwater BMP Maintenance Agreement, which will be signed and recorded as part of this project approval.

### 3. Curb and Gutter, Street, and Inlet Capacity

An 8-ft wide crosspan, in accordance with El Paso's Standard Details, will be constructed at the driveway intersection with Eastonville Road to convey offsite runoff within the road at the north end of the new Phase II parking lot. Another crosspan was constructed for the parking lot constructed with Phase I. Flowmaster V8i was used for the capacity calculations for the driveway/cross pan. Onsite curb/gutter capacity was calculated using UD-Inlet_v4.05. All street and curb calculations are provided in **Appendix C**. The Type-R Inlet capacity within Basins B1 and B2 were also calculated using the UD-Inlet_v4.05 workbook.

### 4. Offsite Channel Capacity

In order to ensure the existing off-site drainageway can sufficiently pass 100-year peak runoff without resulting in adverse site impacts, a normal depth analysis was conducted using FlowMaster, version 8i. 100-year outflow from the upstream detention pond as described in the DBPS was considered in this analysis (see **Appendix D**). Two typical drainageway cross-sections were cut along the project extents. The longitudinal channel slope was estimated based on available contours and the DBPS Selected Alternative (0.25%).

The resulting typical cross-sections are presented in **Figure 4**. The cross-section points on the west bank are taken from survey. Cross section points from the east bank are calculated based on the typical channel cross section found in the *Bennett Ranch* DBPS (See **Appendix D**). It is assumed that some incision of the channel has occurred since the construction of the stabilized channel, and the toe (the invert of the original channel) is equal to the surveyed elevation 5' west of the surveyed flowline. The Manning's n for the channel sections is taken from Table 8-5 of MHFD Volume 1, which is summarized in **Table 6**. The resulting water flow depths and freeboard are presented in **Table 7**.

**Table 7 – Manning's n**

| Location and Cover | Manning's n[1] |
|---|---|
| Main Channel (bankfull channel) | |
| Sand or clay bed | 0.04 |
| Vegetated Overbanks | |
| Native Grasses | 0.05 |
| Willow Stands, woody shrubs | 0.16 |

*Notes:*

*1. Manning's n for assessing water surface elevation and water depth*

A detailed cross section and corresponding segments for each manning's n used can be found in **Appendix C**. Normal flow depths for the channel are as follows:

**Table 8 – Offsite Channel Flow Depth Summary**

| Scenario | 100-yr WSEL (feet) | Flow Depth (feet) | Freeboard (feet) |
|---|---|---|---|
| Cross-section 1 | 6943.90 | 7.14 | 7.10 |
| Cross-section 2 | 6945.87 | 7.87 | 5.13 |

*Notes:*

1. *100-Year Master Planned Flow = 810 cfs, as per the DBPS for 100-year release rate from the upstream pond, see **Appendix D**).*
2. *Freeboard is measured from weir crest elevation. = 6951.0*

**XS 1** **XS 2**

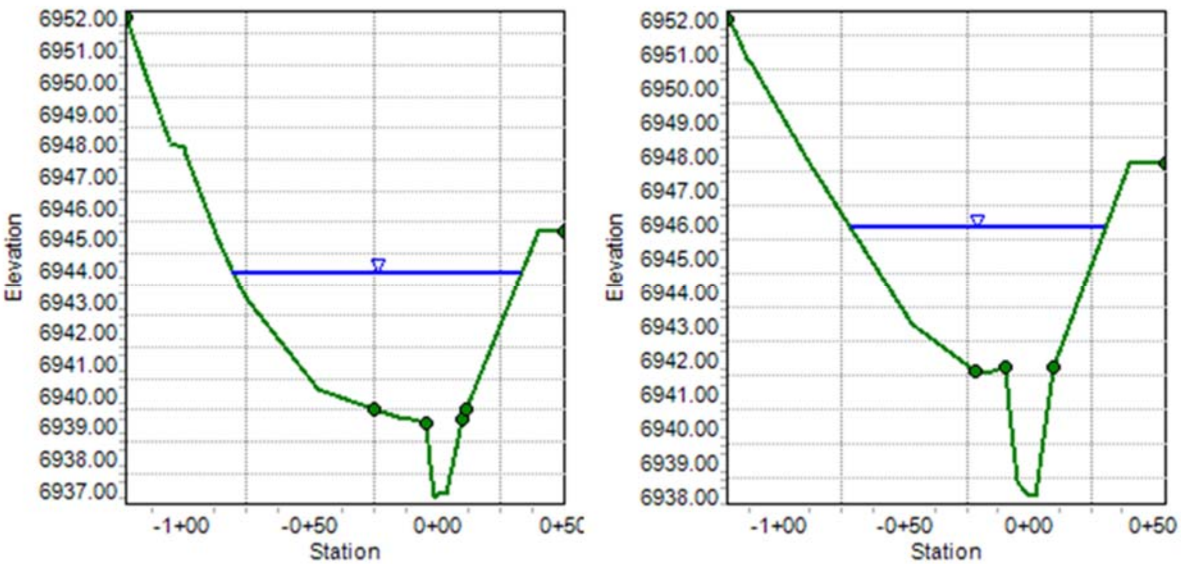



**Figure 4 – Offsite Drainageway Capacity Evaluation**
Cross sections, looking upstream
Q100=810 cfs (release rate of upstream basin)

There is a significant amount of freeboard between 100-yr channel WSEL and the onsite emergency overflow weir crest. The 100-yr WSEL at Section X2 (6945.87), however, is higher than the invert of the detention basin outlet pipe (6942.00). Because of difference in peak timing, it is not anticipated that this will negatively impact the ability for the EDB to drain in 72 hours or less.

The offsite channel can convey 60 cfs without any backwater effect on the pond (WSEL = invert of outlet pipe = 6942.00’). Backflow would not occur into the pond until offsite flow of 538 cfs (WSEL = Invert of pond = 6945). Calculations can be found in **Appendix C**.

## V. DRAINAGE FEES

Drainage and Bridge fees were paid with the Woodmen Hills Filing #10 final plate, therefore no fees are due.

# VI. REFERENCES

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# APPENDIX A – SITE PHOTOS

**Figure A1. Project area looking east.**

**Figure A2. Project area looking south east along utility access towards the drainage easement.**

**Figure A3. Project area looking north along Eastonville Road right-of-way.**

**Figure A4. Project area looking south towards adjacent residential property.**

**Figure A5. Bennett Ranch drainageway looking upstream (north) along eastern extent of the project area.**

# APPENDIX B – HYDROLOGIC ANALYSIS

**Project Name:** Liberty Tree Academy
**Job Number:** 20.995.002
**Subject:** Composite Runoff Coefficients
**Date:** 6/25/2020
**Designed by:** MAS

| Global Parameters | | | | | | | |
|---|---|---|---|---|---|---|---|
| Land Use | % Imp. | $C_2$ | $C_5$ | $C_{10}$ | $C_{25}$ | $C_{50}$ | $C_{100}$ |
| Pasture/ Meadow, Lawn | 0 | 0.02 | 0.08 | 0.15 | 0.25 | 0.30 | 0.35 |
| Playground | 13 | 0.07 | 0.16 | 0.24 | 0.32 | 0.37 | 0.41 |
| Paved, Drive and Walk, Detention | 100 | 0.89 | 0.90 | 0.92 | 0.94 | 0.95 | 0.96 |
| Roofs/Gravel | 90 | 0.71 | 0.73 | 0.75 | 0.78 | 0.80 | 0.81 |

*Type A Soils

| Subbasin | Total Area (acres) | Land Use Area per Sub-Basin | | | | | | | | Composite Imperviousness | Composite Runoff Coefficient | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Pasture/ Meadow, Lawn | | Playground | | Paved, Drive and Walk, Detention | | Roofs/Gravel | | | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
| | | Area (acres) | % | Area (acres) | % | Area (acres) | % | Area (acres) | % | | | | | | | |
| **EXISTING** | | | | | | | | | | | | | | | | |
| ER1 | 0.76 | 0.32 | 42.1% | 0.00 | 0.0% | 0.44 | 57.9% | 0.00 | 0.0% | **57.9%** | **0.52** | **0.55** | **0.60** | **0.65** | **0.68** | **0.70** |
| ER2 | 0.33 | 0.14 | 43.5% | 0.00 | 0.0% | 0.19 | 56.5% | 0.00 | 0.0% | **56.5%** | **0.51** | **0.54** | **0.59** | **0.64** | **0.67** | **0.69** |
| OS1 | 2.39 | 2.39 | 100.0% | 0.00 | 0.0% | 0.00 | 0.0% | 0.00 | 0.0% | **0.0%** | **0.02** | **0.08** | **0.15** | **0.25** | **0.30** | **0.35** |
| OS2 | 8.37 | 8.10 | 96.8% | 0.00 | 0.0% | 0.27 | 3.2% | 0.00 | 0.0% | **3.2%** | **0.05** | **0.11** | **0.17** | **0.27** | **0.32** | **0.37** |
| **PROPOSED** | | | | | | | | | | | | | | | | |
| ER1 | 0.94 | 0.40 | 42.9% | 0.00 | 0.0% | 0.54 | 57.1% | 0.00 | 0.0% | **57.1%** | **0.52** | **0.55** | **0.59** | **0.64** | **0.67** | **0.70** |
| ER2 | 0.45 | 0.22 | 48.4% | 0.00 | 0.0% | 0.23 | 51.6% | 0.00 | 0.0% | **51.6%** | **0.47** | **0.50** | **0.55** | **0.61** | **0.64** | **0.66** |
| OS1 | 0.30 | 0.30 | 100.0% | 0.00 | 0.0% | 0.00 | 0.0% | 0.00 | 0.0% | **0.0%** | **0.02** | **0.08** | **0.15** | **0.25** | **0.30** | **0.35** |
| OS2 | 6.01 | 5.74 | 95.5% | 0.00 | 0.0% | 0.27 | 4.5% | 0.00 | 0.0% | **4.5%** | **0.06** | **0.12** | **0.18** | **0.28** | **0.33** | **0.38** |
| A | 2.57 | 0.32 | 12.3% | 0.12 | 4.5% | 1.43 | 55.5% | 0.71 | 27.7% | **81.0%** | **0.70** | **0.72** | **0.75** | **0.78** | **0.80** | **0.82** |
| B1 | 0.44 | 0.16 | 37.2% | 0.00 | 0.0% | 0.28 | 62.8% | 0.00 | 0.0% | **62.8%** | **0.57** | **0.60** | **0.63** | **0.68** | **0.71** | **0.73** |
| B2 | 0.45 | 0.02 | 5.4% | 0.00 | 0.0% | 0.42 | 94.6% | 0.00 | 0.0% | **94.6%** | **0.84** | **0.86** | **0.88** | **0.90** | **0.91** | **0.93** |
| C | 0.69 | 0.66 | 95.5% | 0.00 | 0.0% | 0.03 | 4.5% | 0.00 | 0.0% | **4.5%** | **0.06** | **0.12** | **0.18** | **0.28** | **0.33** | **0.38** |
| **Pond Total A-C** | **4.15** | **1.16** | **28.0%** | **0.12** | **2.8%** | **2.16** | **52.0%** | **0.71** | **17.2%** | **67.9%** | **0.59** | **0.62** | **0.66** | **0.70** | **0.73** | **0.75** |

**TIME OF CONCENTRATION**

P1, 5-yr: 1.50 in.
P1, 100-yr: 2.52 in.

Location: Liberty Tree Academy
Date: June 25, 2020
Designed by: MAS

| Sub-Basin Data | | | | | | Overland Time (ti) | | | Travel Time 1 (tt) | | | | | Travel Time 2 (tt) | | | | | Tc Check | | | | | Tc | 5-Year Runoff | | | 100-Year Runoff | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Design Pt. | Basin ID | Area | Imperv. | Coefficient "C5" | Coefficient "C100" | Length (300' max) | Slope | ti | Length | Slope | Cv, conveyance factor | Velocity = Cv * Slope^0.5 | tt | Length | Slope | Cv, conveyance factor | Velocity = Cv * Slope^0.5 | tt | tc=ti+tt | Channelized Length | Channelized Slope | tc = (26-17i)+Lt/(60(14i+9)St^0.5 | Minimum tc | Final tc | 5-Year Intensity "I" | $C_5A$ | Total Peak Discharge "Q5" | 100-Year Intensity "I" | $C_{100}A$ | Total Peak Discharge "Q100" |
| | | acres | | | | ft | % | min. | ft | % | | fps | min | | | | | | min | ft | | min | min | min | | | cfs | | | cfs |
| | **EXISTING** | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| ER1 | ER1 | 0.76 | 58% | 0.55 | 0.70 | 21 | 2.5% | 3.4 | 873 | 1.3% | 20 | 2.3 | 6.4 | | | | | | 9.8 | 873 | 1.3% | 23.6 | 5.0 | 9.8 | 4.10 | 0.42 | 1.74 | 6.88 | 0.54 | 3.70 |
| ER2 | ER2 | 0.33 | 57% | 0.54 | 0.69 | 12 | 2.5% | 2.6 | 367 | 1.6% | 20 | 2.6 | 2.4 | | | | | | 5.0 | 367 | 1.6% | 19.2 | 5.0 | 5.0 | 5.09 | 0.18 | 0.92 | 8.55 | 0.23 | 1.97 |
| **ER2** | **ER1+ER2** | **1.10** | **57%** | **0.55** | **0.70** | **21** | **2.5%** | **3.4** | **873** | **1.3%** | **20** | **2.3** | **6.4** | **367** | **1.6%** | **20** | **2.6** | **2.4** | **12.2** | **1240** | **1.4%** | **26.5** | **5.0** | **12.2** | **3.74** | **0.60** | **2.26** | **6.29** | **0.77** | **4.83** |
| **OS1** | **OS1** | **2.39** | **0%** | **0.08** | **0.35** | **175** | **2.0%** | **19.6** | **241** | **2.7%** | **7** | **1.1** | **3.5** | | | | | | **23.1** | **241** | **2.7%** | **28.7** | **5.0** | **23.1** | **2.73** | **0.19** | **0.52** | **4.59** | **0.84** | **3.83** |
| **OS2** | **OS2** | **8.37** | **3%** | **0.11** | **0.37** | **197** | **12.5%** | **11.0** | **1041** | **0.9%** | **15** | **1.4** | **12.1** | | | | | | **23.1** | **1041** | **0.9%** | **44.6** | **5.0** | **23.1** | **2.73** | **0.89** | **2.43** | **4.59** | **3.09** | **14.19** |
| | **PROPOSED** | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| ER1 | ER1 | 0.94 | 57% | 0.55 | 0.70 | 21 | 2.5% | 3.4 | 873 | 1.3% | 20 | 2.3 | 6.4 | | | | | | 9.8 | 873 | 1.3% | 23.8 | 5.0 | 9.8 | 4.09 | 0.52 | 2.11 | 6.87 | 0.66 | 4.52 |
| ER2 | ER2 | 0.45 | 52% | 0.50 | 0.66 | 21 | 2.5% | 3.7 | 367 | 1.6% | 20 | 2.6 | 2.4 | | | | | | 6.1 | 367 | 1.6% | 20.2 | 5.0 | 6.1 | 4.82 | 0.22 | 1.08 | 8.09 | 0.30 | 2.40 |
| **ER2** | **ER1+ER2** | **1.39** | **55%** | **0.53** | **0.69** | **21** | **2.5%** | **3.5** | **873** | **1.3%** | **20** | **2.3** | **6.4** | **367** | **1.6%** | **20** | **2.6** | **2.4** | **12.3** | **1240** | **1.4%** | **27.0** | **5.0** | **12.3** | **3.73** | **0.74** | **2.76** | **6.26** | **0.95** | **5.98** |
| **OS1** | **OS1** | **0.30** | **0%** | **0.08** | **0.35** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **-** | **5.0** | **5.0** | **5.09** | **0.02** | **0.12** | **8.55** | **0.11** | **0.90** |
| **OS2** | **OS2** | **6.01** | **4%** | **0.12** | **0.38** | **197** | **12.5%** | **10.9** | **1041** | **0.9%** | **15.0** | **1.4** | **12.1** | | | | | | **23.0** | **1041** | **0.9%** | **44.0** | **5.0** | **23.0** | **2.74** | **0.70** | **1.92** | **4.60** | **2.27** | **10.44** |
| A | A | 2.57 | 81% | 0.72 | 0.82 | 5 | 2.0% | 1.2 | 740 | 0.6% | 20.0 | 1.5 | 8.3 | | | | | | 9.5 | 740 | 0.6% | 20.3 | 5.0 | 9.5 | 4.14 | 1.85 | 7.66 | 6.96 | 2.11 | 14.65 |
| B1 | B1 | 0.44 | 63% | 0.60 | 0.73 | 7 | 2.5% | 1.8 | 431 | 1.3% | 20.0 | 2.3 | 3.1 | | | | | | 4.9 | 431 | 1.3% | 18.8 | 5.0 | 5.0 | 5.09 | 0.26 | 1.34 | 8.55 | 0.32 | 2.77 |
| B2 | B2 | 0.45 | 95% | 0.86 | 0.93 | 7 | 2.5% | 0.9 | 473 | 1.3% | 20.0 | 2.3 | 3.4 | | | | | | 4.3 | 473 | 1.3% | 13.0 | 5.0 | 5.0 | 5.09 | 0.38 | 1.94 | 8.55 | 0.41 | 3.53 |
| C | C | 0.69 | 4% | 0.12 | 0.38 | 95 | 3.2% | 12.0 | 165 | 0.6% | 20.0 | 1.6 | 1.7 | | | | | | 13.7 | 165 | 0.6% | 28.9 | 5.0 | 13.7 | 3.55 | 0.08 | 0.28 | 5.96 | 0.26 | 1.54 |
| **C** | **A-C** | **4.15** | **68%** | **0.62** | **0.75** | **5** | **2.0%** | **1.6** | **740** | **0.6%** | **20.0** | **1.5** | **8.3** | **140** | **0.6%** | **20.0** | **1.5** | **1.5** | **11.3** | **880** | **0.6%** | **25.0** | **5.0** | **11.3** | **3.86** | **2.57** | **9.93** | **6.48** | **3.10** | **20.11** |

## EDB EXISTING RUNOFF ANALYSIS

Location: Liberty Tree Academy
Date: June 25, 2020
Designed by: MAS

**Basin Name: Existing A-C**

| | |
|---|---|
| Area (ac.) | 4.15 |
| Imperv. (%) | 0% |
| Tc (min.) | 13.27 |

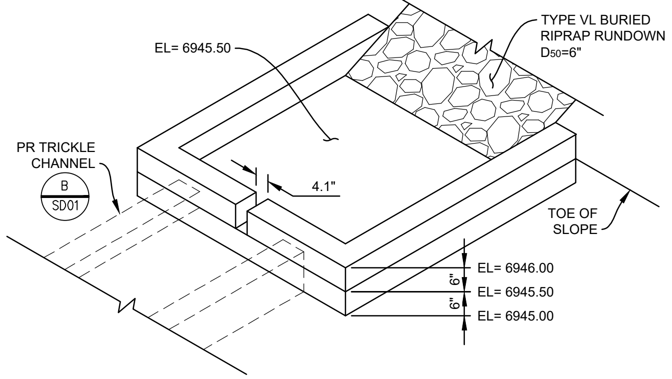
| Design Storm | P1 (in) | Runoff Coefficient | Intensity "I" | Total Peak Discharge "Q10 |
|---|---|---|---|---|
| | | | in/hr | *cfs* |
| **2** | **1.19** | 0.02 | 2.86 | 0.24 |
| **5** | **1.5** | 0.08 | 3.60 | 1.20 |
| **10** | **1.75** | 0.15 | 4.20 | 2.61 |
| **25** | **2** | 0.25 | 4.80 | 4.98 |
| **50** | **2.25** | 0.30 | 5.40 | 6.72 |
| **100** | **2.52** | 0.35 | 6.05 | 8.78 |

*Intensity values from Eq. 5-1 of MHFD V1

# APPENDIX C – HYDRAULIC ANALYSIS

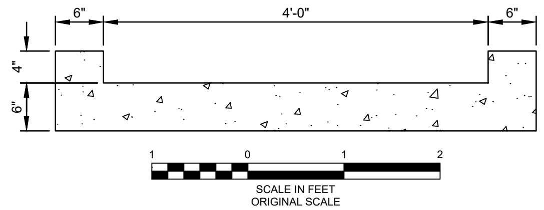
## Extended Detention Basin Design

ORIGINAL POND DESIGN FROM 2018 PLANS

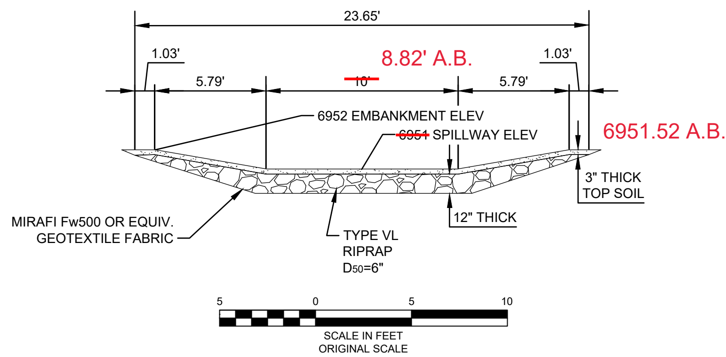



FOREBAY DETAIL (A)

SCALE: N.T.S.
SEE PLAN FOR LIMITS.

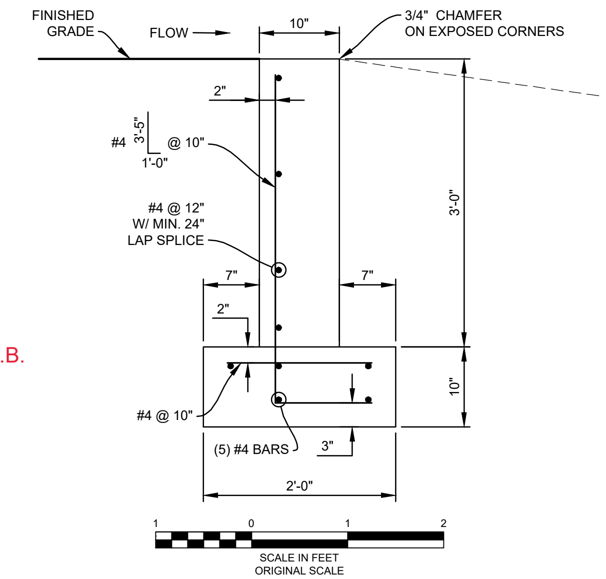



TRICKLE CHANNEL SECTION (B)



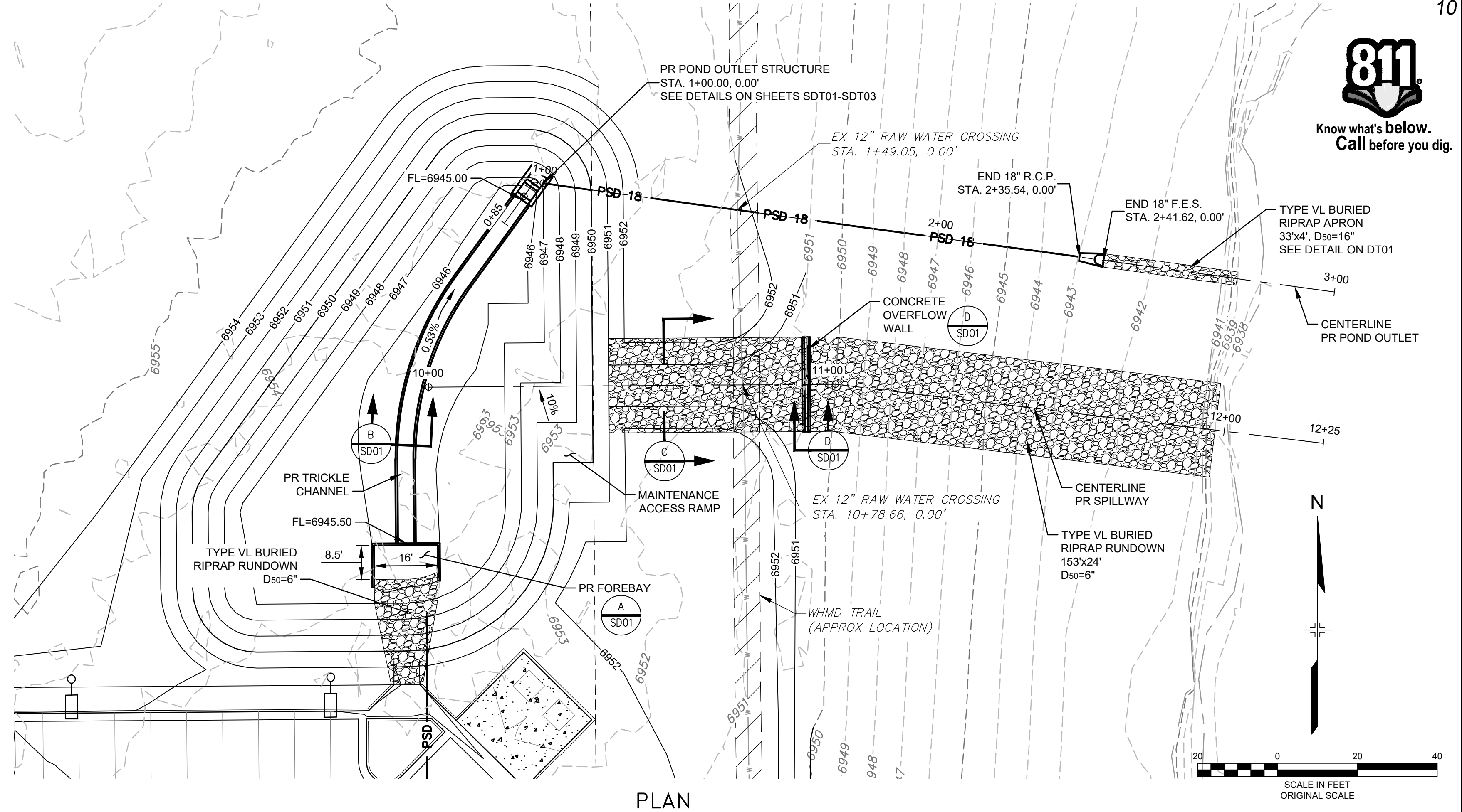

SPILLWAY SECTION (C)

