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:

April 13, 2020

PPR-18-023

Revise to PPR2018

please remove the word "City"

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 City/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

COLORADO REGISTERED PROFESSIONAL ENGINEER
ANDREW J. BECK
48153

Developers Statement:

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

____

Ronnie Wilson, Vice President
Liberty Tree Academy Building Corporation
PO Box 64614
Colorado Springs, CO 80962

Please revise to state the following:
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.

EL PASO COUNTY ONLY:

Filed in accordance with Section 51.1 of the El Paso Land Development Code, as amended

____

Director of Public Works Date

Conditions:

Revise to:
Jennifer Irvine, P.E.
County Engineer / ECM Administrator

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Please locate the drainage plans at the end of your report.

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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 “existing condition” in this report refers to the condition previous to any development and the “proposed condition” includes the buildings proposed with the August 2018 report as well as the new proposed addition and parking area.

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.

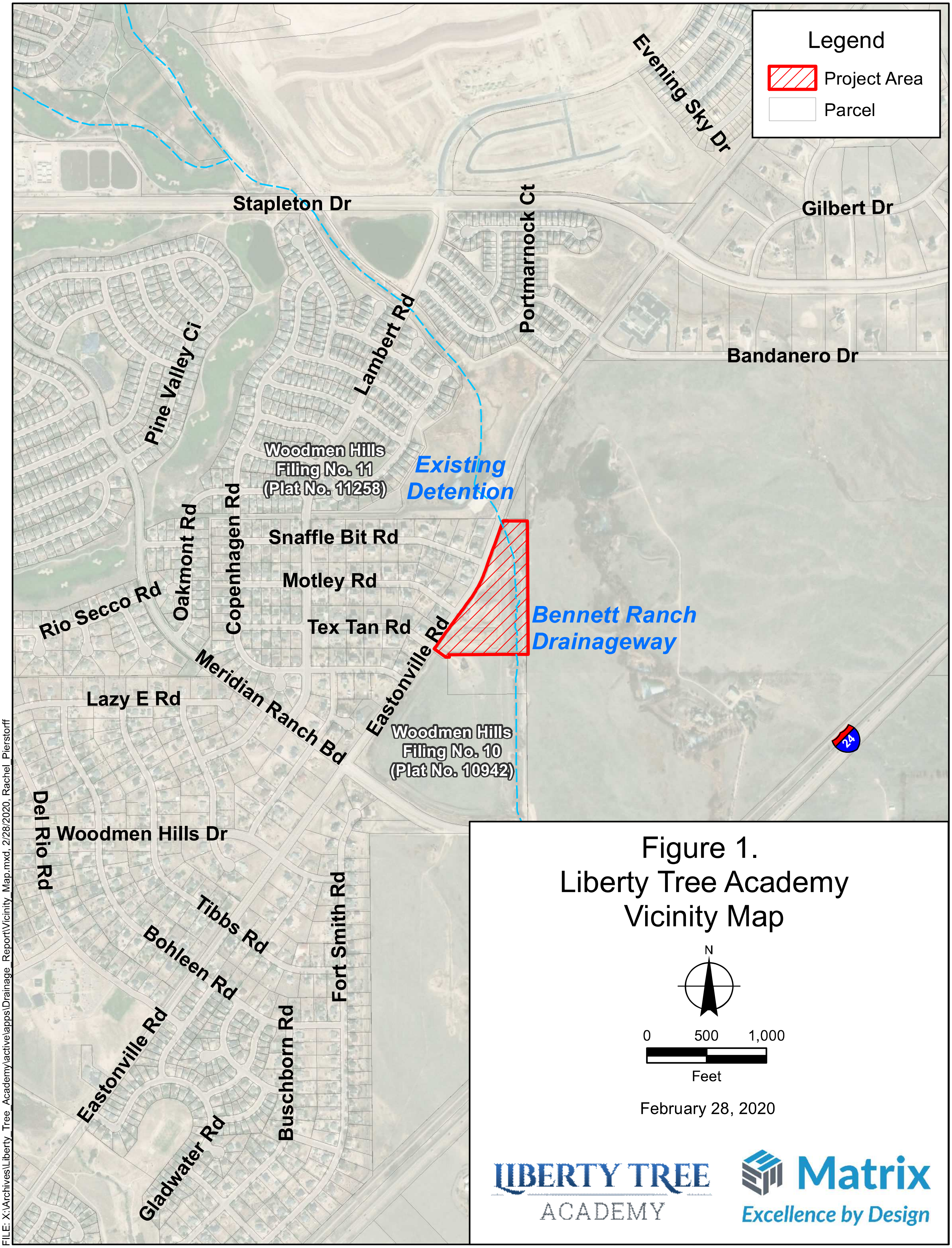

Figure 1.
Liberty Tree Academy
Vicinity Map

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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.

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**. No irrigation facilities exist onsite.

The previously approved site development plan indicated an irrigation system at the landscaped areas. Revise the text accordingly.

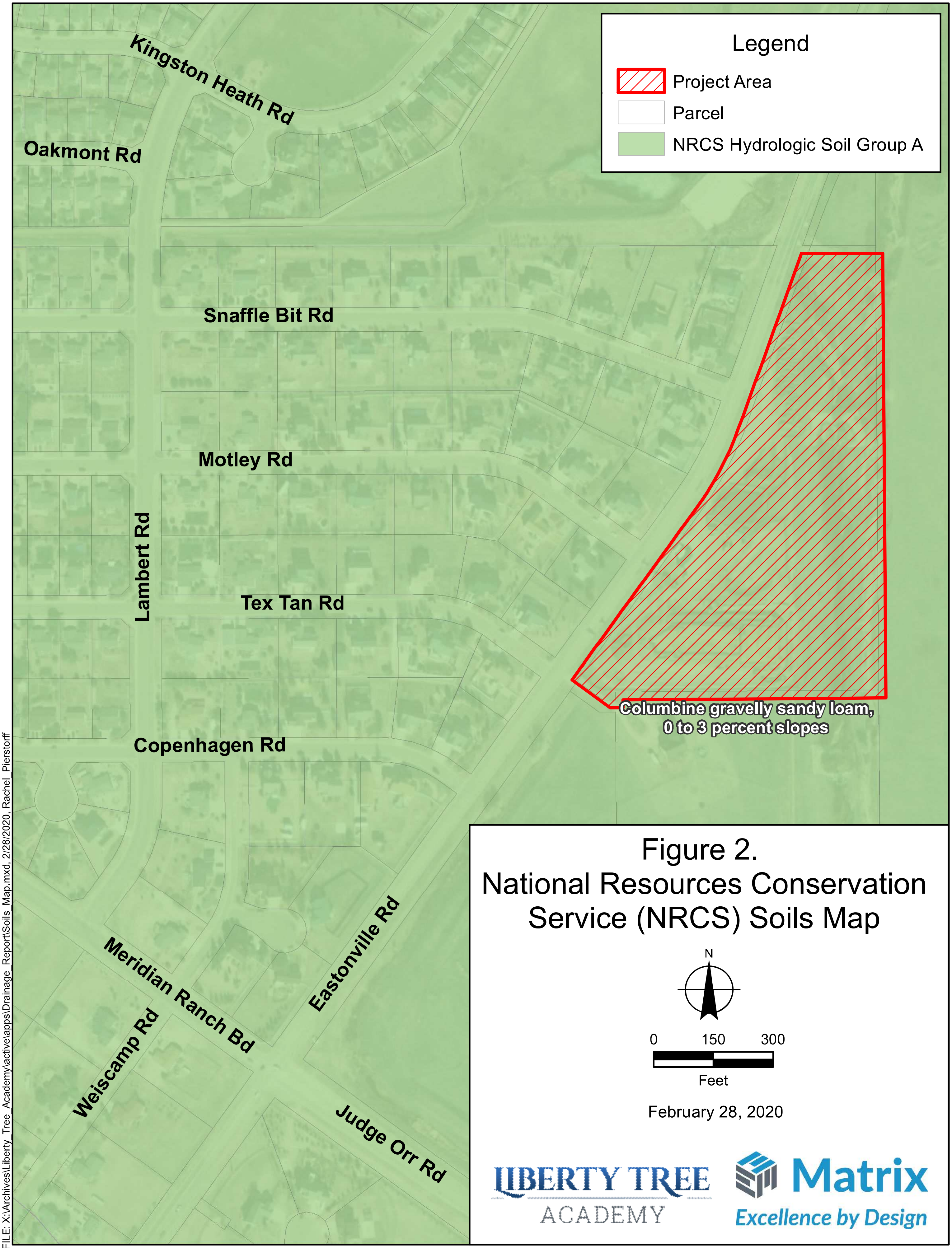

Figure 2.
National Resources Conservation Service (NRCS) Soils Map

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II. DRAINAGE BASINS AND SUB-BASINS

PLease update the FEMA FIRM # and date accordingly.

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 E**.

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 080059 0575 F, Effective March 17, 1997 (see **Appendix E**). The FIRM was revised in the vicinity of the project by Letter of Map Revision (LOMR) Case Number 12-08-0659P, on July 12, 2013. This LOMR extended the floodplain upstream, however, the project is located outside of this boundary.

B. Sub-basin Description

The overall project area in both the existing and proposed condition drain southeast to the Bennett Ranch Drainageway. Existing 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.

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

Eastonville Road

Proposed Basins ER1 and ER2 are 0.94 and 0.45 acres in size, respectively, and consist of existing roadway, lawn, and proposed sidewalk. As in the existing 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 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.

Please prove a statement indicating whether or not Eastonville road has the capacity for the increase in flow

Southern Boundary

In the existing condition, Basin OS1 drains to the property to the south before reaching the Bennett Ranch Drainageway. In the proposed condition, Basin OS1 will be reduced in size from 2.39 acres to 0.30 acres and will remain unvegetated. 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 a proposed Extended Detention Basin (EDB). A separate piped system will convey roof runoff to the detention pond; an underdrain draining directly to the Bennett Ranch Drainageway will serve the playground area. The runoff from the roof and playground are included within Basin A and not calculated separately.

The proposed Basins B1 and B2 are 0.44 and 0.45 acres, respectively, and consist of parking and landscaped area. Runoff from this basin is routed via curb and gutter to Type-R Inlet. The storm system 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 Basin B reaches the EDB via a vegetated swale starting at DP B. 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 will be reduced in size from 8.37 acres to 6.01 acres and will continue to sheet flow into the drainageway. Basin OS2 will remain largely undisturbed except some grading to meet the existing grade and the installation of a riprap rundown. Disturbed areas will be revegetated. The tributary runoff from Basin OS2 will be less in the proposed condition than in the existing condition. Basin OS2 will continue to serve as a drainage easement.

In the paragraph above it indicates that runoff from the B basins will be routed to inlets. The GEC plan shows a storm drain from these inlets that discharge to the pond. A vegetated swale has not been identified at design point B2. Please revise the narrative accordingly.

Please identify this new riprap rundown to be installed in basin OS2 on the drainage plan. The riprap shown on the drainage plan appear to be existing. Please clarify and/or revise accordingly.

C. Conveyance of Offsite Runoff

Basins ER1 and ER2 contain half of offsite Eastonville Road and the proposed sidewalk and landscaped area within the ROW. These basins will continue to drain into Eastonville Road. The total area added by the proposed 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 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

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.

Please also include the previously approved drainage report for phase 1

2. Previous Drainage Studies

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

- El Paso County. 2001. Bennett Ranch Pilot Project Drainage Basin Planning Study. El Paso County. November 2001.
- Federal Emergency Management Agency (FEMA). 1999. Flood Insurance Study El Paso County, Colorado and Unincorporated Areas. Revised August 23, 1999.
- Federal Emergency Management Agency (FEMA). 1997. Flood Insurance Rate Map Number 08041C0575 F. El Paso County, Colorado and Unincorporated Areas. Effective August 17, 1997.
- Federal Emergency Management Agency (FEMA). 2013. Letter of Map Revision Case No. 12-08-0659P, Flood Insurance Rate Map Number 08041C0575 F. El Paso County, Colorado and Unincorporated Areas. Effective July 12, 2013.

The site is not within a FEMA regulatory floodplain (See **Appendix E**). 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 C**.

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 existing 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 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 existing 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 proposed extended detention basin (EDB) in the eastern side of the project area. The extended detention basin will maintain historic outflow to the existing Bennett Ranch Drainageway.

The approved drainage report for phase 1 indicated that the EDB will have to be redesigned to accommodate the increase in imperviousness due to future development. Address in your narrative how the pond is affected by this new development as it was not accounted for in the original design of the pond. Is the pond as constructed adequate? Are any changes required to the pond, outlet structure, orifice plate, etc.? and if so provided the appropriate construction documents. How do the developed flows compare to the previously approved design?

See previous comments and revise accordingly.

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. A vegetated swale is provided from the parking area in Basin B, and vegetation is provided by landscape islands and around the building; this was not considered as a formal water quality measure for our calculations. Developed runoff from the site is routed to an extended detention basin.

There are inlets at design points (B1 and B2) not curb cuts. Please revise accordingly.

2. Stabilize Drainageways

All channelized runoff on the site is conveyed via curb and gutter to a curb cuts at DPs A and B. Riprap rundowns route the runoff to an extended detention basin, and energy 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 E**).

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 D** for extended detention basin design.

