DRAINAGE LETTER REPORT

for

CATHEDRAL ROCK CHURCH
846 STRUTHERS RANCH ROAD
TRACT A, STRUTHERS RANCH SUBDIVISION FILING NO. 2

Prepared for:

Hammers Construction, Inc.
1411 Woolsey Heights
Colorado Springs, CO 80915

September 20, 2024

Prepared by:

JPS ENGINEERING

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JPS Project No. 082401
PCD Filing No. PPR___

CATHEDRAL ROCK CHURCH
DRAINAGE LETTER REPORT
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DRAINAGE STATEMENT

Engineer's Statement:

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the County for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for liability caused by negligent acts, errors or omissions on my part in preparing this report.

John P. Schwab, P.E. #29891

Developer's Statement:

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

By:

Date

Hammers Construction, Inc.
1411 Woolsey Heights, Colorado Springs, CO 80915

El Paso County's Statement

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

Joshua Palmer, P.E. Date
County Engineer / ECM Administrator

Conditions:

I. INTRODUCTION

A. Property Location and Description

Cathedral Rock Church is planning to construct a new church building on the vacant 5.1-acre property at the northeast corner of Struthers Road and Struthers Ranch Road in northern El Paso County, Colorado. The property is described as Tract A, Struthers Ranch Subdivision Filing No. 2 (El Paso County Assessor's Parcel Number 71363-01-013).

The project consists of a new 8,125 square-foot Church Building with associated parking and site improvements. Future phases of site development are anticipated to include a 3,250 square-foot building addition on the east side of the Phase 1 building, an additional future 10,000-square foot building, and expanded parking areas. Additionally, the Church plans to process a minor subdivision to create a separate 1-acre lot reserved for future development in the southwest corner of the site.

The property is bounded by Struthers Road on the southwest side and Struthers Ranch Road on the southeast side. Struthers Road is a fully improved, asphalt-paved arterial public street, and Struthers Ranch Road is a fully improved local public street. Existing platted residential lots are located along the northeast boundary of the parcel (Struthers Ranch Filing No. 2). The north boundary of the site adjoins a vacant, unplatted 6.5-acre property (zoned R-4).

The property is zoned Planned Unit Development (PUD), and the proposed site development is fully consistent with the existing zoning of the site. Access to the site will be provided by the existing private driveway connection to Struthers Ranch Road along the southeast boundary of the site.

The site is located in the Black Forest Creek Drainage Basin, and surface drainage from this site sheet flows southwesterly to an existing public storm sewer system in Struthers Ranch Road, flowing to the existing Struthers Ranch stormwater detention pond on the west side of Struthers Road.

This report is intended to meet the requirements of a site-specific "Letter Type" drainage report in accordance with El Paso County subdivision drainage criteria.

B. References

JPS Engineering, Inc., "Preliminary & Final Drainage Report for Cathedral Rock Commons Commercial," revised March 8, 2023 (approved by El Paso County 3/29/23).

JPS Engineering, Inc., "Final Drainage Report for Struthers Ranch Filing No. 2," revised October 14, 2004 (approved by El Paso County 10/20/04).

JPS Engineering, Inc., "Drainage Letter Report for Struthers Ranch Polaris, Lots 1-2, Struthers Ranch Subdivision Filing No. 4," revised April 7, 2023 (approved by El Paso County 5/4/23).

C. Drainage Analysis Methods and Criteria

ITEM	DESCRIPTION	REFERENCE
Design Storm (initial/major)	5-year/100-year	CS/EPC DCM
Storm Runoff	Rational Method (Area<100acres)	CS/EPC DCM
Major Drainage Basin	Black Forest Creek	
Floodplain Impacts	Parcel is located outside any delineated FEMA floodplains	FIRM
Existing Downstream Facilities	Existing storm sewer system on east side of Struthers Road; Existing detention pond on west side of Struthers Road	

CS/EPC DCM = City of Colorado Springs & El Paso County Drainage Criteria Manual

II. EXISTING / PROPOSED DRAINAGE CONDITIONS

Subdivision Drainage Report

Drainage planning for this site was previously master planned during original development of the Struthers Ranch Subdivision, as detailed in the "Final Drainage Report (FDR) for Struthers Ranch Filing No. 2" by JPS Engineering, dated October 14, 2004 (see excerpts in Appendix A). The project area at the northeast corner of Struthers Road and Struthers Ranch Road was identified as a future commercial development area in the original planning of the subdivision.

According to the original FDR, Basins C (4.75 acres) and E1 (1.5 acres) comprise the future commercial development areas on the north side of Struthers Ranch Road. The previously approved subdivision drainage planning assumed full commercial development within all of Basins C and E1, with runoff coefficients of $C_5 = 0.90$ and $C_{100} = 0.90$, and impervious areas of 95 percent for the entirety of these basins. According to the Rational Method calculations in the original subdivision drainage report, developed peak flows from Basin C were calculated as $Q_5 = 22.2$ cfs and $Q_{100} = 38.5$ cfs, and peak flows from Basins OE1 and E1 (FDR DP#5) were calculated as $Q_5 = 4.6$ cfs and $Q_{100} = 8.9$ cfs (see Appendix A).

As shown on the enclosed Struthers Ranch Subdivision Drainage Plan (Figure D1, Appendix F), the proposed Church building and parking areas lie entirely within Basin C as delineated in the approved "Final Drainage Report for Struthers Ranch Filing No. 2."

The site slopes downward to the southwest, with average grades of 1-4 percent. On-site soils are classified by SCS as type 71, "Pring" series coarse sandy loam soils. These soils have moderately rapid permeability and slow to medium surface runoff characteristics. The soils are classified as hydrologic soils group B.

Developed drainage from this Church site will sheet flow southwesterly to the existing 42" public storm sewer system in Struthers Ranch Road. An existing 10-foot Type R public storm inlet collects street drainage at the northeast corner of Struthers Ranch Road and Struthers Road, and an existing 24" RCP storm sewer was stubbed north from the inlet during initial subdivision development. The existing 42" storm sewer in Struthers Ranch Road flows south along the east side of Struthers Road to a catch basin, where double 48-inch culverts convey developed flows across Struthers Road and into the existing detention pond. The previously approved drainage report for Struthers Ranch Filing No. 2 assumed full commercial development for this basin, which is consistent with the proposed site development. The existing detention pond was sized to account for fully developed flows from this commercial area.

The impervious area for the proposed Cathedral Rock Church development (delineated as Basins A1-A4 within this report, which correlates with Basin C in the FDR) amounts to approximately 65.8 percent of the site (as tabulated on Sh. D1.1 and Appendix B), which is well below the impervious area of 95 percent assumed for full commercial development in the previously approved subdivision drainage report (see Appendix A).

Based on the previous construction of drainage improvements for the Struthers Ranch Subdivision, no significant impact on downstream drainage facilities is anticipated from this site development and replat. Proper erosion control measures will be required for development of the site, including silt fence along downstream property boundaries to minimize off-site transport of construction sediment.

Existing Drainage Conditions

As shown on the enclosed Existing Conditions Drainage Plan (Figure EX1, Appendix F), the site has been delineated as two on-site drainage basins. The majority of the project area has been delineated as Basin A, and the north edge of the site has been delineated as Basin B. The site is impacted by small off-site basin areas (delineated as Basins OA1 and OB1) consisting of the rear sides of the adjoining single-family residential lots (platted as part of Struthers Ranch Filing No 2) along the northeast boundary of the site.

Surface drainage from off-site Basin OA1 (back sides of adjoining developed single-family residential lots along northeast boundary of project site) sheet flows into Basin A, and Basin A sheet flows southwesterly across the property to the existing public storm inlet (10' Type R) on the north side of Struthers Ranch Road. Flows from Basin OA1 combine with Basin A at Design Point #1, with existing peak flows calculated as $Q_5 = 1.4$ cfs and $Q_{100} = 8.5$ cfs.

Drainage from off-site Basin OB1 (back sides of adjoining developed single-family residential lots along northeast boundary of project site) sheet flows southwesterly into Basin B, and Basin B flows southwesterly to the existing curb and gutter along the east side of Struthers Road, ultimately flowing north into the existing public culvert crossing Struthers Road at the southeast corner of Spanish Bit Drive and Struthers Road.

Flows from Basins OB1 and B combine at Design Point #2, with existing peak flows calculated as $Q_5 = 0.7$ cfs and $Q_{100} = 4.1$ cfs.

Developed Drainage Plan

Developed flows have been calculated based on the impervious areas associated with the proposed building and parking improvements. Surface drainage swales and a private storm sewer system will convey developed flows to the proposed Rain Garden A along the south boundary of the site. Site grades will slope to storm inlets and curb openings at selected locations, collecting surface drainage and conveying stormwater to Rain Garden A. The proposed building pads will be graded with protective slopes to provide positive drainage away from the buildings, and the curb, gutter, drainage swales, and private storm sewer system will convey developed flows southwesterly into Rain Garden A.

Basin A

The proposed Church building and the majority of the central parking lot area have been delineated as Basin A1 (2.29-acres), which drains by sheet flow and curb and gutter to a private storm sewer system conveying flows to Rain Garden A. Private Storm Inlet A1.1 (5' Type R) will intercept surface drainage from the northwest side of the parking lot, and Private Storm Sewer A1 (18" HDPE) will convey this flow southeast to Private Storm Inlet A1.2. Storm Inlet A1.2 will intercept surface drainage from the south side of the parking lot, and Private Storm Sewer A1.2 (18" HDPE) will convey the combined flows southeasterly to Private Storm Inlet A2 in the southeast access drive.

Developed peak flows for Basin A1 are calculated as $Q_5 = 7.9$ cfs and $Q_{100} = 15.6$ cfs. Off-site flows from Basin OA1 combine with Basin A1 at Design Point A1.1, with developed peak flows calculated as $Q_5 = 5.6$ cfs and $Q_{100} = 11.2$ cfs.

The southeast access drive and southeast corner of the parking lot have been delineated as Basin A2 (0.86-acre), which drains southwesterly by sheet flow and curb and gutter to the proposed Private Storm Inlet A2 (5' Type R) at the southwest corner of the new access drive. Private Storm Sewer A2 (18" HDPE) will convey the combined flows southwesterly into the forebay at the east end of Rain Garden A.

Developed peak flows for Basin A2 are calculated as $Q_5 = 2.6$ cfs and $Q_{100} = 5.3$ cfs. Off-site flows from Basin OA2 combine with Basin A2 at Design Point A2.1, with developed peak flows calculated as $Q_5 = 2.3$ cfs and $Q_{100} = 5.2$ cfs. Off-site flows from Basins OA1-OA2 combine with Basins A1-A2 at Design Point A2.2, with developed peak flows calculated as $Q_5 = 8.1$ cfs and $Q_{100} = 16.7$ cfs.