SCALE: 1" = 5'



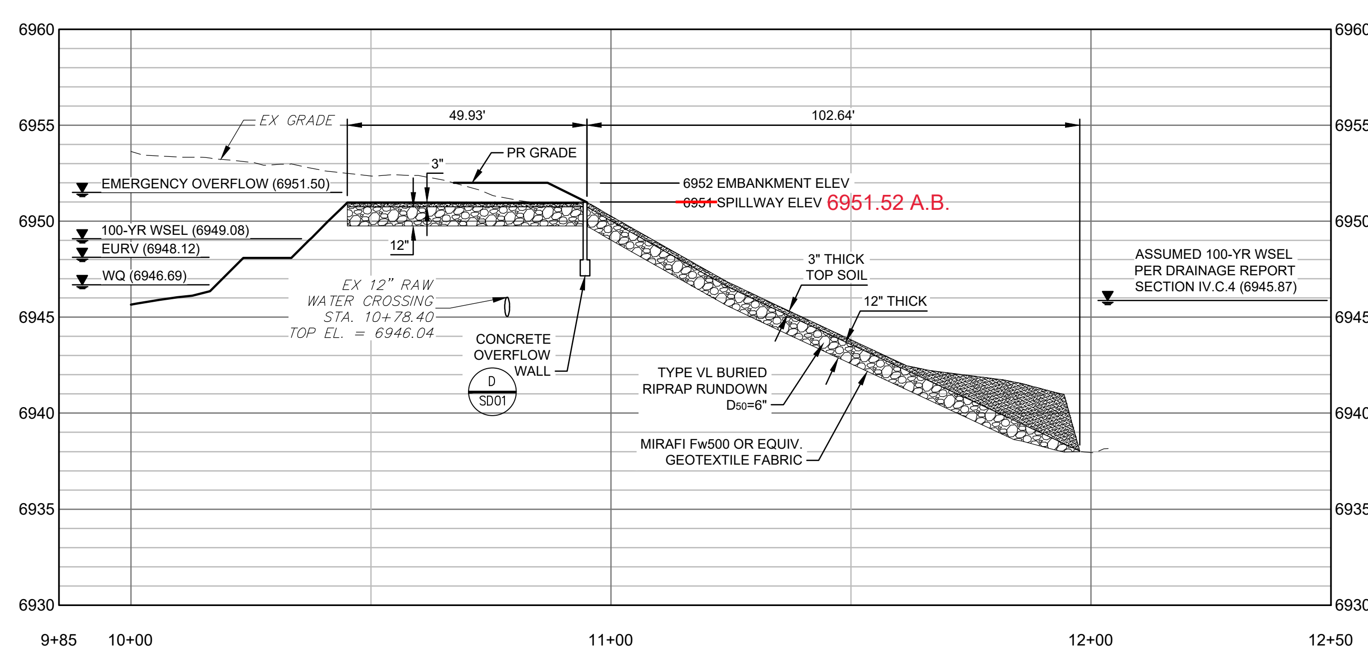

CONCRETE OVERFLOW WALL SECTION (D)

SCALE: 1" = 1'-0"



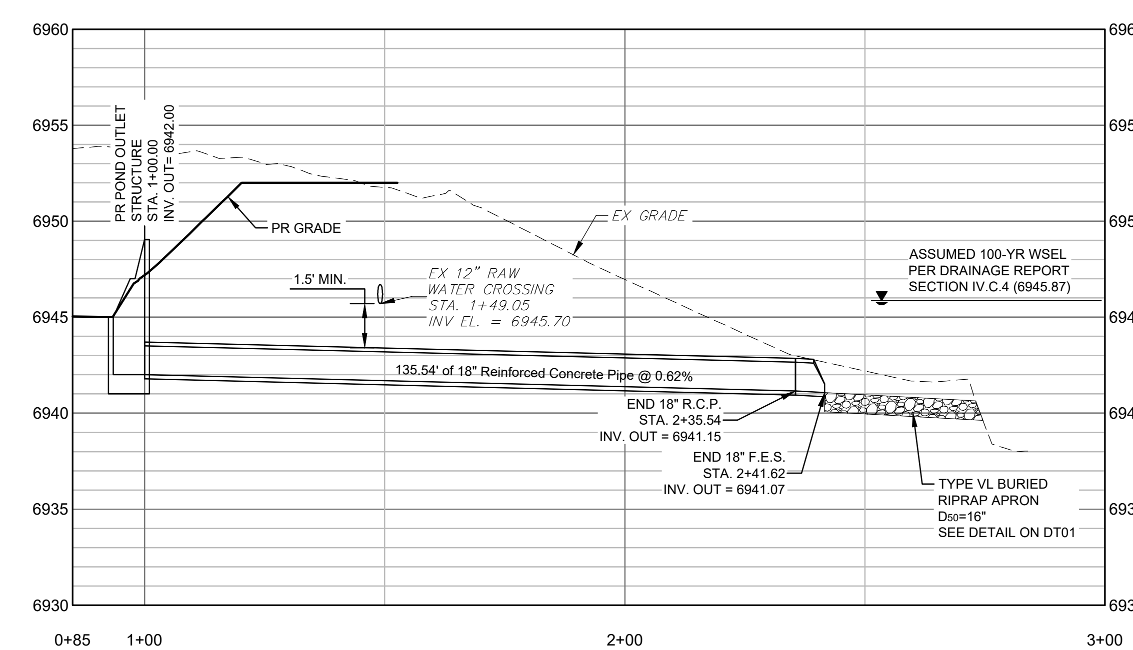

PLAN

SCALE: 1"=20'




SPILLWAY PROFILE

SCALE: 1"=20'(H), 1"=5'(V)




POND OUTLET PROFILE

SCALE: 1"=20'(H), 1"=5'(V)


REFERENCE DRAWINGS
X-995-MDG22x34
X-995-PR-UTIL
X-995-EX-BASE
X-995-EX-MAP
X-995-PR-BASE

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

REVISIONS

COMPUTER FILE MANAGEMENT
FILE NAME: R:\18.995.001 (Liberty Tree Academy)\Dwg\Construction Plans\SD01.dwg
CTB FILE: ----
PLOT DATE: 8/10/2018 11:14 AM
THIS DRAWING IS CURRENT AS OF PLOT DATE AND MAY BE SUBJECT TO CHANGE.

SHEET KEY

PREPARED FOR:
LIBERTY TREE ACADEMY
PREPARED BY:
Matrix DESIGN GROUP
AN EMPLOYEE-OWNED COMPANY

SEAL
24520
8/10/18
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 18.995.001

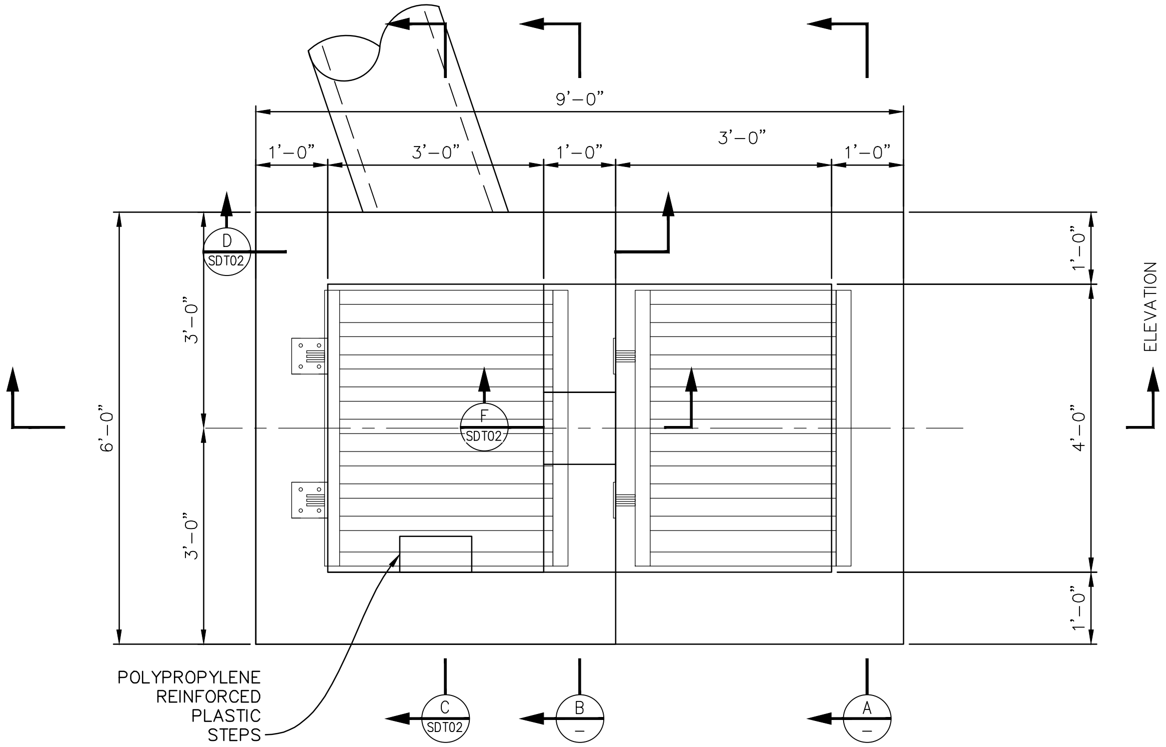
LIBERTY TREE ACADEMY
TOWN OF PEYTON, EL PASO COUNTY
CONSTRUCTION DOCUMENTS, PCD FILE NO. PPR-18-023
POND OUTLET & SPILLWAY PLAN & PROFILE

DESIGNED BY: ACR
DRAWN BY: ACR
CHECKED BY: DRK
SCALE HORIZ. AS SHOWN VERT. 1" = 5'
DATE ISSUED: AUGUST 2018
SHEET 10 OF 29
DRAWING No. SD01

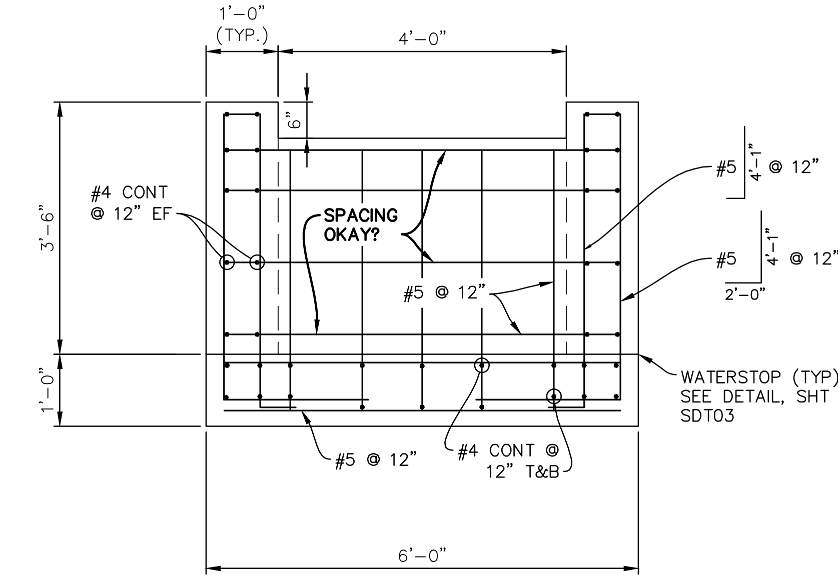
ORIGINAL POND DESIGN FROM 2018 PLANS



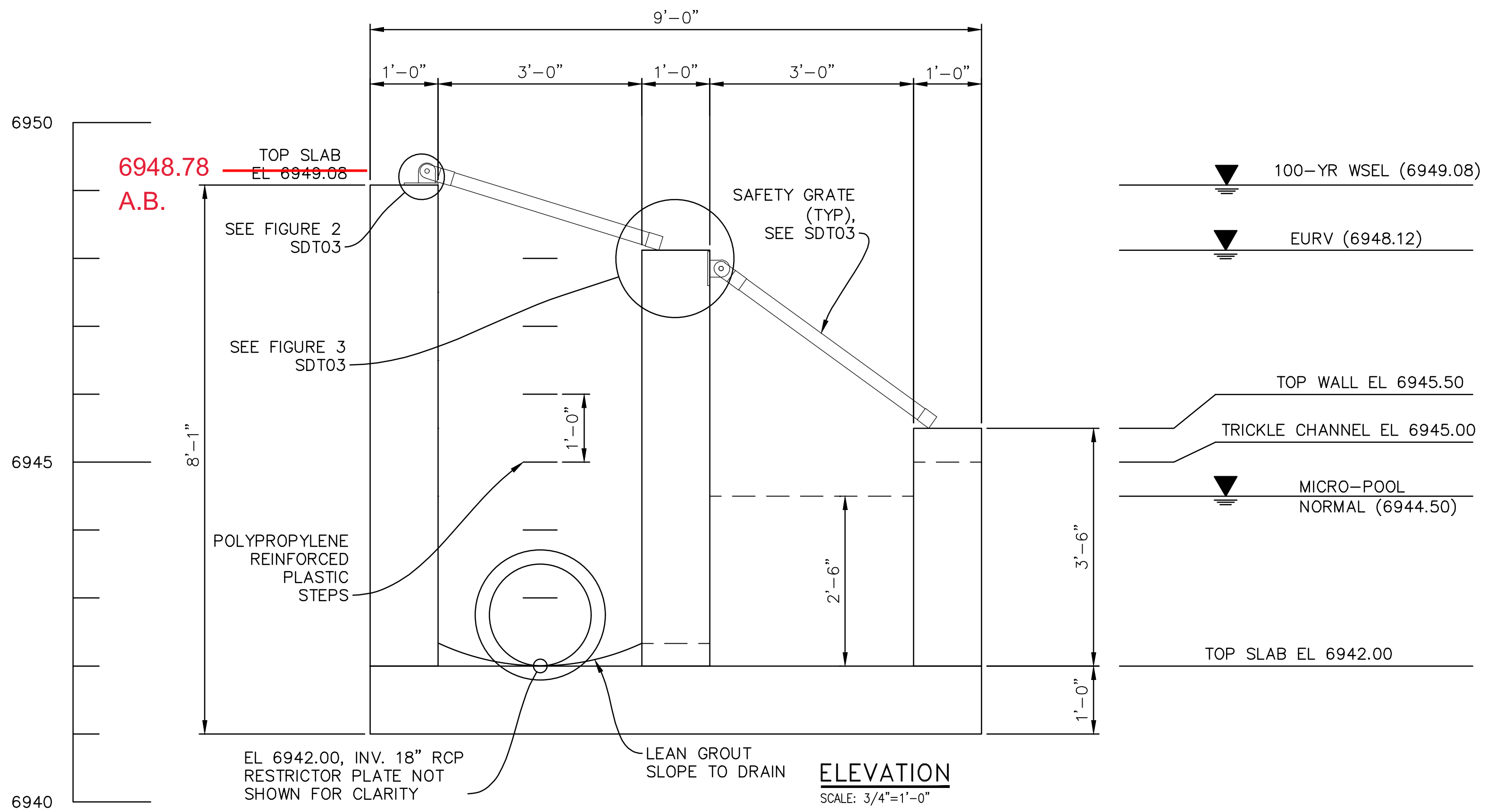
Know what's below. Call before you dig.



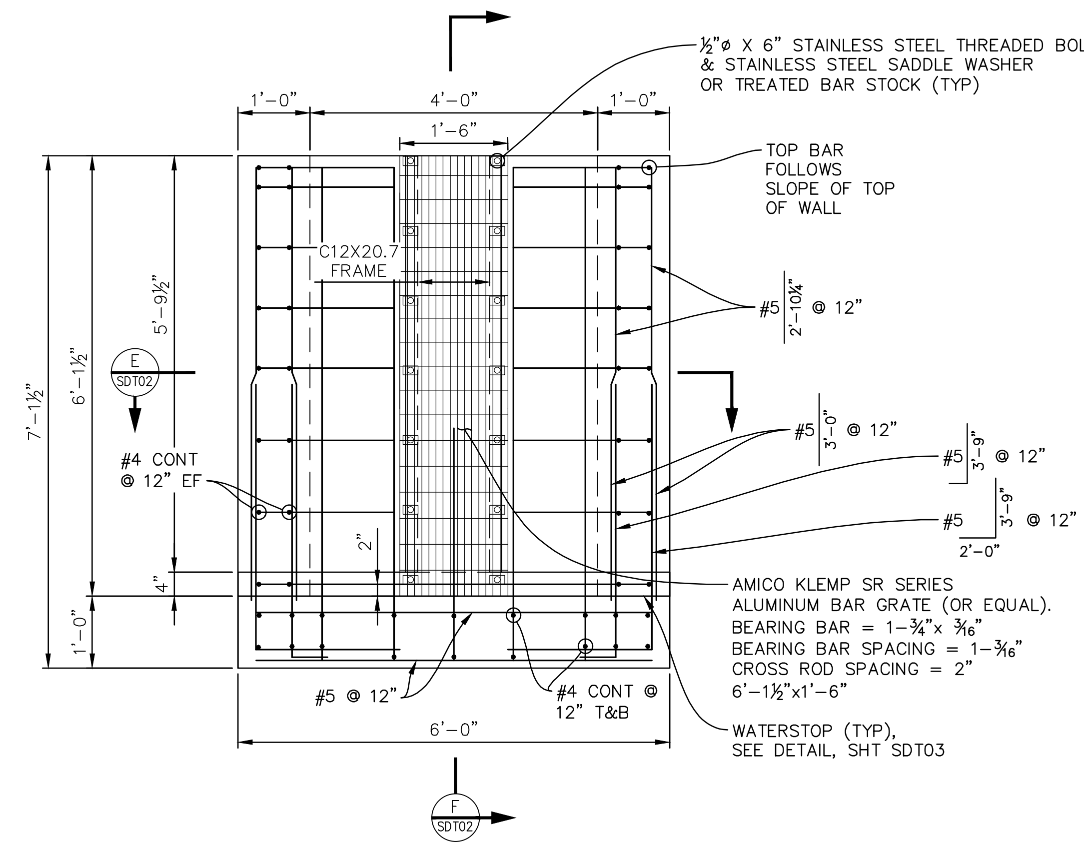

PLAN
SCALE: 3/4"=1'-0"




SECTION A
SCALE: 3/4"=1'-0"




ELEVATION
SCALE: 3/4"=1'-0"




SECTION B
SCALE: 3/4"=1'-0"

REFERENCE DRAWINGS
X-995-MDG22x34

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

REVISIONS

SHEET KEY

COMPUTER FILE MANAGEMENT
FILE NAME: R:\18.995.001 (Liberty Tree Academy)\Dwg\Construction Plans\Structures\SDT01-03.dwg
CTB FILE: ----
PLOT DATE: 8/10/2018 8:34 AM
THIS DRAWING IS CURRENT AS OF PLOT DATE AND MAY BE SUBJECT TO CHANGE.

PREPARED FOR:
LIBERTY TREE ACADEMY

PREPARED BY:
Matrix DESIGN GROUP
AN EMPLOYEE-OWNED COMPANY

SEAL
24520
8/10/18
PROFESSIONAL ENGINEER
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 18.995.001

# LIBERTY TREE ACADEMY

TOWN OF PEYTON, EL PASO COUNTY
CONSTRUCTION DOCUMENTS, PCD FILE NO. PPR-18-023

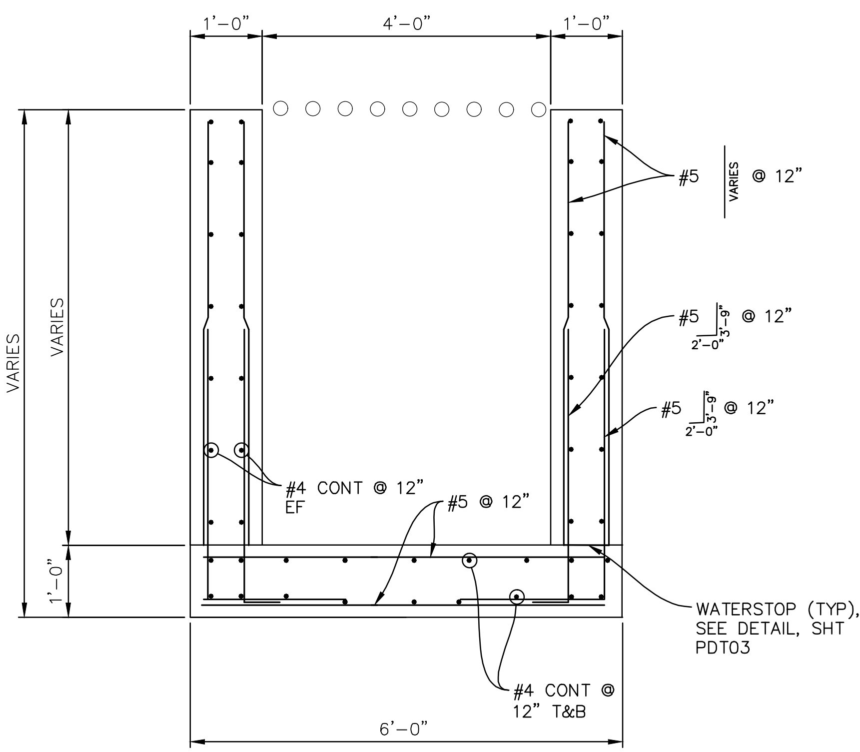
## POND OUTLET STRUCTURE DETAILS

| DESIGNED BY: ACR | SCALE | DATE ISSUED: AUGUST 2018 | DRAWING No. |
|---|---|---|---|
| DRAWN BY: ACR | HORIZ. 3/8" = 1'-0" | SHEET 20 OF 29 | SDT01 |
| CHECKED BY: DRK | VERT. 3/8" = 1'-0" | | |

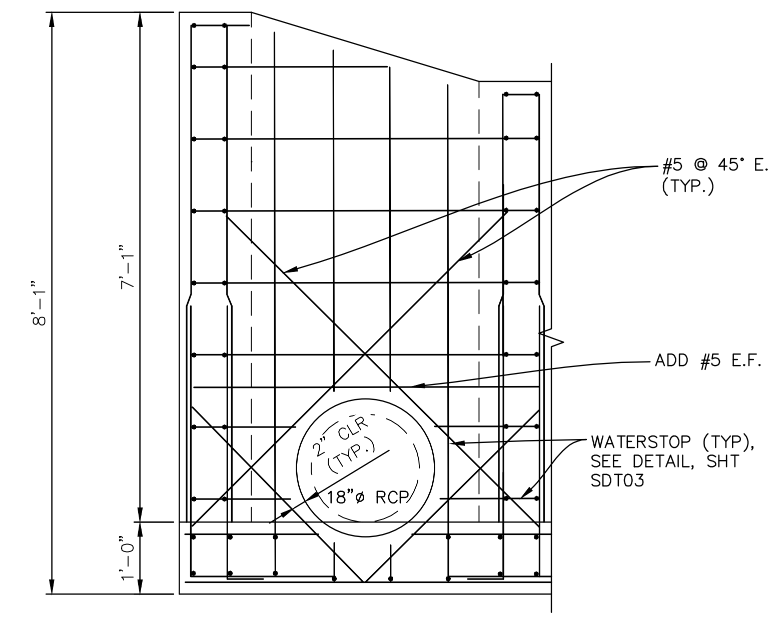
ORIGINAL POND DESIGN FROM 2018 PLANS



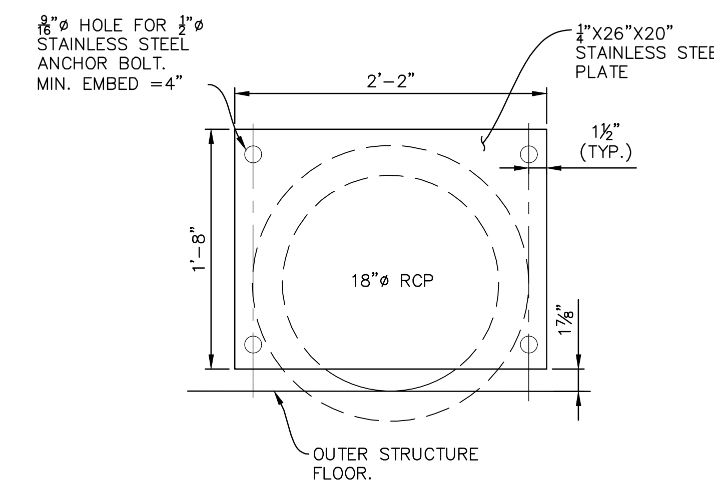
Know what's below. Call before you dig.