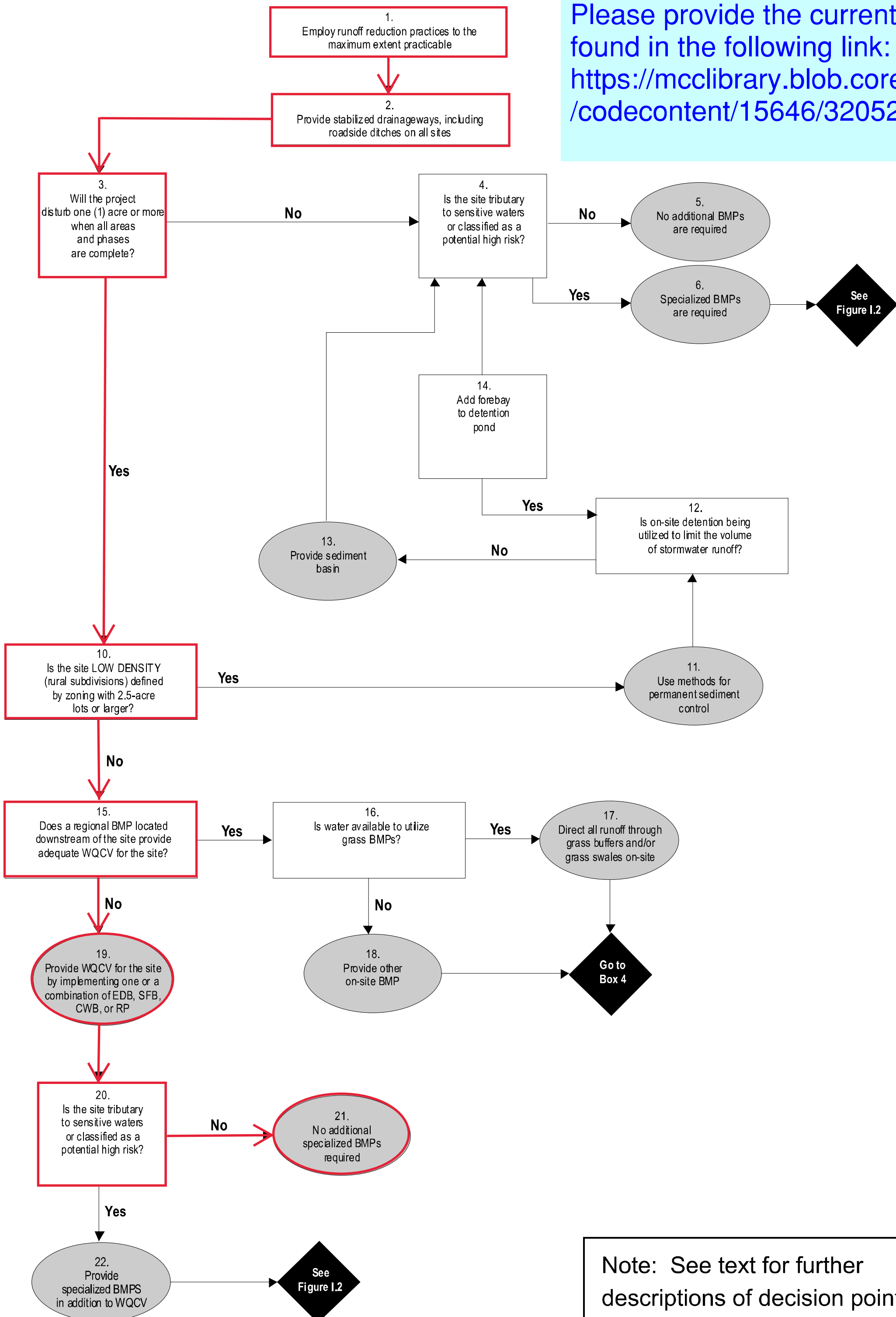
4. Consider Need for Industrial and Commercial BMPs

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

Per ECM appendix I.7 (revised July 2019), 100% of the applicable development site must be captured. Please identify the areas that are not captured and provide justification for not capturing these areas. Please site criteria that allows for these areas to not be captured. (See ECM section I.7.1.B.6 and I.7.1.C.1).

Figure I-1. BMP Requirements Flowchart for New Development and Redevelopment Sites - For Selecting Post-Construction BMPs in Compliance with El Paso County's Stormwater NPDES Permit

Please provide the current flowchart. It can be found in the following link: https://mcclibrary.blob.core.usgovcloudapi.net/codecontent/15646/320527/FigI-1.png



Note: See text for further descriptions of decision points and requirements.

OS2

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 existing condition. Basin O2 will remain undeveloped and drain to the Bennett Ranch Drainageway; any regraded area in basin OS2 will be revegetated.

Basins A, B, and C contain all the proposed 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 detention pond. Runoff from Basin B will be routed via curb and gutter to a curb cut at DP B, and will then be routed via vegetated swale 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 C for detailed hydrology calculations, see Appendix D 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

There are inlets at design points (B1 and B2) not curb cuts. Please revise accordingly.

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.*

east

2. Extended Detention Basin Design

The extended detention basin will be located at the eastern edge of the developed area. The EDB will intercept all developed runoff from the site and convey attenuated flows west to the Bennett Ranch Drainageway. The proposed extended detention basin 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 D.**

<u>Volumes and Release Rates</u>

The basin and outlet structure were sized using MHFD-Detention, version 4.02 in accordance with DCM-V1-Update criteria. The outlet structure utilizes an orifice plate to release the water quality capture volume (WQCV) over 40-hours and the extended urban runoff volume (EURV) in 72-hours. A 2.5' micropool in front of the orifice plate will provide settlement. A drop box and 18-inch pipe with a restrictor plate will attenuate runoff events exceeding the EURV. Outflows will be conveyed under to the existing stabilized channel in Bennett Ranch Drainageway. Total 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)=	0.04	1.22	0.21	0.92	1.18	1.22	1.25	1.29
Calculated Runoff Volume (acre-ft)	0.092	0.354	0.248	0.324	0.385	0.463	0.540	0.632
Maximum Volume Stored (acre-ft)	0.093	0.355	0.229	0.253	0.273	0.338	0.399	0.489
Maximum Ponding Elevation (ft)	6946.94	6948.82	6948.02	6948.18	6948.32	6948.73	6949.08	6949.55

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 C** 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 6949.55’. Placing the emergency overflow weir crest at this elevation would reduce the cover of the waterline parallel to the access road to less than 5’. In order to maintain 5’ of cover over the waterline, it is proposed that the emergency overflow weir elevation be set at 6951.00’. In the condition that the outlet structure became completely clogged, the 100-yr event runoff volume, 0.632 ac-ft, would have a ponded depth of 6950.24’. Total freeboard between the 100-yr WSEL (clogged condition) and weir crest is 0.76’.

The 8.82-ft wide emergency spillway was sized 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 would occur, but would be less than 12”.

Please identify which parking lot

Other Design Components

Runoff will enter the detention pond via the concrete pans within the parking lot in drainage Basin A and flow down a riprap rundown to the forebay. From Basin B, runoff will be captured by Type-R inlets and piped to a riprap rundown, 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 trickle channel 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.

Will the new trickle channel have these same specifications? Please address.

Maintenance access to the pond will be along the existing drainage easement access road via the proposed school fire lane 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. 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 D**. The Type-R Inlet capacity within Basins B1 and B2 were also calculated using the UD-Inlet_v4.05 workbook.

Please be sure to identify this as the new driveway

Matrix

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 E**). 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 E**). 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 D**. 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 E**).*
2. *Freeboard is measured from weir crest elevation. = 6951.0*

XS 1 **XS 2**

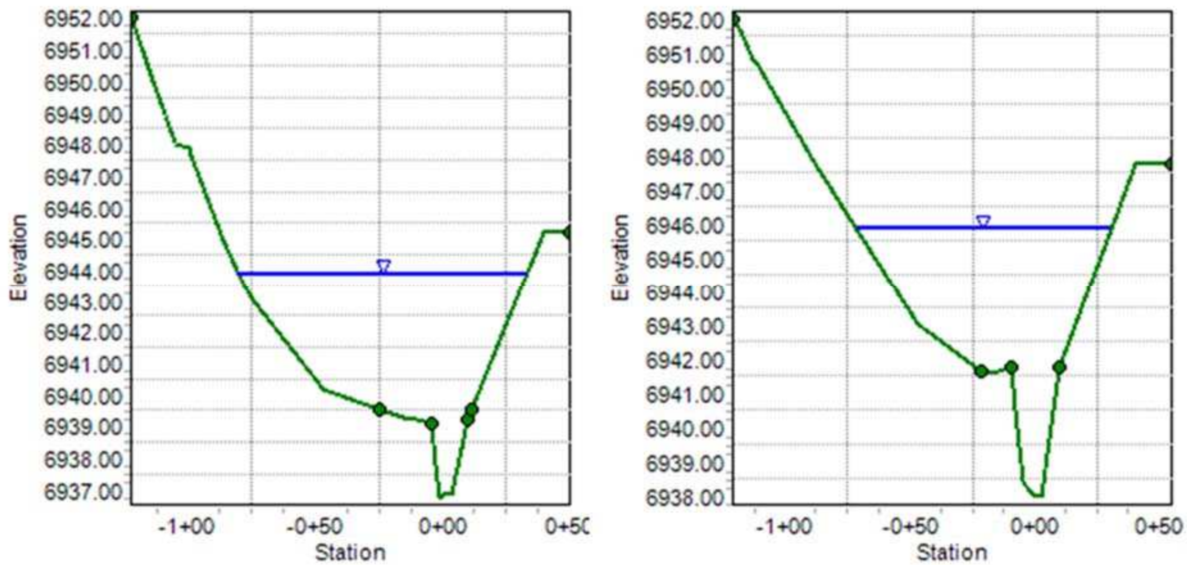


Figure 4 – Offsite Drainageway Capacity Evaluation
Cross sections, looking upstream
Q100=810 cfs (release rate of upstream basin)

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 D**.

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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El Paso County. 2001. Bennett Ranch Pilot Project Drainage Basin Planning Study. El Paso County. November 2001.

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Federal Emergency Management Agency (FEMA). 2013. Letter of Map Revision Case No. 12-08-0659P, Flood Insurance Rate Map Number 08041C0575 F. El Paso County, Colorado and Unincorporated Areas. Effective July 12, 2013.

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Mile High Flood District (MHFD). 2017a. MHFD-Detention, Version 4.02. Mile High Flood District. Last updated January 2020.

APPENDIX A – SITE PHOTOS

Figure A1. Project area looking east.

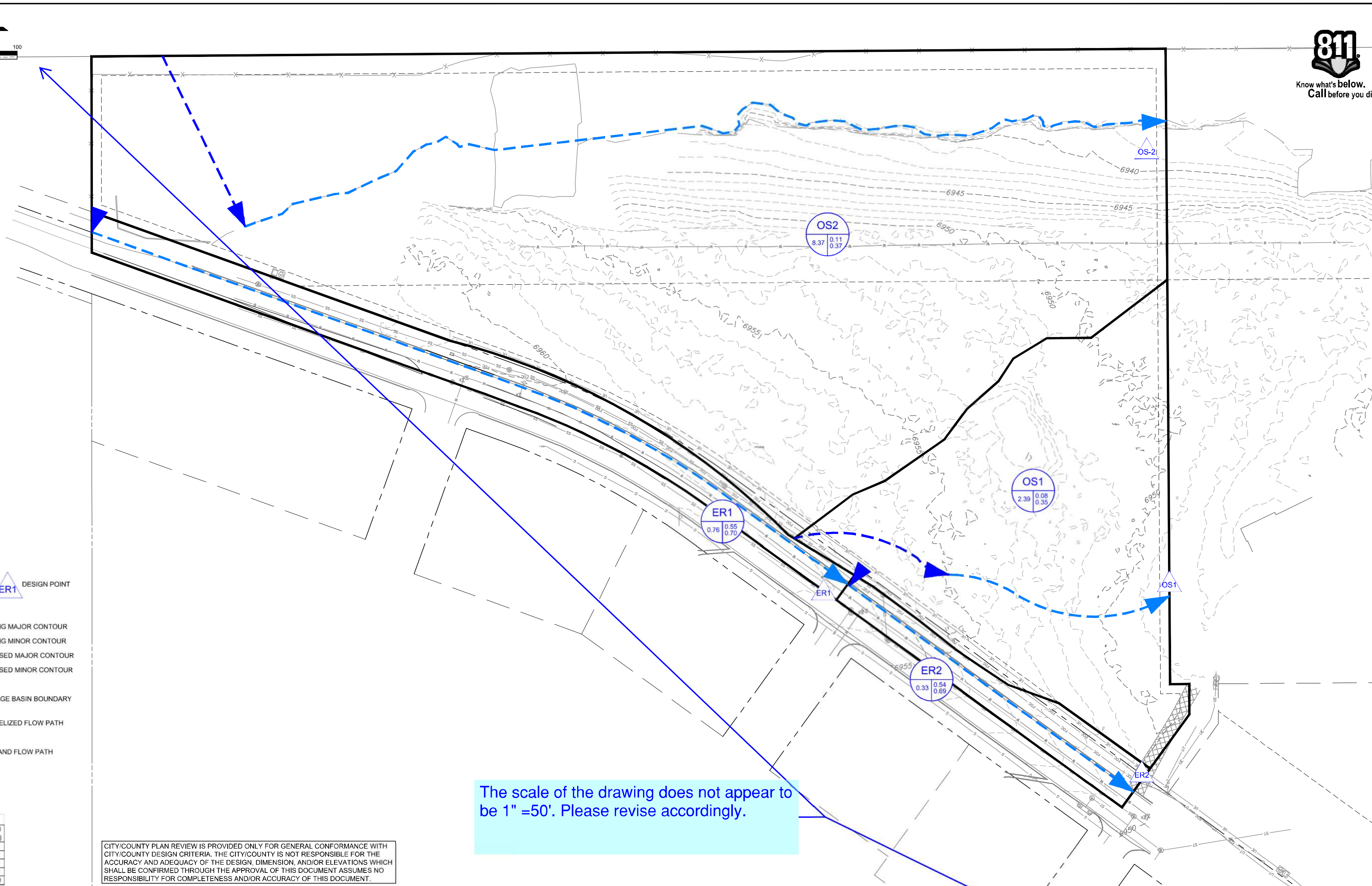
Figure A2. Project area looking south east along utility access towards the drainage easement.