The future development area in the southwest corner of the property has been delineated as Basin A3 (0.96-acre). The Church has plans to process a subdivision to create a separate 1-acre lot in this area for potential future sale and commercial development. Runoff calculations for Basin A3 have assumed an impervious area of 85 percent for future commercial development of this area, with Basin A3 developed peak flows calculated as $Q_5 = 3.6$ cfs and $Q_{100} = 6.7$ cfs.

The proposed Rain Garden A area along the south boundary of the property has been delineated as Basin A4 (0.23-acre), and developed peak flows for Basin A4 are calculated as $Q_5 = 0.1$ cfs and $Q_{100} = 0.7$ cfs.

The 24" RCP discharge pipe from Rain Garden A (along with overflows from the pond spillway) will drain into the existing public storm inlet along the north side of Struthers Ranch Road, flowing into the existing 42-inch RCP public storm sewer in Struthers Ranch Road. The existing public storm sewer system flows south to the existing double 48-inch RCP storm sewer which crosses Struthers Road, draining southwesterly into the existing regional Struthers Ranch Detention Pond ("Detention Pond 11" per Black Forest Creek DBPS).

<u>Combined Flows and Comparison to Subdivision FDR</u>

Developed flows from Basins OA1-OA2, and A1-A4 combine at Design Point #1, with peak flows calculated as $Q_5 = 10.6$ cfs and $Q_{100} = 21.8$ cfs. For comparison with the original Struthers Ranch Subdivision FDR, the developed flows from FDR Basin C (equivalent to Design Point #1 in this report) were calculated as $Q_5 = 22.2$ cfs and $Q_{100} = 38.5$ cfs (significantly higher than the current developed flow calculations). As such, the proposed developed flows are well below the previously master planned developed flows entering the regional detention pond.

Hydrologic and hydraulic calculations for the site are detailed in the appendices (Appendix B and C), and peak flows are identified on Figure D1.1 (Appendix F).

<u>Basin B</u>

The proposed site development plan will minimize developed drainage impacts within Basin B along the north boundary of the site, as developed flows from the church building and parking areas will be conveyed southwesterly to Rain Garden A. Developed peak flows for Basin B are calculated as $Q_5 = 0.3$ cfs and $Q_{100} = 2.3$ cfs. Developed flows from Basins OB1 and B will continue to combine at Design Point #2, with peak flows calculated as $Q_5 = 0.4$ cfs and $Q_{100} = 1.9$ cfs (lower than existing conditions).

III. DRAINAGE PLANNING FOUR STEP PROCESS

El Paso County Drainage Criteria require drainage planning to include a Four Step Process for receiving water protection that focuses on reducing runoff volumes, treating the water quality capture volume (WQCV), stabilizing drainageways, and implementing long-term source controls.

As stated in ECM Appendix I.7., the Four Step Process is applicable to all new and re-development projects with construction activities that disturb 1 acre or greater or that disturb less than 1 acre but are part of a larger common plan of development. The Four Step Process has been implemented as follows in the planning of this project:

Step 1: Employ Runoff Reduction Practices
- Rain Garden: The majority of developed flows will be routed through the on-site Rain Garden water quality facility, which will be vegetated to encourage stormwater infiltration.

Step 2: Stabilize Drainageways
- There are no drainageways directly adjacent to this project site. Implementation of the on-site drainage improvements and Rain Garden will minimize downstream drainage impacts from this site.
- Drainage basin fees were previously paid during recording of the subdivision plat, and these fees provided the applicable cost contribution towards regional drainage improvements.

Step 3: Provide Water Quality Capture Volume (WQCV)
- RG: The majority of the developed site will drain through an on-site Private Rain Garden (RG) along the south boundary of the property. The Rain Garden will capture and slowly release the WQCV over an extended release period.

Step 4: Consider Need for Industrial and Commercial BMPs
- No industrial uses are proposed for this site.
- The property owner will implement a Stormwater Management Plan including proper housekeeping practices and spill containment procedures.
- On-site developed drainage will be routed through the Rain Garden to minimize introduction of contaminants to the County's public drainage system.

IV. FLOODPLAIN IMPACTS

According to the FEMA floodplain map for this area, El Paso County FIRM Panel No. 08041C0287G, dated December 7, 2018, the site is located beyond the limits of any delineated floodplains.

V. STORMWATER DETENTION AND WATER QUALITY

Stormwater detention for this site is provided in the existing regional stormwater detention pond constructed during initial development of the Struthers Ranch Subdivision. The Struthers Ranch Homeowners Association is the owner of the existing Struthers Ranch Detention Pond located within Tract C, Struthers Ranch Filing No. 2. There currently appears to be a need for removal of excess vegetation within the pond to ensure proper operation of the detention facilities. The developer will need to coordinate with the HOA to ensure that the required maintenance is performed on the existing regional detention pond.

An on-site private Rain Garden will be constructed to meet stormwater quality requirements for this site in accordance with current El Paso County drainage criteria. As detailed in the Rain Garden calculations in Appendix D, the required Water Quality Capture Volume (WQCV) has been calculated as 0.13 acre-feet. The water quality capture volume has been calculated based on the actual impervious area of the proposed church site development within Basins A1-A2, along with the typical single-family residential impervious area of 40% within the adjoining developed Basins OA1-OA2, and a conservative estimated impervious area of 85% for the anticipated future commercial development within Basin A3. Water quality calculations have also accounted for future building improvements and future parking expansion areas within Basins A1-A2 as noted on the Developed Drainage Plan.

The proposed Rain Garden has been designed utilizing the Denver Mile High Flood District's "UD-BMP_v3.07" software package. Calculations and details for the proposed Rain Garden are enclosed in Appendix D, and design parameters for the Rain Garden are summarized as follows:

Water Quality Facility (RG)	Tributary Drainage Basins	Tributary Area (ac)	Impervious Percentage	Min. WQCV (cf)	Design Volume (af)
A	A1-A4	4.79	65.8	3,579	3,817

The proposed on-site Rain Garden A provides a storage volume of 3,817 cubic feet, which meets the required WQCV volume.

The proposed Rain Garden will include a concrete forebay for erosion control at the entry. The outlet structure has been designed with a water quality orifice plate to maintain a 40-hour release of the WQCV. The Rain Garden will have a vegetated bottom to encourage infiltration of stormwater prior to discharging into the downstream public drainage system.

The new on-site Rain Garden will be privately owned and maintained by the property owner, and maintenance access will be provided from the access drive at the southeast corner of the site.

VI. PUBLIC IMPROVEMENTS / DRAINAGE BASIN FEES

No public drainage improvements are required or proposed for this project. As detailed in Appendix E, the proposed private Rain Garden A has an estimated cost of approximately $19,267.

The site lies completely within the Black Forest Creek Drainage Basin. Applicable drainage basin fees were paid at the time of original platting of Struthers Ranch Filing No. 2, so no drainage basin fees or bridge fees are applicable at this time.

VII. SUMMARY

The developed drainage patterns for the proposed Cathedral Rock Church site development on Tract A, Struthers Ranch Filing No. 2 will remain consistent with the established drainage plan for this subdivision. The grading and drainage plan for the proposed church site development fully conforms to the approved drainage plan for Struthers Ranch Filing No. 2.

Developed flows from the site will drain through a Private Rain Garden water quality facility along the south boundary of the property prior to discharging into the existing downstream public storm sewer system. Stormwater detention is provided by the existing Struthers Ranch Detention Pond which was designed to accept fully developed flows from the commercial area encompassing this site. The proposed on-site Rain Garden will meet current stormwater quality requirements for this site. Construction and proper maintenance of the on-site drainage facilities and Rain Garden, in conjunction with proper erosion control practices, will ensure that this developed site has no significant adverse drainage impact on downstream or surrounding areas.

APPENDIX A

EXCERPTS FROM SUBDIVISION DRAINAGE REPORT

FINAL DRAINAGE REPORT

for

STRUTHERS RANCH FILING NO. 2

Prepared for:

WL Homes LLC
8610 Explorer Drive, Suite 300
Colorado Springs, CO 80920

November 6, 2003
Revised April 12, 2004
Revised May 7, 2004
Revised May 25, 2004
Revised September 3, 2004
Revised October 14, 2004

Prepared by:

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2. Developed Drainage Conditions

The developed drainage basins and projected flows are shown in Figure D1, and preliminary hydrologic calculations are enclosed in Appendix B. The developed site has been divided into five major basins (A-E) and five design points (DP1-DP5), as shown on the enclosed Drainage Plan (Sheets D1 and D1.02). Hydrologic flow schematics and calculations are enclosed in Appendix B.

Struthers Ranch Filing No. 2 is located within parts of Basins C-F at the northwest corner of the site. The majority of developed areas ultimately flow to the proposed detention pond at Design Point No. 4. The internal road gutters of sub-basins D1-D10 will be graded to drain southwesterly through the interior road system. Storm inlets will be constructed in the interior roads as required to intercept developed flows exceeding the allowable street capacity. Storm sewer outfalls will be extended to the proposed detention pond.

To minimize the impacts of developed drainage from Struthers Ranch, flows from Basins C, D, and F will be routed through the proposed detention pond. Off-site Basins OC1 and OD1 will combine with flows from on-site Sub-basins D1-D10, C, E2, E3, and F at the proposed detention pond (Design Point #4), with developed flows of Q_5 = 66 cfs and Q_{100} = 191 cfs (SCS Method). The detention pond will discharge historic flows to the existing swale at the southerly site boundary, flowing into the existing 48-inch culvert crossing I-25. The proposed 48-inch RCP discharge pipe from the detention pond will be released to a riprap apron, flowing to an existing stable grass-lined swale across a parcel owned by the U.S Air Force Academy, ultimately crossing I-25 through the existing 48-inch CMP culvert.

The proposed site layout will significantly reduce the amount of developed flow reaching the existing 3.5'x2' culvert (Structure #11) at the westerly site boundary (Design Point #3). Flows from Sub-basin E4 (Q_5 = 1.9 cfs and Q_{100} = 3.7 cfs) represent the westerly side of the proposed Struthers Road draining to the existing culvert crossing I-25.

Basin E1 represents the small developed area at the northwest corner of the site, draining to the existing 4'x4' box culvert at Design Point #5. The proposed grading scheme for the commercial area north of Struthers Ranch Road will direct the majority of developed flows into Basin C, ultimately flowing to the proposed detention pond. As a result, developed flow impacts to the Jackson Creek Basin at the northwest corner of the site will be minimized. Estimated developed peak flows of Q_5 = 4.6 cfs and Q_{100} = 8.9 cfs at Design Point #5 remain within the capacity of the existing culvert.