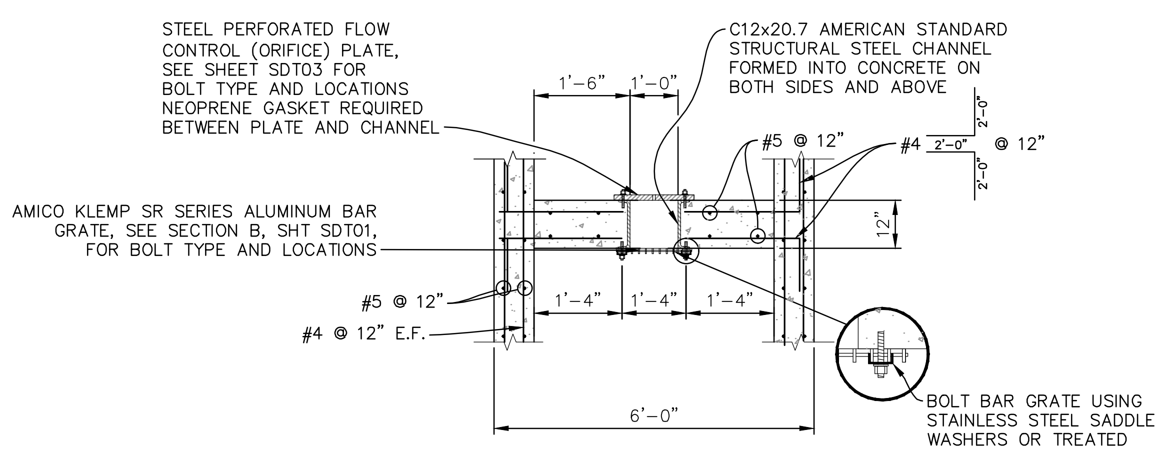

SECTION C SDT01
SCALE: 3/4"=1'-0"



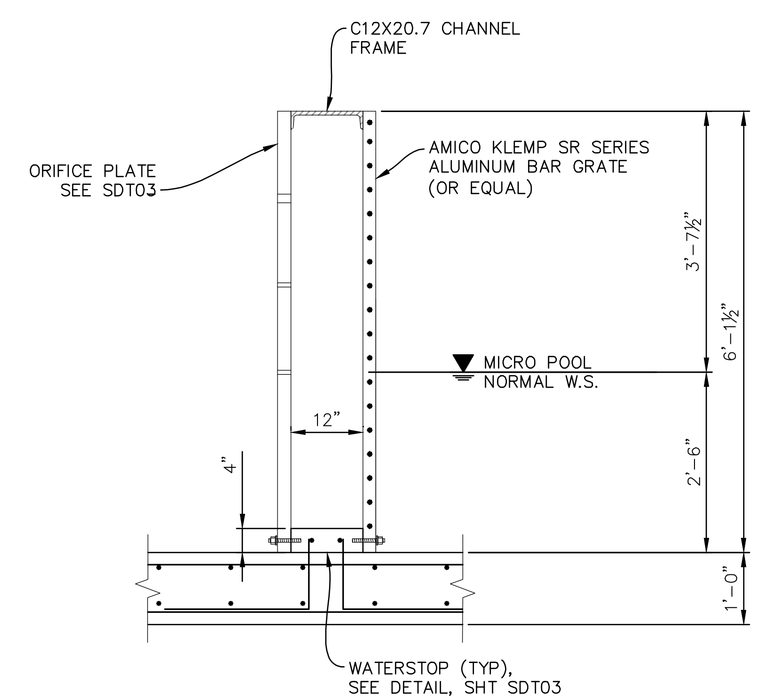

SECTION D SDT01
SCALE: 3/4"=1'-0"




RESTRICTOR PLATE DETAIL
SCALE: 1-1/2"=1"-0"




SECTION E SDT01
SCALE: 1/2"=1'-0"




SECTION F SDT01
SCALE: 3/4"=1'-0"

REFERENCE DRAWINGS
X-995-MDG22x34

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

REVISIONS

COMPUTER FILE MANAGEMENT
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SHEET KEY

PREPARED FOR:
LIBERTY TREE ACADEMY

PREPARED BY:
Matrix DESIGN GROUP
AN EMPLOYEE-OWNED COMPANY

SEAL
24520
8/10/18
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 18.995.001

LIBERTY TREE ACADEMY
TOWN OF PEYTON, EL PASO COUNTY
CONSTRUCTION DOCUMENTS, PCD FILE NO. PPR-18-023

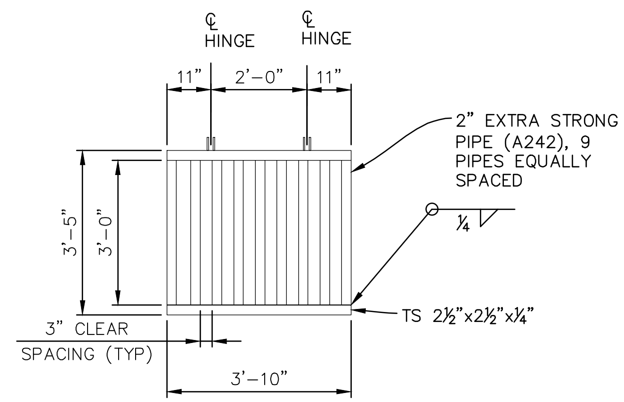
POND OUTLET STRUCTURE DETAILS

DESIGNED BY: ACR
DRAWN BY: ACR
CHECKED BY: DRK
SCALE
HORIZ.
VERT.
DATE ISSUED: AUGUST 2018
SHEET 21 OF 29
DRAWING No. SDT02

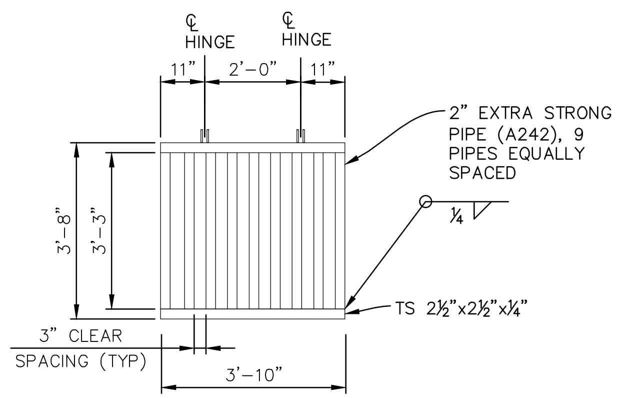
ORIGINAL POND DESIGN FROM 2018 PLANS



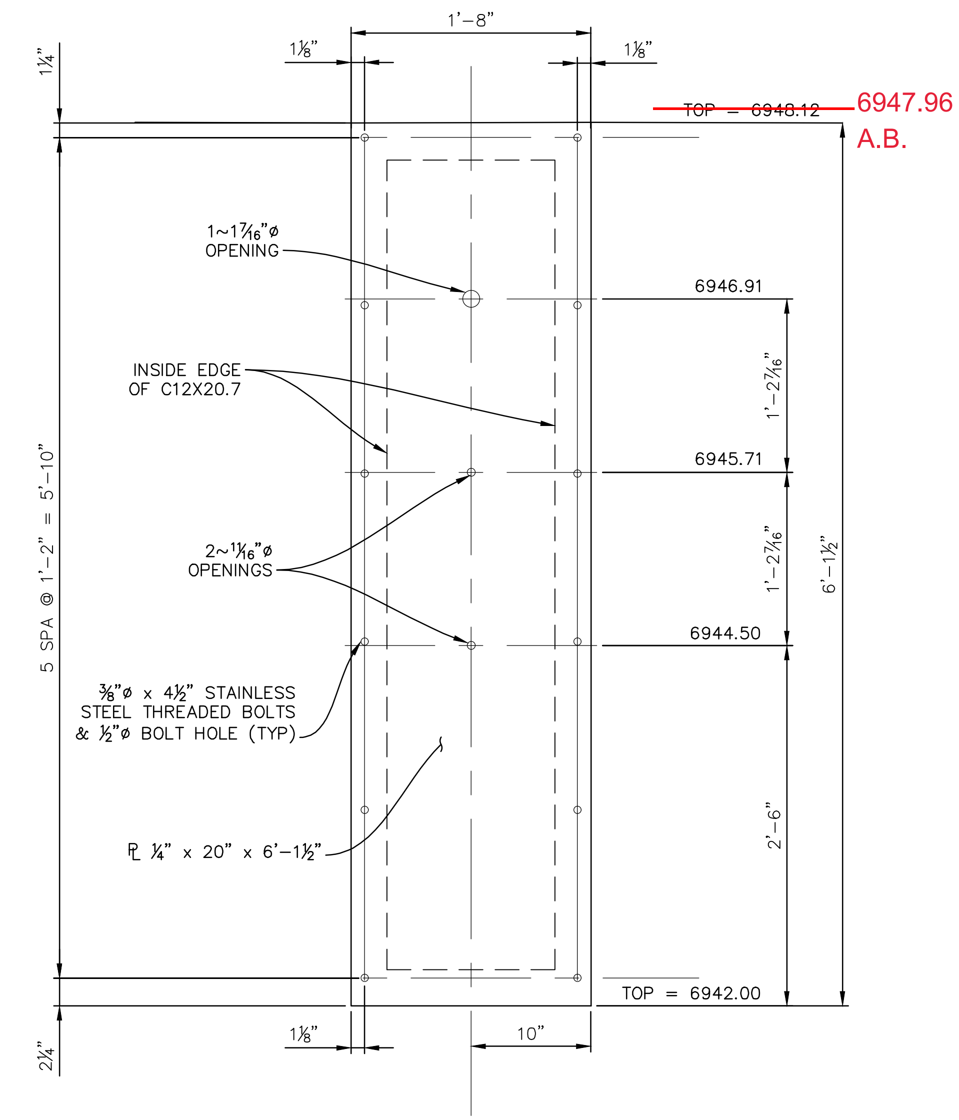
Know what's **below.** **Call** before you dig.



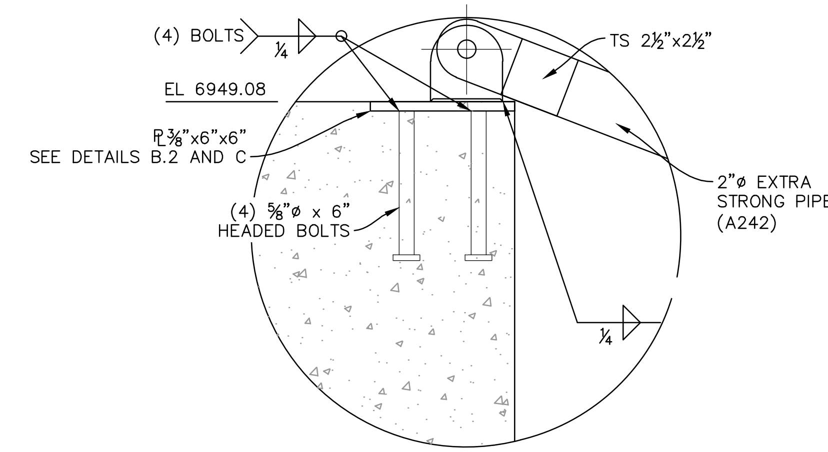

## UPPER SAFETY GRATE PLAN
½"=1'–0"



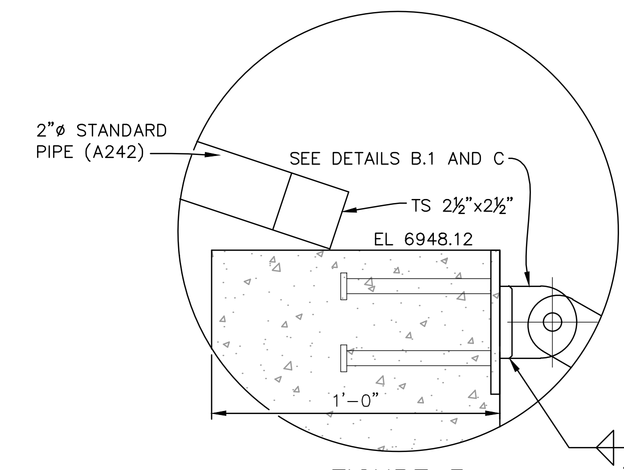

## LOWER SAFETY GRATE PLAN
1/2"=1'–0"



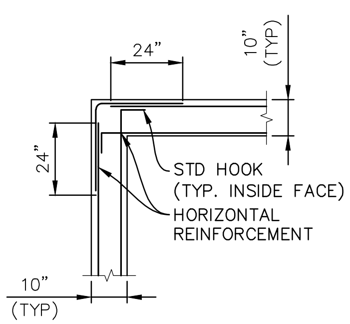

## ORIFICE PLATE
1–1/2"=1'–0"



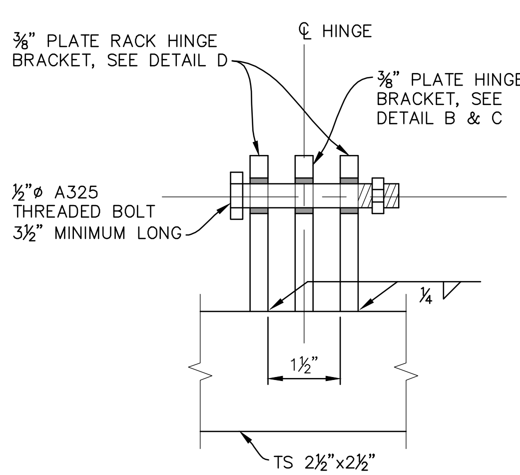

## FIGURE 2
SAFETY GRATE HINGE
3"=1'–0"



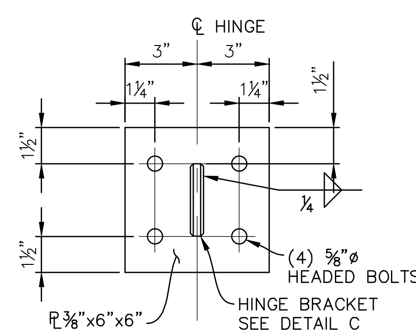

## FIGURE 3
SAFETY GRATE HINGE
3"=1'–0"



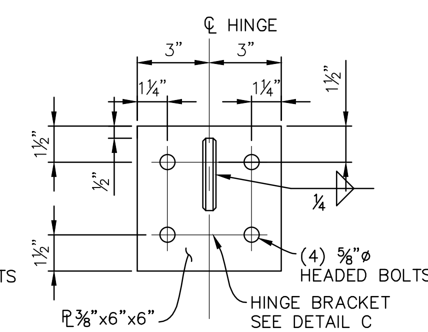

## CORNER REINFORCING DETAIL
PLAN VIEW
3/8"=1'–0"



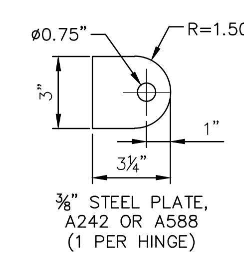

## DETAIL A
6"=1'–0"



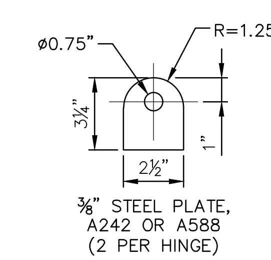

## DETAIL B.1
(LOWER SAFETY GRATE ONLY)
3"=1'–0"



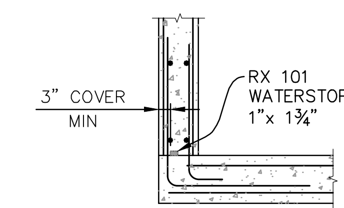

## DETAIL B.2
(UPPER SAFETY GRATE ONLY)
3"=1'–0"




## DETAIL C
## HINGE BRACKET
3"=1'–0"




## DETAIL D
## RACK–HINGE BRACKET
3"=1'–0"




NOTES:

WATERSTOP SHALL BE VOLCLAY WATERSTOP RX 101 OR EARTHSHIELD JP 648 OR APPROVED EQUAL.

PLACE WATERSTOP FOR ENTIRETY OF EACH HORIZONTAL & VERTICAL CONSTRUCTION JOINT.

FOLLOW MANUFACTURER'S INSTALLATION INSTRUCTIONS.

## WATERSTOP DETAIL

REFERENCE DRAWINGS
X-995-MDG22x34

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

REVISIONS

COMPUTER FILE MANAGEMENT
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PREPARED FOR:
LIBERTY TREE ACADEMY

PREPARED BY:
Matrix DESIGN GROUP
AN EMPLOYEE-OWNED COMPANY

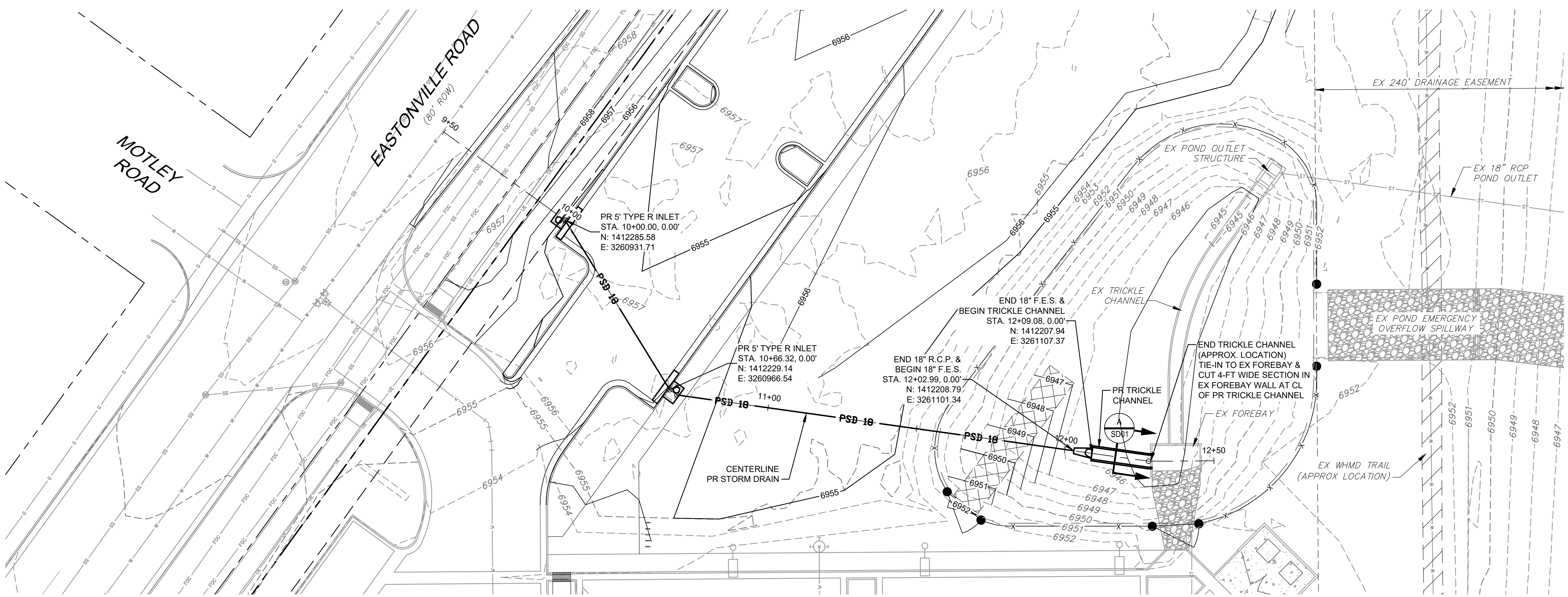
SEAL
COLORADO REGISTERED PROFESSIONAL ENGINEER 24520 8/10/18
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 18.995.001

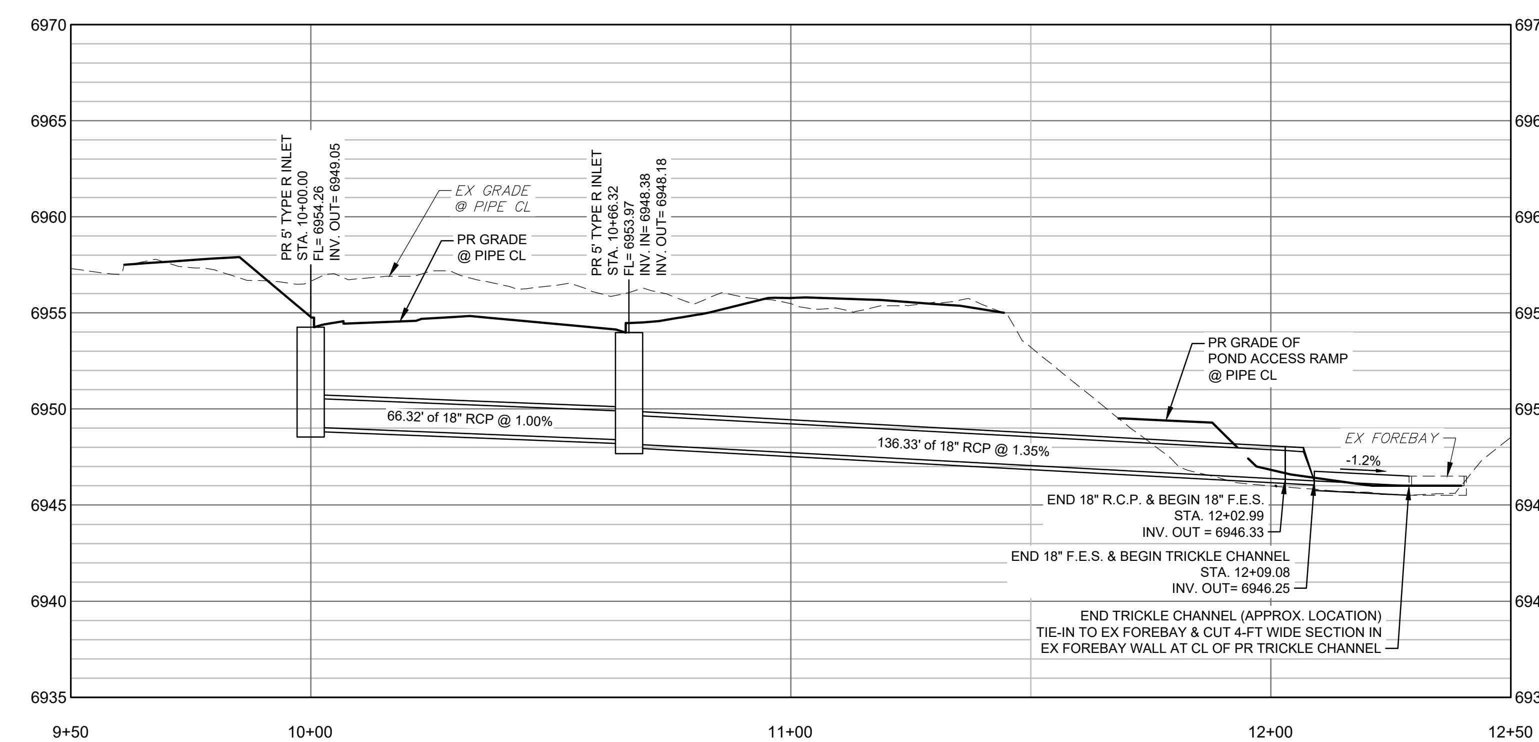
LIBERTY TREE ACADEMY
TOWN OF PEYTON, EL PASO COUNTY
CONSTRUCTION DOCUMENTS, PCD FILE NO. PPR-18-023

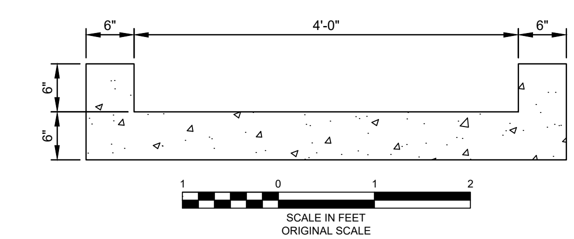
POND OUTLET STRUCTURE DETAILS

DESIGNED BY: ACR
DRAWN BY: ACR
CHECKED BY: DRK
SCALE: HORIZ. AS SHOWN; VERT. AS SHOWN
DATE ISSUED: AUGUST 2018
SHEET 22 OF 29
DRAWING No. STD03

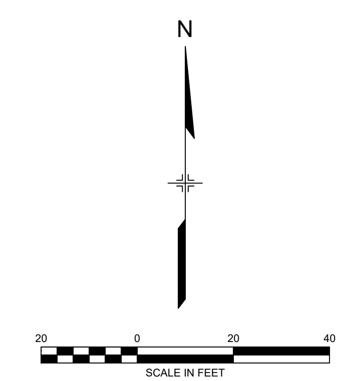
811 Know what's below. Call before you dig.