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 – DRAINAGE PLANS



The scale of the drawing does not appear to be 1" =50'. Please revise accordingly.

LEGEND

AREA (AC) — 2.35 | A | 0.76 C5 | 0.85 C100 — 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

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

REVISIONS

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CTB FILE: ----
PLOT DATE: 4/10/2020 10:28 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
Excellence by Design

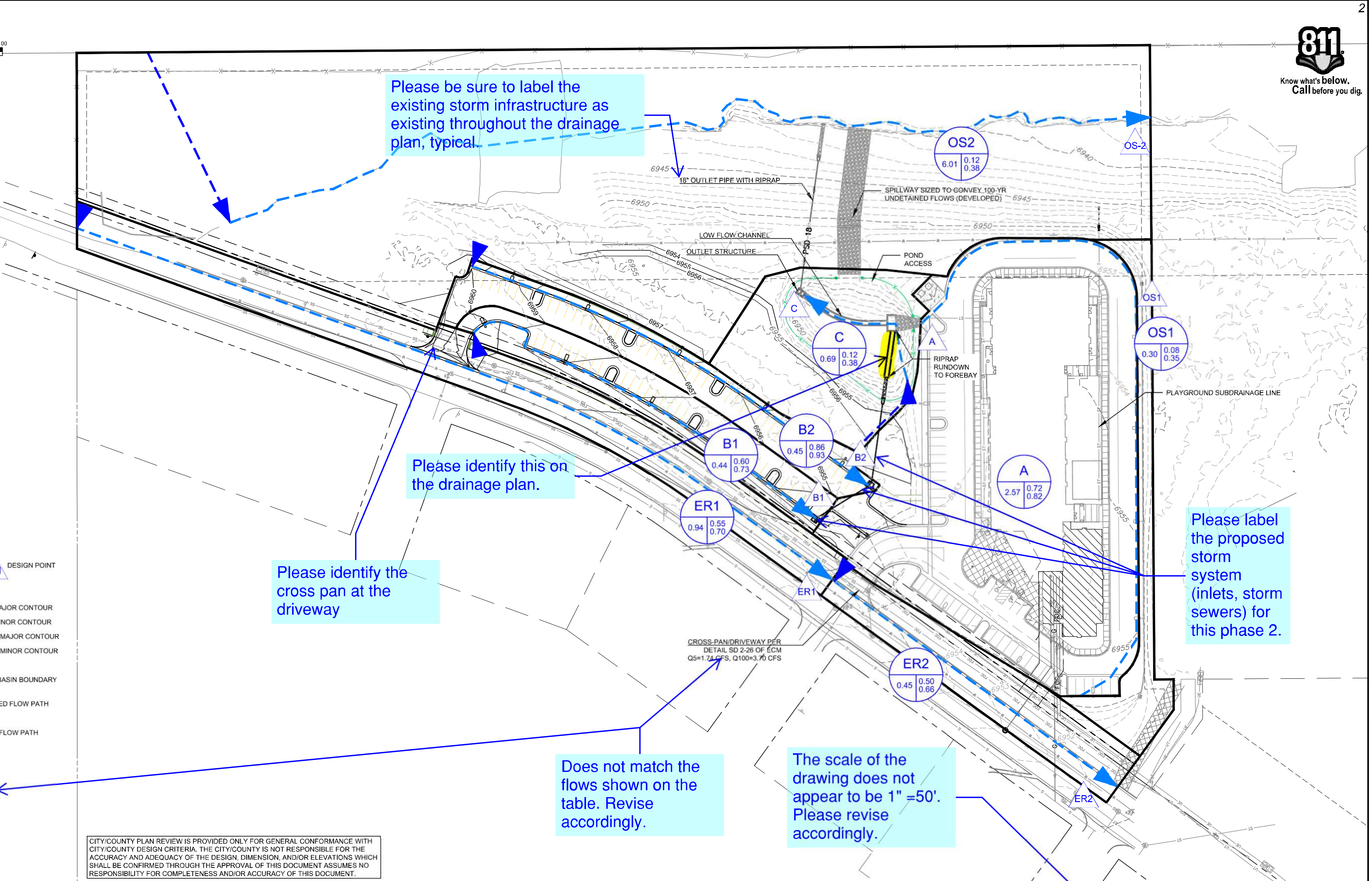
PRELIMINARY
THIS DRAWING HAS NOT BEEN APPROVED BY GOVERNING AGENCIES AND IS SUBJECT TO CHANGE

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
DRAWN BY: MAS
CHECKED BY: DK
SCALE HORIZ. 1"=50' VERT N/A
DATE ISSUED: 4/10/2020
SHEET 1 OF 2
DRAWING No. A-1



LEGEND

- A — BASIN LABEL (AREA (AC) 2.35; C5 0.76; C100 0.85)
- 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

REVISIONS

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THIS DRAWING IS CURRENT AS OF PLOT DATE AND MAY BE SUBJECT TO CHANGE.

SHEET KEY

PREPARED FOR:
LIBERTY TREE ACADEMY

PREPARED BY:
Matrix
Excellence by Design

PRELIMINARY
THIS DRAWING HAS NOT BEEN APPROVED BY GOVERNING AGENCIES AND IS SUBJECT TO CHANGE

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
DRAWN BY: MAS
CHECKED BY: DK
SCALE HORIZ. 1"=50' VERT N/A
DATE ISSUED: 4/10/2020
SHEET 2 OF 2
DRAWING No. A-2

APPENDIX C – HYDROLOGIC ANALYSIS

Project Name: Liberty Tree Academy
Job Number: 20.995.002
Subject: Composite Runoff Coefficients
Date: 4/10/2020
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

Location: Liberty Tree Academy
Date: April 10, 2020
Designed by: MAS

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

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: April 10, 2020
Designed by: MAS

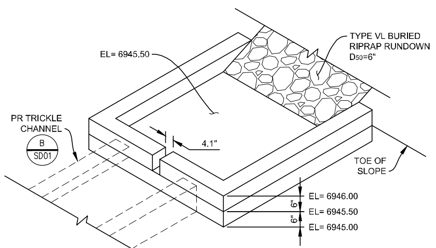
Basin Name:	Existing A-C
Area (ac.)	4.15
Imperv. (%)	0%
Tc (min.)	13.27

Design Storm	P1 (in)	Runoff Coefficient	Intensity "I"	Total Peak Discharge "Q100
			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

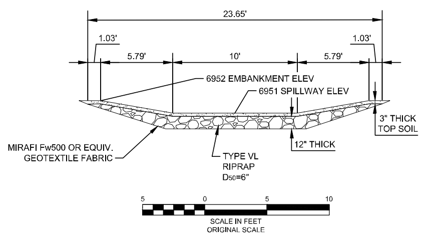
*Intensity values from Eq. 5-1 of MHFD V1

APPENDIX D – HYDRAULIC ANALYSIS

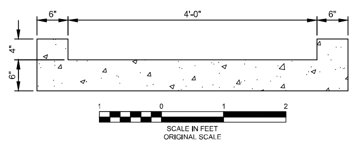
Extended Detention Basin Design



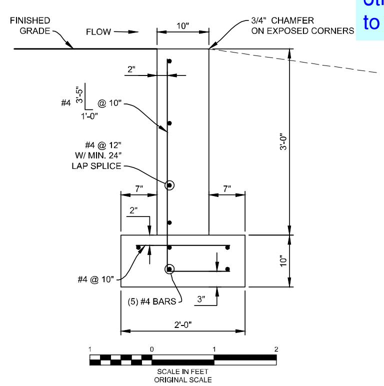
FOREBAY DETAIL (A)
SCALE: N.T.S.
SEE PLAN FOR LIMITS.



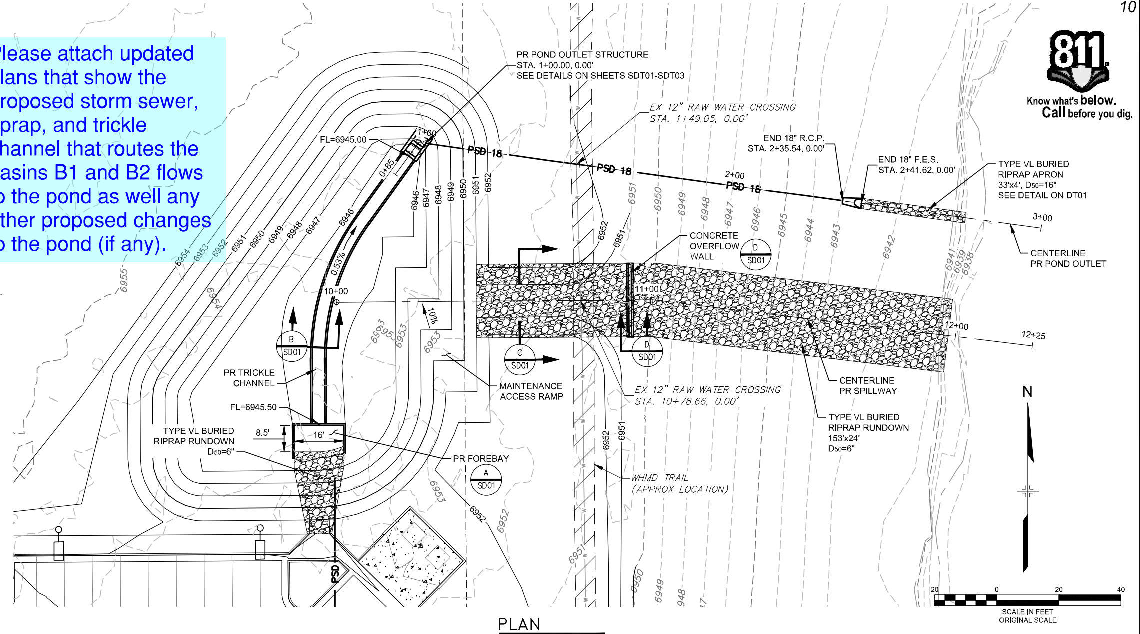
SPILLWAY SECTION (C)
SCALE: 1" = 5'



TRICKLE CHANNEL SECTION (B)

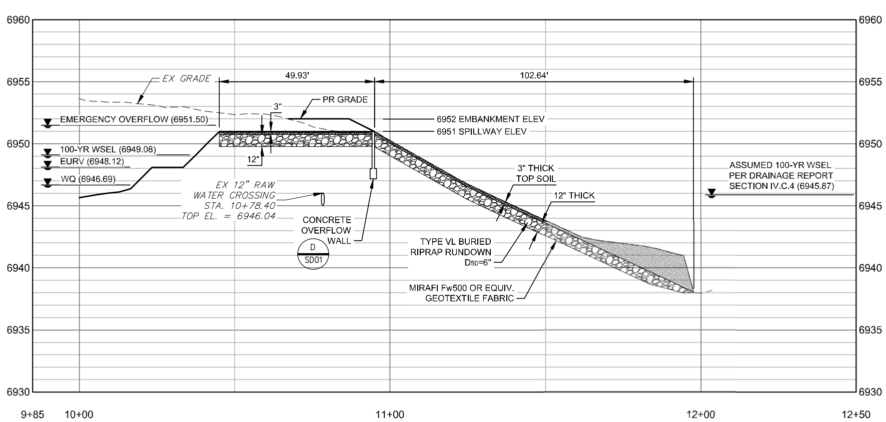


CONCRETE OVERFLOW WALL SECTION (D)
SCALE: 1" = 1'-0"

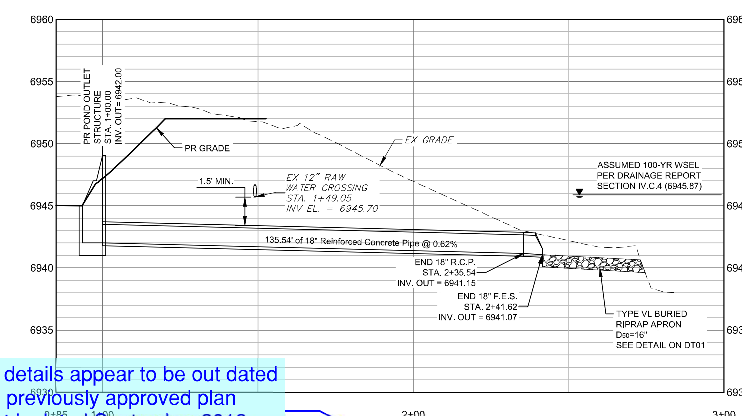


Please attach updated plans that show the proposed storm sewer, riprap, and trickle channel that routes the basins B1 and B2 flows to the pond as well any other proposed changes to the pond (if any).

PLAN
SCALE: 1"=20'



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



These pond details appear to be out dated plans as the previously approved plan (phase 1) set is dated September 2018

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.