C. Comparison of Developed to Historic Discharges

Based on the hydrologic calculations in Appendix B, the total undetained developed flow from the site will exceed historic flow from the parcel. Projected increases in developed flows will be mitigated by routing flows through a proposed on-site stormwater detention pond. The comparison of developed to historic discharges at key design points is summarized as follows:

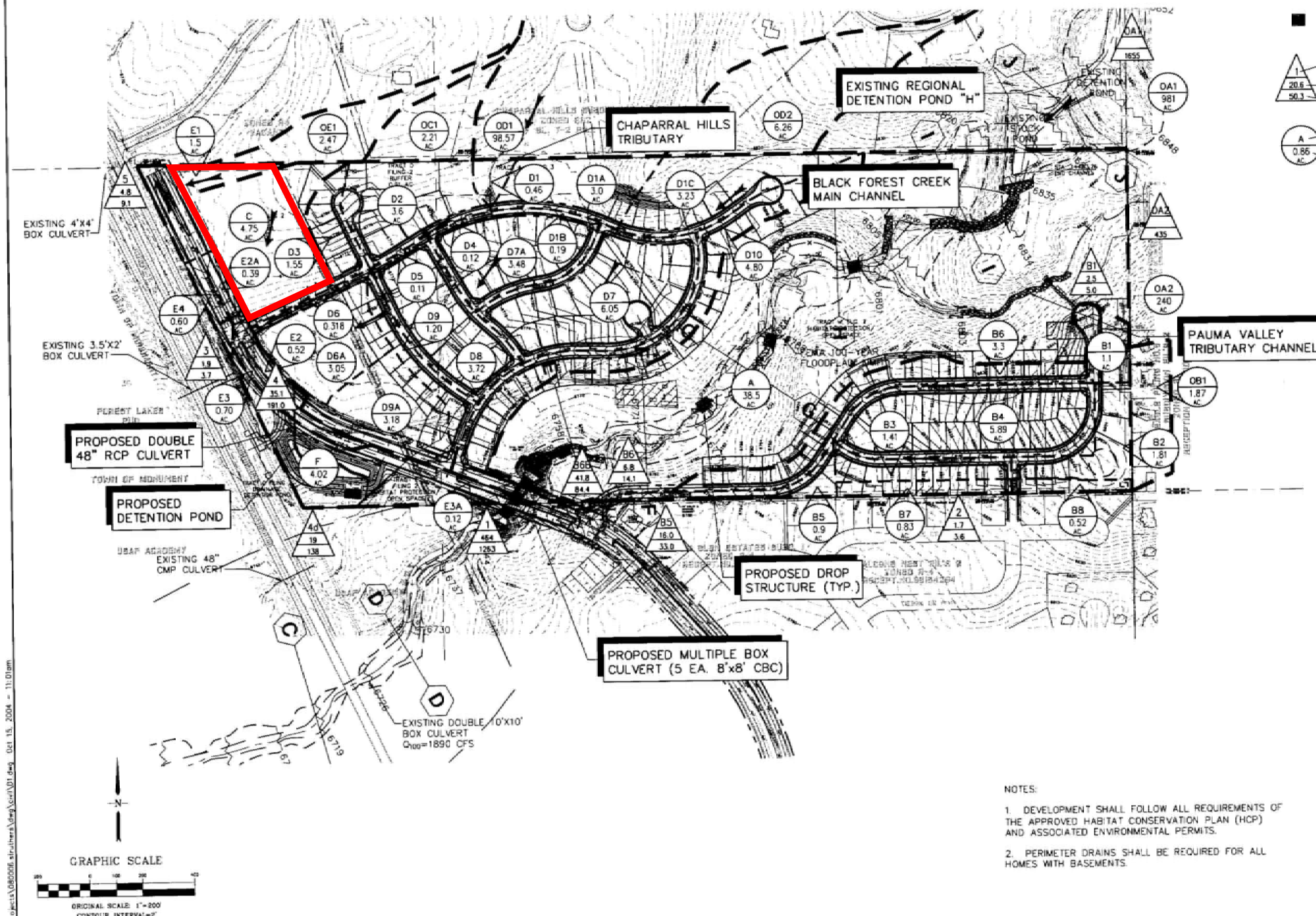
Design Point	Historic Flow			Developed Flow			Comparison of Developed to Historic Flow (Q_5%/Q_{100}%)
	Area (ac)	Q_5 (cfs)	Q_{100} (cfs)	Area (ac)	Q_5 (cfs)	Q_{100} (cfs)	
1 (SCS)	1,266	473	1,281	1,274	464	1,263	98% / 99% (decrease)
2	15.1	9.3	22.4	1.4	1.7	3.6	18% / 16% (decrease)
3	16.0	9.9	24.0	0.6	1.9	3.7	19% / 15% (decrease)
4 (SCS)	133.6	50	148	155.4	66	191	132% / 129% (increase)
5	6.8	8	9.2	4.0	4.6	8.9	121% / 99% (increase)

D. Detention Ponds

The total developed storm runoff downstream of Struthers Ranch will be maintained at historic levels by routing flows through the proposed on-site detention pond located at the westerly boundary of the Struthers Ranch property (equivalent to "Detention Pond #11" as identified in the DBPS). The proposed detention facility will be sized to attenuate peak flows through the pond, based on the difference between outflow and inflow hydrographs. Flows from Basins C and D will be routed through the proposed detention pond at Design Point #4. The pond will be designed to "over-detain" to account for release of developed flows from Basins A and B, ensuring that the net discharge from the overall site will be maintained below historic levels.

As depicted on Sheet C1.02 (Appendix A), the proposed interim access connection from the I-25 Frontage Road to Struthers Road will bisect the pond, providing for a forebay at the upstream end of the pond. Once the interim access to the frontage road is abandoned, the maintenance access road will remain, and the forebay will continue to serve as a water quality enhancement feature. A detailed pond routing analysis utilizing the "Intelisolve Hydraflow" software package is enclosed in Appendix C1, resulting in the following pond design parameters:

Pond	Pond Inflow (Q_5 / Q_{100}, cfs)	Pond Outflow (Q_5 / Q_{100}, cfs)	Pond Volume (ac-ft)
DP4 ("Pond #11")	35 / 191	19.3 / 138.4	4.7



LEGEND

- FILING LIMITS
- MAJOR BASIN BOUNDARY
- MINOR BASIN BOUNDARY
- 6520 EXISTING CONTOUR
- FLOWLINE
- PROPOSED FLOW DIRECTION ARROW
- PROPOSED DROP STRUCTURE
- DESIGN POINT
- Q_5 (cfs)
- Q_{100}(cfs)
- BASIN DESIGNATION
- BASIN AREA (ACRES)

NOTES:

1. DEVELOPMENT SHALL FOLLOW ALL REQUIREMENTS OF THE APPROVED HABITAT CONSERVATION PLAN (HCP) AND ASSOCIATED ENVIRONMENTAL PERMITS.

2. PERIMETER DRAINS SHALL BE REQUIRED FOR ALL HOMES WITH BASEMENTS.

STRUTHERS RANCH SUBDIVISION

DEVELOPED DRAINAGE PLAN

No.	REVISION	BY	DATE
E	EPC COMMENTS	JPS	4/8/04
F	EPC COMMENTS	JPS	5/7/04
G	EPC COMMENTS	JPS	5/25/04
H	EPC COMMENTS	JPS	9/2/04
I	RE-SUBMITTAL TO EPC	JPS	9/30/04

HORZ. SCALE: 1"=200'	DRAWN: MJP
VERT. SCALE: N/A	DESIGNED: JPS
SURVEYED: PINNACLE	CHECKED: JPS
CREATED: 9/11/00	LAST MODIFIED: 10/15/04
PROJECT NO: 080006	MODIFIED BY: MJP
SHEET: D1	

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TABLE 5-1

RECOMMENDED AVERAGE RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	"C" FREQUENCY 10 A&B*	10 C&D*	100 A&B*	100 C&D*
Business					
Commercial Areas	95	0.90	0.90	0.90	0.90
Neighborhood Areas	70	0.75	0.75	0.80	0.80
Residential					
1/8 Acre or less	65	0.60	0.70	0.70	0.80
1/4 Acre	40	0.50	0.60	0.60	0.70
1/3 Acre	30	0.40	0.50	0.55	0.60
1/2 Acre	25	0.35	0.45	0.45	0.55
1 Acre	20	0.30	0.40	0.40	0.50
Industrial					
Light Areas	80	0.70	0.70	0.80	0.80
Heavy Areas	90	0.80	0.80	0.90	0.90
Parks and Cemeteries	7	0.30	0.35	0.55	0.60
Playgrounds	13	0.30	0.35	0.60	0.65
Railroad Yard Areas	40	0.50	0.55	0.60	0.65
Undeveloped Areas					
Historic Flow Analysis-Greenbelts, Agricultural	2	0.15	0.25	0.20	0.30
Pasture/Meadow	0	0.25	0.30	0.35	0.45
Forest	0	0.10	0.15	0.15	0.20
Exposed Rock	100	0.90	0.90	0.95	0.95
Offsite Flow Analysis (when land use not defined)	45	0.55	0.60	0.65	0.70
Streets					
Paved	100	0.90	0.90	0.95	0.95
Gravel	80	0.80	0.80	0.85	0.85
Drive and Walks	100	0.90	0.90	0.95	0.95
Roofs	90	0.90	0.90	0.95	0.95
Lawns	0	0.25	0.30	0.35	0.45

* Hydrologic Soil Group

9/30/90

(EPC-DCM)

STRUTHERS RANCH
COMPOSITE RUNOFF COEFFICIENTS

DEVELOPED CONDITIONS

5-YEAR C VALUES

BASIN	TOTAL AREA (AC)	SOIL TYPE	(AC)	SUB-AREA 1 DEVELOPMENT/ COVER	C	AREA (AC)	SUB-AREA 2 DEVELOPMENT/ COVER	C	(AC)	SUB-AREA 3 DEVELOPMENT/ COVER	C	WEIGHTED C VALUE
OA1	981	B	981	0.25-AC LOTS	0.5							0.500
OA2	240	B	240	0.25-AC LOTS	0.5							0.500
OA1,OA2	1221											0.500
B1	1.1	B	1.1	0.25-AC LOTS	0.5							0.500
A	38.5	B	38.5	OPEN SPACE	0.25							0.250
OA1,OA2,B1,A	1260.6											0.492
OB1	1.87	B	1.87	0.25-AC LOTS	0.5							0.500
B2	1.81	B	1.81	0.25-AC LOTS	0.5							0.500
B3	1.4	B	1.41	0.25-AC LOTS	0.5							0.500
B4	5.8	B	5.8	0.25-AC LOTS	0.5							0.500
OB1,B2-B4	10.9											0.500
B5	0.9	B	0.9	0.25-AC LOTS	0.5							0.500
OB1,B2-B5	11.8											0.500
B6	3.3	B	3.3	0.25-AC LOTS	0.5							0.500
OB1,B2-B6	15.1											0.500
OA1,OA2,A,B1-B6	1275.7											0.492
B7	0.83	B	0.83	0.25-AC LOTS	0.5							0.500
B8	0.52	B	0.52	0.25-AC LOTS	0.5							0.500
B7,B8	1.4											0.500
E4	0.6	B	0.34	PAVED	0.9	0.3	LANDSCAPE	0.25				0.618
OD1	98.57	B	98.57	5-AC LOTS	0.3							0.300
D1	0.46	B	0.46	MEADOW	0.25							0.250
OD1,D1	99.03											0.300