TRICKLE CHANNEL SECTION (A)




REFERENCE DRAWINGS
X-995.002-MDG22x34
X-995.002-PR-BASE_PH-2
X-995.002-EX-BASE
X-995.002-EX-BASE_PH-1
X-995.002-EX-MAP
X-995.002-PR-UTIL_PH-2

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

REVISIONS

COMPUTER FILE MANAGEMENT
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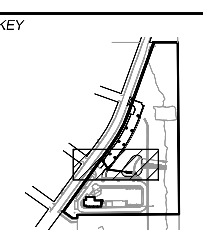
SHEET KEY



PREPARED FOR:
LIBERTY TREE ACADEMY
PREPARED BY:
Matrix
Excellence by Design

SEAL
FOR AND ON BEHALF OF
MATRIX DESIGN GROUP, INC.
PROJECT No. 20.995.002

# LIBERTY TREE ACADEMY - PHASE 2

TOWN OF PEYTON, EL PASO COUNTY
CONSTRUCTION DOCUMENTS, PCD FILE NO. PPR-20-018

## STORM DRAIN PLAN & PROFILE

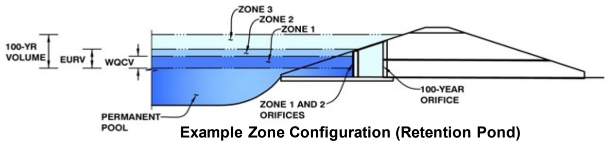
| DESIGNED BY: ACR | SCALE | DATE ISSUED: DECEMBER 2020 | DRAWING No. |
|---|---|---|---|
| DRAWN BY: ACR | HORIZ. 1" = 20' | | SD01 |
| CHECKED BY: DRK | VERT. 1" = 5' | SHEET 8 OF 19 | |

# DETENTION BASIN STAGE-STORAGE TABLE BUILDER

*MHFD-Detention, Version 4.02 (February 2020)*

**Project:** **Liberty Tree - Phase II Improvements**

**Basin ID:** **Extended Detention Basin**




**Example Zone Configuration (Retention Pond)**

Watershed Information

| | | |
|---|---|---|
| Selected BMP Type = | **EDB** | |
| Watershed Area = | 4.15 | acres |
| Watershed Length = | 765 | ft |
| Watershed Length to Centroid = | 185 | ft |
| Watershed Slope = | 0.010 | ft/ft |
| Watershed Imperviousness = | 67.90% | percent |
| Percentage Hydrologic Soil Group A = | 100.0% | percent |
| Percentage Hydrologic Soil Group B = | 0.0% | percent |
| Percentage Hydrologic Soil Groups C/D = | 0.0% | percent |
| Target WQCV Drain Time = | 40.0 | hours |
| Location for 1-hr Rainfall Depths = | User Input | |

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

| | | | Optional User Overrides | |
|---|---|---|---|---|
| Water Quality Capture Volume (WQCV) = | 0.092 | acre-feet | | acre-feet |
| Excess Urban Runoff Volume (EURV) = | 0.354 | acre-feet | | acre-feet |
| 2-yr Runoff Volume (P1 = 1.19 in.) = | 0.248 | acre-feet | 1.19 | inches |
| 5-yr Runoff Volume (P1 = 1.5 in.) = | 0.324 | acre-feet | 1.50 | inches |
| 10-yr Runoff Volume (P1 = 1.75 in.) = | 0.385 | acre-feet | 1.75 | inches |
| 25-yr Runoff Volume (P1 = 2 in.) = | 0.463 | acre-feet | 2.00 | inches |
| 50-yr Runoff Volume (P1 = 2.25 in.) = | 0.540 | acre-feet | 2.25 | inches |
| 100-yr Runoff Volume (P1 = 2.52 in.) = | 0.632 | acre-feet | 2.52 | inches |
| 500-yr Runoff Volume (P1 = 3.14 in.) = | 0.835 | acre-feet | 3.14 | inches |
| Approximate 2-yr Detention Volume = | 0.231 | acre-feet | | |
| Approximate 5-yr Detention Volume = | 0.301 | acre-feet | | |
| Approximate 10-yr Detention Volume = | 0.362 | acre-feet | | |
| Approximate 25-yr Detention Volume = | 0.435 | acre-feet | | |
| Approximate 50-yr Detention Volume = | 0.478 | acre-feet | | |
| Approximate 100-yr Detention Volume = | 0.522 | acre-feet | | |

Define Zones and Basin Geometry

| | | |
|---|---|---|
| Zone 1 Volume (WQCV) = | 0.092 | acre-feet |
| Zone 2 Volume (EURV - Zone 1) = | 0.262 | acre-feet |
| Zone 3 Volume (100-year - Zones 1 & 2) = | 0.168 | acre-feet |
| Total Detention Basin Volume = | 0.522 | acre-feet |

Depth Increment = ft

| Stage - Storage Description | Stage (ft) | Optional Override Stage (ft) | Length (ft) | Width (ft) | Area ($ft^2$) | Optional Override Area ($ft^2$) | Area (acre) | Volume ($ft^3$) | Volume (ac-ft) |
|---|---|---|---|---|---|---|---|---|---|
| **Top of Micropool** | -- | 0.00 | -- | -- | -- | 12 | 0.000 | | |
| **6,945.00** | -- | 0.50 | -- | -- | -- | 94 | 0.002 | 27 | 0.001 |
| **6,946.00** | -- | 1.50 | -- | -- | -- | 1,900 | 0.044 | 1,024 | 0.023 |
| **6,947.00** | -- | 2.50 | -- | -- | -- | 4,666 | 0.107 | 4,306 | 0.099 |
| **6,948.00** | -- | 3.50 | -- | -- | -- | 6,031 | 0.138 | 9,655 | 0.222 |
| **6,949.00** | -- | 4.50 | -- | -- | -- | 7,516 | 0.173 | 16,428 | 0.377 |
| **6,950.00** | -- | 5.50 | -- | -- | -- | 9,217 | 0.212 | 24,795 | 0.569 |
| **6,951.00** | -- | 6.50 | -- | -- | -- | 11,057 | 0.254 | 34,932 | 0.802 |
| **6,952.00** | -- | 7.50 | -- | -- | -- | 12,709 | 0.292 | 46,815 | 1.075 |
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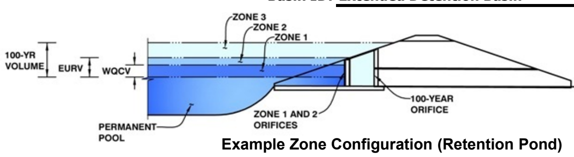
# DETENTION BASIN OUTLET STRUCTURE DESIGN

*MHFD-Detention, Version 4.02 (February 2020)*

**Project:** **Liberty Tree - Phase II Improvements**

**Basin ID:** **Extended Detention Basin**




**Example Zone Configuration (Retention Pond)**

| | Estimated Stage (ft) | Estimated Volume (ac-ft) | Outlet Type |
|---|---|---|---|
| Zone 1 (WQCV) | 2.44 | 0.092 | Orifice Plate |
| Zone 2 (EURV) | 4.37 | 0.262 | Orifice Plate |
| Zone 3 (100-year) | 5.28 | 0.168 | Weir&Pipe (Restrict) |
| Total (all zones) | | 0.522 | |

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

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

Calculated Parameters for Underdrain

| | | |
|---|---|---|
| Underdrain Orifice Area = | | $ft^2$ |
| Underdrain Orifice Centroid = | | feet |

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

| | | |
|---|---|---|
| Invert of Lowest Orifice = | 0.00 | ft (relative to basin bottom at Stage = 0 ft) |
| Depth at top of Zone using Orifice Plate = | 4.92 | ft (relative to basin bottom at Stage = 0 ft) |
| Orifice Plate: Orifice Vertical Spacing = | N/A | inches |
| Orifice Plate: Orifice Area per Row = | N/A | inches |

Calculated Parameters for Plate

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

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

| | Row 1 (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 | 1.21 | 2.41 | | | | | |
| Orifice Area (sq. inches) | 0.44 | 0.44 | 1.62 | | | | | |

| | 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)

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

Calculated Parameters for Vertical Orifice

| | Not Selected | Not Selected | |
|---|---|---|---|
| Vertical Orifice Area = | N/A | N/A | $ft^2$ |
| Vertical Orifice Centroid = | N/A | N/A | feet |

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

| | Zone 3 Weir | Not Selected | |
|---|---|---|---|
| Overflow Weir Front Edge Height, Ho = | 3.46 | N/A | ft (relative to basin bottom at Stage = 0 ft) |
| Overflow Weir Front Edge Length = | 4.00 | N/A | feet |
| Overflow Weir Grate Slope = | 3.66 | N/A | H:V |
| Horiz. Length of Weir Sides = | 3.00 | N/A | feet |
| Overflow Grate Open Area % = | 70% | N/A | %, grate open area/total area |
| Debris Clogging % = | 50% | N/A | % |

Calculated Parameters for Overflow Weir

| | Zone 3 Weir | Not Selected | |
|---|---|---|---|
| Height of Grate Upper Edge, $H_t$ = | 4.28 | N/A | feet |
| Overflow Weir Slope Length = | 3.11 | N/A | feet |
| Grate Open Area / 100-yr Orifice Area = | 88.83 | N/A | |
| Overflow Grate Open Area w/o Debris = | 8.71 | N/A | $ft^2$ |
| Overflow Grate Open Area w/ Debris = | 4.35 | 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 = | 2.50 | N/A | ft (distance below basin bottom at Stage = 0 ft) |
| Outlet Pipe Diameter = | 18.00 | N/A | inches |
| Restrictor Plate Height Above Pipe Invert = | 1.88 | | inches |

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

| | Zone 3 Restrictor | Not Selected | |
|---|---|---|---|
| Outlet Orifice Area = | 0.10 | N/A | $ft^2$ |
| Outlet Orifice Centroid = | 0.09 | N/A | feet |
| Half-Central Angle of Restrictor Plate on Pipe = | 0.66 | N/A | radians |

User Input: Emergency Spillway (Rectangular or Trapezoidal)

| | | |
|---|---|---|
| Spillway Invert Stage= | 7.02 | ft (relative to basin bottom at Stage = 0 ft) |
| Spillway Crest Length = | 8.82 | feet |
| Spillway End Slopes = | 4.00 | H:V |
| Freeboard above Max Water Surface = | 0.40 | feet |

Calculated Parameters for Spillway

| | | |
|---|---|---|
| Spillway Design Flow Depth= | 0.50 | feet |
| Stage at Top of Freeboard = | 7.92 | feet |
| Basin Area at Top of Freeboard = | 0.29 | acres |
| Basin Volume at Top of Freeboard = | 1.07 | acre-ft |

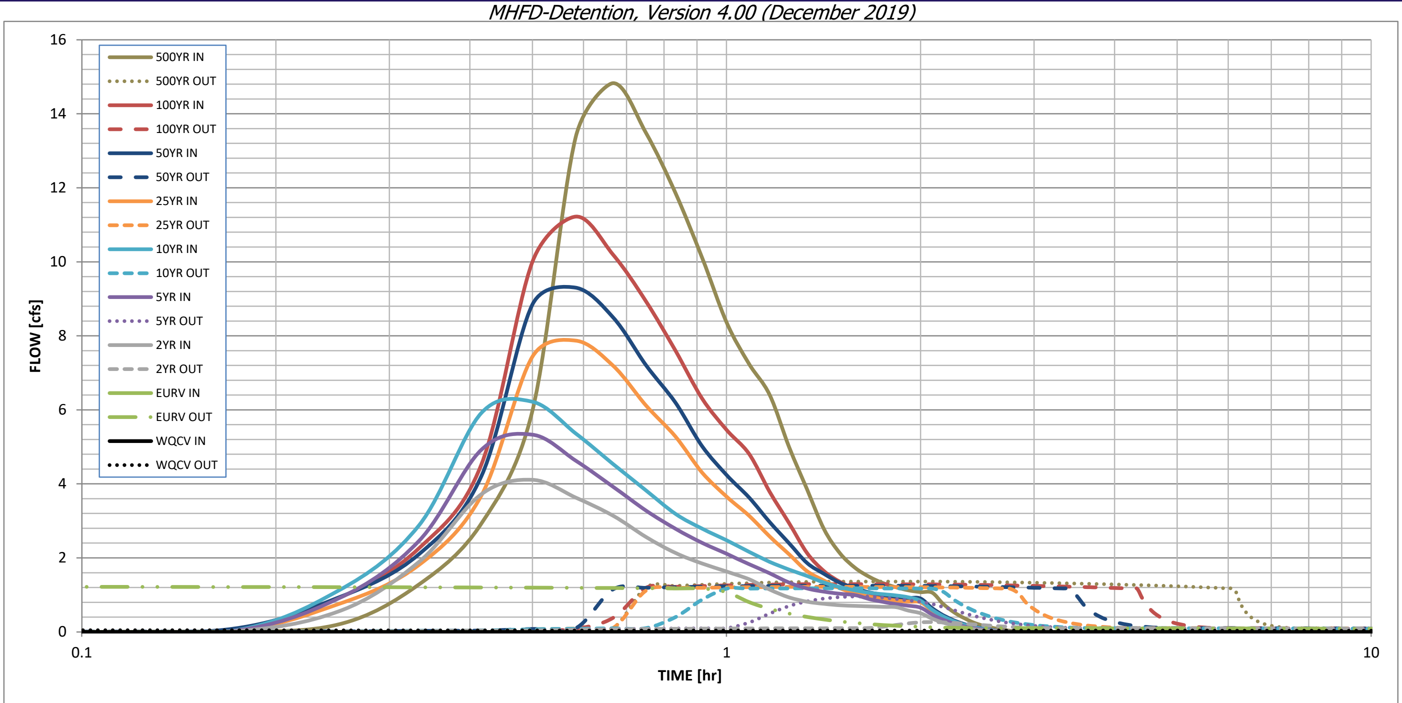
Routed Hydrograph Results

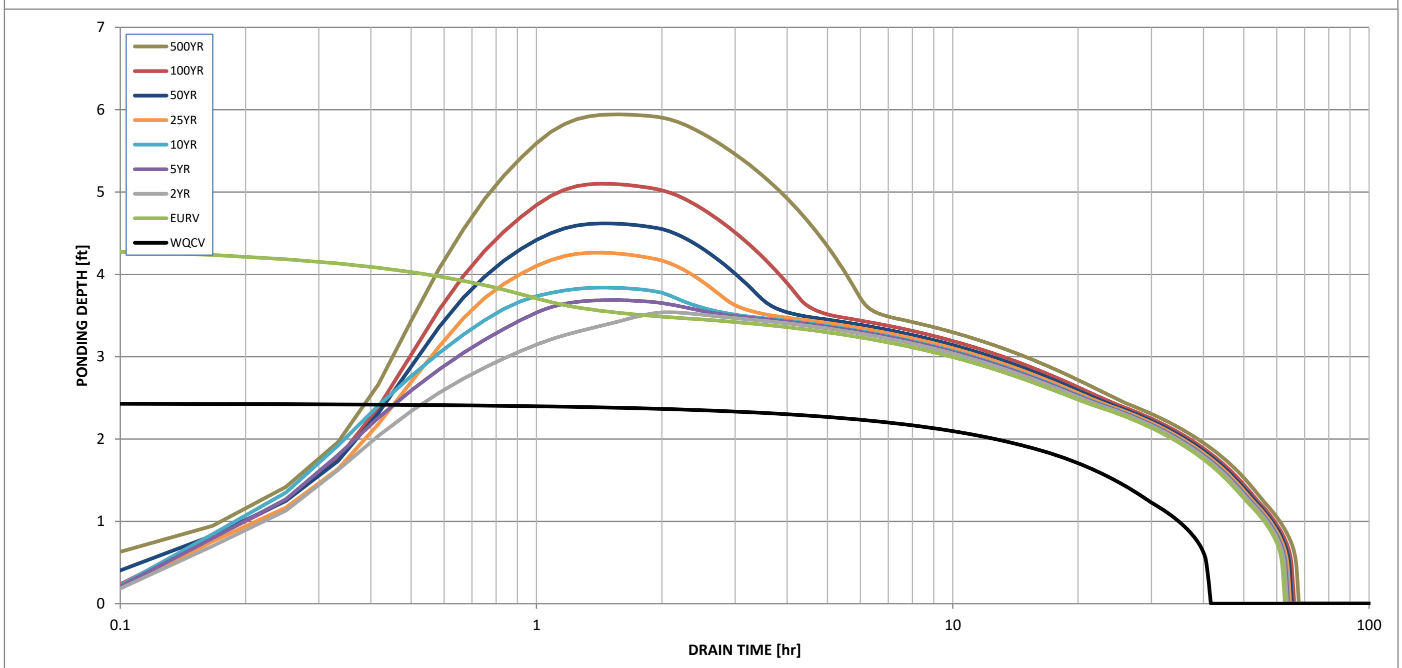
*The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).*

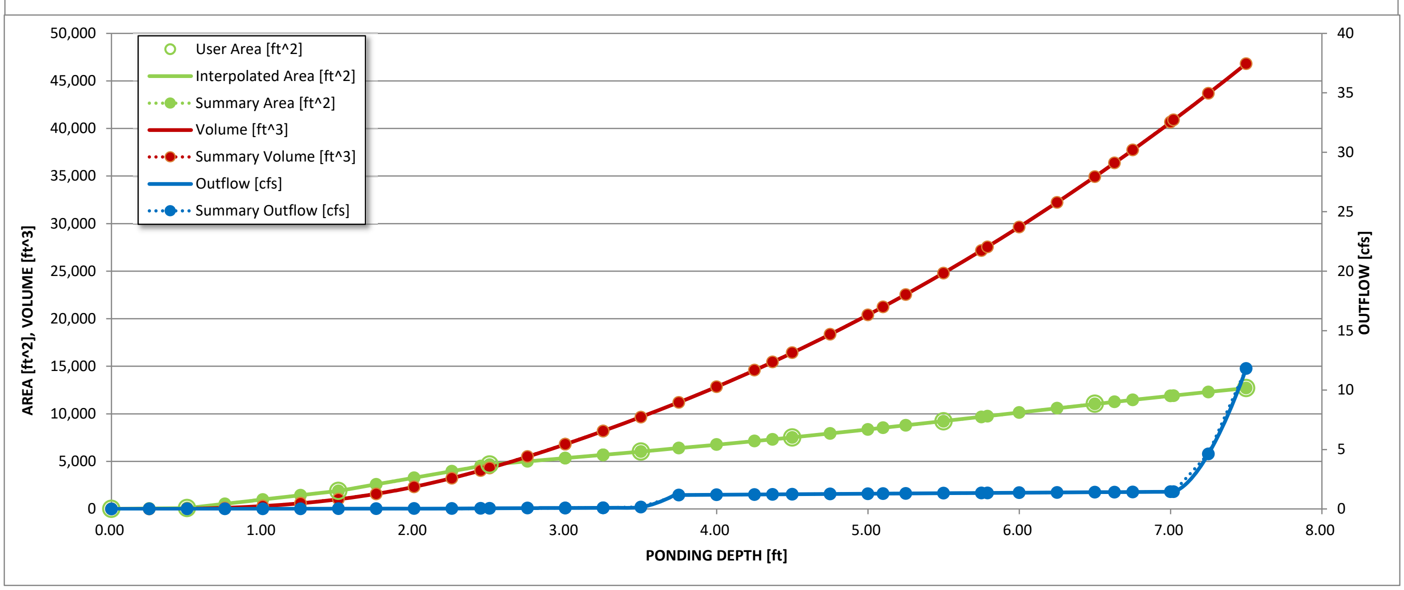
| Design Storm Return Period = | WQCV | EURV | 2 Year | 5 Year | 10 Year | 25 Year | 50 Year | 100 Year | 500 Year |
|---|---|---|---|---|---|---|---|---|---|
| One-Hour Rainfall Depth (in) = | N/A | N/A | 1.19 | 1.50 | 1.75 | 2.00 | 2.25 | 2.52 | 3.14 |
| CUHP Runoff Volume (acre-ft) = | 0.092 | 0.354 | 0.248 | 0.324 | 0.385 | 0.463 | 0.540 | 0.632 | 0.835 |
| Inflow Hydrograph Volume (acre-ft) = | N/A | N/A | 0.248 | 0.324 | 0.385 | 0.463 | 0.540 | 0.632 | 0.835 |
| CUHP Predevelopment Peak Q (cfs) = | N/A | N/A | 0.03 | 0.06 | 0.08 | 0.75 | 1.51 | 2.46 | 4.40 |
| OPTIONAL Override Predevelopment Peak Q (cfs) = | N/A | N/A | | | | | | | |
| Predevelopment Unit Peak Flow, q (cfs/acre) = | N/A | N/A | 0.01 | 0.01 | 0.02 | 0.18 | 0.36 | 0.59 | 1.06 |
| Peak Inflow Q (cfs) = | N/A | N/A | 4.11 | 5.33 | 6.22 | 7.87 | 9.30 | 11.22 | 14.83 |
| Peak Outflow Q (cfs) = | 0.05 | 1.23 | 0.27 | 0.96 | 1.18 | 1.22 | 1.25 | 1.29 | 1.36 |
| Ratio Peak Outflow to Predevelopment Q = | N/A | N/A | N/A | 16.01 | 14.31 | 1.62 | 0.83 | 0.53 | 0.31 |
| Structure Controlling Flow = | Plate | Outlet Plate 1 | Overflow Weir 1 | Overflow Weir 1 | Outlet Plate 1 | Outlet Plate 1 | Outlet Plate 1 | Outlet Plate 1 | Outlet Plate 1 |
| Max Velocity through Grate 1 (fps) = | N/A | 0.13 | 0.02 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 |
| Max Velocity through Grate 2 (fps) = | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Time to Drain 97% of Inflow Volume (hours) = | 39 | 53 | 56 | 55 | 54 | 53 | 52 | 51 | 50 |
| Time to Drain 99% of Inflow Volume (hours) = | **40** | 59 | 61 | 60 | 60 | 60 | 60 | 60 | 60 |
| Maximum Ponding Depth (ft) = | 2.44 | 4.37 | 3.54 | 3.69 | 3.84 | 4.27 | 4.62 | 5.10 | 5.95 |
| Area at Maximum Ponding Depth (acres) = | 0.10 | 0.17 | 0.14 | 0.14 | 0.15 | 0.16 | 0.18 | 0.20 | 0.23 |
| Maximum Volume Stored (acre-ft) = | 0.093 | 0.355 | 0.227 | 0.247 | 0.271 | 0.337 | 0.398 | 0.488 | 0.666 |

# DETENTION BASIN OUTLET STRUCTURE DESIGN

*MHFD-Detention, Version 4.00 (December 2019)*



# DETENTION BASIN OUTLET STRUCTURE DESIGN

*Outflow Hydrograph Workbook Filename:*

Inflow Hydrographs

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

| | SOURCE | CUHP | CUHP | CUHP | CUHP | CUHP | CUHP | CUHP | CUHP | CUHP |
|---|---|---|---|---|---|---|---|---|---|---|
| Time Interval | TIME | WQCV [cfs] | EURV [cfs] | 2 Year [cfs] | 5 Year [cfs] | 10 Year [cfs] | 25 Year [cfs] | 50 Year [cfs] | 100 Year [cfs] | 500 Year [cfs] |
| 5.00 min | 0:00:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 0:05:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 0:10:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.06 | 0.01 | 0.20 |
| | 0:15:00 | 0.00 | 0.00 | 0.55 | 0.90 | 1.12 | 0.75 | 0.93 | 0.91 | 1.29 |
| | 0:20:00 | 0.00 | 0.00 | 1.90 | 2.46 | 2.88 | 1.81 | 2.10 | 2.26 | 2.93 |
| | 0:25:00 | 0.00 | 0.00 | 3.72 | 4.92 | 5.92 | 3.69 | 4.21 | 4.53 | 5.98 |
| | 0:30:00 | 0.00 | 0.00 | 4.11 | 5.33 | 6.22 | 7.44 | 8.85 | 10.00 | 13.37 |
| | 0:35:00 | 0.00 | 0.00 | 3.63 | 4.63 | 5.37 | 7.87 | 9.30 | 11.22 | 14.83 |
| | 0:40:00 | 0.00 | 0.00 | 3.14 | 3.93 | 4.54 | 7.20 | 8.50 | 10.20 | 13.49 |
| | 0:45:00 | 0.00 | 0.00 | 2.57 | 3.28 | 3.82 | 6.12 | 7.21 | 8.94 | 11.86 |
| | 0:50:00 | 0.00 | 0.00 | 2.13 | 2.79 | 3.19 | 5.28 | 6.19 | 7.60 | 10.11 |
| | 0:55:00 | 0.00 | 0.00 | 1.85 | 2.41 | 2.79 | 4.31 | 5.02 | 6.31 | 8.37 |
| | 1:00:00 | 0.00 | 0.00 | 1.63 | 2.11 | 2.48 | 3.66 | 4.24 | 5.46 | 7.25 |
| | 1:05:00 | 0.00 | 0.00 | 1.43 | 1.83 | 2.17 | 3.14 | 3.63 | 4.81 | 6.41 |
| | 1:10:00 | 0.00 | 0.00 | 1.15 | 1.59 | 1.89 | 2.58 | 2.97 | 3.79 | 5.00 |
| | 1:15:00 | 0.00 | 0.00 | 0.94 | 1.34 | 1.69 | 2.10 | 2.40 | 2.95 | 3.85 |
| | 1:20:00 | 0.00 | 0.00 | 0.82 | 1.18 | 1.51 | 1.64 | 1.86 | 2.13 | 2.77 |
| | 1:25:00 | 0.00 | 0.00 | 0.76 | 1.09 | 1.33 | 1.39 | 1.57 | 1.64 | 2.12 |
| | 1:30:00 | 0.00 | 0.00 | 0.72 | 1.03 | 1.21 | 1.18 | 1.33 | 1.35 | 1.73 |
| | 1:35:00 | 0.00 | 0.00 | 0.70 | 0.99 | 1.12 | 1.04 | 1.18 | 1.17 | 1.49 |
| | 1:40:00 | 0.00 | 0.00 | 0.69 | 0.89 | 1.06 | 0.95 | 1.07 | 1.04 | 1.32 |
| | 1:45:00 | 0.00 | 0.00 | 0.68 | 0.81 | 1.02 | 0.89 | 1.00 | 0.96 | 1.21 |
| | 1:50:00 | 0.00 | 0.00 | 0.67 | 0.75 | 0.99 | 0.85 | 0.95 | 0.90 | 1.13 |
| | 1:55:00 | 0.00 | 0.00 | 0.58 | 0.71 | 0.94 | 0.82 | 0.92 | 0.87 | 1.08 |
| | 2:00:00 | 0.00 | 0.00 | 0.51 | 0.66 | 0.84 | 0.80 | 0.90 | 0.85 | 1.06 |
| | 2:05:00 | 0.00 | 0.00 | 0.36 | 0.47 | 0.60 | 0.57 | 0.64 | 0.61 | 0.76 |
| | 2:10:00 | 0.00 | 0.00 | 0.26 | 0.33 | 0.42 | 0.41 | 0.46 | 0.43 | 0.54 |
| | 2:15:00 | 0.00 | 0.00 | 0.18 | 0.23 | 0.30 | 0.28 | 0.32 | 0.30 | 0.38 |
| | 2:20:00 | 0.00 | 0.00 | 0.12 | 0.15 | 0.20 | 0.19 | 0.22 | 0.21 | 0.26 |
| | 2:25:00 | 0.00 | 0.00 | 0.08 | 0.10 | 0.13 | 0.13 | 0.14 | 0.14 | 0.17 |
| | 2:30:00 | 0.00 | 0.00 | 0.05 | 0.07 | 0.09 | 0.09 | 0.10 | 0.09 | 0.11 |
| | 2:35:00 | 0.00 | 0.00 | 0.03 | 0.04 | 0.05 | 0.05 | 0.06 | 0.05 | 0.07 |
| | 2:40:00 | 0.00 | 0.00 | 0.01 | 0.02 | 0.02 | 0.03 | 0.03 | 0.03 | 0.03 |
| | 2:45:00 | 0.00 | 0.00 | 0.00 | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| | 2:50:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 2:55:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:00:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:05:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:10:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:15:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:20:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:25:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:30:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:35:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:40:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:45:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:50:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 3:55:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:00:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:05:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:10:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:15:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:20:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:25:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:30:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:35:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:40:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:45:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:50:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 4:55:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:00:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:05:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:10:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:15:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:20:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:25:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:30:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:35:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:40:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:45:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:50:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 5:55:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 6:00:00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

# DETENTION BASIN OUTLET STRUCTURE DESIGN

*MHFD-Detention, Version 4.02 (February 2020)*

Summary Stage-Area-Volume-Discharge Relationships

The user can create a summary S-A-V-D by entering the desired stage increments and the remainder of the table will populate automatically.