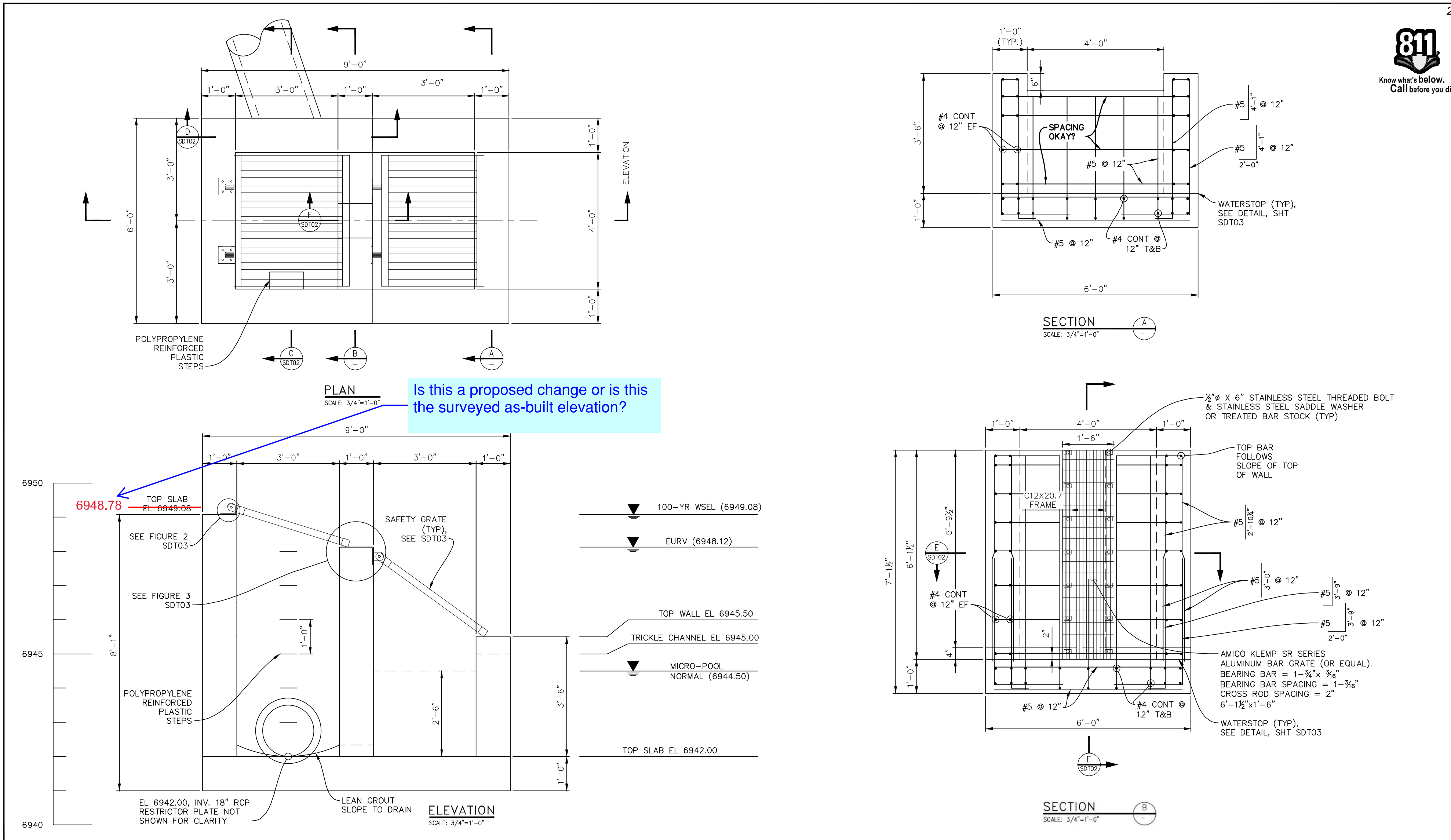
SHEET KEY

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



REFERENCE DRAWINGS	No.	DATE	DESCRIPTION	BY
X-995-MDG22x34				

REVISIONS

COMPUTER FILE MANAGEMENT
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CTB FILE: ---
PLOT DATE: 8/10/2018 8:34 AM
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SHEET KEY

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

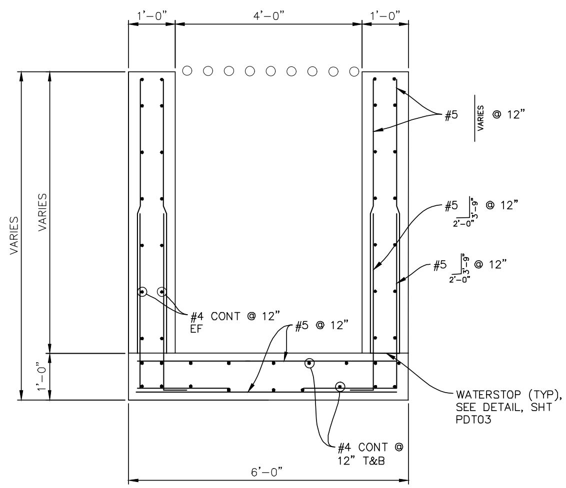
SEAL
24520
8/10/18
PROFESSIONAL ENGINEER
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC.
PROJECT No. 18.995.001

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

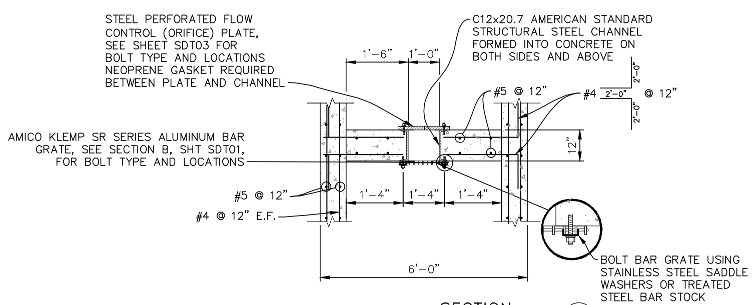
POND OUTLET STRUCTURE DETAILS

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

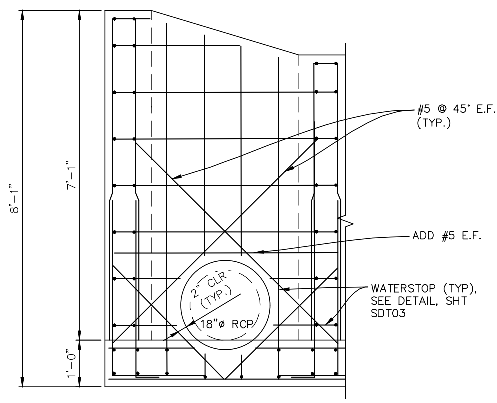
Know what's below. Call before you dig.



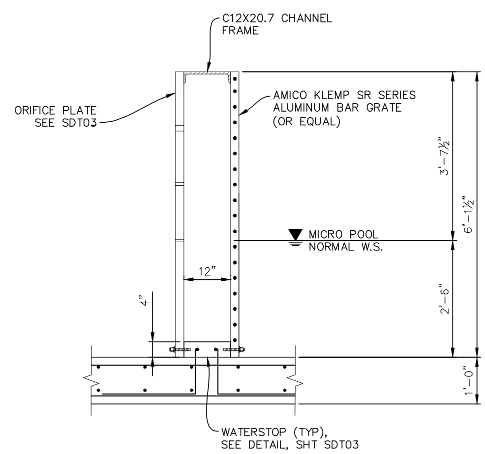
SECTION C SDT01
SCALE: 3/4"=1'-0"



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



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



SECTION F SDT01
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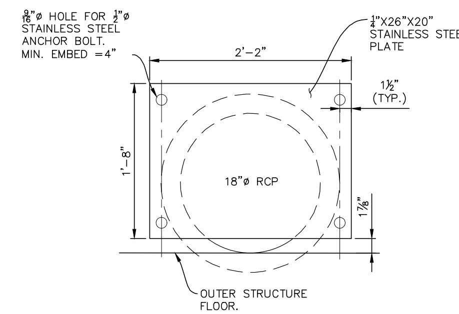

RESTRICTOR PLATE DETAIL
SCALE: 1-1/2"=1'-0"

REFERENCE DRAWINGS	No.	DATE	DESCRIPTION	BY
X-995-MDG22x34				

REVISIONS

COMPUTER FILE MANAGEMENT
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CTB FILE: ---
PLOT DATE: 8/10/2018 8:35 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

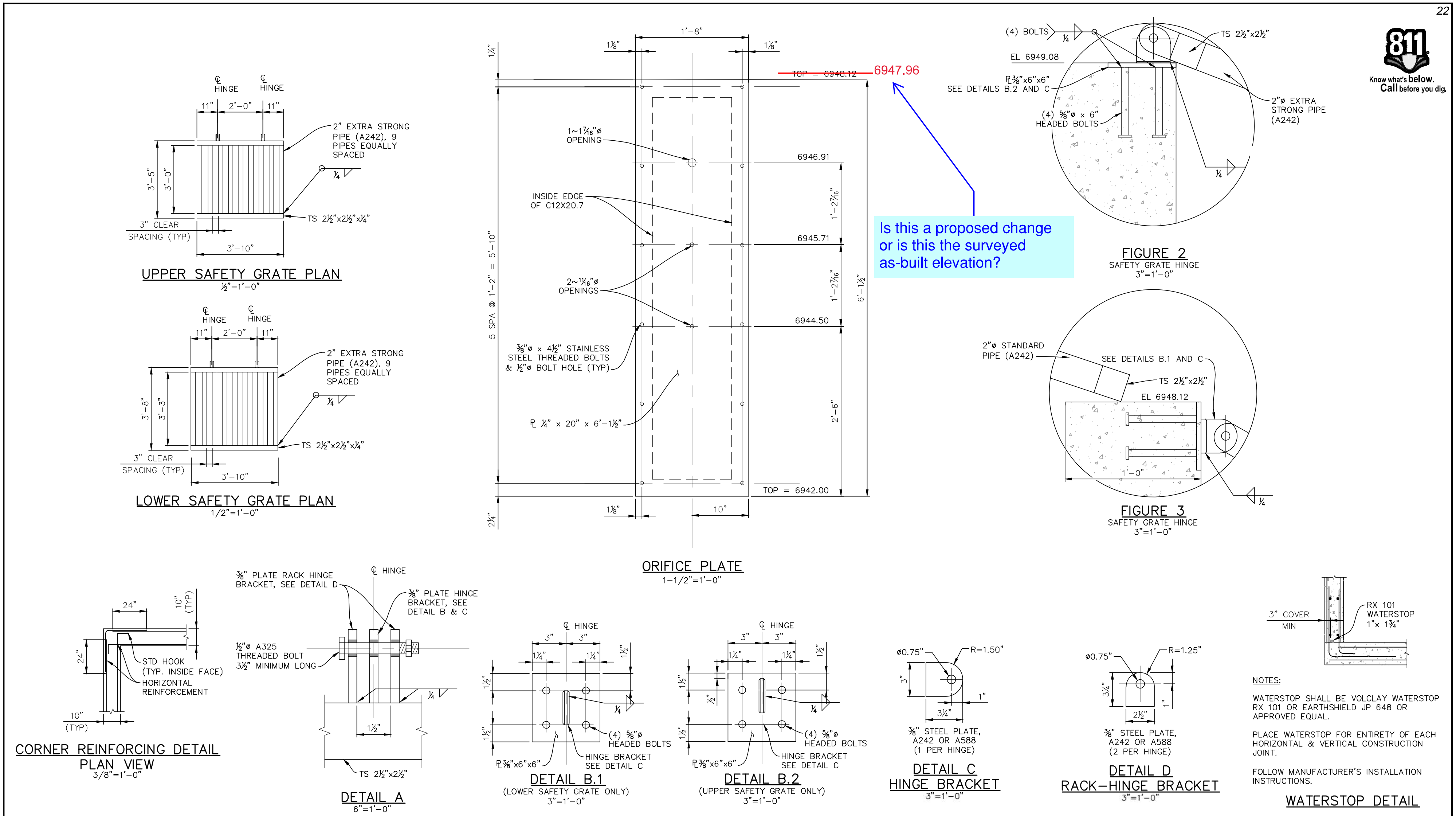
SEAL
24520 8/10/18 PROFESSIONAL ENGINEER
FOR AND ON BEHALF OF MATRIX DESIGN GROUP, INC. PROJECT No. 18.995.001

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.		
CHECKED BY: DRK	VERT.	SHEET 21 OF 29	SDT02



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.

REFERENCE DRAWINGS: X-995-MDG22x34

No.	DATE	DESCRIPTION	BY

REVISIONS

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 10:16 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
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

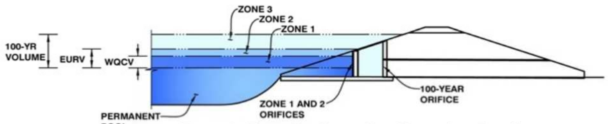
DESIGNED BY: ACR | DRAWN BY: ACR | CHECKED BY: DRK
SCALE: HORIZ. AS SHOWN, VERT. AS SHOWN
DATE ISSUED: AUGUST 2018
SHEET 22 OF 29
DRAWING No. STD03

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	--	--	--	0	0.000		
6,945.00	--	0.50	--	--	--	12	0.000	3	0.000
6,946.00	--	1.50	--	--	--	1,876	0.043	947	0.022
6,947.00	--	2.50	--	--	--	4,873	0.112	4,322	0.099
6,948.00	--	3.50	--	--	--	6,225	0.143	9,871	0.227
6,949.00	--	4.50	--	--	--	7,678	0.176	16,823	0.386
6,950.00	--	5.50	--	--	--	9,232	0.212	25,278	0.580
6,951.00	--	6.50	--	--	--	11,057	0.254	35,423	0.813
6,952.00	--	7.50	--	--	--	12,699	0.292	47,301	1.086
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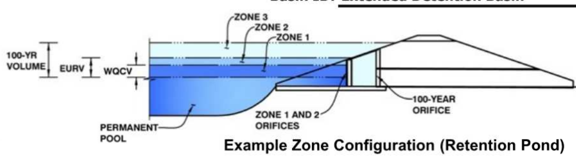
The area does not match the previously approved design. Please address the discrepancy.