OD2	6.26	B	6.26	5-AC LOTS	0.3							0.300
D1C	3.23	B	1.5	0.25-AC LOTS	0.5	1.7	OPEN SPACE	0.25				0.366
OD2,D1C	9.49											0.322
D1A	3.00	B	0.8	0.25-AC LOTS	0.5	2.2	PARK / OS	0.25				0.317
OD2,D1C,D1A	12.49											0.321
D1B	0.19	B	0.19	ROADWAY	0.9							0.900
OD1,D1,D1A,D1B	111.71											0.303
D4	0.12	B	0.12	ROADWAY	0.9							0.900
OD1,D1,D1A,D1B,D4	111.83											0.304
D5	0.11	B	0.11	ROADWAY	0.9							0.900
OD1,D1,D1A,D1B,D4,D5	111.94											0.304
D6	0.32	B	0.318	ROADWAY	0.9							0.900
OD1,D1,D1A,D1B,D4-D6	112.26											0.306
OC1	2.21	B	2.21	5-AC LOTS	0.3							0.300
D2	3.60	B	3.6	0.25-AC LOTS	0.5							0.500
OC1,D2	5.81											0.424
E2A	0.39	B	0.3	PAVED	0.9	0.1	LANDSCAPE	0.25				0.750
D3	1.55	B	1.55	0.25-AC LOTS	0.5							0.500
C	4.75	B	4.75	COMMERCIAL	0.9							0.900
D3,C	6.30											0.802
OD1,OC1,C,E2A,D1-D6	124.76											0.338
E2	0.52	B	0.4	PAVED	0.9	0.1	LANDSCAPE	0.25				0.750
D6A	3.00	B	3	COMMERCIAL	0.9							0.900
OD1,OC1,C,D1-D6A	128.28											0.350
D7A	3.48	B	3.48	0.25-AC LOTS	0.5							0.500
D7	6.05	B	6.05	0.25-AC LOTS	0.5							0.500
D7A,D7	9.53											0.500
D8	3.72	B	3.72	0.25-AC LOTS	0.5							0.500
D7A,D7,D8	13.25											0.500
D9	1.20	B	1.2	0.25-AC LOTS	0.5							0.500
D7A-D9	14.45											0.500
E3A	0.12	B	0.12	MEDIAN	0.25							0.250
D10	4.80	B	4.8	0.25-AC LOTS	0.5							0.500
D7A-D10,E3A	19.37											0.498
D9A	3.18	B	3.18	COMMERCIAL	0.9							0.900
D7A-D10,E3A	22.55											0.555
E3	0.70	B	0.5	PAVED	0.9	0.2	LANDSCAPE	0.25				0.714
F	4.02	B	4.02	OPEN SPACE	0.25							0.250
OD1,C,D1-D10,E2-E3,F	155.55	B										0.379
OE1	2.47	B	2.47	5-AC LOTS	0.3							0.300
E1	1.5	B	1.5	COMMERCIAL	0.9							0.900
OE1,E1	4.0											0.527

STRUTHERS RANCH
COMPOSITE RUNOFF COEFFICIENTS

DEVELOPED CONDITIONS

100-YEAR C VALUES

BASIN	TOTAL AREA (AC)	SOIL TYPE	(AC)	SUB-AREA 1 DEVELOPMENT/ COVER	C	AREA (AC)	SUB-AREA 2 DEVELOPMENT/ COVER	C	(AC)	SUB-AREA 3 DEVELOPMENT/ COVER	C	WEIGHTED C VALUE
OA1	981	B	981	0.25-AC LOTS	0.6							0.600
OA2	240	B	240	0.25-AC LOTS	0.6							0.600
OA1,OA2	1221											0.600
B1	1.1	B	1.1	0.25-AC LOTS	0.6							0.600
A	38.5	B	38.5	OPEN SPACE	0.35							0.350
OA1,OA2,B1,A	1260.6											0.592
OB1	1.87	B	1.87	0.25-AC LOTS	0.6							0.600
B2	1.81	B	1.81	0.25-AC LOTS	0.6							0.600
B3	1.4	B	1.41	0.25-AC LOTS	0.6							0.600
B4	5.8	B	5.8	0.25-AC LOTS	0.6							0.600
OB1,B2-B4	10.9											0.600
B5	0.9	B	0.9	0.25-AC LOTS	0.6							0.600
OB1,B2-B5	11.8											0.600
B6	3.3	B	3.3	0.25-AC LOTS	0.6							0.600
OB1,B2-B6	15.1											0.600
OA1,OA2,A,B1-B6	1275.7											0.592
B7	0.83	B	0.83	0.25-AC LOTS	0.6							0.600
B8	0.52	B	0.52	0.25-AC LOTS	0.6							0.600
B7,B8	1.4											0.600
E4	0.6	B	0.34	PAVED	0.95	0.3	LANDSCAPE	0.35				0.690
OD1	98.57	B	98.57	5-AC LOTS	0.4							0.400
D1	0.46	B	0.46	MEADOW	0.35							0.350
OD1,D1	99.03											0.400

OD2	6.26	B	6.26	5-AC LOTS	0.4							0.400
D1C	3.23	B	1.5	0.25-AC LOTS	0.6	1.7	OPEN SPACE	0.35				0.466
OD2,D1C	9.49											0.422
D1A	3.00	B	0.8	0.25-AC LOTS	0.6	2.2	PARK / OS	0.35				0.417
OD2,D1C,D1A	12.49											0.421
D1B	0.19	B	0.19	ROADWAY	0.95							0.950
OD1,D1,D1A,D1B	111.71											0.403
D4	0.12	B	0.12	ROADWAY	0.95							0.950
OD1,D1,D1A,D1B,D4	111.83											0.404
D5	0.11	B	0.11	ROADWAY	0.95							0.950
OD1,D1,D1A,D1B,D4,D5	111.94											0.404
D6	0.32	B	0.318	ROADWAY	0.95							0.950
OD1,D1,D1A,D1B,D4-D6	112.26											0.406
OC1	2.21	B	2.21	5-AC LOTS	0.4							0.400
D2	3.60	B	3.6	0.25-AC LOTS	0.6							0.600
OC1,D2	5.81											0.524
E2A	0.39	B	0.3	PAVED	0.95	0.1	LANDSCAPE	0.35				0.812
D3	1.55	B	1.55	0.25-AC LOTS	0.6							0.600
C	4.75	B	4.75	COMMERCIAL	0.9							0.900
D3,C	6.30											0.826
OD1,OC1,C,E2A,D1-D6	124.76											0.434
E2	0.52	B	0.4	PAVED	0.95	0.1	LANDSCAPE	0.35				0.812
D6A	3.00	B	3	COMMERCIAL	0.9							0.900
OD1,OC1,C,D1-D6A	128.28											0.443
D7A	3.48	B	3.48	0.25-AC LOTS	0.6							0.600
D7	6.05	B	6.05	0.25-AC LOTS	0.6							0.600
D7A,D7	9.53											0.600
D8	3.72	B	3.72	0.25-AC LOTS	0.6							0.600
D7A,D7,D8	13.25											0.600
D9	1.20	B	1.2	0.25-AC LOTS	0.6							0.600
D7A-D9	14.45											0.600
E3A	0.12	B	0.12	MEDIAN	0.35							0.350
D10	4.80	B	4.8	0.25-AC LOTS	0.6							0.600
D7A-D10,E3A	19.37											0.598
D9A	3.18	B	3.18	COMMERCIAL	0.9							0.900
D7A-D10,E3A	22.55											0.641
E3	0.70	B	0.5	PAVED	0.95	0.2	LANDSCAPE	0.35				0.779
F	4.02	B	4.02	OPEN SPACE	0.35							0.350
OD1,C,D1-D10,E2-E3,F	155.55	B										0.471
OE1	2.47	B	2.47	5-AC LOTS	0.4							0.400
E1	1.5	B	1.5	COMMERCIAL	0.9							0.900
OE1,E1	4.0											0.589