The user should graphically compare the summary S-A-V-D table to the full S-A-V-D table in the chart to confirm it captures all key transition points.

| Stage - Storage Description | Stage [ft] | Area [ft $^2$] | Area [acres] | Volume [ft $^3$] | Volume [ac-ft] | Total Outflow [cfs] |
|---|---|---|---|---|---|---|
| Micropool=6944.5 | 0.00 | 12 | 0.000 | 0 | 0.000 | 0.00 |
| 6944.75 | 0.25 | 53 | 0.001 | 8 | 0.000 | 0.01 |
| 6945 | 0.50 | 94 | 0.002 | 27 | 0.001 | 0.01 |
| 6945.25 | 0.75 | 546 | 0.013 | 107 | 0.002 | 0.01 |
| 6945.5 | 1.00 | 997 | 0.023 | 299 | 0.007 | 0.01 |
| 6945.75 | 1.25 | 1,448 | 0.033 | 605 | 0.014 | 0.02 |
| 6946 | 1.50 | 1,900 | 0.044 | 1,024 | 0.023 | 0.03 |
| 6946.25 | 1.75 | 2,591 | 0.059 | 1,585 | 0.036 | 0.03 |
| 6946.5 | 2.00 | 3,283 | 0.075 | 2,319 | 0.053 | 0.03 |
| 6946.75 | 2.25 | 3,974 | 0.091 | 3,226 | 0.074 | 0.04 |
| WQCV=6946.94 | 2.44 | 4,500 | 0.103 | 4,031 | 0.093 | 0.05 |
| 6947 | 2.50 | 4,666 | 0.107 | 4,306 | 0.099 | 0.06 |
| 6947.25 | 2.75 | 5,007 | 0.115 | 5,515 | 0.127 | 0.07 |
| 6947.5 | 3.00 | 5,349 | 0.123 | 6,810 | 0.156 | 0.09 |
| 6947.75 | 3.25 | 5,690 | 0.131 | 8,190 | 0.188 | 0.10 |
| 6947.5 | 3.00 | 5,349 | 0.123 | 6,810 | 0.156 | 0.09 |
| 6947.75 | 3.25 | 5,690 | 0.131 | 8,190 | 0.188 | 0.10 |
| 6948 | 3.50 | 6,031 | 0.138 | 9,655 | 0.222 | 0.16 |
| 6948.25 | 3.75 | 6,402 | 0.147 | 11,209 | 0.257 | 1.17 |
| 6948.5 | 4.00 | 6,774 | 0.155 | 12,856 | 0.295 | 1.19 |
| 6948.75 | 4.25 | 7,145 | 0.164 | 14,596 | 0.335 | 1.22 |
| EURV=6948.87 | 4.37 | 7,323 | 0.168 | 15,464 | 0.355 | 1.23 |
| 6949 | 4.50 | 7,516 | 0.173 | 16,428 | 0.377 | 1.24 |
| 6949.25 | 4.75 | 7,941 | 0.182 | 18,361 | 0.421 | 1.26 |
| 6949.5 | 5.00 | 8,366 | 0.192 | 20,399 | 0.468 | 1.28 |
| 100 YR=6949.6 | 5.10 | 8,536 | 0.196 | 21,244 | 0.488 | 1.29 |
| 6949.75 | 5.25 | 8,792 | 0.202 | 22,544 | 0.518 | 1.31 |
| 6950 | 5.50 | 9,217 | 0.212 | 24,795 | 0.569 | 1.33 |
| 6950.25 | 5.75 | 9,677 | 0.222 | 27,156 | 0.623 | 1.35 |
| 100 YR Clogged=6950.29 | 5.79 | 9,751 | 0.224 | 27,545 | 0.632 | 1.35 |
| 6950.5 | 6.00 | 10,137 | 0.233 | 29,633 | 0.680 | 1.37 |
| 6950.75 | 6.25 | 10,597 | 0.243 | 32,225 | 0.740 | 1.39 |
| 6951 | 6.50 | 11,057 | 0.254 | 34,932 | 0.802 | 1.41 |
| 500 YR Clogged=6951.13 | 6.63 | 11,272 | 0.259 | 36,383 | 0.835 | 1.42 |
| 6951.25 | 6.75 | 11,470 | 0.263 | 37,748 | 0.867 | 1.43 |
| 6951.5 | 7.00 | 11,883 | 0.273 | 40,667 | 0.934 | 1.45 |
| AB Weir Crest El.=6951.52 | 7.02 | 11,916 | 0.274 | 40,905 | 0.939 | 1.45 |
| 6951.75 | 7.25 | 12,296 | 0.282 | 43,689 | 1.003 | 4.63 |
| 6952 | 7.50 | 12,709 | 0.292 | 46,815 | 1.075 | 11.82 |

For best results, include the stages of all grade slope changes (e.g. ISV and Floor) from the S-A-V table on Sheet 'Basin'.

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

# Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.07, March 2018)



**Designer:** MAS
**Company:** Matrix
**Date:** June 25, 2020
**Project:** Liberty Tree Academy
**Location:** Liberty Tree EDB Forebay

| | |
|---|---|
| 1. Basin Storage Volume | |
| A) Effective Imperviousness of Tributary Area, $I_a$ | $I_a$ = 67.9 % |
| B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) | i = 0.679 |
| C) Contributing Watershed Area | Area = 4.150 ac |
| D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm | $d_6$ = in |
| E) Design Concept (Select EURV when also designing for flood control) | Choose One: ○ Water Quality Capture Volume (WQCV); ● Excess Urban Runoff Volume (EURV) |
| F) Design Volume (WQCV) Based on 40-hour Drain Time ($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area)$) | $V_{DESIGN}$= 0.092 ac-ft |
| G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume ($V_{WQCV\ OTHER} = (d_6*(V_{DESIGN}/0.43))$) | $V_{DESIGN\ OTHER}$= ac-ft |
| H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired) | $V_{DESIGN\ USER}$= ac-ft |
| I) NRCS Hydrologic Soil Groups of Tributary Watershed | |
| i) Percentage of Watershed consisting of Type A Soils | $HSG_A$ = % |
| ii) Percentage of Watershed consisting of Type B Soils | $HSG_B$ = % |
| iii) Percentage of Watershed consisting of Type C/D Soils | $HSG_{C/D}$ = % |
| J) Excess Urban Runoff Volume (EURV) Design Volume<br>For HSG A: $EURV_A = 1.68 * i^{1.28}$<br>For HSG B: $EURV_B = 1.36 * i^{1.08}$<br>For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$ | $EURV_{DESIGN}$ = ac-f t |
| K) User Input of Excess Urban Runoff Volume (EURV) Design Volume (Only if a different EURV Design Volume is desired) | $EURV_{DESIGN\ USER}$= ac-f t |
| 2. Basin Shape: Length to Width Ratio (A basin length to width ratio of at least 2:1 will improve TSS reduction.) | L : W = 2.0 : 1 |
| 3. Basin Side Slopes | |
| A) Basin Maximum Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred) | Z = 4.00 ft / ft |
| 4. Inlet | |
| A) Describe means of providing energy dissipation at concentrated inflow locations: | |
| 5. Forebay | |
| A) Minimum Forebay Volume ($V_{FMIN}$ = 2% of the WQCV) | $V_{FMIN}$ = 0.002 ac-ft |
| B) Actual Forebay Volume | $V_F$ = 0.002 ac-ft |
| C) Forebay Depth ($D_F$ = 18 inch maximum) | $D_F$ = 6.0 in |
| D) Forebay Discharge | |
| i) Undetained 100-year Peak Discharge | $Q_{100}$ = 20.20 cfs |
| ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$) | $Q_F$ = 0.40 cfs |
| E) Forebay Discharge Design | Choose One: ○ Berm With Pipe; ● Wall with Rect. Notch; ○ Wall with V-Notch Weir — Flow too small for berm w/ pipe |
| F) Discharge Pipe Size (minimum 8-inches) | Calculated $D_P$ = in |
| G) Rectangular Notch Width | Calculated $W_N$ = 5.3 in |

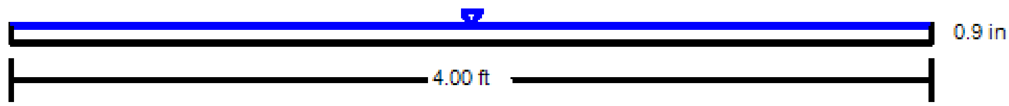
# Cross Section for Trickle Channel

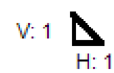
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | |
|---|---|
| Roughness Coefficient | 0.013 |
| Channel Slope | 0.005 ft/ft |
| Normal Depth | 0.9 in |
| Bottom Width | 4.00 ft |
| Discharge | 0.40 cfs |



# Spillway Capacity Calculation
## Spillway Section

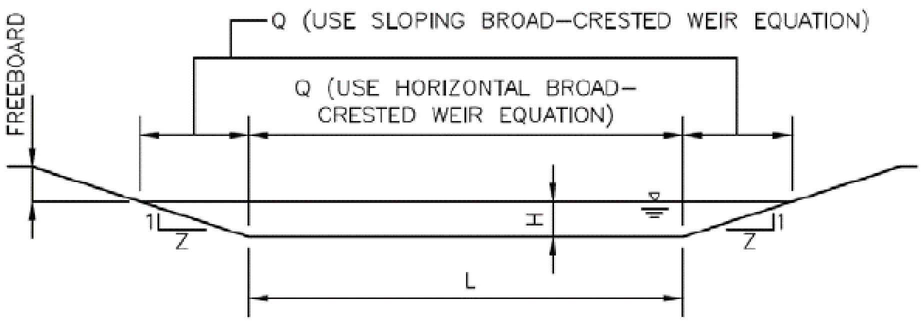



*Fig. 12-20 of UDFCD V1*

Horizontal Broad Crested Weir: Q=$C_{BCW}LH^{1.5}$ *Eq. 12-20 of UDFCD V1*

Sloping Broad-Crested Weir: Q=$\left(\frac{2}{5}\right)C_{BCW}ZH^{2.5}$ *Eq. 12-21 of UDFCD V1*

| | | | |
|---|---|---|---|
| 100-yr Undetained Runoff | Q = | 20.11 | cfs |
| Side slope 1 (horizontal: vertical) | Z1 = | 4.00 | |
| Side slope 2 (horizontal: vertical) | Z2 = | 4.00 | |
| Broad Crested Weir Coefficient | $C_{BCW}$ = | 3.0 | |
| Head above Weir Crest | H = | 0.71 | ft |

Total Required Length $L=\dfrac{Q-\left(\frac{2}{5}\right)C_{BCW}Z_1Z1H^{2.5}-\left(\frac{2}{5}\right)C_{BCW}Z_22H^{2.5}}{CH^{1.5}}$

| | | | |
|---|---|---|---|
| Bottom Length of Weir | Lbottom = | 8.82 | ft |
| Water spread at Q100 | Ltop= | 14.53 | |
| Unit Discharge | q = | 1.38 | cfs/ft |

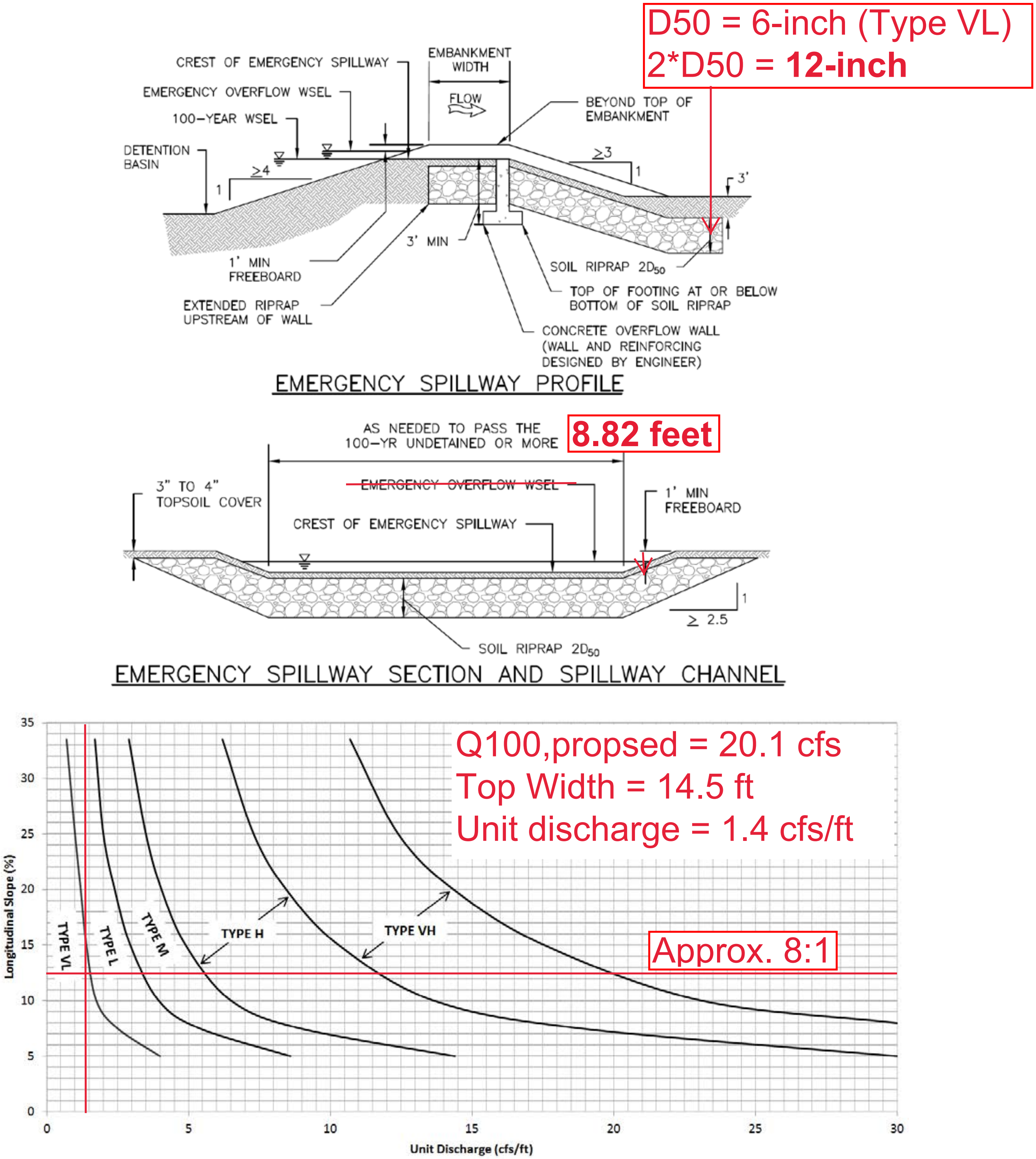
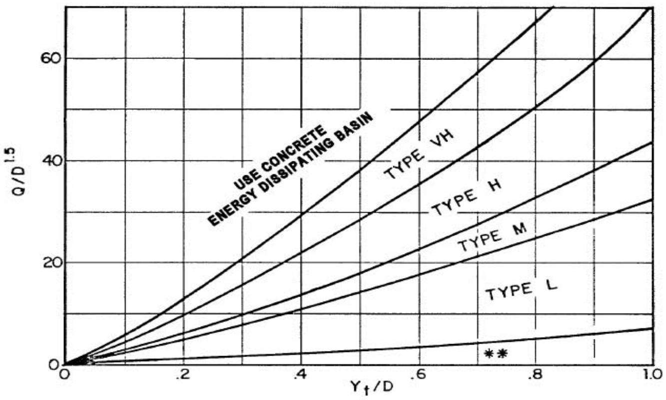



**Figure 12-21. Embankment protection details and rock sizing chart** (adapted from Arapahoe County)

## 18" RCP Outfall Protection

| | Q (cfs) | Dc (ft) | Yt/Dt | Yt | $Q/D^{1.5}$ | Selected Riprap |
|---|---|---|---|---|---|---|
| Basin B | 6.30 | 1.5 | 0.4 | 0.6 | 3.43 | L |




Use $D_a$ instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of 3D downstream.

**Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D_{2.5} \leq 6.0$)**

## SOIL RIPRAP SIZING

| | | |
|---|---|---|
| Mile High Flood District | (grey) | = Calculated Value |
| Volume I - August 2018 | (green) | = Manually Entered Value |
| Chapter 8 - Open Channels | (blue) | = Referenced from Flowmaster |
| 8.1.2 - Steep Slope Conditions | (white) | = Other / Published Value |

***USACE Steep Slope Riprap Equation***

U.S. Army Corps of Engineers. 1994. Hydraulic Design of Flood Control Channels.

EM 1110-2-1601, Change 1. June 30.

Steep Slope Method (Recommended for slopes from 2% to 20%)

***S =*** 0.25 ***Channel Slope (ft/ft)***

***q = Peak Unit Discharge (cfs/ft)***

100-Year = 1.8

***$D_{50}$ = Median Stone Size (ft)***

100-Year = 0.64 ***Type L riprap, $D_{50}$ 0.75 ft to be used**

***$F_{fc}$ =*** 1.25

***Flow Concentration Factor (1.25 recommednded in EM 1601)***

***g =*** 32.2

***Gravity (ft/s$^2$)***

***$F_{fc}$ x q= Peak Unit Discharge by Flow Concentration Factor (cfs/ft)***

100-Year = 2.30

***$D_{30}$ = Rock Diameter for which 30% is smaller by mass (ft)***

100-Year = 0.50

$$D_{30} = \frac{1.95\ S^{0.555}\ q^{2/3}}{g^{1/3}}$$

where

$S$ = slope of bed

$q$ = unit discharge

# **Curb and Gutter / Street Capacity**

**Existing Gutter Capacity Determination**

1. Calculate upstream runoff along the gutter line.

The gutter line on the southeast side of Eastonville extends from the proposed driveway to the north side of the Bennett Ranch drainageway crossing.


*Google Streetview at the upstream end of the gutter line (north side of Bennett Ranch drainageway crossing).*

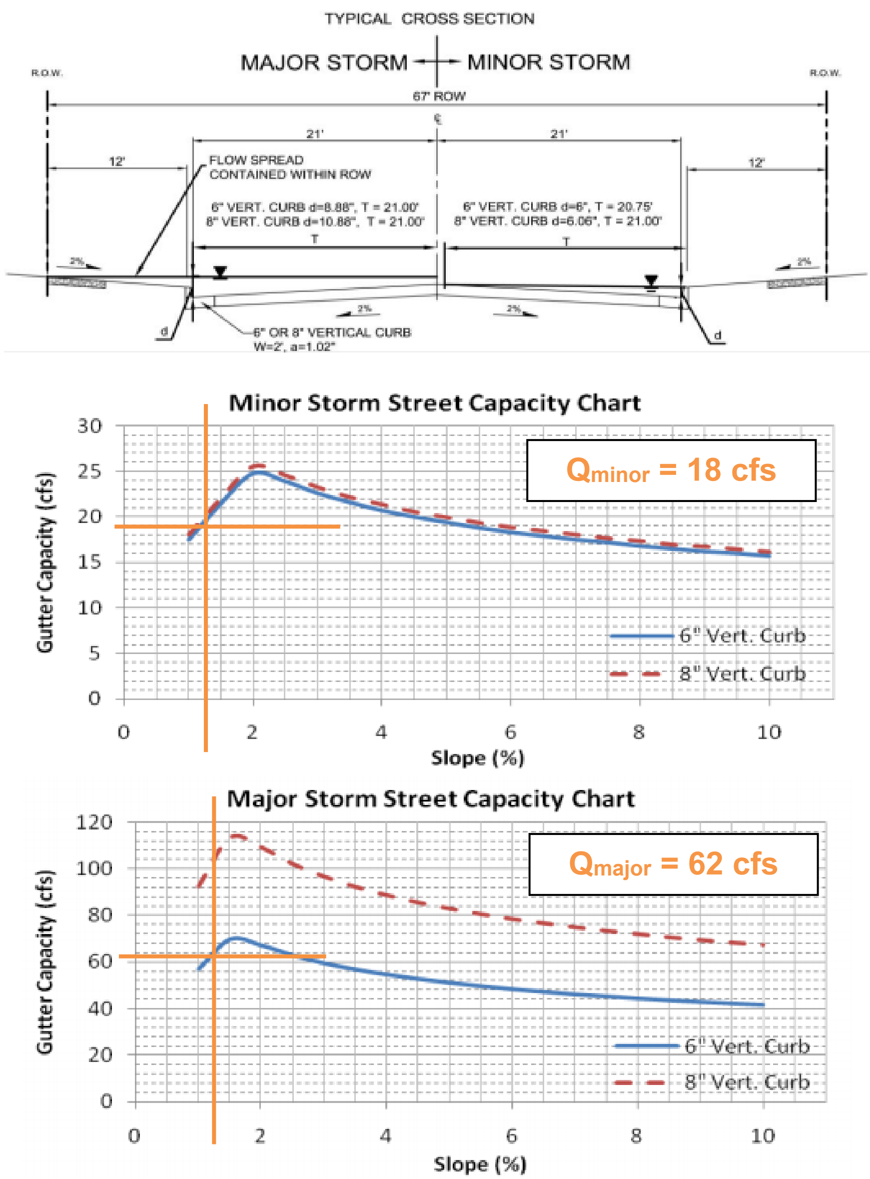

*Google Streetview at the upstream end of the gutter line (south side of Bennett Ranch drainageway crossing). Transition from block to 6 inch curb.*

The roadway is crowned in the center with two 20 ft wide lanes plus 2 ft wide gutters on each side as per survey, aerial, and CDOT data. To determine the drainage area to the project driveway, the length was measured along the flow line from the driveway to the upstream end of the gutter line (960 ft). Tributary area is basin ER1 at DP ER1, and Basins ER1 and ER2 at DP ER2 at the southern end of the Liberty Tree site. In the existing condition includes the road, curb and gutter as well as some vegetated areas sloped towards the road. In the proposed condition the tributary area is the road, curb and gutter, landscaped area, and proposed sidewalk. Runoff was calculated using the Rational Method (see **Appendix B, Composite Runoff Coefficients**).

2. Calculate maximum allowable flow in gutter based on El Paso criteria for minor arterials.

Gutter capacity was determined using the street capacity charts in Chapter 7 of DCM-V1-Update. The street is a minor arterial, however, the typical cross-section in Figure 7-5 for Collectors with Parking applies to this roadway (6” vertical curve, d = 6”, $T_{max}$ = 22’ (20’ travel lane with 2’ wide gutter)).

**Figure 7-5. Street Capacity Charts Collector (with Parking)**




These charts shall only be used for the standard street sections as shown. The capacity shown is based on ½ the street section as calculated by the UD-Inlet spreadsheets. Minor storm capacities are based on no crown overtopping, curb height or maximum allowable spread widths. Major storm capacities are based on flow being containing within the public right-of-way, including conveyance capacity behind the curb. The UDFCD Safety Reduction Factor was applied. An ‘$n_{STREET}$’ of 0.016 and ‘$n_{BACK}$’ of 0.020 was used. Calculations were done using UD-Inlet 3.00.xls, March, 2011.

| Parameter | Value | Note |
|---|---|---|
| Flow spread, T (ft) | 21 | Length from crown to gutterline (based on CDOT lane width) plus 1 ft gutter width. |
| Longitudinal slope, S (ft/ft) | 0.013 | Measured from the 5958 to 5951 contour. |
| Manning's n, n | 0.016 | From Figure 7-5 of DCM-V1-Update. |
| Minor gutter capacity, Q (cfs) | **18** | Using Figure 7-5 of DCM-V1-Update. |
| 5-year Q (cfs) @ DP ER2 | **2.8** | From Proposed Conditions, Rational Calculations |
| Major gutter capacity, Q (cfs) | **62** | Using Figure 7-5 of DCM-V1-Update. |
| 100-year Q (cfs) @ DP ER2 | **6.0** | From Existing Conditions, Rational Calculations |

As summarized in the table above, runoff in the minor and major event will be contained within the R.O.W. of Eastonville Road without entering the site.

3. Calculate maximum allowable flow in cross pan/driveway section.

The driveway section geometry was determined from El Paso County Detail SD 2-26 for Typical Cross Pan Layout Detail. Depth from the flowline of the cross pan to the crown of the road is 6.8 inches; the corresponding maximum allowable flow is 51.0 cfs, as shown in the following FlowMasterV8i calculation sheets. The minor and major events will be sufficiently to conveyed in the proposed cross pan along Eastonville Road without entering the site.