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)	4.32	0.262	Orifice Plate
Zone 3 (100-year)	5.22	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)	0.37	0.37	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

The crest length differs from the previously approved design. Please address any discrepancies/differences from the previously approved design and/or constructed pond and revise as necessary.

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=	6.50	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 =	7.40	feet
Basin Area at Top of Freeboard =	0.29	acres
Basin Volume at Top of Freeboard =	1.06	acre-ft

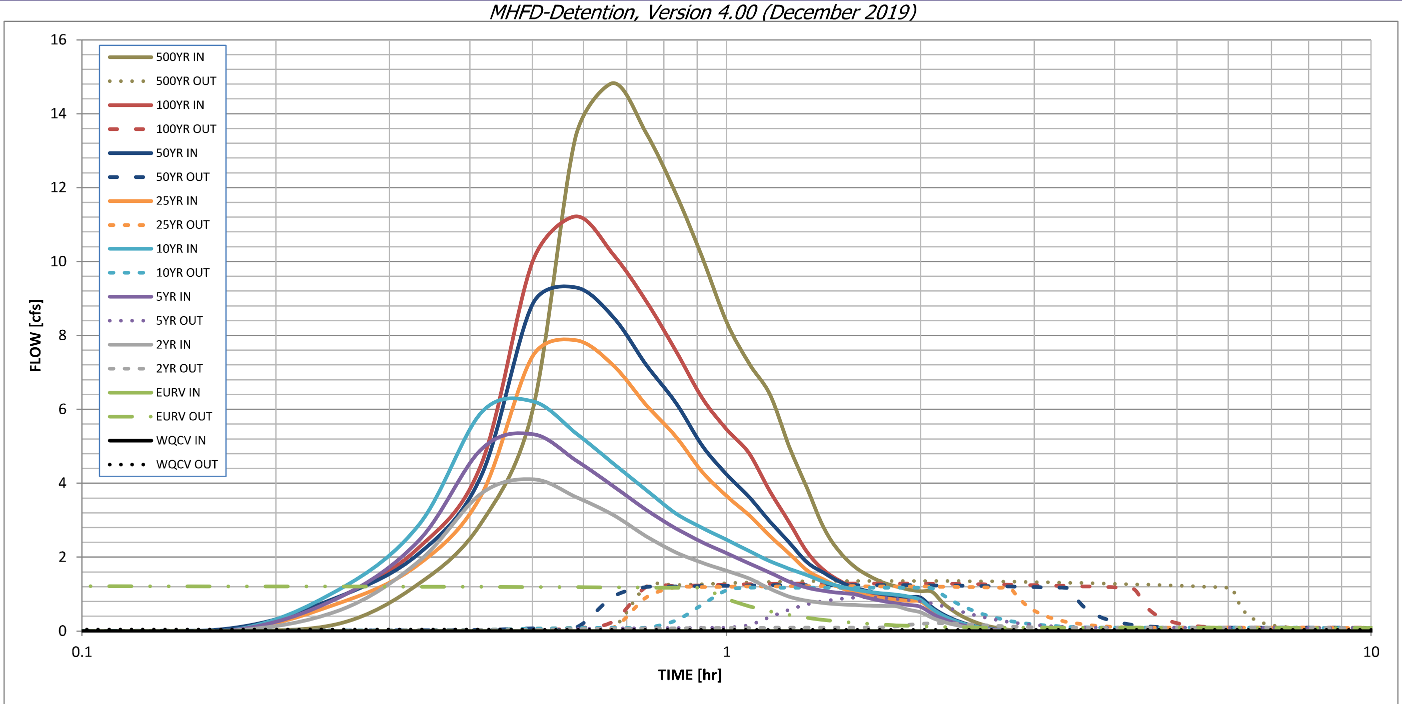
Routed Hydrograph Results

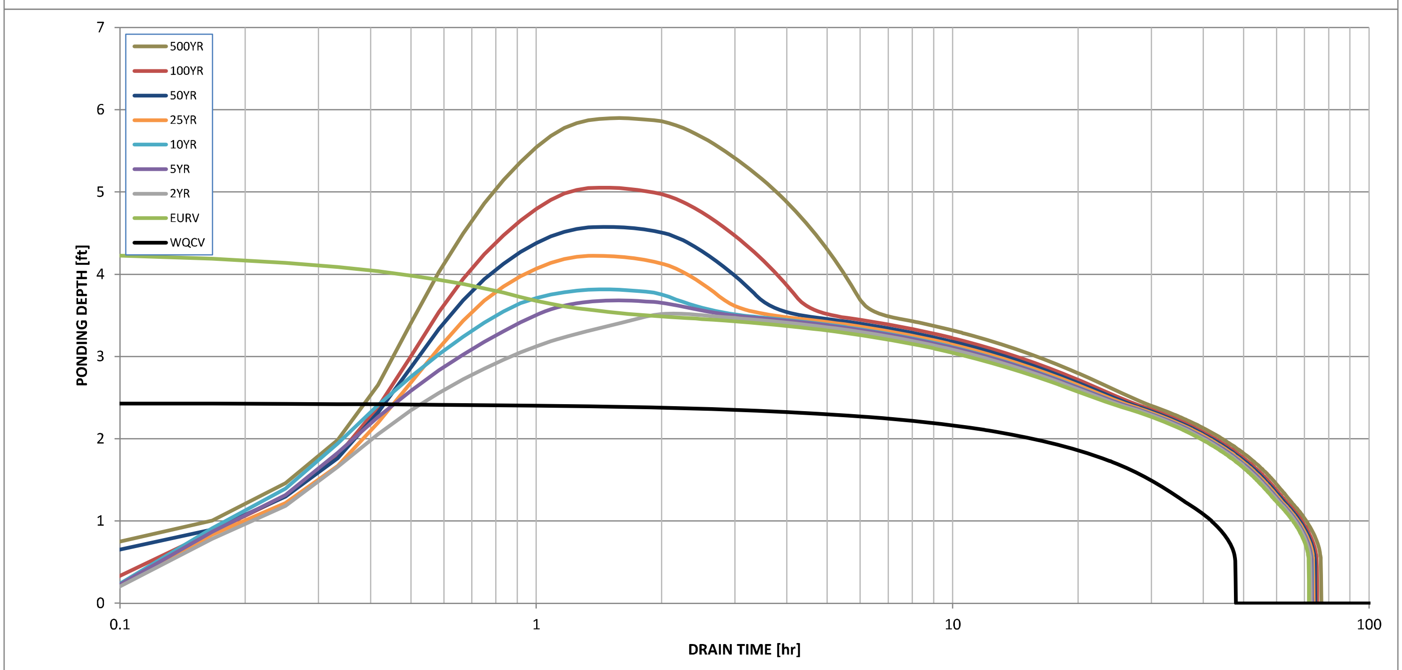
The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

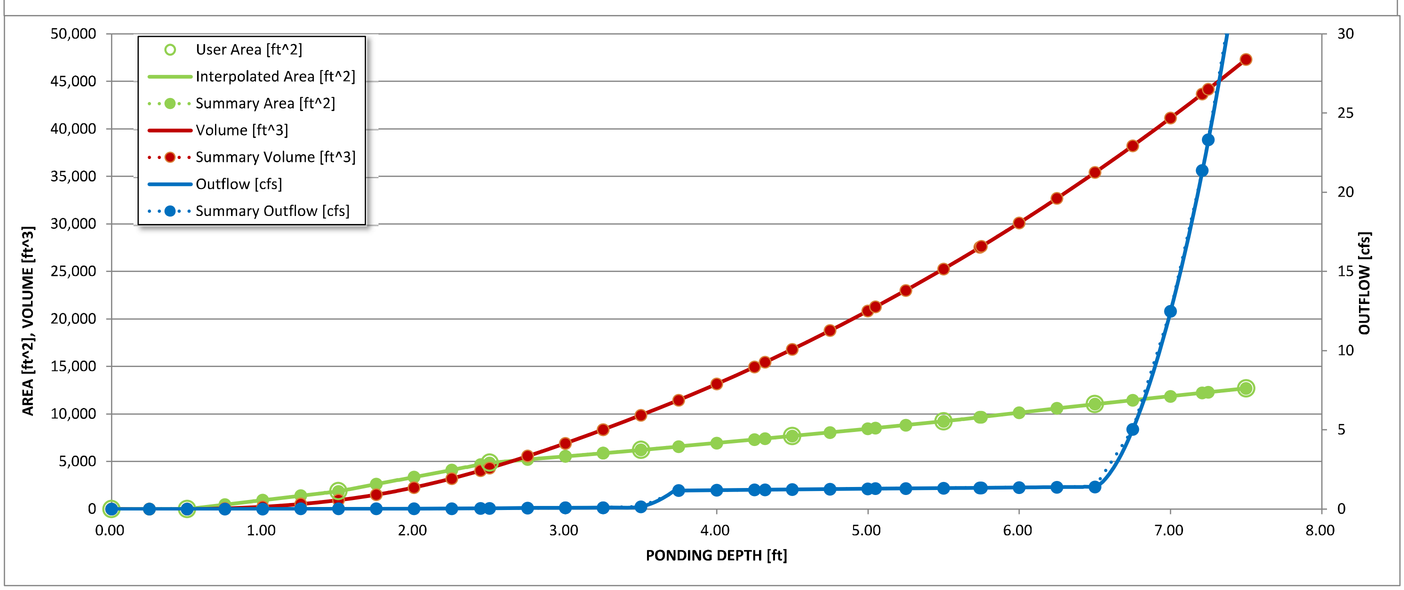
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	N/A	N/A	1.19	1.50	1.75	2.00	2.25	2.52	3.14
CUHP Runoff Volume (acre-ft) =	0.092	0.354	0.248	0.324	0.385	0.463	0.540	0.632	0.835
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.248	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) =	0.04	1.22	0.21	0.92	1.18	1.22	1.25	1.29	1.36
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	15.27	14.28	1.62	0.83	0.52	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	0.01	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) =	45	61	65	63	61	60	59	59	57
Time to Drain 99% of Inflow Volume (hours) =	**47**	68	70	69	69	68	68	68	68
Maximum Ponding Depth (ft) =	2.44	4.32	3.52	3.68	3.82	4.23	4.58	5.05	5.90
Area at Maximum Ponding Depth (acres) =	0.11	0.17	0.14	0.15	0.15	0.17	0.18	0.20	0.23
Maximum Volume Stored (acre-ft) =	0.093	0.355	0.229	0.253	0.273	0.338	0.399	0.489	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	0	0.000	0	0.000	0.00
6944.75	0.25	6	0.000	1	0.000	0.01
6945	0.50	12	0.000	3	0.000	0.01
6945.25	0.75	478	0.011	64	0.001	0.01
6945.5	1.00	944	0.022	242	0.006	0.01
6945.75	1.25	1,410	0.032	536	0.012	0.02
6946	1.50	1,876	0.043	947	0.022	0.02
6946.25	1.75	2,625	0.060	1,510	0.035	0.03
6946.5	2.00	3,375	0.077	2,260	0.052	0.03
6946.75	2.25	4,124	0.095	3,197	0.073	0.03
WQCV=6946.94	2.44	4,693	0.108	4,035	0.093	0.04
6947	2.50	4,873	0.112	4,322	0.099	0.05
6947.25	2.75	5,211	0.120	5,582	0.128	0.07
6947.5	3.00	5,549	0.127	6,927	0.159	0.08
6947.75	3.25	5,887	0.135	8,357	0.192	0.09
6947.5	3.00	5,549	0.127	6,927	0.159	0.08
6947.75	3.25	5,887	0.135	8,357	0.192	0.09
6948	3.50	6,225	0.143	9,871	0.227	0.15
6948.25	3.75	6,589	0.151	11,473	0.263	1.17
6948.5	4.00	6,952	0.160	13,165	0.302	1.19
6948.75	4.25	7,315	0.168	14,949	0.343	1.22
EURV=6948.82	4.32	7,417	0.170	15,464	0.355	1.22
6949	4.50	7,678	0.176	16,823	0.386	1.24
6949.25	4.75	8,067	0.185	18,791	0.431	1.26
6949.5	5.00	8,455	0.194	20,856	0.479	1.28
100 YR=6949.55	5.05	8,533	0.196	21,281	0.489	1.29
6949.75	5.25	8,844	0.203	23,019	0.528	1.31
6950	5.50	9,232	0.212	25,278	0.580	1.33
100 YR Clogged=6950.24	5.74	9,670	0.222	27,546	0.632	1.35
6950.25	5.75	9,689	0.222	27,643	0.635	1.35
6950.5	6.00	10,145	0.233	30,122	0.692	1.37
6950.75	6.25	10,601	0.243	32,716	0.751	1.39
Spillway El.=6951	6.50	11,057	0.254	35,423	0.813	1.41
6951.25	6.75	11,468	0.263	38,239	0.878	5.04
6951.5	7.00	11,878	0.273	41,157	0.945	12.50
Act. Spill. 100yr WSEL=6951.71	7.21	12,223	0.281	43,687	1.003	21.37
6951.75	7.25	12,288	0.282	44,178	1.014	23.33
6952	7.50	12,699	0.292	47,301	1.086	37.55

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:	**April 13, 2020**
Project:	**Liberty Tree Academy**
Location:	**DP A & B Forebay**

1. Basin Storage Volume

A) Effective Imperviousness of Tributary Area, I_a — I_a = 80.4 %

B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) — i = 0.804

C) Contributing Watershed Area — Area = 3.460 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)

EURV should be selected

The watershed area is previously indicated as 4.15 acres. Revise accordingly.