STRUTHERS RANCH
RATIONAL METHOD - DRAINAGE CALCULATIONS

DEVELOPED FLOWS

BASIN	DESIGN POINT	AREA (AC)	C 5-YEAR[7]	C 100-YEAR[7]	OVERLAND LENGTH (FT)	SLOPE (%)	Tco[1] (MIN)	CHANNEL LENGTH (FT)	CONVEYANCE COEFFICIENT K	SLOPE (%)	SCS[2] VELOCITY (FT/S)	Tt[3] (MIN)	TOTAL Tc[4] (MIN)	INTENSITY[5] 5-YR (IN/HR)	INTENSITY[5] 100-YR (IN/HR)	PEAK FLOW Q5[6] (CFS)	PEAK FLOW Q100[6] (CFS)
OA1		981.00	0.500	0.600	300	5.4	10.7	11900	1.50	5.4	3.49	56.9	67.6	1.50	2.65	735.75	1559.79
OA2		240.00	0.500	0.600	300	5.5	10.6	620	1.50	5.5	3.52	2.9	13.5	3.60	6.10	432.00	878.40
OA1,OA2	OA1	1221.00	0.500	0.600									67.6	1.50	2.65	915.75	1941.39
B1	B1	1.10	0.500	0.600	250	12.8	7.3	0				0.0	7.3	4.50	7.60	2.48	5.02
A		38.50	0.250	0.350	0		0.0	2730	1.50	3.2	2.68	17.0	17.0	3.20	5.50	30.80	74.11
OA1,OA2,B1,A		1260.60	0.492	0.592									84.5	1.50	2.65	930.32	1977.63
B2		1.81	0.500	0.600	150	5.3	7.6	450	2.00	4.9	4.43	1.7	9.3	4.10	7.10	3.71	7.71
B3		1.41	0.500	0.600	0		0.0	700	2.00	3	3.46	3.4	3.4	5.20	9.00	3.67	7.61
B4		5.89	0.500	0.600	0		0.0	1180	2.00	3.7	3.85	5.1	5.1	5.20	9.00	15.31	31.81
OB1,B2,B3,B4	B3	9.11	0.500	0.600									12.7	3.70	6.20	16.85	33.89
B5		0.90	0.500	0.600	0		0.0	1000	2.00	3.3	3.63	4.6	4.6	5.20	9.00	2.34	4.86
OB1,B2-B5	B5	10.01	0.500	0.600									17.2	3.20	5.50	16.02	33.03
B6	B6	3.30	0.500	0.600	0		0.0	2100	2.00	3.7	3.85	9.1	9.1	4.10	7.10	6.77	14.06
OB1,B2-B6	B6A	13.31	0.500	0.600									17.2	3.20	5.50	21.30	43.92
B6A	B6B															41.80	84.40
OA1,OA2,A,B1-B6	1	1273.9	0.492	0.592									84.5	1.50	2.65	940.15	1998.51
B7		0.83	0.500	0.600	150	4.0	8.3	0				0.0	8.3	4.25	7.50	1.76	3.74
B8		0.52	0.500	0.600	850	5.5	17.8	0				0.0	17.8	3.10	5.20	0.81	1.62
B7,B8	2	1.35	0.500	0.600									26.2	2.50	4.40	1.69	3.56
E4	3	0.60	0.618	0.690	0		0.0	450	1.50	5.5	3.52	2.1	2.1	5.20	9.00	1.93	3.73
OD1		98.57	0.300	0.400	1000	10.0	21.2	3300	1.50	3.9	2.96	18.6	39.7	1.90	3.40	56.18	134.06
D1		0.46	0.250	0.350	0		0.0	180	1.50	2.5	2.37	1.3	1.3				
OD1,D1	D1	99.03	0.300	0.400									41.0	1.90	3.40	56.45	134.68
OD2		6.26	0.300	0.400	1000	3.5	30.0	0				0.0	30.0	2.35	4.10	4.41	10.27
D1C		3.23	0.366	0.466	0		0.0	700	2.00	3.4	3.69	3.2	3.2				
OD2,D1C	D1C	9.49	0.322	0.422									33.2	2.20	3.85	6.72	15.42
D1A		3.00	0.317	0.417	0		0.0	370	2.00	2.7	3.29	1.9	1.9				
OD2,D1C,D1A	D1A	12.49	0.321	0.421									35.0	2.10	3.75	8.42	19.72
D1B	D1B	0.19	0.900	0.950	0		0.0	420	2.00	1.6	2.53	2.8	2.8	5.20	9.00	0.89	1.62
OD1,D1,D1A,D1B	D1A1	111.71	0.303	0.403									41.0	1.90	3.40	64.31	153.07
D4	D4	0.12	0.900	0.950	0		0.0	700	2.00	1.56	2.50	4.7	4.7	5.20	9.00	0.56	1.03
OD1,D1,D1A,D1B,D4	D4A	111.83	0.304	0.404									45.7	1.75	3.20	64.87	154.09
D5	D5	0.11	0.900	0.950	0		0.0	250	2.00	3.27	3.62	1.2	1.2	5.20	9.00	0.51	0.94
OD1,D1,D1A,D1B,D4,D5	D5A	111.94	0.304	0.404									46.8	1.70	3.15	65.39	155.03
D6	D6	0.32	0.900	0.950	0		0.0	480	2.00	4.44	4.21	1.9	1.9	5.20	9.00	1.49	2.72
OD1,D1,D1A,D1B,D4-D6	D6A1	112.26	0.306	0.406									48.7	1.70	3.00	66.88	157.75

BASIN	DESIGN POINT	AREA (AC)	C 5-YEAR[7]	C 100-YEAR[7]	OVERLAND LENGTH (FT)	SLOPE (%)	Tco[1] (MIN)	CHANNEL LENGTH (FT)	CONVEYANCE COEFFICIENT K	SLOPE (%)	SCS[2] VELOCITY (FT/S)	Tt[3] (MIN)	TOTAL Tc[4] (MIN)	INTENSITY[5] 5-YR (IN/HR)	INTENSITY[5] 100-YR (IN/HR)	PEAK FLOW Q5[6] (CFS)	PEAK FLOW Q100[6] (CFS)
OC1		2.21	0.300	0.400	550	3.3	22.7					0.0	22.7	2.70	4.70	1.79	4.15
D2		3.60	0.500	0.600	0		0.0	600	2.00	3.6	3.79	2.6	2.6				
OC1,D2	D2	5.81	0.424	0.524									25.3	2.60	4.50	6.40	13.70
E2A	E2A	0.39	0.750	0.812	0		0.0	300	1.50	4	3.00	1.7	1.7	5.20	9.00	1.52	2.85
D3		1.55	0.500	0.600	0		0.0	580	2.00	4.3	4.15	2.3	2.3	5.20	9.00	4.03	8.37
C		4.75	0.900	0.900	0		0.0	750	2.00	3.3	3.63	3.4	3.4	5.20	9.00	22.23	38.48
D3,C	C	6.30	0.802	0.826									5.8	5.00	8.50	25.26	44.23
OD1,OC1,E2A,C,D1-D6	C1	124.76	0.338	0.434									48.7	1.70	3.00	71.69	162.43
E2		0.52	0.750	0.812	0		0.0	300	1.50	4	3.00	1.7	1.7	5.20	9.00	2.03	3.80
D6A	D6A	3.00	0.900	0.900	0		0.0	470	2.00	3.4	3.69	2.1	2.1	5.20	9.00	14.04	24.30
OD1,OC1,C,D1-D6A	D6A2	128.28	0.350	0.443									50.8	1.60	2.90	71.84	164.80
D7A	D7A	3.48	0.500	0.600	0		0.0	950	2.00	1.68	2.59	6.1	6.1	5.00	8.50	8.70	17.75
D7		6.05	0.500	0.600	0		0.0	1244	2.00	2.17	2.95	7.0	7.0	4.60	8.00	13.92	29.04
D7A,D7	D7	9.53	0.500	0.600									7.0	4.60	8.00	21.92	45.74
D8	D8	3.72	0.500	0.600	0		0.0	225	2.00	3.4	3.69	1.0	1.0	5.20	9.00	9.67	20.09
D7A-D8	D8A	13.25	0.500	0.600									8.1	4.40	7.50	29.15	59.63
D9	D9	1.20	0.500	0.600	0		0.0	210	2.00	3.4	3.69	0.9	0.9	5.20	9.00	3.12	6.48
D7A-D9	D9A	14.45	0.500	0.600									9.0	4.20	7.20	30.35	62.42
E3A	E3A	0.12	0.250	0.350	0		0.0	220	1.50	4.3	3.11	1.2	1.2	5.20	9.00	0.16	0.38
D10	D10	4.80	0.500	0.600	300	4.0	11.8	1820	2.00	3	3.46	8.8	20.5	2.95	5.05	7.08	14.54
D10A	D10A	0.23	0.500	0.600	0		0.0	200	1.50	0.5	1.06	3.1	3.1	5.20	9.00	0.60	1.24
D7A-D10,E3A	D10B	19.37	0.498	0.598									20.5	2.95	5.05	28.46	58.50
D9A		3.18	0.900	0.900	0		0.0	620	1.50	0.5	1.06	9.7	9.7	5.20	9.00	14.88	25.76
D7A-D10,E3A	D9B	22.55	0.555	0.641									30.3	2.30	4.05	28.79	58.54
E3	E3	0.70	0.714	0.779	0		0.0	620	1.50	0.8	1.34	7.7	7.7	4.40	7.50	2.20	4.09
F		4.02	0.250	0.350	0		0.0	570	1.50	1.0	1.50	6.3	6.3	5.00	8.50	5.03	11.96
OD1,OC1,C,D1-D10,E2-E3,F	4	155.55	0.379	0.471									50.8	1.60	2.90	94.32	212.46
OE1		2.47	0.300	0.400	850	2.8	29.8	0				0.0	29.8	2.35	4.10	1.74	4.05
E1		1.50	0.900	0.900	0		0.0	700	2.00	2.3	3.03	3.8	3.8	5.20	9.00	7.02	12.15
OE1,E1	5	3.97	0.527	0.589									33.6	2.20	3.80	4.60	8.89

1) OVERLAND FLOW Tco = (1.87*(1.1-RUNOFF COEFFICIENT)*(OVERLAND FLOW LENGTH^(0.5)/(SLOPE^(0.333))
2) SCS VELOCITY = K * ((SLOPE(%))^0.5)
K = 0.70 FOR MEADOW / FOREST
K = 1.0 FOR BARE SOIL
K = 1.5 FOR GRASS CHANNEL
K = 2.0 FOR PAVEMENT
3) GUTTER/SWALE FLOW, TRAVEL TIME, Tt = (CHANNEL LENGTH/ SCS VELOCITY) / 60 SEC
4) Tc = Tco + Tt
*** IF TOTAL TIME OF CONCENTRATION IS LESS THAN 5 MINUTES, THEN 5 MINUTES IS USED
5) INTENSITY BASED ON I-D-F CURVE IN EL PASO COUNTY DRAINAGE CRITERIA MANUAL
6) Q = CiA
7) WEIGHTED AVERAGE C VALUES FOR COMBINED BASINS

APPENDIX B

HYDROLOGIC CALCULATIONS

Hydrologic Soil Group—El Paso County Area, Colorado
(Cathedral Rock Church)

Map Scale: 1:1,530 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 20, 40, 80, 120

Feet: 0, 50, 100, 200, 300

N

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

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

- Streams and Canals

Transportation

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

Background

- Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 21, Aug 24, 2023

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

Date(s) aerial images were photographed: Jun 9, 2021—Jun 12, 2021

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

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
68	Peyton-Pring complex, 3 to 8 percent slopes	B	1.3	25.1%
71	Pring coarse sandy loam, 3 to 8 percent slopes	B	3.9	74.9%
Totals for Area of Interest			**5.2**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

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.

$$t_c = t_i + t_t \qquad \text{(Eq. 6-7)}$$

Where:

t_c = time of concentration (min)

t_i = overland (initial) flow time (min)

t_t = travel time in the ditch, channel, gutter, storm sewer, etc. (min)

3.2.1 Overland (Initial) Flow Time

The overland flow time, t_i, may be calculated using Equation 6-8.

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}} \qquad \text{(Eq. 6-8)}$$

Where:

t_i = overland (initial) flow time (min)
C_5 = runoff coefficient for 5-year frequency (see Table 6-6)
L = length of overland flow (300 ft maximum for non-urban land uses, 100 ft maximum for urban land uses)
S = average basin slope (ft/ft)

Note that in some urban watersheds, the overland flow time may be very small because flows quickly concentrate and channelize.

3.2.2 Travel Time

For catchments with overland and channelized flow, the time of concentration needs to be considered in combination with the travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the overland travel time, t_t, can be estimated with the help of Figure 6-25 or Equation 6-9 (Guo 1999).

$$V = C_v S_w^{0.5} \qquad \text{(Eq. 6-9)}$$

Where:

V = velocity (ft/s)

C_v = conveyance coefficient (from Table 6-7)

S_w = watercourse slope (ft/ft)

Table 6-7. Conveyance Coefficient, C_v

Type of Land Surface	C_v
Heavy meadow	2.5
Tillage/field	5
Riprap (not buried)*	6.5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

* For buried riprap, select C_v value based on type of vegetative cover.