# Cross Pan and Driveway Section

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.01300 | ft/ft |
| Normal Depth | 0.57 | ft |

Section Definitions

| Station (ft) | Elevation (ft) |
|---|---|
| -0+22 | 0.57 |
| -0+02 | 0.17 |
| 0+00 | 0.00 |
| 0+06 | 0.17 |
| 0+36 | 0.76 |

Roughness Segment Definitions

| Start Station | Ending Station | Roughness Coefficient |
|---|---|---|
| (-0+22, 0.57) | (-0+02, 0.17) | 0.016 |
| (-0+02, 0.17) | (0+06, 0.17) | 0.013 |
| (0+06, 0.17) | (0+36, 0.76) | 0.016 |

## Options

| | |
|---|---|
| Current Roughness Weighted Method | Pavlovskii's Method |
| Open Channel Weighting Method | Pavlovskii's Method |
| Closed Channel Weighting Method | Pavlovskii's Method |

## Results

| | | |
|---|---|---|
| Discharge | 50.95 | $ft^3/s$ |
| Elevation Range | 0.00 to 0.76 ft | |
| Flow Area | 11.87 | $ft^2$ |
| Wetted Perimeter | 48.02 | ft |
| Hydraulic Radius | 0.25 | ft |
| Top Width | 48.00 | ft |

## Cross Pan and Driveway Section

### Results

| | | | |
|---|---|---|---|
| Normal Depth | | 0.57 | ft |
| Critical Depth | | 0.65 | ft |
| Critical Slope | | 0.00522 | ft/ft |
| Velocity | | 4.29 | ft/s |
| Velocity Head | | 0.29 | ft |
| Specific Energy | | 0.85 | ft |
| Froude Number | | 1.52 | |
| Flow Type | Supercritical | | |

### GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | ft |
| Length | 0.00 | ft |
| Number Of Steps | 0 | |

### GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | ft |
| Profile Description | | |
| Profile Headloss | 0.00 | ft |
| Downstream Velocity | Infinity | ft/s |
| Upstream Velocity | Infinity | ft/s |
| Normal Depth | 0.57 | ft |
| Critical Depth | 0.65 | ft |
| Channel Slope | 0.01300 | ft/ft |
| Critical Slope | 0.00522 | ft/ft |

# Cross Pan and Driveway Section

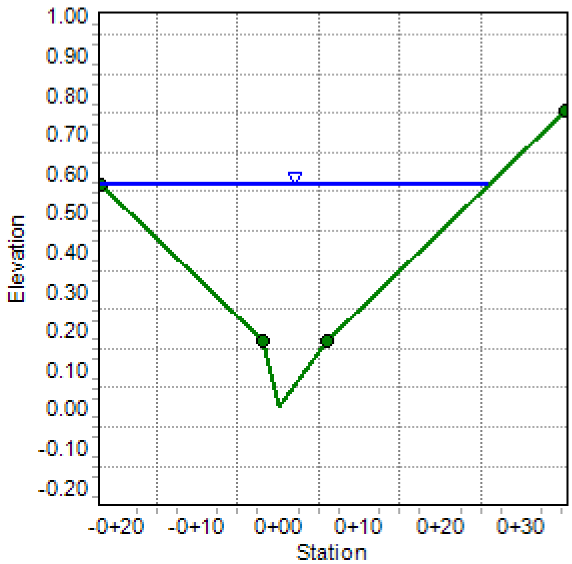
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.01300 | ft/ft |
| Normal Depth | 0.57 | ft |
| Discharge | 50.95 | ft³/s |

## Cross Section Image



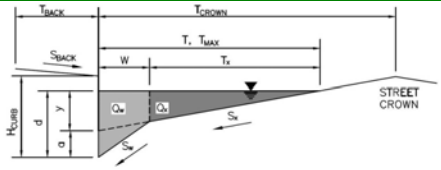
Version 4.05 Released March 2017

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

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

**Project:** Liberty Tree Academy

**Inlet ID:** Parking Lot - 1' Gutter Vertical Catch Curb




**Gutter Geometry (Enter data in the blue cells)**

| | | | | |
|---|---|---|---|---|
| Maximum Allowable Width for Spread Behind Curb | $T_{BACK}$ = | 20.0 | | ft |
| Side Slope Behind Curb (leave blank for no conveyance credit behind curb) | $S_{BACK}$ = | 0.020 | | ft/ft |
| Manning's Roughness Behind Curb (typically between 0.012 and 0.020) | $n_{BACK}$ = | 0.016 | | |
| Height of Curb at Gutter Flow Line | $H_{CURB}$ = | 6.00 | | inches |
| Distance from Curb Face to Street Crown | $T_{CROWN}$ = | 30.0 | | ft |
| Gutter Width | W = | 1.00 | | ft |
| Street Transverse Slope | $S_X$ = | 0.020 | | ft/ft |
| Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) | $S_W$ = | 0.083 | | ft/ft |
| Street Longitudinal Slope - Enter 0 for sump condition | $S_O$ = | 0.008 | | ft/ft |
| Manning's Roughness for Street Section (typically between 0.012 and 0.020) | $n_{STREET}$ = | 0.016 | | |
| | | Minor Storm | Major Storm | |
| Max. Allowable Spread for Minor & Major Storm | $T_{MAX}$ = | 30.0 | 30.0 | ft |
| Max. Allowable Depth at Gutter Flowline for Minor & Major Storm | $d_{MAX}$ = | 6.0 | 12.0 | inches |
| Allow Flow Depth at Street Crown (leave blank for no) | | ☐ | ☑ | check = yes |
| | | Minor Storm | Major Storm | |
| **MINOR STORM Allowable Capacity is based on Depth Criterion** | | | | |
| **MAJOR STORM Allowable Capacity is based on Depth Criterion** | $Q_{allow}$ = | **16.9** | **143.0** | **cfs** |

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

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

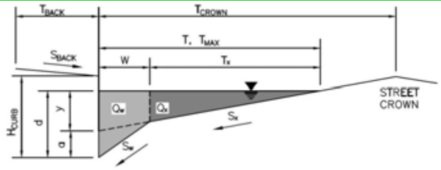
Version 4.05 Released March 2017

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

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

**Project:** **Liberty Tree Academy**

**Inlet ID:** **Parking Lot - 2' Gutter Vertical Catch Curb**




**Gutter Geometry (Enter data in the blue cells)**

| | | | | |
|---|---|---|---|---|
| Maximum Allowable Width for Spread Behind Curb | $T_{BACK}$ = | 20.0 | | ft |
| Side Slope Behind Curb (leave blank for no conveyance credit behind curb) | $S_{BACK}$ = | 0.020 | | ft/ft |
| Manning's Roughness Behind Curb (typically between 0.012 and 0.020) | $n_{BACK}$ = | 0.016 | | |
| Height of Curb at Gutter Flow Line | $H_{CURB}$ = | 6.00 | | inches |
| Distance from Curb Face to Street Crown | $T_{CROWN}$ = | 30.0 | | ft |
| Gutter Width | W = | 2.00 | | ft |
| Street Transverse Slope | $S_X$ = | 0.020 | | ft/ft |
| Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) | $S_W$ = | 0.083 | | ft/ft |
| Street Longitudinal Slope - Enter 0 for sump condition | $S_O$ = | 0.008 | | ft/ft |
| Manning's Roughness for Street Section (typically between 0.012 and 0.020) | $n_{STREET}$ = | 0.016 | | |
| | | Minor Storm | Major Storm | |
| Max. Allowable Spread for Minor & Major Storm | $T_{MAX}$ = | 30.0 | 30.0 | ft |
| Max. Allowable Depth at Gutter Flowline for Minor & Major Storm | $d_{MAX}$ = | 6.0 | 12.0 | inches |
| Allow Flow Depth at Street Crown (leave blank for no) | | ☐ | ☑ | check = yes |
| | | Minor Storm | Major Storm | |
| **MINOR STORM Allowable Capacity is based on Depth Criterion** | | | | |
| **MAJOR STORM Allowable Capacity is based on Depth Criterion** | **$Q_{allow}$ =** | **11.9** | **126.2** | **cfs** |

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

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

## **Storm System Capacity**

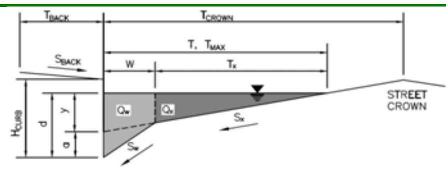
Version 4.05 Released March 2017

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

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

**Project:** Liberty Tree - Phase II Improvements

**Inlet ID:** B1




**Gutter Geometry (Enter data in the blue cells)**

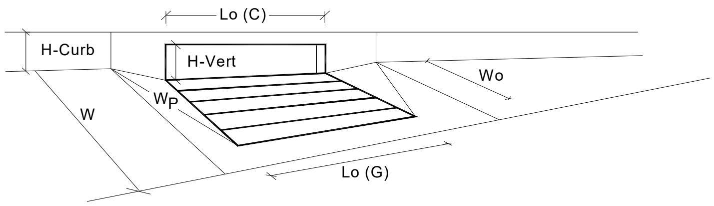
| | | | |
|---|---|---|---|
| Maximum Allowable Width for Spread Behind Curb | $T_{BACK}$ = | 6.5 | ft |
| Side Slope Behind Curb (leave blank for no conveyance credit behind curb) | $S_{BACK}$ = | 0.020 | ft/ft |
| Manning's Roughness Behind Curb (typically between 0.012 and 0.020) | $n_{BACK}$ = | 0.016 | |
| Height of Curb at Gutter Flow Line | $H_{CURB}$ = | 6.00 | inches |
| Distance from Curb Face to Street Crown | $T_{CROWN}$ = | 30.0 | ft |
| Gutter Width | W = | 2.00 | ft |
| Street Transverse Slope | $S_X$ = | 0.020 | ft/ft |
| Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) | $S_W$ = | 0.083 | ft/ft |
| Street Longitudinal Slope - Enter 0 for sump condition | $S_O$ = | 0.000 | ft/ft |
| Manning's Roughness for Street Section (typically between 0.012 and 0.020) | $n_{STREET}$ = | 0.016 | |

| | | Minor Storm | Major Storm | |
|---|---|---|---|---|
| Max. Allowable Spread for Minor & Major Storm | $T_{MAX}$ = | 30.0 | 30.0 | ft |
| Max. Allowable Depth at Gutter Flowline for Minor & Major Storm | $d_{MAX}$ = | 3.6 | 4.6 | inches |
| Check boxes are not applicable in SUMP conditions | | ☐ | ☐ | |

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

# INLET IN A SUMP OR SAG LOCATION

**Version 4.05 Released March 2017**




| | | | | | |
|---|---|---|---|---|---|
| **Design Information (Input)** | | | MINOR | MAJOR | |
| Type of Inlet | CDOT Type R Curb Opening | Type = | CDOT Type R Curb Opening | | |
| Local Depression (additional to continuous gutter depression 'a' from above) | | $a_{local}$ = | 3.00 | 3.00 | inches |
| Number of Unit Inlets (Grate or Curb Opening) | | No = | 1 | 1 | |
| Water Depth at Flowline (outside of local depression) | | Ponding Depth = | 3.6 | 4.6 | 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 | |

| | | | | |
|---|---|---|---|---|
| **Low Head Performance Reduction (Calculated)** | | MINOR | MAJOR | |
| Depth for Grate Midwidth | $d_{Grate}$ = | N/A | N/A | ft |
| Depth for Curb Opening Weir Equation | $d_{Curb}$ = | 0.13 | 0.22 | ft |
| Combination Inlet Performance Reduction Factor for Long Inlets | $RF_{Combination}$ = | 0.46 | 0.59 | |
| Curb Opening Performance Reduction Factor for Long Inlets | $RF_{Curb}$ = | 1.00 | 1.00 | |
| Grated Inlet Performance Reduction Factor for Long Inlets | $RF_{Grate}$ = | N/A | N/A | |

| | | | | |
|---|---|---|---|---|
| | | MINOR | MAJOR | |
| **Total Inlet Interception Capacity (assumes clogged condition)** | $Q_a$ = | **1.4** | **2.8** | **cfs** |
| **Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)** | $Q_{PEAK\ REQUIRED}$ = | 1.3 | 2.8 | cfs |

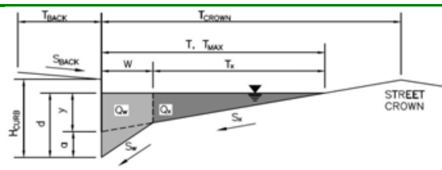
Version 4.05 Released March 2017

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

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

**Project:** Liberty Tree - Phase II Improvements

**Inlet ID:** B2




**Gutter Geometry (Enter data in the blue cells)**

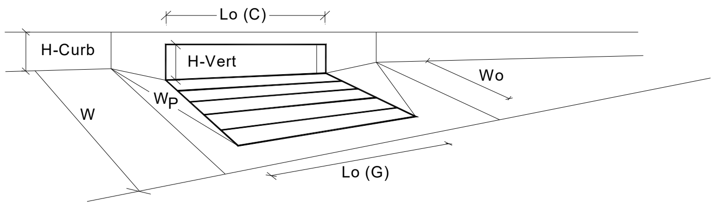
| | | | |
|---|---|---|---|
| Maximum Allowable Width for Spread Behind Curb | $T_{BACK}$ = | 6.5 | ft |
| Side Slope Behind Curb (leave blank for no conveyance credit behind curb) | $S_{BACK}$ = | 0.020 | ft/ft |
| Manning's Roughness Behind Curb (typically between 0.012 and 0.020) | $n_{BACK}$ = | 0.016 | |
| Height of Curb at Gutter Flow Line | $H_{CURB}$ = | 6.00 | inches |
| Distance from Curb Face to Street Crown | $T_{CROWN}$ = | 30.0 | ft |
| Gutter Width | W = | 2.00 | ft |
| Street Transverse Slope | $S_X$ = | 0.020 | ft/ft |
| Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft) | $S_W$ = | 0.083 | ft/ft |
| Street Longitudinal Slope - Enter 0 for sump condition | $S_O$ = | 0.000 | ft/ft |
| Manning's Roughness for Street Section (typically between 0.012 and 0.020) | $n_{STREET}$ = | 0.016 | |

| | | Minor Storm | Major Storm | |
|---|---|---|---|---|
| Max. Allowable Spread for Minor & Major Storm | $T_{MAX}$ = | 30.0 | 30.0 | ft |
| Max. Allowable Depth at Gutter Flowline for Minor & Major Storm | $d_{MAX}$ = | 4.0 | 5.0 | inches |
| Check boxes are not applicable in SUMP conditions | | ☐ | ☐ | |

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

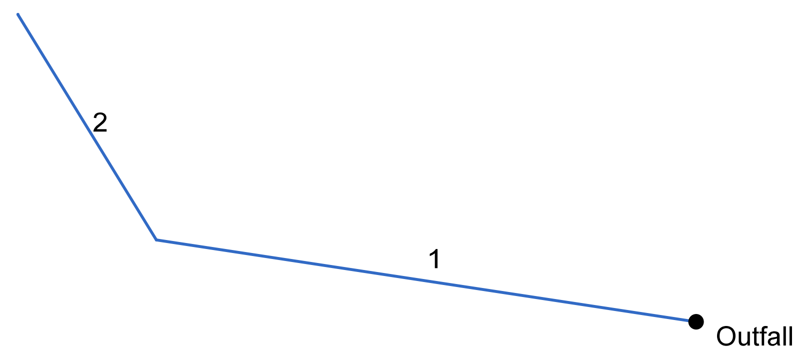
# INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017




| | | MINOR | MAJOR | |
|---|---|---|---|---|
| **Design Information (Input)** | | | | |
| Type of Inlet — CDOT Type R Curb Opening | Type = | CDOT Type R Curb Opening | | |
| Local Depression (additional to continuous gutter depression 'a' from above) | $a_{local}$ = | 3.00 | 3.00 | inches |
| Number of Unit Inlets (Grate or Curb Opening) | No = | 1 | 1 | |
| Water Depth at Flowline (outside of local depression) | Ponding Depth = | 4.0 | 5.0 | 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 | |
| **Low Head Performance Reduction (Calculated)** | | MINOR | MAJOR | |
| Depth for Grate Midwidth | $d_{Grate}$ = | N/A | N/A | ft |
| Depth for Curb Opening Weir Equation | $d_{Curb}$ = | 0.17 | 0.25 | ft |
| Combination Inlet Performance Reduction Factor for Long Inlets | $RF_{Combination}$ = | 0.52 | 0.65 | |
| Curb Opening Performance Reduction Factor for Long Inlets | $RF_{Curb}$ = | 1.00 | 1.00 | |
| Grated Inlet Performance Reduction Factor for Long Inlets | $RF_{Grate}$ = | N/A | N/A | |
| | | MINOR | MAJOR | |
| **Total Inlet Interception Capacity (assumes clogged condition)** | $Q_a$ = | **2.0** | **3.6** | **cfs** |
| Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK) | $Q_{PEAK\ REQUIRED}$ = | 1.9 | 3.5 | cfs |

# Hydraflow Storm Sewers Extension for Autodesk® AutoCAD® Civil 3D® Plan




| Project File: 100-YR STORM.stm | Number of lines: 2 | Date: 6/25/2020 |
|---|---|---|

# Storm Sewer Summary Report



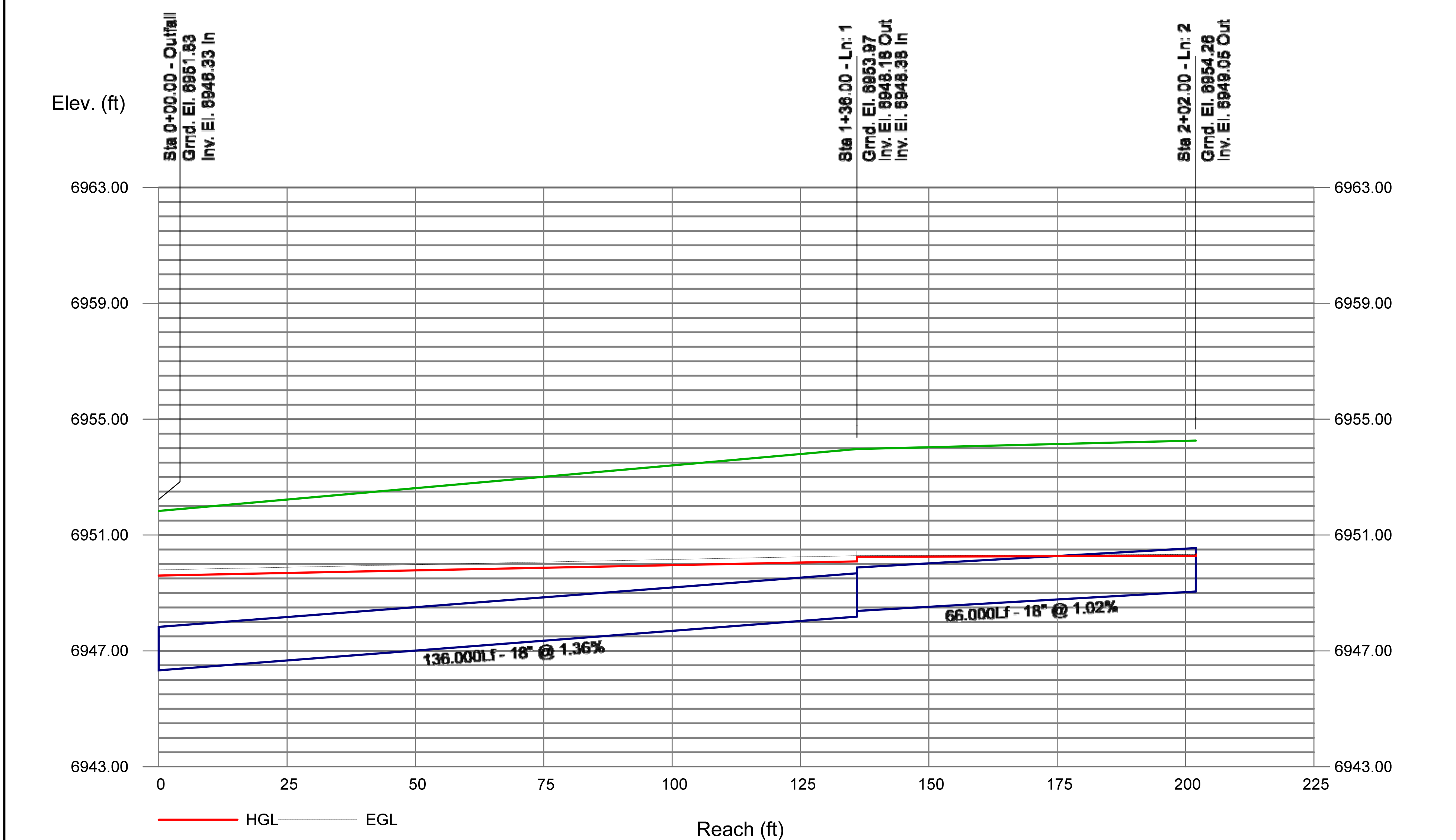
| Line No. | Line ID | Flow rate (cfs) | Line Size (in) | Line shape | Line length (ft) | Invert EL Dn (ft) | Invert EL Up (ft) | Line Slope (%) | HGL Down (ft) | HGL Up (ft) | Minor loss (ft) | HGL Junct (ft) | Dns Line No. | Junction Type |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Pipe - (2) | 6.30 | 18 | Cir | 136.000 | 6946.33 | 6948.18 | 1.360 | 6949.60* | 6950.09* | 0.16 | 6950.25 | End | None |
| 2 | Pipe - (1) | 2.77 | 18 | Cir | 66.000 | 6948.38 | 6949.05 | 1.015 | 6950.25 | 6950.28 | 0.05 | 6950.33 | 1 | None |

| Project File: 100-YR STORM.stm | Number of lines: 2 | Run Date: 6/25/2020 |
|---|---|---|

NOTES: Return period = 100 Yrs. ; *Surcharged (HGL above crown).

# Storm Sewer Profile

Proj. file: 100-YR STORM.stm



## **Bennet Ranch Drainageway Capacity**

# Worksheet for X1

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Discharge | 810.00 | ft³/s |
| Section Definitions | | |

| Station (ft) | Elevation (ft) |
|---|---|
| -1+22 | 6952.00 |
| -1+16 | 6950.54 |
| -1+13 | 6950.00 |
| -1+10 | 6949.29 |
| -1+09 | 6949.00 |
| -1+09 | 6948.91 |
| -1+06 | 6948.39 |
| -1+05 | 6948.06 |
| -1+05 | 6948.00 |
| -1+00 | 6947.87 |
| -1+00 | 6947.87 |
| -0+99 | 6947.65 |
| -0+86 | 6944.96 |
| -0+78 | 6943.42 |
| -0+74 | 6942.94 |
| -0+47 | 6940.17 |
| -0+25 | 6939.53 |
| -0+16 | 6939.30 |
| -0+10 | 6939.19 |
| -0+04 | 6939.09 |
| -0+03 | 6937.77 |
| -0+02 | 6937.26 |
| -0+02 | 6936.77 |
| -0+01 | 6936.76 |
| 0+00 | 6936.87 |
| 0+03 | 6936.85 |
| 0+04 | 6936.83 |

# Worksheet for X1

## Input Data

| Station (ft) | Elevation (ft) |
|---|---|
| 0+04 | 6936.87 |
| 0+10 | 6939.19 |
| 0+12 | 6939.53 |
| 0+40 | 6945.19 |
| 0+50 | 6945.19 |

| Start Station | Ending Station | Roughness Coefficient |
|---|---|---|
| (-1+22, 6952.00) | (-0+25, 6939.53) | 0.050 |
| (-0+25, 6939.53) | (-0+04, 6939.09) | 0.160 |
| (-0+04, 6939.09) | (0+10, 6939.19) | 0.040 |
| (0+10, 6939.19) | (0+12, 6939.53) | 0.160 |
| (0+12, 6939.53) | (0+50, 6945.19) | 0.050 |

## Options

| | |
|---|---|
| Current Roughness Weighted Method | Pavlovskii's Method |
| Open Channel Weighting Method | Pavlovskii's Method |
| Closed Channel Weighting Method | Pavlovskii's Method |

## Results

| | | |
|---|---|---|
| Normal Depth | 7.14 | ft |
| Elevation Range | 6936.76 to 6952.00 ft | |
| Flow Area | 396.96 | ft² |
| Wetted Perimeter | 115.86 | ft |
| Hydraulic Radius | 3.43 | ft |
| Top Width | 113.90 | ft |
| Normal Depth | 7.14 | ft |
| Critical Depth | 4.13 | ft |
| Critical Slope | 0.08816 | ft/ft |

# Worksheet for X1

## Results

| | | | |
|---|---|---|---|
| Velocity | | 2.04 | ft/s |
| Velocity Head | | 0.06 | ft |
| Specific Energy | | 7.20 | ft |
| Froude Number | | 0.19 | |
| Flow Type | Subcritical | | |

## GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | ft |
| Length | 0.00 | ft |
| Number Of Steps | 0 | |

## GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | ft |
| Profile Description | | |
| Profile Headloss | 0.00 | ft |
| Downstream Velocity | Infinity | ft/s |
| Upstream Velocity | Infinity | ft/s |
| Normal Depth | 7.14 | ft |
| Critical Depth | 4.13 | ft |
| Channel Slope | 0.00250 | ft/ft |
| Critical Slope | 0.08816 | ft/ft |

# Cross Section for X1

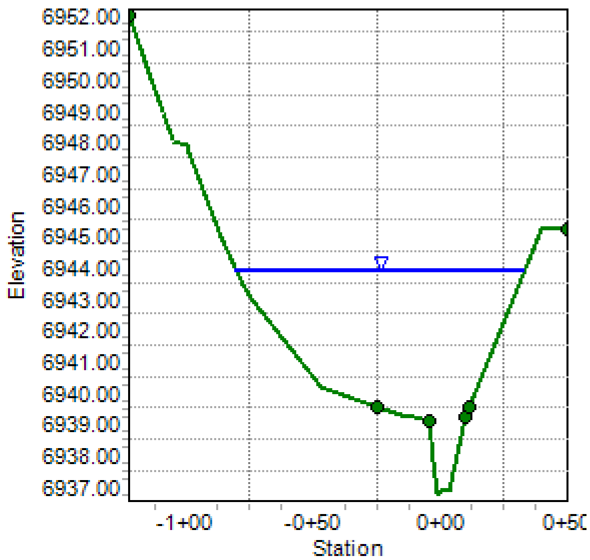
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Normal Depth | 7.14 | ft |
| Discharge | 810.00 | ft³/s |