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} = 0.095 ac-ft

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

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

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

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

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

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

3. Basin Side Slopes

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

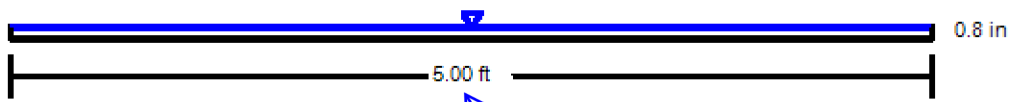
F) Discharge Pipe Size (minimum 8-inches) — Calculated D_P = ___ in

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	0.8 in
Bottom Width	5.00 ft
Discharge	0.40 cfs



A 4 ft. trickle channel is indicated on the CD's. Revise accordingly.

V: 1
H: 1

Spillway Capacity Calculation

Spillway Section

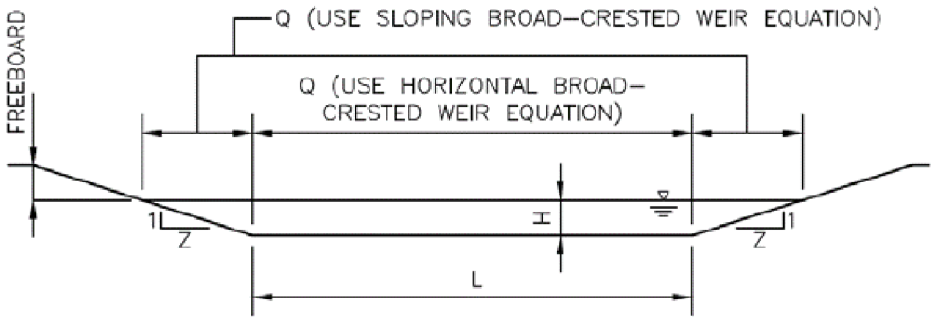


Fig. 12-20 of UDFCD V1

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

See previous comment regarding the crest length. Additionally the next page indicates 10 ft

Total Required Length L=$\frac{Q-\left(\frac{2}{5}\right)C_{BCW}Z_1 Z1H^{2.5}-\left(\frac{2}{5}\right)C_{BCW}Z_2 2H^{2.5}}{CH^{1.5}}$

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

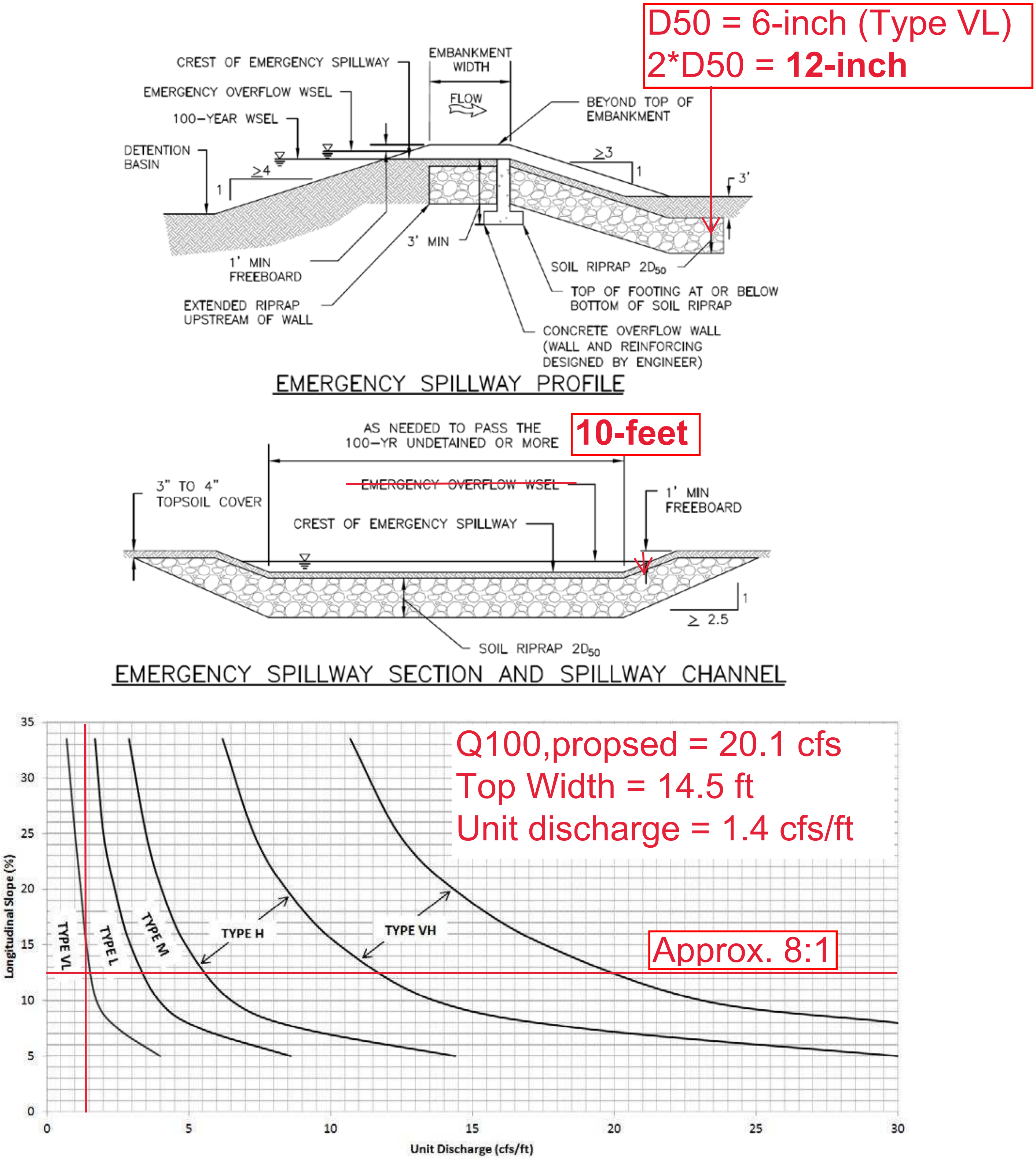
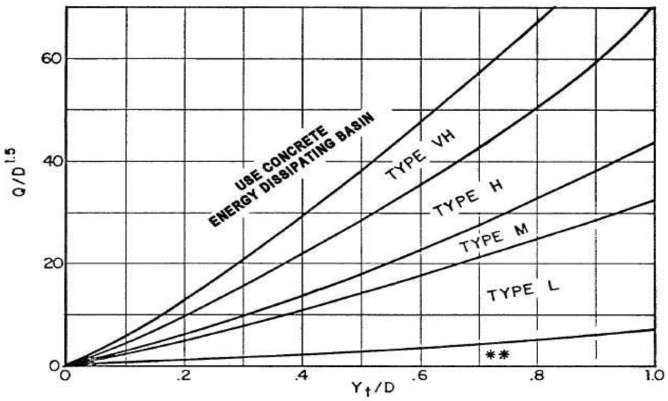


Figure 12-21. Embankment protection details and rock sizing chart (adapted from Arapahoe County)

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 $Q/D_{2.5} \leq 6.0$)

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).

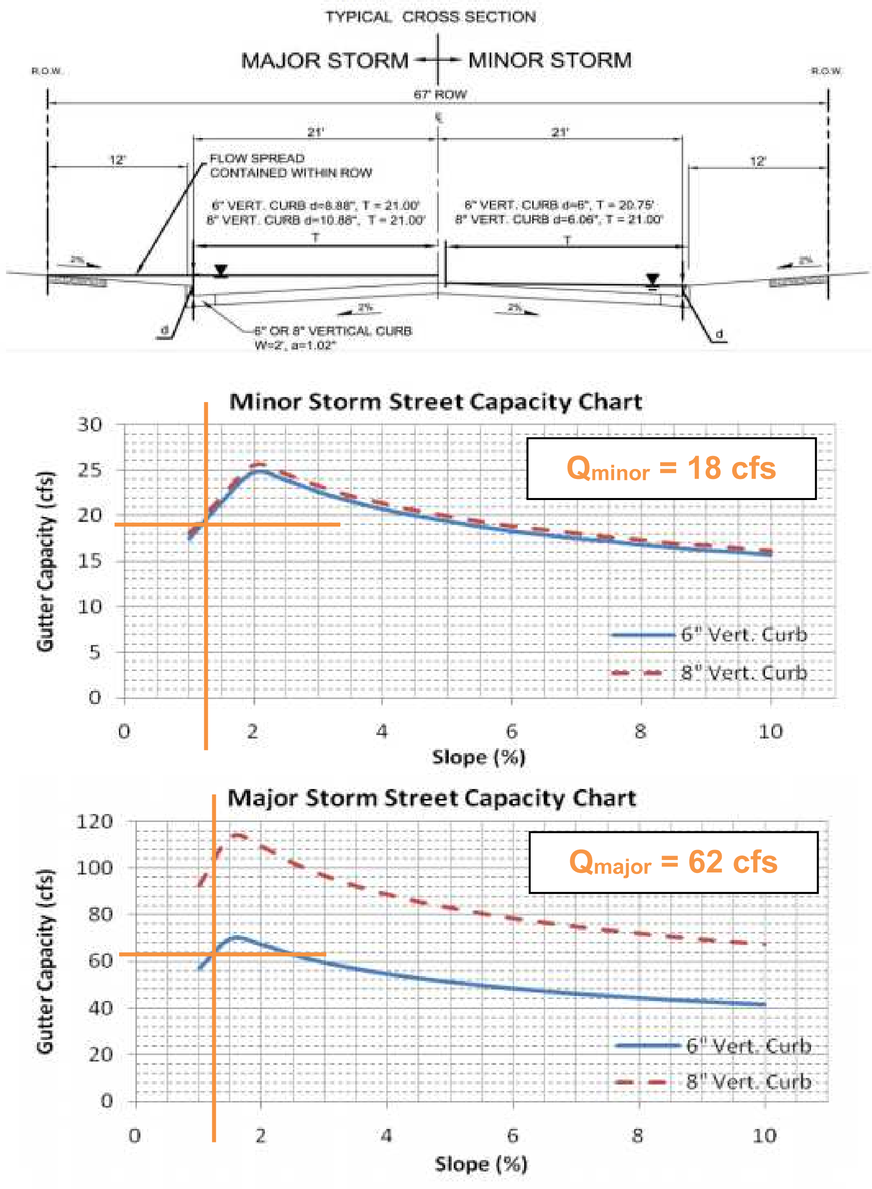
Google Streetview at the upstream end of the gutter line (south side of Bennett Ranch drainageway crossing). Transition from block to 6 inch curb.

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 and 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 C, 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)	**1.7**	From Proposed Conditions, Rational Calculations
Major gutter capacity, Q (cfs)	**62**	Using Figure 7-5 of DCM-V1-Update.
100-year Q (cfs)	**3.7**	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.

Existing or proposed? Note that per the proposed conditions calculations the runoff is 2.11cfs and 4.52cfs for the 5&100 yr flows at DP ER1. Please revise accordingly.