The travel time is calculated by dividing the flow distance (in feet) by the velocity calculated using Equation 6-9 and converting units to minutes.

The time of concentration (t_c) is then the sum of the overland flow time (t_i) and the travel time (t_t) per Equation 6-7.

3.2.3 First Design Point Time of Concentration in Urban Catchments

Using this procedure, the time of concentration at the first design point (typically the first inlet in the system) in an urbanized catchment should not exceed the time of concentration calculated using Equation 6-10. The first design point is defined as the point where runoff first enters the storm sewer system.

$$t_c = \frac{L}{180} + 10 \qquad \text{(Eq. 6-10)}$$

Where:

t_c = maximum time of concentration at the first design point in an urban watershed (min)

L = waterway length (ft)

Equation 6-10 was developed using the rainfall-runoff data collected in the Denver region and, in essence, represents regional "calibration" of the Rational Method. Normally, Equation 6-10 will result in a lesser time of concentration at the first design point and will govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches.

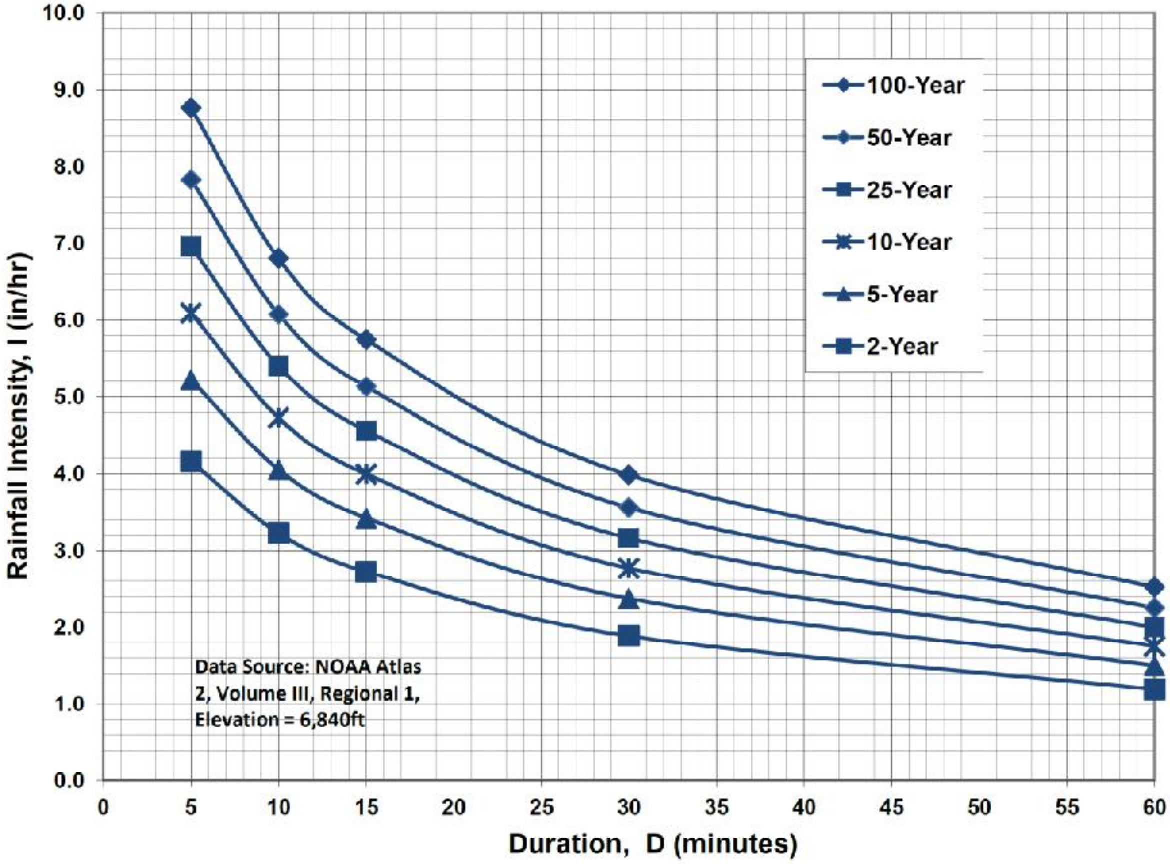
3.2.4 Minimum Time of Concentration

If the calculations result in a t_c of less than 10 minutes for undeveloped conditions, it is recommended that a minimum value of 10 minutes be used. The minimum t_c for urbanized areas is 5 minutes.

3.2.5 Post-Development Time of Concentration

As Equation 6-8 indicates, the time of concentration is a function of the 5-year runoff coefficient for a drainage basin. Typically, higher levels of imperviousness (higher 5-year runoff coefficients) correspond to shorter times of concentration, and lower levels of imperviousness correspond to longer times of

Figure 6-5. Colorado Springs Rainfall Intensity Duration Frequency



IDF Equations

$$I_{100} = -2.52 \ln(D) + 12.735$$

$$I_{50} = -2.25 \ln(D) + 11.375$$

$$I_{25} = -2.00 \ln(D) + 10.111$$

$$I_{10} = -1.75 \ln(D) + 8.847$$

$$I_{5} = -1.50 \ln(D) + 7.583$$

$$I_{2} = -1.19 \ln(D) + 6.035$$

Note: Values calculated by equations may not precisely duplicate values read from figure.

CATHEDRAL ROCK CHURCH
COMPOSITE RUNOFF COEFFICIENTS

DEVELOPED CONDITIONS

5-YEAR C VALUES

BASIN	TOTAL AREA (AC)	(AC)	SUB-AREA 1 DEVELOPMENT/ COVER	C	AREA (AC)	SUB-AREA 2 DEVELOPMENT/ COVER	C	(AC)	SUB-AREA 3 DEVELOPMENT/ COVER	C	WEIGHTED C VALUE
OA1	0.05	0.00	SF RESIDENTIAL	0.3							0.300
A1	2.29	1.632	PAVED/IMPERVIOUS	0.9	0.66	LANDSCAPED	0.08				0.664
OA1,A1	2.34										0.657
OA2	0.40	0.00	SF RESIDENTIAL	0.3							0.300
A2	0.86	0.523	PAVED/IMPERVIOUS	0.9	0.34	LANDSCAPED	0.08				0.579
OA2,A2	1.26										0.490
OA1-OA2,A1-A2	3.60										0.598
A3	0.96	0.82	PAVED/IMPERVIOUS	0.9	0.14	LANDSCAPED	0.08				0.777
A1-A3	4.56										0.636
A4	0.23	0.00	PAVED/IMPERVIOUS	100	0.23	LANDSCAPED	0.08				0.080
A1-A4	**4.79**										**0.609**
OB1	0.13	0.00	SF RESIDENTIAL	0.3							0.300
B	0.74	0.00	PAVED/IMPERVIOUS	0.9	0.74	LANDSCAPED	0.08				0.080
OB1,B	0.87										0.113

100-YEAR C VALUES

BASIN	TOTAL AREA (AC)	(AC)	SUB-AREA 1 DEVELOPMENT/ COVER	C	AREA (AC)	SUB-AREA 2 DEVELOPMENT/ COVER	C	(AC)	SUB-AREA 3 DEVELOPMENT/ COVER	C	WEIGHTED C VALUE
OA1	0.05	0.00	SF RESIDENTIAL	0.5							0.500
A1	2.29	1.632	PAVED/IMPERVIOUS	0.96	0.66	LANDSCAPED	0.35				0.785
OA1,A1	2.34										0.779
OA2	0.40	0.00	SF RESIDENTIAL	0.5							0.500
A2	0.86	0.523	PAVED/IMPERVIOUS	0.96	0.34	LANDSCAPED	0.35				0.721
OA2,A2	1.26										0.651
OA1-OA2,A1-A2	3.60										0.734
A3	0.96	0.82	PAVED/IMPERVIOUS	0.96	0.14	LANDSCAPED	0.35				0.869
A1-A3	4.56										0.762
A4	0.23	0.00	PAVED/IMPERVIOUS	100	0.23	LANDSCAPED	0.35				0.350
A1-A4	**4.79**										**0.742**
OB1	0.13	0.00	SF RESIDENTIAL	0.5							0.500
B	0.74	0.00	PAVED/IMPERVIOUS	0.96	0.74	LANDSCAPED	0.35				0.350
OB1,B	0.87										0.372

CATHEDRAL ROCK CHURCH
RATIONAL METHOD

EXISTING CONDITIONS

BASIN	DESIGN POINT	AREA (AC)	C 5-YEAR	C 100-YEAR	Overland Flow LENGTH (FT)	Overland Flow SLOPE (FT/FT)	Overland Flow Tco (1) (MIN)	Channel flow CHANNEL LENGTH (FT)	Channel flow CONVEYANCE COEFFICIENT C	Channel flow SLOPE (FT/FT)	Channel flow SCS (2) VELOCITY (FT/S)	Channel flow Tt (3) (MIN)	TOTAL Tc (4) (MIN)	TOTAL Tc (4) (MIN)	INTENSITY (5) 5-YR (IN/HR)	INTENSITY (5) 100-YR (IN/HR)	PEAK FLOW Q5 (6) (CFS)	PEAK FLOW Q100 (6) (CFS)
OA1		0.40	0.300	0.500	100	0.020	11.6					0.0	11.6	11.6	3.90	6.55	0.47	1.31
Tt OA1 to DP1								430	15	0.056	3.55	2.0						
A		3.36	0.080	0.350	100	0.070	9.8	430	15	0.047	3.25	2.2	12.0	12.0	3.86	6.48	1.04	7.62
OA1,A	1	3.76	0.103	0.366									13.6	13.6	3.66	6.15	1.42	8.46
OB1		0.18	0.300	0.500	100	0.020	11.6	30	15	0.02	2.12	0.2	11.9	11.9	3.87	6.50	0.21	0.59
B		1.72	0.080	0.350			0.0	535	15	0.034	2.77	3.2	3.2	5.0	5.17	8.68	0.71	5.22
OB1,B	2	1.90	0.101	0.364									15.1	15.1	3.51	5.90	0.67	4.08