## Cross Section Image



# Worksheet for X2

## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Discharge | 810.00 | ft³/s |

Section Definitions

| Station (ft) | Elevation (ft) |
|---|---|
| -1+21 | 6952.00 |
| -1+18 | 6951.62 |
| -1+13 | 6950.84 |
| -1+11 | 6950.71 |
| -0+89 | 6947.86 |
| -0+87 | 6947.61 |
| -0+85 | 6947.40 |
| -0+47 | 6943.05 |
| -0+46 | 6943.02 |
| -0+22 | 6941.67 |
| -0+17 | 6941.62 |
| -0+10 | 6941.78 |
| -0+08 | 6940.48 |
| -0+05 | 6938.40 |
| -0+03 | 6938.21 |
| 0+00 | 6938.00 |
| 0+02 | 6938.02 |
| 0+03 | 6938.02 |
| 0+10 | 6941.78 |
| 0+40 | 6947.78 |
| 0+55 | 6947.78 |

Roughness Segment Definitions

| Start Station | Ending Station | Roughness Coefficient |
|---|---|---|

# Worksheet for X2

## Input Data

| Start Station | Ending Station | Roughness Coefficient |
|---|---|---|
| (-1+21, 6952.00) | (-0+22, 6941.67) | 0.050 |
| (-0+22, 6941.67) | (-0+10, 6941.78) | 0.160 |
| (-0+10, 6941.78) | (0+10, 6941.78) | 0.040 |
| (0+10, 6941.78) | (0+55, 6947.78) | 0.050 |

## Options

| | |
|---|---|
| Current Roughness Weighted Method | Pavlovskii's Method |
| Open Channel Weighting Method | Pavlovskii's Method |
| Closed Channel Weighting Method | Pavlovskii's Method |

## Results

| | | | |
|---|---|---|---|
| Normal Depth | | 7.87 | ft |
| Elevation Range | 6938.00 to 6952.00 ft | | |
| Flow Area | | 346.08 | ft² |
| Wetted Perimeter | | 104.77 | ft |
| Hydraulic Radius | | 3.30 | ft |
| Top Width | | 102.19 | ft |
| Normal Depth | | 7.87 | ft |
| Critical Depth | | 4.97 | ft |
| Critical Slope | | 0.06270 | ft/ft |
| Velocity | | 2.34 | ft/s |
| Velocity Head | | 0.09 | ft |
| Specific Energy | | 7.96 | ft |
| Froude Number | | 0.22 | |
| Flow Type | Subcritical | | |

## GVF Input Data

| | | |
|---|---|---|
| Downstream Depth | 0.00 | ft |
| Length | 0.00 | ft |
| Number Of Steps | 0 | |

## GVF Output Data

| | | |
|---|---|---|
| Upstream Depth | 0.00 | ft |

## Worksheet for X2

### GVF Output Data

| | | |
|---|---|---|
| Profile Description | | |
| Profile Headloss | 0.00 | ft |
| Downstream Velocity | Infinity | ft/s |
| Upstream Velocity | Infinity | ft/s |
| Normal Depth | 7.87 | ft |
| Critical Depth | 4.97 | ft |
| Channel Slope | 0.00250 | ft/ft |
| Critical Slope | 0.06270 | ft/ft |

# Cross Section for X2

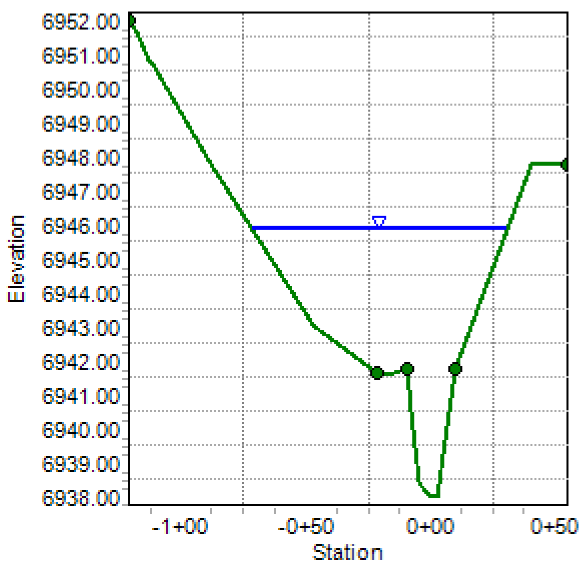
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Normal Depth | 7.87 | ft |
| Discharge | 810.00 | ft³/s |

## Cross Section Image



# Cross Section for X2, WSEL = 6942

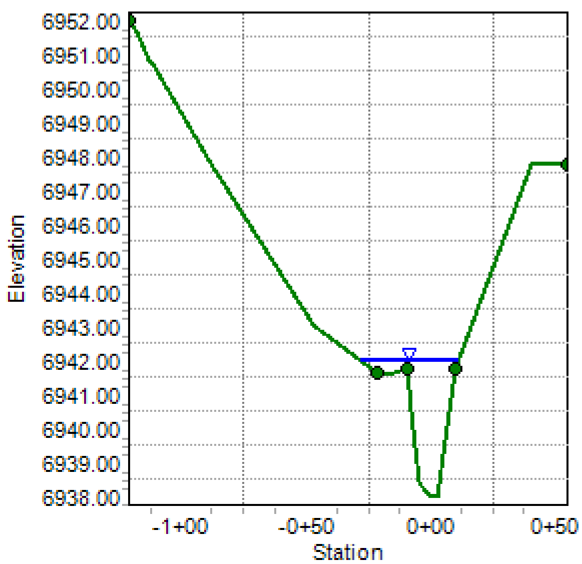
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Normal Depth | 4.00 | ft |
| Discharge | 59.97 | ft³/s |

## Cross Section Image



# Cross Section for X2, WSEL = 6945

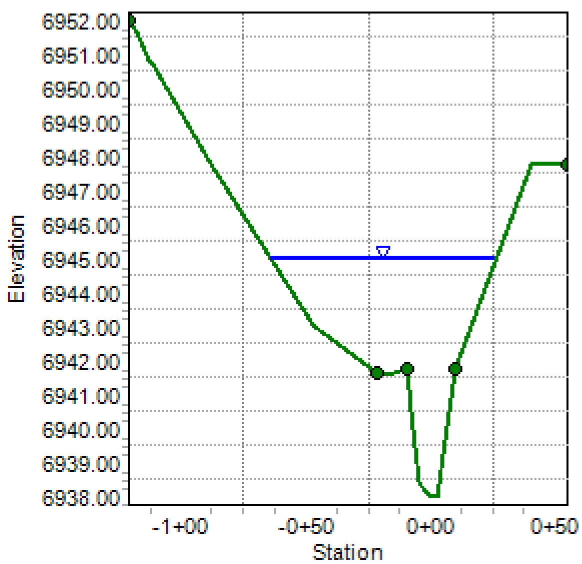
## Project Description

| | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Discharge |

## Input Data

| | | |
|---|---|---|
| Channel Slope | 0.00250 | ft/ft |
| Normal Depth | 7.00 | ft |
| Discharge | 537.83 | ft³/s |

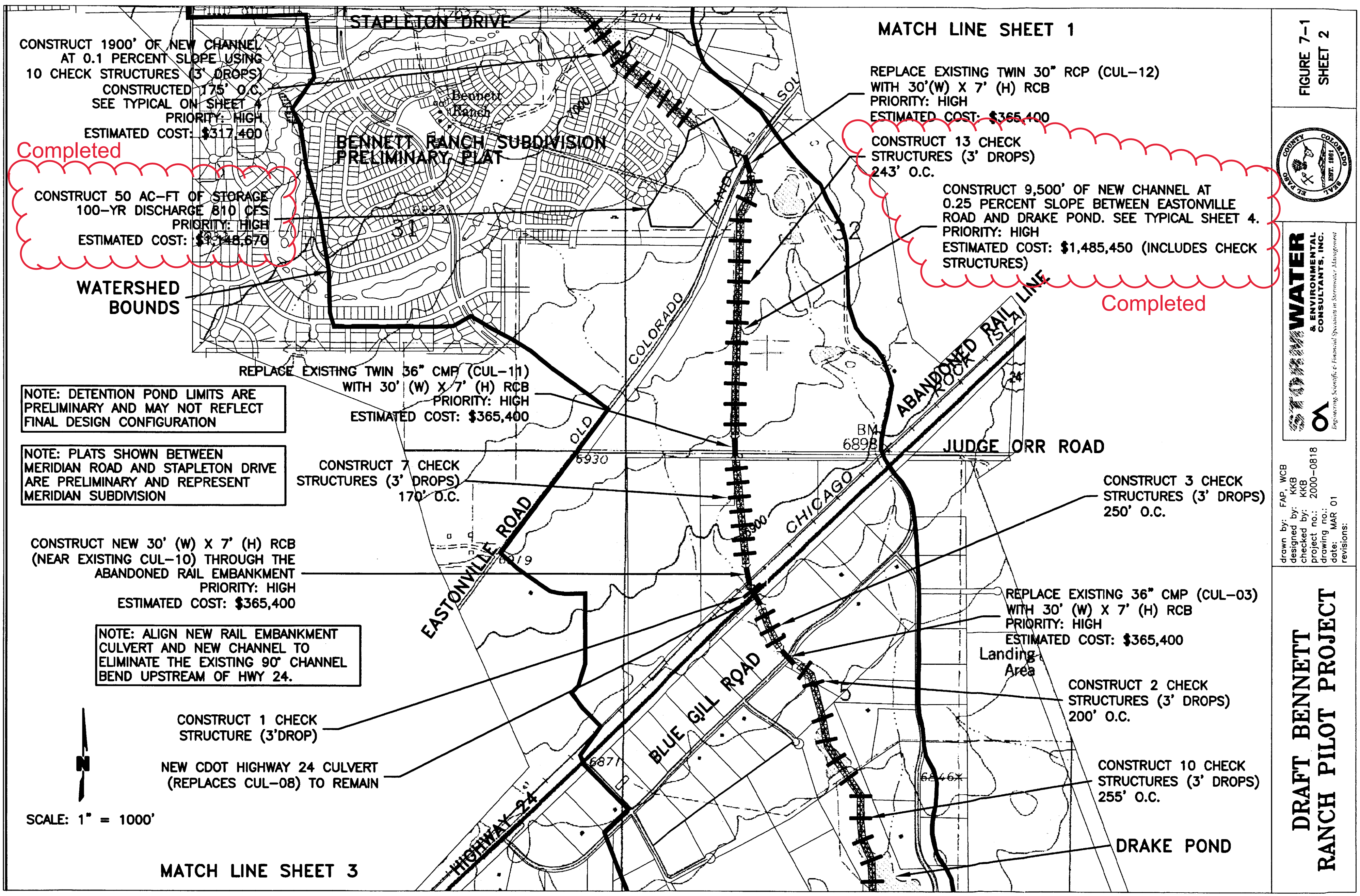
## Cross Section Image

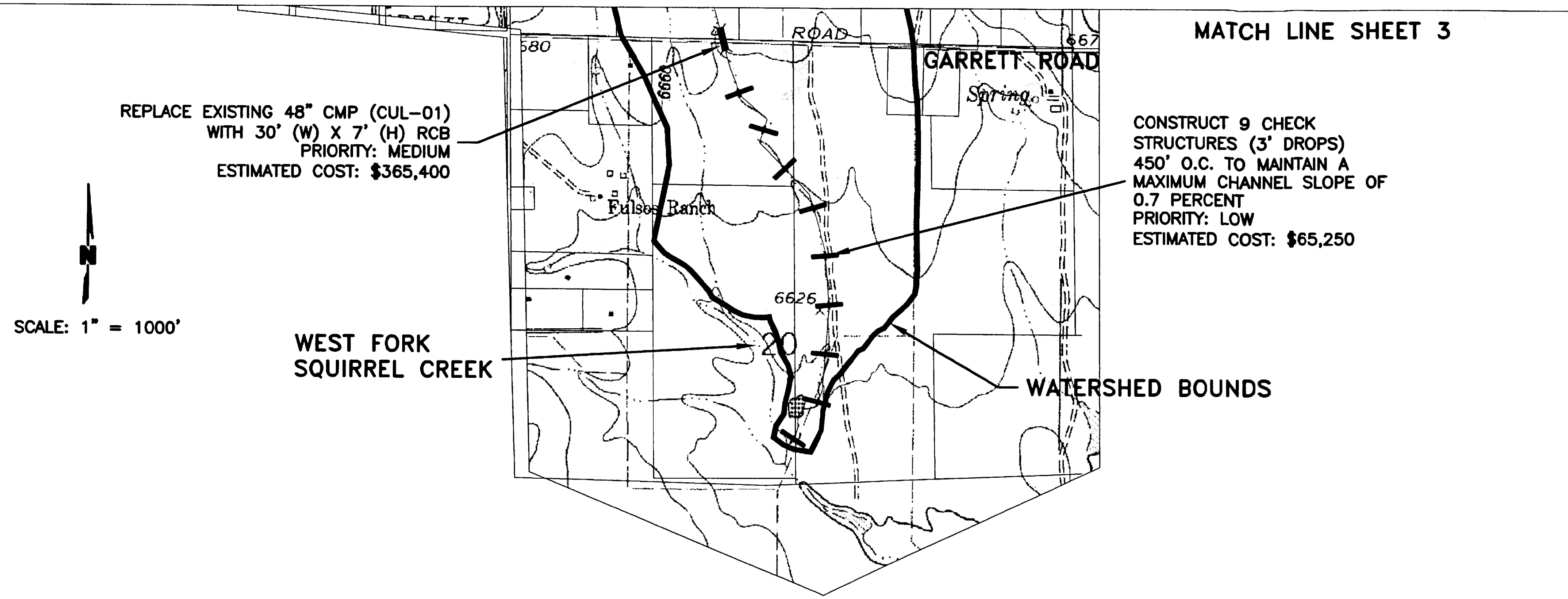


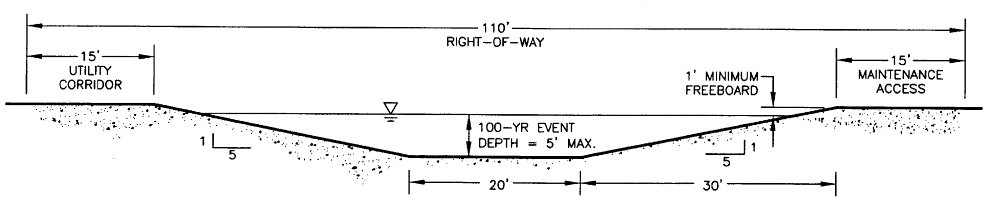
# APPENDIX D – REFERENCED DOCUMENTS

**Selected Pages from the Bennett Ranch**

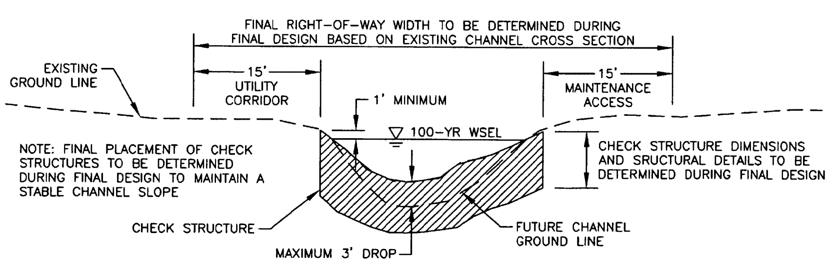
**Drainage Basin Planning Study**


MATCH LINE SHEET 1
STAPLETON DRIVE
CONSTRUCT 1900' OF NEW CHANNEL AT 0.1 PERCENT SLOPE USING 10 CHECK STRUCTURES (3' DROPS) CONSTRUCTED 175' O.C. SEE TYPICAL ON SHEET 4 PRIORITY: HIGH ESTIMATED COST: $317,400
Bennett Ranch
BENNETT RANCH SUBDIVISION PRELIMINARY PLAT
Completed
CONSTRUCT 50 AC-FT OF STORAGE 100-YR DISCHARGE 810 CFS PRIORITY: HIGH ESTIMATED COST: $1,148,670
WATERSHED BOUNDS
REPLACE EXISTING TWIN 30" RCP (CUL-12) WITH 30'(W) X 7' (H) RCB PRIORITY: HIGH ESTIMATED COST: $365,400
CONSTRUCT 13 CHECK STRUCTURES (3' DROPS) 243' O.C.
CONSTRUCT 9,500' OF NEW CHANNEL AT 0.25 PERCENT SLOPE BETWEEN EASTONVILLE ROAD AND DRAKE POND. SEE TYPICAL SHEET 4. PRIORITY: HIGH ESTIMATED COST: $1,485,450 (INCLUDES CHECK STRUCTURES)
Completed
REPLACE EXISTING TWIN 36" CMP (CUL-11) WITH 30' (W) X 7' (H) RCB PRIORITY: HIGH ESTIMATED COST: $365,400
NOTE: DETENTION POND LIMITS ARE PRELIMINARY AND MAY NOT REFLECT FINAL DESIGN CONFIGURATION
NOTE: PLATS SHOWN BETWEEN MERIDIAN ROAD AND STAPLETON DRIVE ARE PRELIMINARY AND REPRESENT MERIDIAN SUBDIVISION
OLD COLORADO
ABANDONED RAIL LINE
ROCK ISLAND
BM 6898
JUDGE ORR ROAD
CONSTRUCT 7 CHECK STRUCTURES (3' DROPS) 170' O.C.
CONSTRUCT 3 CHECK STRUCTURES (3' DROPS) 250' O.C.
CHICAGO
EASTONVILLE ROAD
CONSTRUCT NEW 30' (W) X 7' (H) RCB (NEAR EXISTING CUL-10) THROUGH THE ABANDONED RAIL EMBANKMENT PRIORITY: HIGH ESTIMATED COST: $365,400
REPLACE EXISTING 36" CMP (CUL-03) WITH 30' (W) X 7' (H) RCB PRIORITY: HIGH ESTIMATED COST: $365,400
NOTE: ALIGN NEW RAIL EMBANKMENT CULVERT AND NEW CHANNEL TO ELIMINATE THE EXISTING 90° CHANNEL BEND UPSTREAM OF HWY 24.
Landing Area
BLUE GILL ROAD
CONSTRUCT 2 CHECK STRUCTURES (3' DROPS) 200' O.C.
CONSTRUCT 1 CHECK STRUCTURE (3'DROP)
NEW CDOT HIGHWAY 24 CULVERT (REPLACES CUL-08) TO REMAIN
CONSTRUCT 10 CHECK STRUCTURES (3' DROPS) 255' O.C.
N
SCALE: 1" = 1000'
HIGHWAY 24
DRAKE POND
MATCH LINE SHEET 3
FIGURE 7-1 SHEET 2
EL PASO COUNTY COLORADO SEAL
STORMWATER & ENVIRONMENTAL CONSULTANTS, INC.
drawn by: FAP, WCB
designed by: KKB
checked by: KKB
project no.: 2000-0818
drawing no.:
date: MAR 01
revisions:
DRAFT BENNETT RANCH PILOT PROJECT

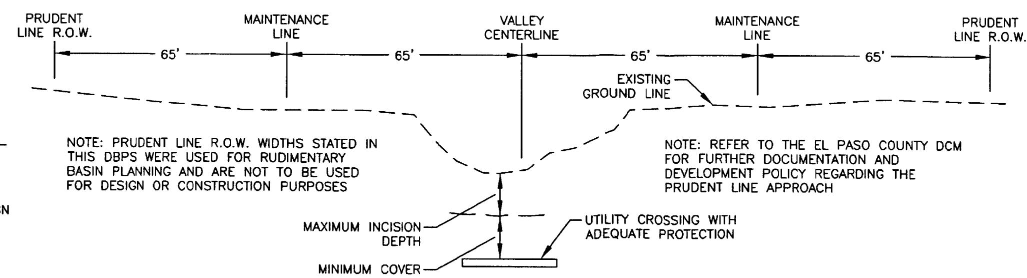



TYPICAL NEW CHANNEL CROSS SECTION
STAPLETON DRIVE TO DRAKE POND
SCALE: 1"=10'




TYPICAL EXISTING CHANNEL CROSS SECTION WITH CHECK STRUCTURES
SUNNYSLOPE DRIVE TO WEST FORK SQUIRREL CREEK
SCALE: 1"=10'




TYPICAL PRUDENT LINE CHANNEL
SNOWBRUSH DRIVE TO MERIDIAN ROAD
NOT TO SCALE

FIGURE 7-1
SHEET 4

EL PASO COUNTY COLORADO SEAL EST. 1861

STORMWATER
& ENVIRONMENTAL
CONSULTANTS, INC.
Engineering, Scientific & Financial Specialists in Stormwater Management

drawn by: FAP, WCB
designed by: KKB
checked by: KKB
project no.: 2000-0818
drawing no.:
date: MAR 01
revisions:

DRAFT BENNETT
RANCH PILOT PROJECT

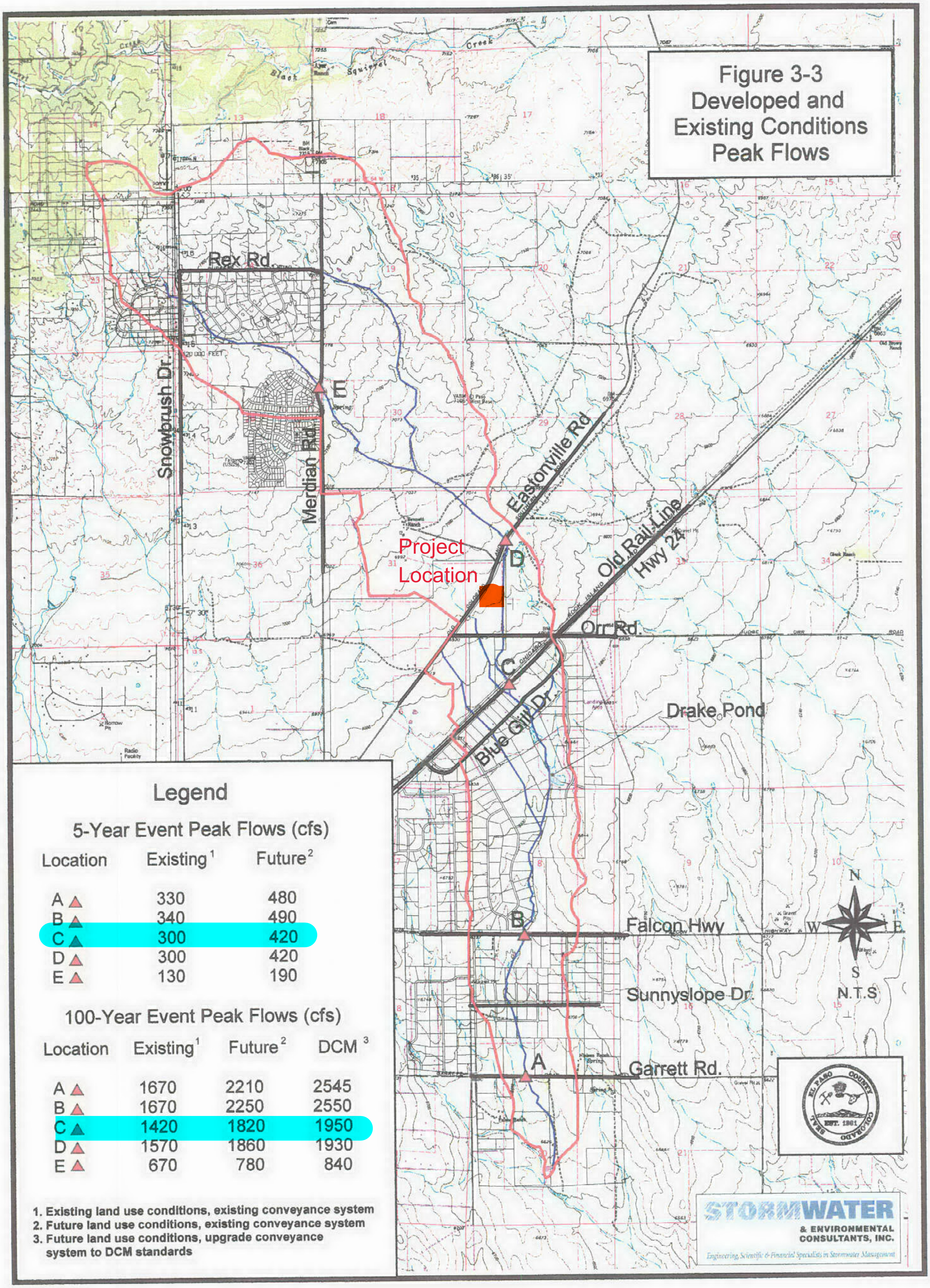



Figure 3-3
Developed and
Existing Conditions
Peak Flows

## Legend

5-Year Event Peak Flows (cfs)

| Location | Existing [1] | Future [2] |
|---|---|---|
| A | 330 | 480 |
| B | 340 | 490 |
| C | 300 | 420 |
| D | 300 | 420 |
| E | 130 | 190 |

100-Year Event Peak Flows (cfs)

| Location | Existing [1] | Future [2] | DCM [3] |
|---|---|---|---|
| A | 1670 | 2210 | 2545 |
| B | 1670 | 2250 | 2550 |
| C | 1420 | 1820 | 1950 |
| D | 1570 | 1860 | 1930 |
| E | 670 | 780 | 840 |

1. Existing land use conditions, existing conveyance system
2. Future land use conditions, existing conveyance system
3. Future land use conditions, upgrade conveyance system to DCM standards

# FEMA Flood Maps

# National Flood Hazard Layer FIRMette

FEMA

104°35'30"W 38°57'53"N


104°34'53"W 38°57'25"N

0 250 500 1,000 1,500 2,000 Feet

1:6,000

## Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

**SPECIAL FLOOD HAZARD AREAS**
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

**OTHER AREAS OF FLOOD HAZARD**
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

**OTHER AREAS**
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

**GENERAL STRUCTURES**
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

**OTHER FEATURES**
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

**MAP PANELS**
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 6/22/2020 at 1:22 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

## Mile High Flood District Standards

**Table EDB-4. EDB component criteria**

| | **On-Site EDBs for Watersheds up to 1 Impervious Acre[1]** | **EDBs with Watersheds between 1 and 2 Impervious Acres[1]** | **EDBs with Watersheds up to 5 Impervious Acres** | **EDBs with Watersheds over 5 Impervious Acres** | **EDBs with Watersheds over 20 Impervious Acres** |
|---|---|---|---|---|---|
| Forebay Release and Configuration | EDBs should not be used for watersheds with less than 1 impervious acre. | Release 2% of the undetained 100-year peak discharge by way of a wall/notch configuration | Release 2% of the undetained 100-year peak discharge by way of a wall/notch configuration | Release 2% of the undetained 100-year peak discharge by way of a wall/notch configuration | Release 2% of the undetained 100-year peak discharge by way of a wall/notch or berm/pipe[2] configuration |
| Minimum Forebay Volume | | 1% of the WQCV | 2% of the WQCV | 3% of the WQCV | 3% of the WQCV |
| Maximum Forebay Depth | | 12 inches | 18 inches | 18 inches | 30 inches |
| Trickle Channel Capacity | | ≥ the maximum possible forebay outlet capacity | ≥ the maximum possible forebay outlet capacity | ≥ the maximum possible forebay outlet capacity | ≥ the maximum possible forebay outlet capacity |
| Micropool | | Area ≥ 10 $ft^2$ | Area ≥ 10 $ft^2$ | Area ≥ 10 $ft^2$ | Area ≥ 10 $ft^2$ |
| Initial Surcharge Volume | | Depth ≥ 4 inches | Depth ≥ 4 inches | Depth ≥ 4 in. Volume ≥ 0.3% WQCV | Depth ≥ 4 in. Volume ≥ 0.3% WQCV |

[1] EDBs are not recommended for sites with less than 2 impervious acres. Consider a sand filter or rain garden.