Off-site Drainageway Analysis

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)

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

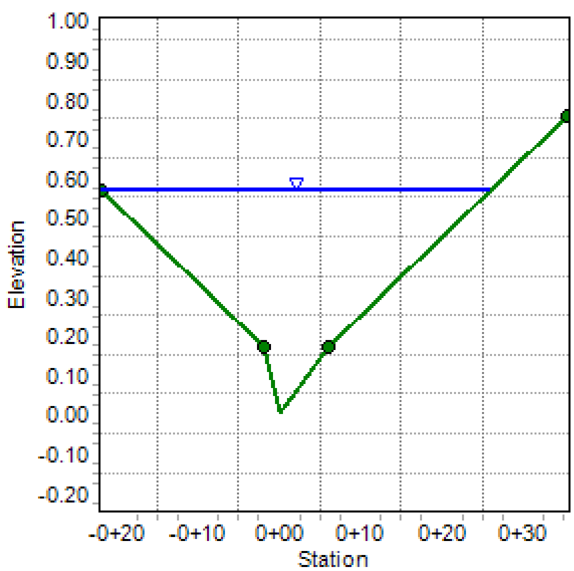
Project Description

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



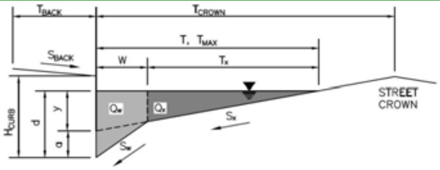
Version 4.05 Released March 2017

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'

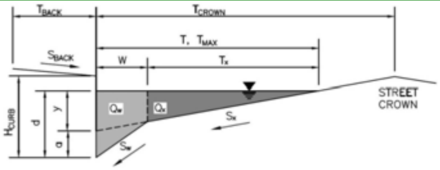
Version 4.05 Released March 2017

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

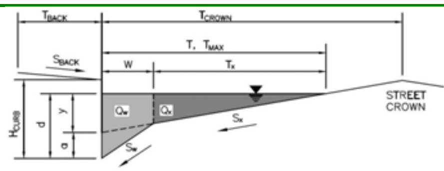
Version 4.05 Released March 2017

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)

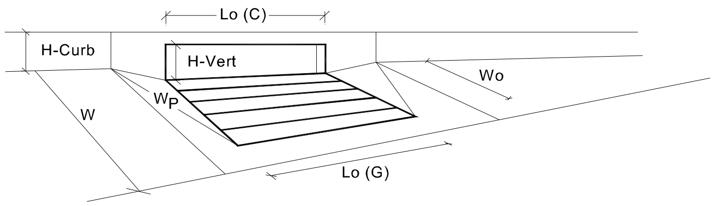
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	6.5	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
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

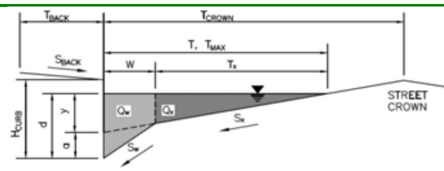
Version 4.05 Released March 2017

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)

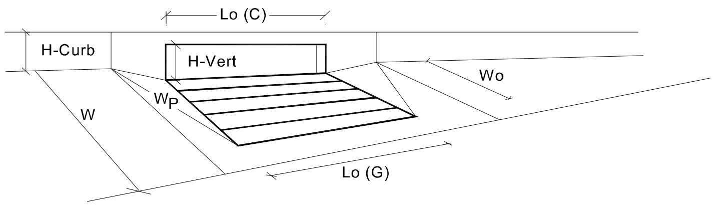
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	6.5	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
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



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 =	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

Worksheet for Storm System - B Basins

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Please show how this flow was determined for basins B1 and B2.

Roughness Coefficient	0.013
Channel Slope	0.005 ft/ft
Diameter	18.0 in
Discharge	4.11 cfs

Results

Normal Depth	9.6 in
Flow Area	1.0 ft²
Wetted Perimeter	2.4 ft
Hydraulic Radius	4.7 in
Top Width	1.50 ft
Critical Depth	9.3 in
Percent Full	53.1 %
Critical Slope	0.005 ft/ft
Velocity	4.31 ft/s
Velocity Head	0.29 ft
Specific Energy	1.09 ft
Froude Number	0.952
Maximum Discharge	7.99 cfs
Discharge Full	7.43 cfs
Slope Full	0.002 ft/ft
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 in
Length	0.0 ft
Number Of Steps	0

GVF Output Data

Upstream Depth	0.0 in
Profile Description	N/A
Profile Headloss	0.00 ft
Average End Depth Over Rise	0.0 %
Normal Depth Over Rise	0.0 %
Downstream Velocity	0.00 ft/s
Upstream Velocity	0.00 ft/s
Normal Depth	9.6 in
Critical Depth	9.3 in
Channel Slope	0.005 ft/ft
Critical Slope	0.005 ft/ft

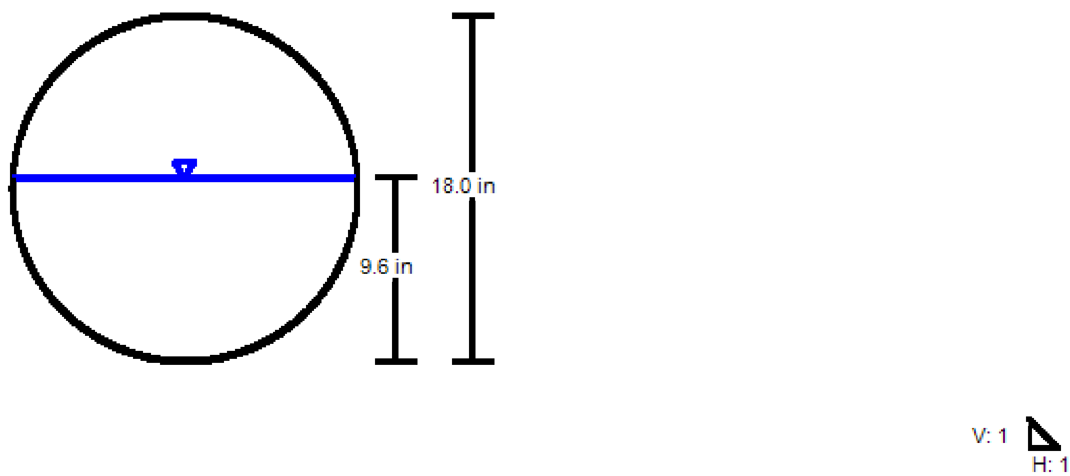
Cross Section for Storm System - B Basins

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013
Channel Slope	0.005 ft/ft
Normal Depth	9.6 in
Diameter	18.0 in
Discharge	4.11 cfs



SOIL RIPRAP SIZING - WILDLIFE CROSSING RUNDOWN

Mile High Flood District		= Calculated Value
Volume I - August 2018		= Manually Entered Value
Chapter 8 - Open Channels		= Referenced from Flowmaster
8.1.2 - Steep Slope Conditions		= 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

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_{50} = Median Stone Size (ft)

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

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

where

S = slope of bed

q = unit discharge

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	ft^3/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

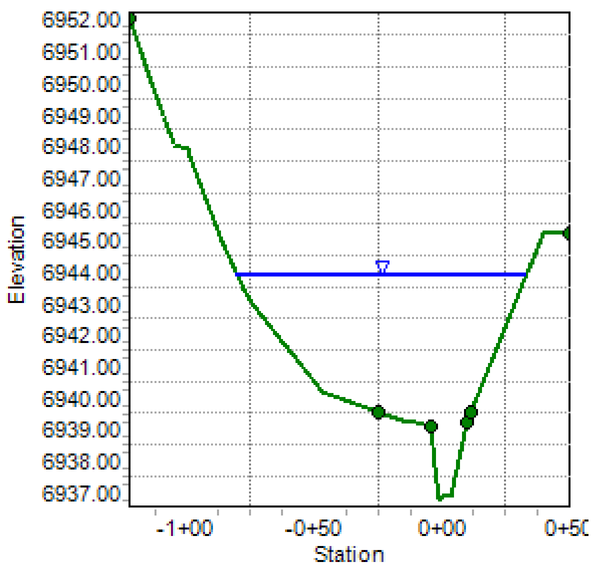
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00250	ft/ft
Normal Depth	7.14	ft
Discharge	810.00	ft^3/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