DEVELOPED CONDITIONS

BASIN	DESIGN POINT	AREA (AC)	C 5-YEAR	C 100-YEAR	Overland Flow LENGTH (FT)	Overland Flow SLOPE (FT/FT)	Overland Flow Tco (1) (MIN)	Channel flow CHANNEL LENGTH (FT)	Channel flow CONVEYANCE COEFFICIENT C	Channel flow SLOPE (FT/FT)	Channel flow SCS (2) VELOCITY (FT/S)	Channel flow Tt (3) (MIN)	TOTAL Tc (4) (MIN)	TOTAL Tc (4) (MIN)	INTENSITY (5) 5-YR (IN/HR)	INTENSITY (5) 100-YR (IN/HR)	PEAK FLOW Q5 (6) (CFS)	PEAK FLOW Q100 (6) (CFS)
OA1		0.05	0.300	0.500	60	0.020	9.0					0.0	9.0	9.0	4.29	7.20	0.06	0.18
A1		2.29	0.664	0.785	40	0.050	3.0	400	20	0.032	3.58	1.9	4.8	5.0	5.17	8.68	7.86	15.60
Tt DP-A1 to A2.2								250	20	0.041	4.05	1.0						
OA1,A1	A1.1	2.34	0.657	0.779									13.8	13.8	3.64	6.12	5.60	11.15
OA2		0.40	0.300	0.500	100	0.020	11.6					0.0	11.6	11.6	3.90	6.55	0.47	1.31
Tt OA2 to A2.1								340	20	0.065	5.10	1.1						
A2		0.86	0.579	0.721	90	0.100	4.2	255	20	0.051	4.52	0.9	5.1	5.1	5.13	8.61	2.55	5.34
OA2,A2	A2.1	1.26	0.490	0.651									12.7	12.7	3.77	6.32	2.33	5.19
OA1-OA2,A1-A2	A2.2	3.60	0.598	0.734									12.7	12.7	3.77	6.32	8.11	16.71
A3		0.96	0.777	0.869	100	0.010	5.9	160	20	0.069	5.25	0.5	6.4	6.4	4.79	8.05	3.58	6.71
OA1-OA2,A1-A3	A3.1	4.56	0.636	0.762									6.4	6.4	4.79	8.05	13.90	27.97
A4		0.23	0.080	0.350			0.0	185	20	0.022	2.97	1.0	1.0	5.0	5.17	8.68	0.10	0.70
OA1-OA2,A1-A4	**1**	**4.79**	**0.609**	**0.742**									**13.8**	**13.8**	**3.65**	**6.13**	**10.64**	**21.77**
OB1		0.13	0.300	0.500	100	0.020	11.6	30	15	0.02	2.12	0.2	11.9	11.9	3.87	6.50	0.15	0.42
B		0.74	0.080	0.350			0.0	535	15	0.034	2.77	3.2	3.2	5.0	5.17	8.68	0.31	2.25
OB1,B	2	0.87	0.113	0.372									15.1	15.1	3.51	5.90	0.35	1.91

1) OVERLAND FLOW Tco = (0.395*(1.1-RUNOFF COEFFICIENT)*(OVERLAND FLOW LENGTH^(0.5)/(SLOPE^(0.333))
2) SCS VELOCITY = C * ((SLOPE(FT/FT)^0.5)
C = 2.5 FOR HEAVY MEADOW
C = 5 FOR TILLAGE/FIELD
C = 7 FOR SHORT PASTURE AND LAWNS
C = 10 FOR NEARLY BARE GROUND
C = 15 FOR GRASSED WATERWAY
C = 20 FOR PAVED AREAS AND SHALLOW PAVED SWALES

3) MANNING'S CHANNEL TRAVEL TIME = L/V (WHEN CHANNEL VELOCITY IS KNOWN)
4) Tc = Tco + Tt
*** IF TOTAL TIME OF CONCENTRATION IS LESS THAN 5 MINUTES, THEN 5 MINUTES IS USED
5) INTENSITY BASED ON I-D-F EQUATIONS IN CITY OF COLORADO SPRINGS DRAINAGE CRITERIA MANUAL
$I_5 = -1.5 * \ln(Tc) + 7.583$
$I_{100} = -2.52 * \ln(Tc) + 12.735$
6) Q = CiA

APPENDIX C

HYDRAULIC CALCULATIONS

CATHEDRAL ROCK CHURCH
STORM INLET SIZING SUMMARY

	BASIN FLOW			INLET FLOW						
INLET	DP	Q5 FLOW (CFS)	Q100 FLOW (CFS)	INLET FLOW % OF BASIN	Q5 FLOW (CFS)	Q100 FLOW (CFS)		INLET CONDITION / TYPE	INLET SIZE (FT)	INLET CAPACITY (CFS)
A1.1	A1	7.9	15.6	50	4.0	7.8		SUMP TYPE R	5'	9.7
A1.2	A1	7.9	15.6	40	3.2	6.2		SUMP TYPE R	5'	8.1
A2	A2	2.6	5.3	100	2.6	5.3		SUMP TYPE R	5'	11.7

MHFD-Inlet, Version 5.03 (August 2023)

INLET MANAGEMENT

Worksheet Protected

INLET NAME	Inlet A1.1	Inlet A1.2	Inlet A2
Site Type (Urban or Rural)	URBAN	URBAN	URBAN
Inlet Application (Street or Area)	STREET	STREET	STREET
Hydraulic Condition	In Sump	In Sump	In Sump
Inlet Type	CDOT Type R Curb Opening	CDOT Type R Curb Opening	CDOT Type R Curb Opening

USER-DEFINED INPUT

User-Defined Design Flows

Minor Q_{Known} (cfs)	4.0	3.2	2.6
Major Q_{Known} (cfs)	7.8	6.2	5.3

Bypass (Carry-Over) Flow from Upstream

Inlets must be organized from upstream (left) to downstream (right) in order for bypass flows to be linked.

Receive Bypass Flow from:	No Bypass Flow Received	No Bypass Flow Received	No Bypass Flow Received
Minor Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0
Major Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0

Watershed Characteristics

Subcatchment Area (acres)			
Percent Impervious			
NRCS Soil Type			

Watershed Profile

Overland Slope (ft/ft)			
Overland Length (ft)			
Channel Slope (ft/ft)			
Channel Length (ft)			

Minor Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

Major Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

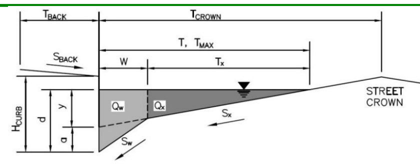
CALCULATED OUTPUT

Minor Total Design Peak Flow, Q (cfs)	**4.0**	**3.2**	**2.6**
Major Total Design Peak Flow, Q (cfs)	**7.8**	**6.2**	**5.3**
Minor Flow Bypassed Downstream, Q_b (cfs)	N/A	N/A	N/A
Major Flow Bypassed Downstream, Q_b (cfs)	N/A	N/A	N/A

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

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

Project: Cathedral Rock Church
Inlet ID: Inlet A1.1



Gutter Geometry:

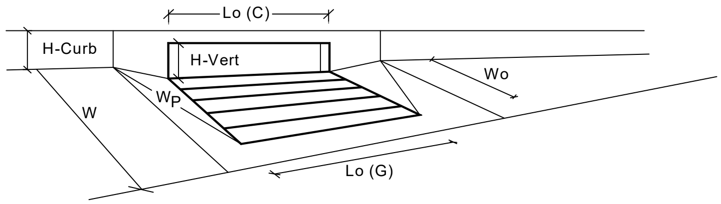
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	4.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.013	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	50.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.011	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} =	50.0	50.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is not applicable to Sump Condition MAJOR STORM Allowable Capacity is not applicable to Sump Condition	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

MHFD-Inlet, Version 5.03 (August 2023)

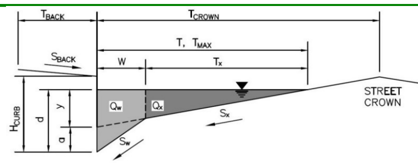


		MINOR	MAJOR	
Design Information (Input) CDOT Type R Curb Opening				
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	8.3	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
Open Area 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	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.33	0.53	ft
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	1.00	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**5.4**	**9.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)	$Q_{PEAK\ REQUIRED}$ =	4.0	7.8	cfs

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

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

Project: Cathedral Rock Church
Inlet ID: Inlet A1.2



Gutter Geometry:

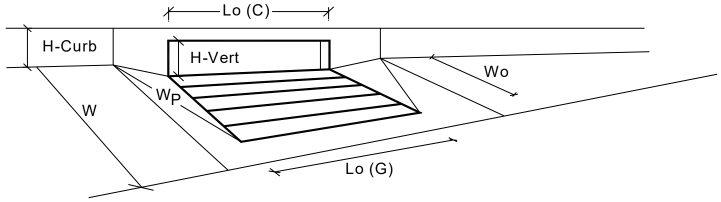
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	4.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	42.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.011	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} =	42.0	42.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is not applicable to Sump Condition				
MAJOR STORM Allowable Capacity is not applicable to Sump Condition	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

MHFD-Inlet, Version 5.03 (August 2023)



Design Information (Input)	CDOT Type R Curb Opening	MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	7.3	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
Open Area 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	Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	

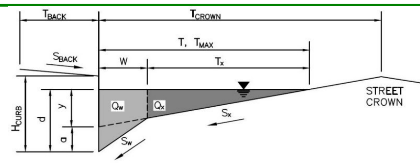
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	0.33	0.44	ft
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	1.00	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	N/A	N/A	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**5.4**	**8.1**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)	$Q_{PEAK\ REQUIRED}$ =	3.2	6.2	cfs

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

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

Project: Cathedral Rock Church
Inlet ID: Inlet A2



Gutter Geometry:

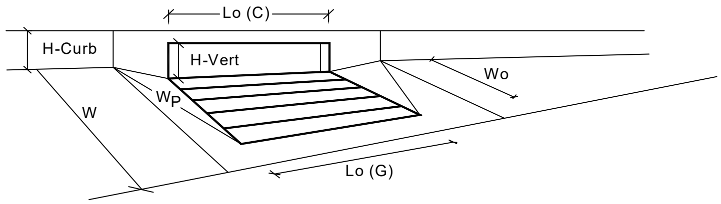
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	4.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	36.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.022	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} =	36.0	36.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is not applicable to Sump Condition				
MAJOR STORM Allowable Capacity is not applicable to Sump Condition	Q_{allow} =	**SUMP**	**SUMP**	**cfs**

INLET IN A SUMP OR SAG LOCATION

MHFD-Inlet, Version 5.03 (August 2023)



Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	11.0	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
Open Area 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		Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)		W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)		C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)		C_w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)		C_o (C) =	0.67	0.67	
Low Head Performance Reduction (Calculated)			MINOR	MAJOR	
Depth for Grate Midwidth		d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation		d_{Curb} =	0.33	0.75	ft
Grated Inlet Performance Reduction Factor for Long Inlets		RF_{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	1.00	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	N/A	N/A	
			MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)		**Q_a =**	**5.4**	**11.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)		$Q_{PEAK\ REQUIRED}$ =	2.6	5.3	cfs

CATHEDRAL ROCK CHURCH
STORM SEWER SIZING SUMMARY

	PIPE FLOW				PIPE CAPACITY		
PIPE	DESIGN POINT	Q5 FLOW (CFS)	Q100 FLOW (CFS)		PIPE SIZE	MIN. PIPE SLOPE	PIPE CAPACITY (CFS)
A1.1	A1.1	4.0	7.8		18	1.0%	10.5
A1.2	A1.1-A1.2	7.1	14.0		18	2.0%	14.9
A2	A1.1-A2	9.7	19.3		18	3.5%	19.7