[2] Round up to the first standard pipe size (minimum 8 inches).

5. **Forebay Design**: The forebay provides an opportunity for larger particles to settle out in an area that can be easily maintained. The length of the flow path through the forebay should be maximized, and the slope minimized to encourage settling. The appropriate size of the forebay may be as much a function of the level of development in the tributary area as it is a percentage of the WQCV. When portions of the watershed may remain disturbed for an extended period of time, the forebay size will need to be increased due to the potentially high sediment load. Refer to Table EDB-4 for a design criteria summary. When using this table, the designer should consider increasing the size of the forebay if the watershed is not fully developed.

   The forebay outlet should be sized to release 2% of the undetained peak 100-year discharge. A soil riprap berm with 3:1 sideslopes (or flatter) and a pipe outlet or a concrete wall with a notch outlet should be constructed between the forebay and the main EDB. It is recommended that the berm/pipe configuration be reserved for watersheds in excess of 20 impervious acres to accommodate the minimum recommended pipe diameter of 8 inches. When using the berm/pipe configuration, round up to the nearest standard pipe size and use a minimum diameter of 8 inches. The floor of the forebay should be concrete or lined with grouted boulders to define sediment removal limits. With either configuration, soil riprap should also be provided on the downstream side of the forebay berm or wall if the downstream grade is lower than the top of the berm or wall. The forebay will overtop frequently so this protection is necessary for erosion control. All soil riprap in the area of the forebay should be seeded and erosion control fabric should be placed to retain the seed in this high flow area.

6. **Trickle Channel:** Convey low flows from the forebay to the micropool with a trickle channel. The trickle channel should have a minimum flow capacity equal to the maximum release from the forebay outlet.

   - **Concrete Trickle Channels:** A concrete trickle channel will help to establish the bottom of the basin long-term and may also facilitate regular sediment removal. It can be a "V" shaped concrete drain pan or a concrete channel with curbs. A flat-bottom channel facilitates maintenance. A slope between 0.4% - 1% is recommended to encourage settling while reducing the potential for low points within the pan.

   - **Soft-bottom Trickle Channels**: When designed and maintained properly, soft-bottom trickle channels can allow for an attractive alternative to concrete. They can also improve water quality. However, they are not appropriate for all sites. Be aware, maintenance of soft bottom trickle channels requires mechanical removal of sediment and vegetation. Additionally, this option provides mosquito habitat. For this reason, UDFCD recommends that they be considered on a case-by-case basis and with the approval of the local jurisdiction. It is recommended that soft bottom trickle channels be designed with a consistent longitudinal slope from forebay to micropool and that they not meander. This geometry will allow for reconstruction of the original design when sediment removal in the trickle channel is necessary. The trickle channel may also be located along the toe of the slope if a straight channel is not desired. The recommended minimum depth of a soft bottom trickle channel is 1.5 feet. This depth will help limit potential wetland growth to the trickle channel, preserving the bottom of the basin.

     Riprap and soil riprap lined trickle channels are not recommended due to past maintenance experiences, where the riprap was inadvertently removed along with the sediment during maintenance.

7. **Micropool and Outlet Structure**: Locate the outlet structure in the embankment of the EDB and provide a permanent micropool directly in front of the structure. Submerge the well screen to the bottom of the micropool. This will reduce clogging of the well screen because it allows water to flow though the well screen below the elevation of the lowest orifice even when the screen above the water surface is plugged. This will prevent shallow ponding in front of the structure, which provides a breeding ground for mosquitoes (large shallow puddles tend to produce more mosquitoes than a smaller, deeper permanent pond).

   Micropool side slopes may be vertical walls or stabilized slopes of 3:1 (horizontal:vertical). For watersheds with less than 5 impervious acres, the micropool can be located inside the outlet structure (refer to Figures OS-7 and OS-8 provided in Fact Sheet T-12). The micropool should be at least 2.5 feet in depth with a minimum surface area of 10 square feet. The bottom should be concrete unless a baseflow is present or anticipated or if groundwater is anticipated. Riprap is not recommended because it complicates maintenance operations.

   Basins with micropools have fewer mosquitoes. Micropools reduce shallow wet areas where breeding is most favorable.

   Where possible, place the outlet in an inconspicuous location as shown in Photo EDB-3. This urban EDB utilizes landscaped parking lot islands connected by a series of culverts (shown in Photo EDB-4) to provide the required water quality and flood control volumes.

   The outlet should be designed to release the WQCV over a 40-hour period. Draining a volume of water over a specified time can be done through an orifice plate as detailed in Fact Sheet T-12. Use reservoir routing calculations as discussed in the *Storage* Chapter of Volume 2 to assist in the design. Two workbooks tools have been developed by UDFCD for this purpose, UD-FSD and UD-Detention. Both are available at www.udfcd.org. UD-FSD is recommended for a typical EDB full spectrum detention design. UD-Detention uses the same methodology and can be used for a full spectrum detention basin or a WQCV only design. It also allows for a wider range of outlet controls should the user want to specify something beyond what is shown in Fact Sheet T-12.

   Refer to BMP Fact Sheet T-12 for schematics pertaining to structure geometry, grates, trash racks, orifice plate, and all other necessary components.

   The outlet may have flared or parallel wing walls as shown in Figures EDB-1 and EDB-2, respectively. Either configuration should be recessed into the embankment to minimize its profile. Additionally, the trash rack should be sloped with the basin side-slopes.

From MHFD, Volume 1:

**Table 8-5. Recommended roughness values**

| Location and Cover | When Assessing Velocity, Froude No., Shear Stress | When Assessing Water Surface Elevation and Water Depth |
|---|---|---|
| Main Channel (bankfull channel) | | |
| Sand or clay bed | 0.03 | 0.04 |
| Gravel or cobble bed | 0.035 | 0.07 |
| Vegetated Overbanks | | |
| Turfgrass sod | 0.03 | 0.04 |
| Native grasses | 0.032 | 0.05 |
| Herbaceous wetlands (few or no willows) | 0.06 | 0.12 |
| Willow stands, woody shrubs | 0.07 | 0.16 |

(Source: Chow 1959, USDA 1954, Barnes 1967, Arcement and Schneider 1989, Jarrett 1985)

# 4.0 Intensity-Duration Curves for Rational Method

To develop depth-duration curves or intensity-duration curves for the Rational Method of runoff analysis take the 1-hour depth(s) obtained from NOAA Atlas 14 and apply Equation 5-1 for the duration (or durations) of interest:

$$I = \frac{28.5\,P_1}{\left(10 + T_d\right)^{0.786}}$$ Equation 5-1

Where:

$I$ = rainfall intensity (inches per hour)

$P_1$ = 1-hour point rainfall depth (inches)

$T_d$ = storm duration (minutes)

### 2.4.1 Initial or Overland Flow Time

The initial or overland flow time, $t_i$, may be calculated using Equation 6-3:

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L_i}}{S_o^{0.33}}$$ Equation 6-3

Where:

$t_i$ = overland (initial) flow time (minutes)
$C_5$ = runoff coefficient for 5-year frequency (from Table 6-4)
$L_i$ = length of overland flow (ft)
$S_o$ = average slope along the overland flow path (ft/ft).

Equation 6-3 is adequate for distances up to 300 feet in urban areas and 500 feet in rural areas. Note that in a highly urbanized catchment, the overland flow length is typically shorter than 300 feet due to effective man-made drainage systems that collect and convey runoff.

### 2.4.2 Channelized Flow Time

The channelized flow time (travel time) is calculated using the hydraulic properties of the conveyance element. The channelized flow time, $t_t$, is estimated by dividing the length of conveyance by the velocity. The following equation, Equation 6-4 (Guo 2013), can be used to determine the flow velocity in conjunction with Table 6-2 for the conveyance factor.

$$t_t = \frac{L_t}{60K\sqrt{S_o}} = \frac{L_t}{60V_t}$$ Equation 6-4

Where:

$t_t$ = channelized flow time (travel time, min)
$L_t$ = waterway length (ft)
$S_o$ = waterway slope (ft/ft)
$V_t$ = travel time velocity (ft/sec) = $K\sqrt{S_o}$
$K$ = NRCS conveyance factor (see Table 6-2).

**Table 6-2. NRCS Conveyance factors, K**

| Type of Land Surface | Conveyance Factor, K |
|---|---|
| Heavy meadow | 2.5 |
| Tillage/field | 5 |
| Short pasture and lawns | 7 |
| Nearly bare ground | 10 |
| Grassed waterway | 15 |
| Paved areas and shallow paved swales | 20 |

The time of concentration, $t_c$, is the sum of the initial (overland) flow time, $t_i$, and the channelized flow time, $t_t$, as per Equation 6-2.

### 2.4.3 First Design Point Time of Concentration in Urban Catchments

Equation 6-4 was solely determined by the waterway characteristics and using a set of empirical formulas. A calibration study between the Rational Method and the Colorado Urban Hydrograph Procedure (CUHP) suggests that the time of concentration shall be the lesser of the values calculated by Equation 6-2 and Equation 6-5 (Guo and Urbonas 2013).

$$t_c = (26 - 17i) + \frac{L_t}{60(14i + 9)\sqrt{S_t}} \qquad \text{Equation 6-5}$$

Where:

$t_c$ = minimum time of concentration for first design point when less than $t_c$ from Equation 6-1.
$L_t$ = length of channelized flow path (ft)
$i$ = imperviousness (expressed as a decimal)
$S_t$ = slope of the channelized flow path (ft/ft).

Equation 6-5 is the regional time of concentration that warrants the best agreement on peak flow predictions between the Rational Method and CUHP when the imperviousness of the tributary area is greater than 20 percent. It was developed using the UDFCD database that includes 295 sample urban catchments under 2-, 5-, 10-, 50, and 100-yr storm events (MacKenzie 2010). It suggests that both initial flow time and channelized flow velocity are directly related to the catchment's imperviousness (Guo and MacKenzie 2013).

The first design point is defined as a node where surface runoff enters the storm drain system. For example, all inlets are "first design points" because inlets are designed to accept flow into the storm drain.

Typically, but not always, Equation 6-5 will result in a lesser time of concentration at the first design point and will govern in an urbanized watershed. For subsequent design points, add the travel time for each relevant segment downstream.

### 2.4.4 Minimum Time of Concentration

Use a minimum $t_c$ value of 5 minutes for urbanized areas and a minimum $t_c$ value of 10 minutes for areas that are not considered urban. Use minimum values even when calculations result in a lesser time of concentration.

### 2.4.5 Common Errors in Calculating Time of Concentration

A common mistake in urbanized areas is to assume travel velocities that are too slow. Another common error is to not check the runoff peak resulting from only part of the catchment. Sometimes a lower portion of the catchment or a highly impervious area produces a larger peak than that computed for the whole catchment. This error is most often encountered when the catchment is long or the upper portion contains grassy open land and the lower portion is more developed.

**El Paso County Drainage Criteria Manual**

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

**Table 6-2. Rainfall Depths for Colorado Springs**

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

Where Z= 6,840 ft/100

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

### 2.2 Design Storms

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

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

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

**Table 6-6. Runoff Coefficients for Rational Method**
(Source: UDFCD 2001)

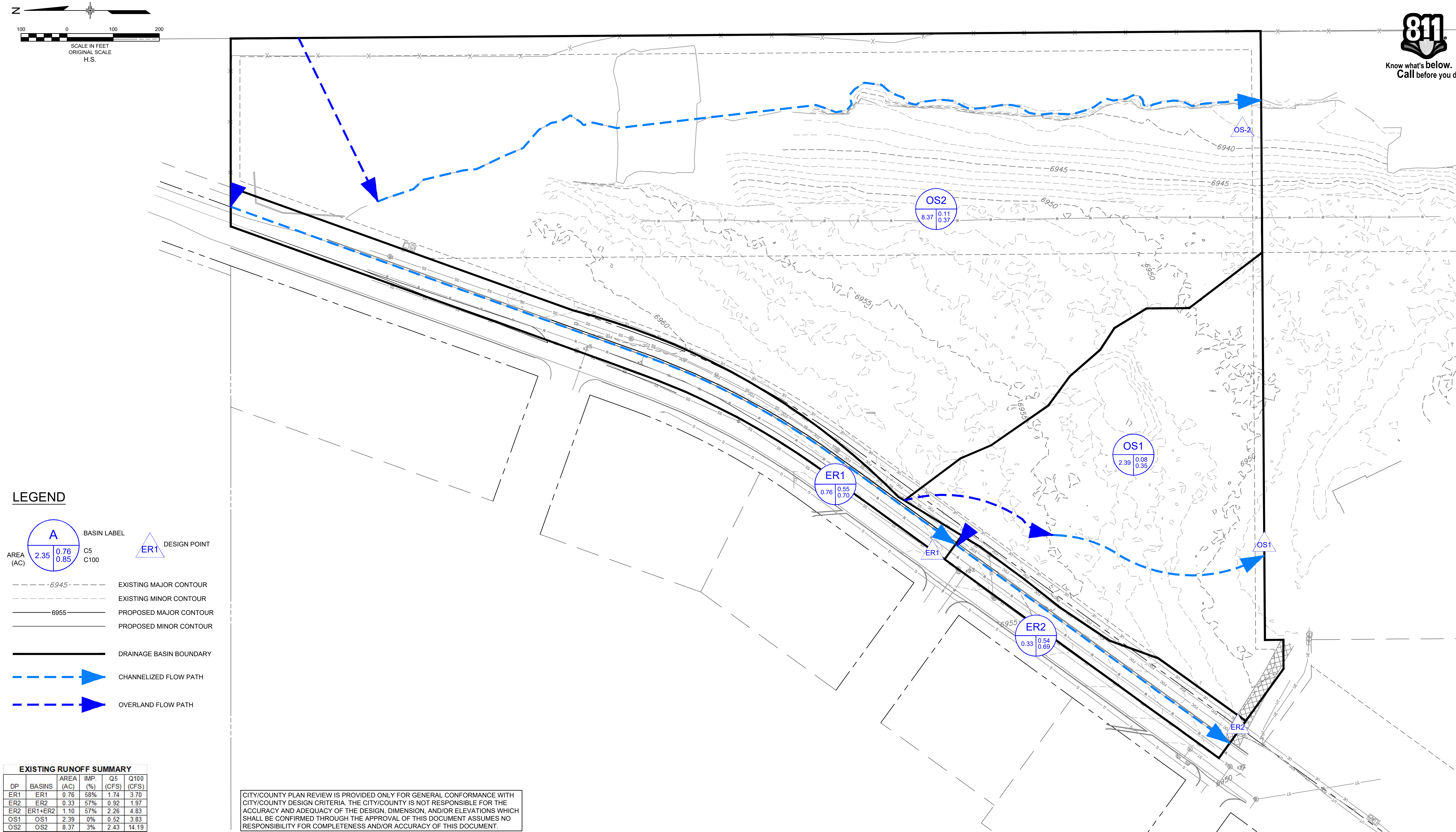
| Land Use or Surface Characteristics | Percent Impervious | Runoff Coefficients | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 2-year | | 5-year | | 10-year | | 25-year | | 50-year | | 100-year | |
| | | HSG A&B | HSG C&D | HSG A&B | HSG C&D | HSG A&B | HSG C&D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

## 3.2 Time of Concentration

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

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

# APPENDIX E – DRAINAGE PLANS




## LEGEND



**EXISTING RUNOFF SUMMARY**

| DP | BASINS | AREA (AC) | IMP. (%) | Q5 (CFS) | Q100 (CFS) |
|---|---|---|---|---|---|
| ER1 | ER1 | 0.76 | 58% | 1.74 | 3.70 |
| ER2 | ER2 | 0.33 | 57% | 0.92 | 1.97 |
| ER2 | ER1+ER2 | 1.10 | 57% | 2.26 | 4.83 |
| OS1 | OS1 | 2.39 | 0% | 0.52 | 3.83 |
| OS2 | OS2 | 8.37 | 3% | 2.43 | 14.19 |

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REFERENCE DRAWINGS
X-995-EX-BASE
X-995.002-EX-MAP
X-995.002-EX-BASE
X-995.002-MDG22x34

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

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SHEET KEY

PREPARED FOR:
LIBERTY TREE ACADEMY
PREPARED BY:
Matrix
Excellence by Design

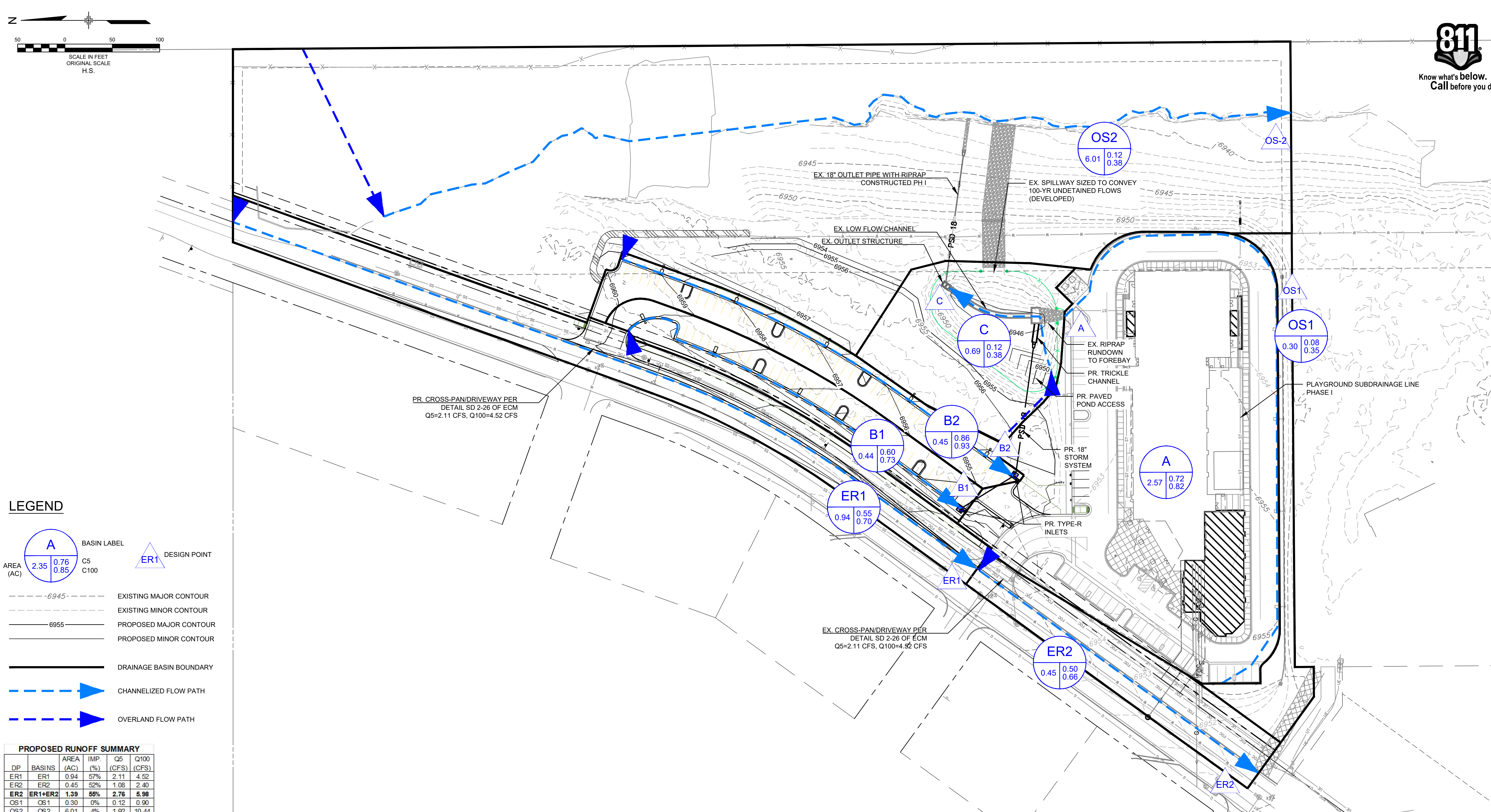
SEAL
12/11/2020
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 20.995.002

# LIBERTY TREE

DRAINAGE REPORT
PHASE II IMPROVEMENTS

## FIGURE A-1
## EXISTING DRAINAGE PLAN

| | | | | | |
|---|---|---|---|---|---|
| DESIGNED BY: | MAS | SCALE | DATE ISSUED: | 8/7/2020 | DRAWING No. |
| DRAWN BY: | MAS | HORIZ.1"=100' (HS) | SHEET | 1 OF 2 | A-1 |
| CHECKED BY: | AJB | VERT. N/A | | | |




## LEGEND

| | |
|---|---|
| A (AREA (AC) 2.35, C5 0.76, C100 0.85) | BASIN LABEL |
| ER1 | DESIGN POINT |
| - - - - 6945 - - - - | EXISTING MAJOR CONTOUR |
| - - - - - - - - | EXISTING MINOR CONTOUR |
| ——— 6955 ——— | PROPOSED MAJOR CONTOUR |
| ———————— | PROPOSED MINOR CONTOUR |
| ———————— | DRAINAGE BASIN BOUNDARY |
| - - - - ▶ | CHANNELIZED FLOW PATH |
| - - - - ▶ | OVERLAND FLOW PATH |

**PROPOSED RUNOFF SUMMARY**

| DP | BASINS | AREA (AC) | IMP. (%) | Q5 (CFS) | Q100 (CFS) |
|---|---|---|---|---|---|
| ER1 | ER1 | 0.94 | 57% | 2.11 | 4.52 |
| ER2 | ER2 | 0.45 | 52% | 1.08 | 2.40 |
| **ER2** | **ER1+ER2** | **1.39** | **55%** | **2.76** | **5.98** |
| OS1 | OS1 | 0.30 | 0% | 0.12 | 0.90 |
| OS2 | OS2 | 6.01 | 4% | 1.92 | 10.44 |
| A | A | 2.57 | 81% | 7.66 | 14.65 |
| B1 | B1 | 0.44 | 63% | 1.34 | 2.77 |
| B2 | B2 | 0.45 | 95% | 1.94 | 3.53 |
| C | C | 0.69 | 4% | 0.28 | 1.54 |
| **C** | **A-C** | **4.15** | **68%** | **9.93** | **20.11** |

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REFERENCE DRAWINGS

X-995-EX-BASE
X-995-PR-UTIL
X-995.002-PR-BASE_PH-2
X-995.002-EX-MAP
X-995.002-EX-BASE
X-995.002-EX-BASE_PH-1
X-995.002-MDG22x34

| No. | DATE | DESCRIPTION | BY |
|---|---|---|---|

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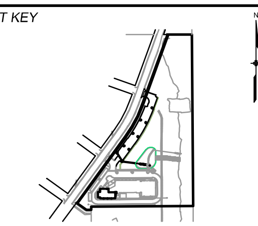

PREPARED FOR:
LIBERTY TREE ACADEMY

PREPARED BY:
Matrix
Excellence by Design

SEAL

COLORADO REGISTERED PROFESSIONAL ENGINEER 24520 12/11/2020

FOR AND ON BEHALF OF
MATRIX DESIGN GROUP, INC.
PROJECT No. 20.995.002

# LIBERTY TREE

DRAINAGE REPORT
PHASE II IMPROVEMENTS

## FIGURE A-2
## PROPOSED DRAINAGE PLAN

| | | | |
|---|---|---|---|
| DESIGNED BY: MAS | SCALE | DATE ISSUED: 8/7/2020 | DRAWING No. |
| DRAWN BY: MAS | HORIZ.1"=100' (HS) | SHEET 2 OF 2 | A-2 |
| CHECKED BY: AJB | VERT. N/A | | |