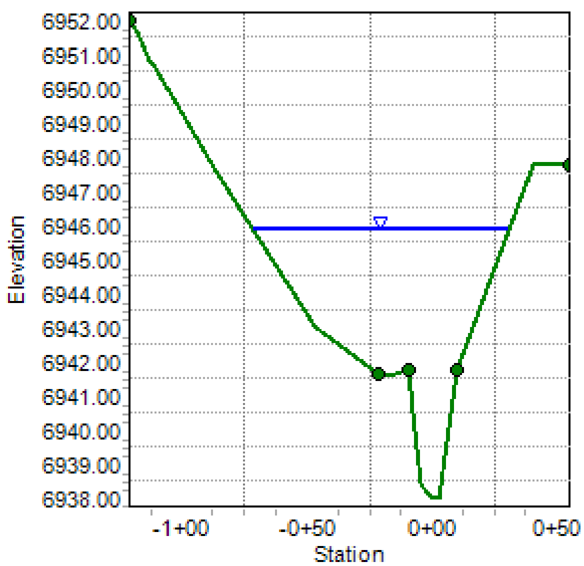
Project Description

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

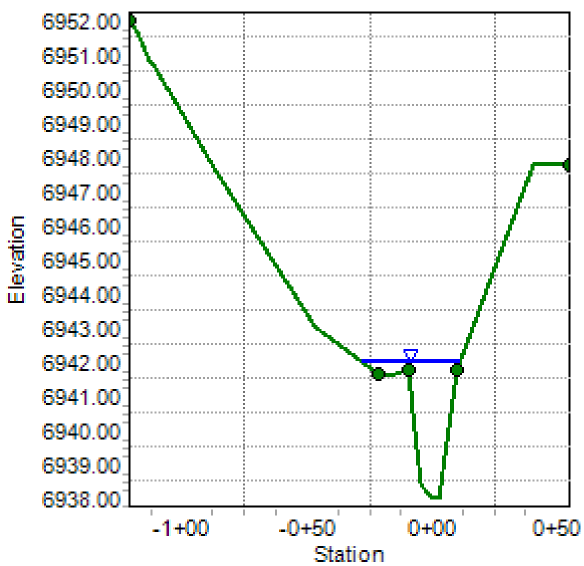
Project Description

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

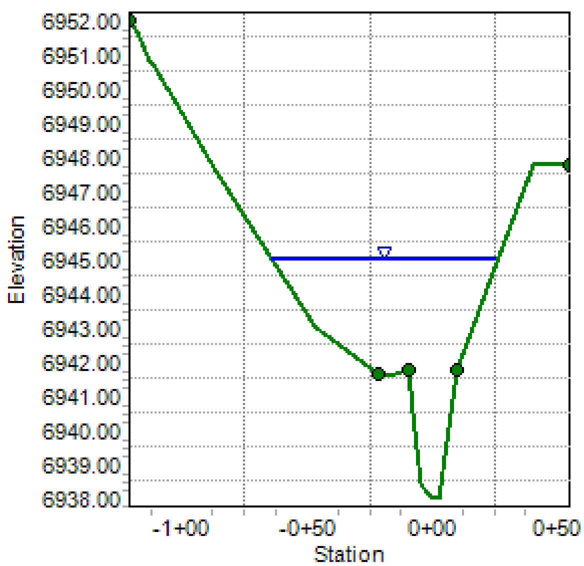
Project Description

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 E – REFERENCED DOCUMENTS

Selected Pages from the Bennett Ranch

Drainage Basin Planning Study

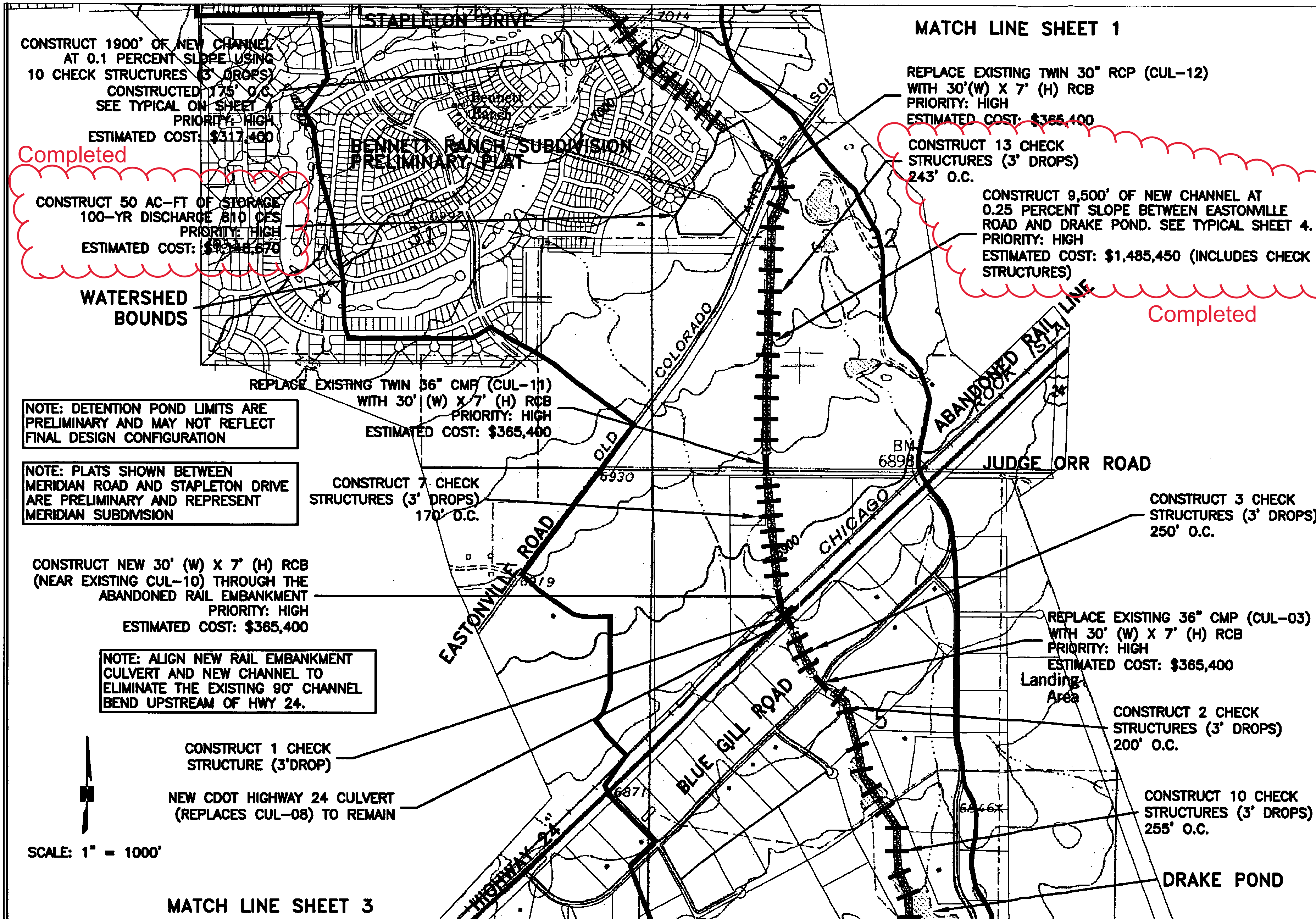


FIGURE 7-1
SHEET 2

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

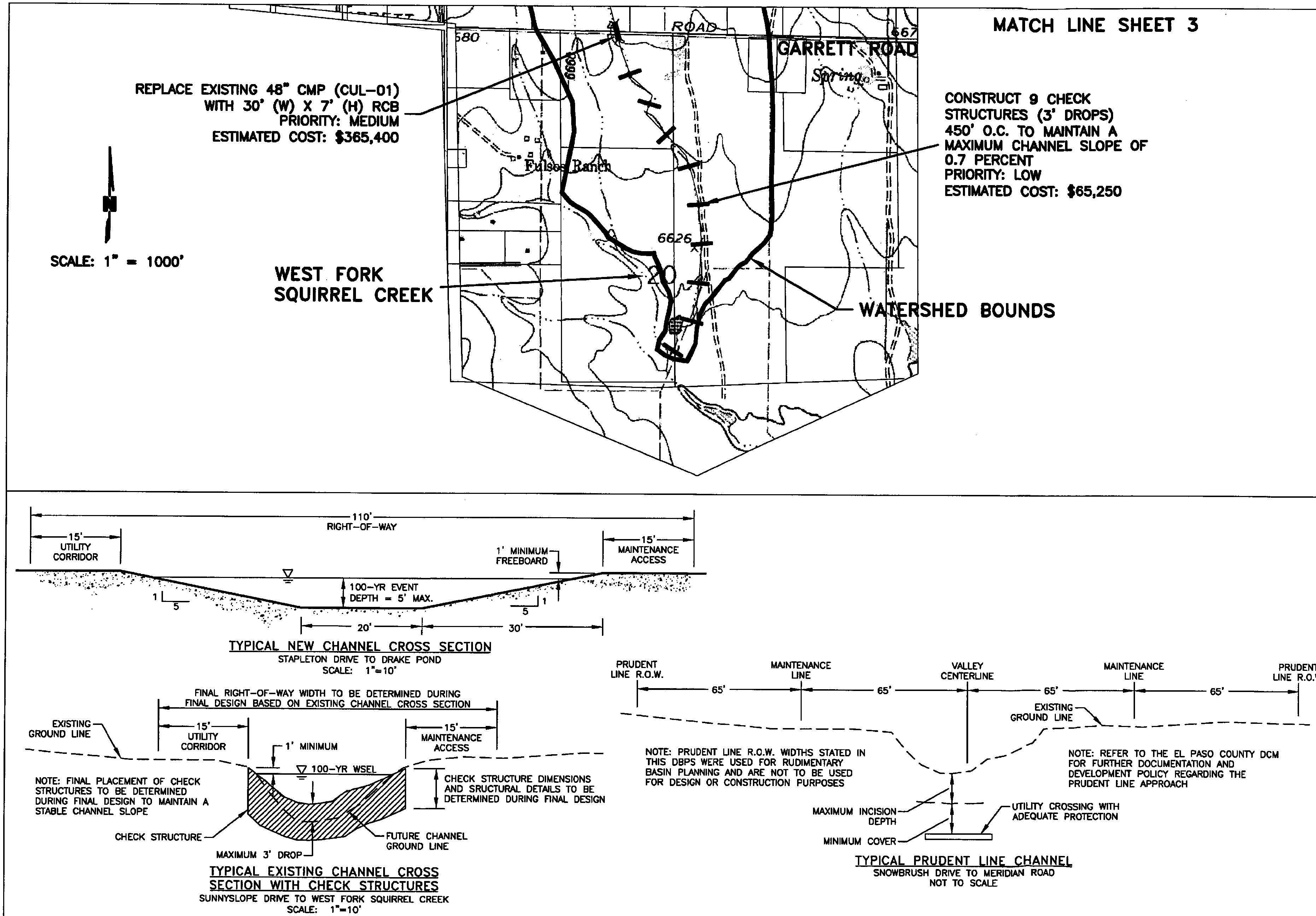


FIGURE 7-1
SHEET 4

EL PASO COUNTY COLORADO SEAL EST. 1861

STORMWATER & ENVIRONMENTAL CONSULTANTS, INC.
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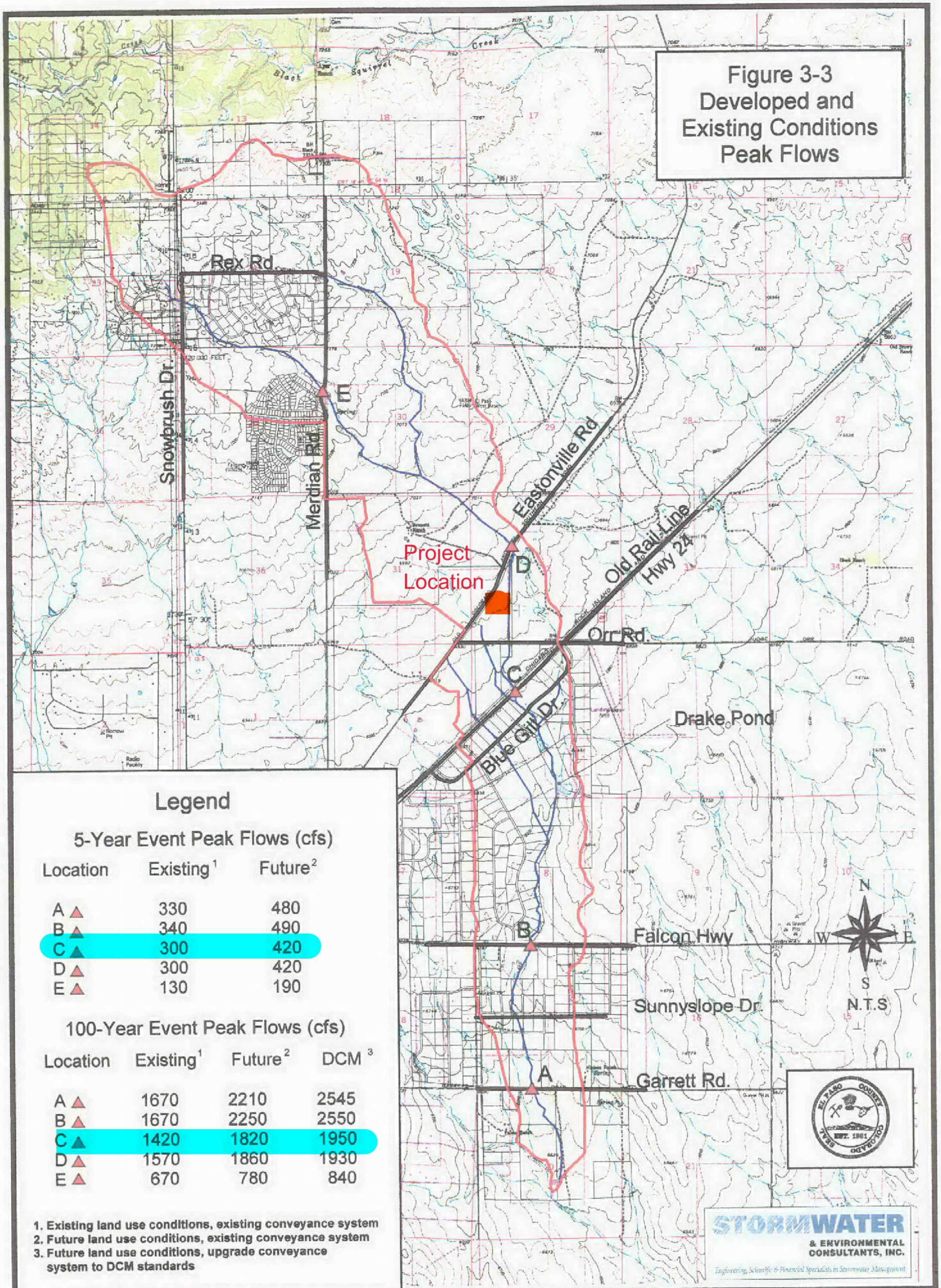
Figure 3-3
Developed and
Existing Conditions
Peak Flows
Rex Rd
Snowbrush Dr.
Meridian Rd
Eastonville Rd
Old Rail Line
Hwy 24
Project
Location
Orr Rd.
Blue Gill Dr
Drake Pond
Falcon Hwy
Sunnyslope Dr.
Garrett Rd.
A
B
C
D
E
N
S
W
E
N.T.S
Legend
5-Year Event Peak Flows (cfs)
Location Existing 1 Future 2
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
STORMWATER
& ENVIRONMENTAL CONSULTANTS, INC.
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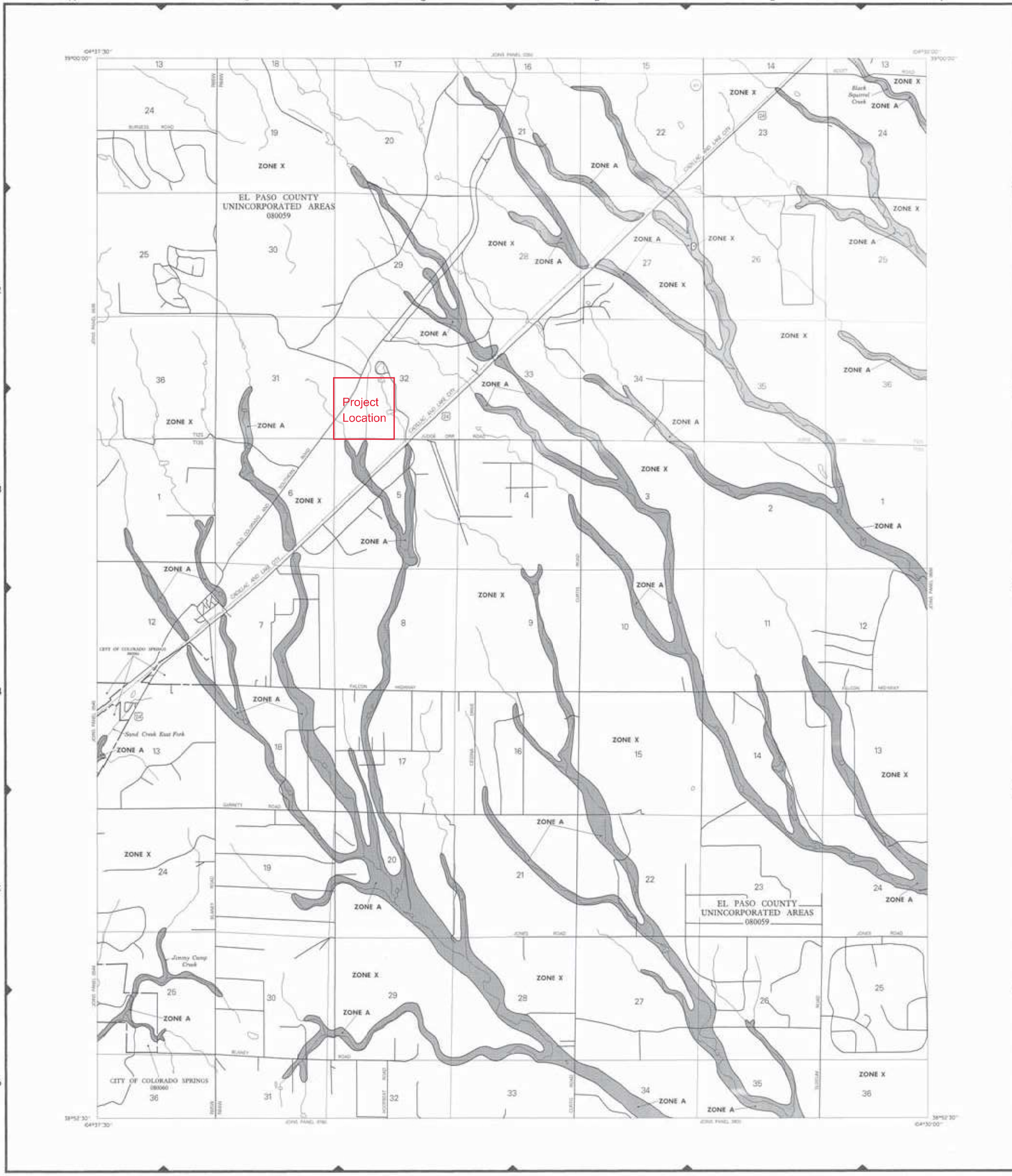
FEMA Flood Maps



LEGEND

NOTES

MAP REPOSITORY
Refer to Repository Listing on Map Index

EFFECTIVE DATE OF
COUNTYWIDE FLOOD INSURANCE RATE MAP
MARCH 17, 1997

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL:

APPROXIMATE SCALE IN FEET

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

EL PASO COUNTY,
COLORADO AND
INCORPORATED AREAS

PANEL 575 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)

MAP NUMBER
08041C0575 F

EFFECTIVE DATE:
MARCH 17, 1997

Federal Emergency Management Agency

Urban Drainage and Flood Control 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.

Off-site Drainageway Analysis

From UDFCD, 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:

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

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.