ASSUMPTIONS:
1. STORM DRAIN PIPE ASSUMED TO BE RCP OR HDPE

Hydraulic Analysis Report

Project Data

Project Title: Project - Cathedral Rock Church

Designer: JPS

Project Date: Monday, September 16, 2024

Project Units: U.S. Customary Units

Notes:

Channel Analysis: SD-A1.1

Notes:

Input Parameters

Channel Type: Circular

Pipe Diameter: 1.5000 ft

Longitudinal Slope: 0.0100 ft/ft

Manning's n: 0.0130

Depth: 1.5000 ft

Result Parameters

Flow: 10.5043 cfs

Area of Flow: 1.7671 ft^2

Wetted Perimeter: 4.7124 ft

Hydraulic Radius: 0.3750 ft

Average Velocity: 5.9442 ft/s

Top Width: 0.0000 ft

Froude Number: 0.0000

Critical Depth: 1.2451 ft

Critical Velocity: 6.6989 ft/s

Critical Slope: 0.0098 ft/ft

Critical Top Width: 1.13 ft

Calculated Max Shear Stress: 0.9360 lb/ft^2

Calculated Avg Shear Stress: 0.2340 lb/ft^2

Channel Analysis: SD-A1.2

Notes:

Input Parameters

Channel Type: Circular

Pipe Diameter: 1.5000 ft

Longitudinal Slope: 0.0200 ft/ft

Manning's n: 0.0130

Depth: 1.5000 ft

Result Parameters

Flow: 14.8554 cfs

Area of Flow: 1.7671 ft^2

Wetted Perimeter: 4.7124 ft

Hydraulic Radius: 0.3750 ft

Average Velocity: 8.4064 ft/s

Top Width: 0.0000 ft

Froude Number: 0.0000

Critical Depth: 1.4026 ft

Critical Velocity: 8.6445 ft/s

Critical Slope: 0.0173 ft/ft

Critical Top Width: 0.74 ft

Calculated Max Shear Stress: 1.8720 lb/ft^2

Calculated Avg Shear Stress: 0.4680 lb/ft^2

Channel Analysis: SD-A2

Notes:

Input Parameters

Channel Type: Circular

Pipe Diameter: 1.5000 ft

Longitudinal Slope: 0.0350 ft/ft

Manning's n: 0.0130

Depth: 1.5000 ft

Result Parameters

Flow: 19.6518 cfs

Area of Flow: 1.7671 ft^2

Wetted Perimeter: 4.7124 ft

Hydraulic Radius: 0.3750 ft

Average Velocity: 11.1207 ft/s

Top Width: 0.0000 ft

Froude Number: 0.0000

Critical Depth: 1.4652 ft

Critical Velocity: 11.1873 ft/s

Critical Slope: 0.0311 ft/ft

Critical Top Width: 0.45 ft

Calculated Max Shear Stress: 3.2760 lb/ft^2

Calculated Avg Shear Stress: 0.8190 lb/ft^2

APPENDIX D

RAIN GARDEN CALCULATIONS

CATHEDRAL ROCK CHURCH
COMPOSITE IMPERVIOUS AREAS

IMPERVIOUS AREAS

BASIN	TOTAL AREA (AC)	(AC)	SUB-AREA 1 DEVELOPMENT/ COVER	PERCENT IMPERVIOUS	AREA (AC)	SUB-AREA 2 DEVELOPMENT/ COVER	PERCENT IMPERVIOUS	(AC)	SUB-AREA 3 DEVELOPMENT/ COVER	PERCENT IMPERVIOUS	WEIGHTED % IMP
OA1	0.05	0.00	SF RESIDENTIAL	40							40.000
A1	2.29	1.632	PAVED/IMPERVIOUS	100	0.66	LANDSCAPED	0.00				71.266
OA1,A1	2.34										70.598
OA2	0.40	0.00	SF RESIDENTIAL	40							40.000
A2	0.86	0.523	PAVED/IMPERVIOUS	100	0.34	LANDSCAPED	0.00				60.814
OA2,A2	1.26										54.206
OA1-OA2,A1-A2	3.60										64.861
A3	0.96	0.82	PAVED/IMPERVIOUS	100	0.14	LANDSCAPED	0.00				85.000
A1-A3	4.56										69.101
A4	0.23	0.00	PAVED/IMPERVIOUS	100	0.23	LANDSCAPED	0.00				0.000
A1-A4	**4.79**										**65.783**
OB1	0.13	0.00	SF RESIDENTIAL	40							40.000
B	0.74	0.00	PAVED/IMPERVIOUS	100	0.74	LANDSCAPED	0.00				0.000
OB1,B	0.87										5.977

Design Procedure Form: Rain Garden (RG)

UD-BMP (Version 3.07, March 2018)

Designer: JPS
Company: JPS
Date: September 14, 2024
Project: Cathedral Rock Church - Rain Garden A
Location: Tract A, Struthers Ranch Filing No. 2

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a (100% if all paved and roofed areas upstream of rain garden)	I_a = 65.8 %
B) Tributary Area's Imperviousness Ratio ($i = I_a/100$)	i = 0.658
C) Water Quality Capture Volume (WQCV) for a 12-hour Drain Time (WQCV= 0.8 * (0.91* i^3 - 1.19 * i^2 + 0.78 * i)	WQCV = 0.21 watershed inches
D) Contributing Watershed Area (including rain garden area)	Area = 208,652 sq ft
E) Water Quality Capture Volume (WQCV) Design Volume Vol = (WQCV / 12) * Area	V_{WQCV} = 3,579 cu ft
F) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = in
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume	$V_{WQCV\ OTHER}$ = cu ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{WQCV\ USER}$ = cu ft
2. Basin Geometry	
A) WQCV Depth (12-inch maximum)	D_{WQCV} = 12 in
B) Rain Garden Side Slopes (Z = 4 min., horiz. dist per unit vertical) (Use "0" if rain garden has vertical walls)	Z = 4.00 ft / ft
C) Mimimum Flat Surface Area	A_{Min} = 2746 sq ft
D) Actual Flat Surface Area	A_{Actual} = 2829 sq ft
E) Area at Design Depth (Top Surface Area)	A_{Top} = 4804 sq ft
F) Rain Garden Total Volume (V_T= ((A_{Top} + A_{Actual}) / 2) * Depth)	V_T= 3,817 cu ft
3. Growing Media	Choose One (●) 18" Rain Garden Growing Media ( ) Other (Explain):
4. Underdrain System	
A) Are underdrains provided?	Choose One (●) YES ( ) NO
B) Underdrain system orifice diameter for 12 hour drain time	
i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y = 2.0 ft
ii) Volume to Drain in 12 Hours	Vol_{12} = 3,579 cu ft
iii) Orifice Diameter, 3/8" Minimum	D_O = 1 3/8 in

Design Procedure Form: Rain Garden (RG)

Designer:	**JPS**
Company:	**JPS**
Date:	**September 14, 2024**
Project:	**Cathedral Rock Church - Rain Garden A**
Location:	**Tract A, Struthers Ranch Filing No. 2**

5. Impermeable Geomembrane Liner and Geotextile Separator Fabric A) Is an impermeable liner provided due to proximity of structures or groundwater contamination?	Choose One ○ YES ◉ NO
6. Inlet / Outlet Control A) Inlet Control	Choose One ○ Sheet Flow- No Energy Dissipation Required ◉ Concentrated Flow- Energy Dissipation Provided
7. Vegetation	Choose One ○ Seed (Plan for frequent weed control) ◉ Plantings ○ Sand Grown or Other High Infiltration Sod
8. Irrigation A) Will the rain garden be irrigated?	Choose One ○ YES ○ NO

Notes:

RAIN GARDEN A FOREBAY CALCULATION

Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.07, March 2018)

Designer: JPS
Company: JPS
Date: September 9, 2024
Project: Cathedral Rock Church - Rain Garden A
Location: Tract A, Struthers Ranch Filing No. 2

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a	I_a = 65.8 %
B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	i = 0.658
C) Contributing Watershed Area	Area = 4.790 ac
D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = in
E) Design Concept (Select EURV when also designing for flood control)	Choose One: (●) Water Quality Capture Volume (WQCV) ( ) Excess Urban Runoff Volume (EURV)
F) Design Volume (WQCV) Based on 40-hour Drain Time ($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area$)	V_{DESIGN}= 0.103 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 ii) Percentage of Watershed consisting of Type B Soils iii) Percentage of Watershed consisting of Type C/D Soils	HSG_A = % HSG_B = % $HSG_{C/D}$ = %
J) Excess Urban Runoff Volume (EURV) Design Volume For HSG A: $EURV_A = 1.68 * i^{1.28}$ For HSG B: $EURV_B = 1.36 * i^{1.08}$ For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$	$EURV_{DESIGN}$ = ac-f t
K) User Input of Excess Urban Runoff Volume (EURV) Design Volume (Only if a different EURV Design Volume is desired)	$EURV_{DESIGN\ USER}$= ac-f t
2. Basin Shape: Length to Width Ratio (A basin length to width ratio of at least 2:1 will improve TSS reduction.)	L : W = 4.0 : 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:	Concrete Forebay
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 = 12.0 in
D) Forebay Discharge	
i) Undetained 100-year Peak Discharge	Q_{100} = 21.80 cfs
ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$)	Q_F = 0.44 cfs
E) Forebay Discharge Design	Choose One: ( ) Berm With Pipe (●) Wall with Rect. Notch ( ) Wall with V-Notch Weir — Flow too small for berm w/ pipe
F) Discharge Pipe Size (minimum 8-inches)	Calculated D_P = in
G) Rectangular Notch Width	Calculated W_N = 4.0 in

PROPOSED FOREBAY VOLUME:
= (12'L x 8'W x 12" DEEP)
= .0022 AF

Cathedral Rock Church
Rain Garden A Spillway

Figure 13-12c. Emergency Spillway Protection

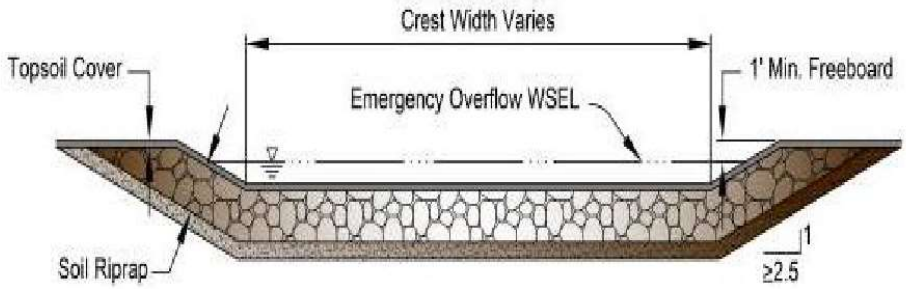
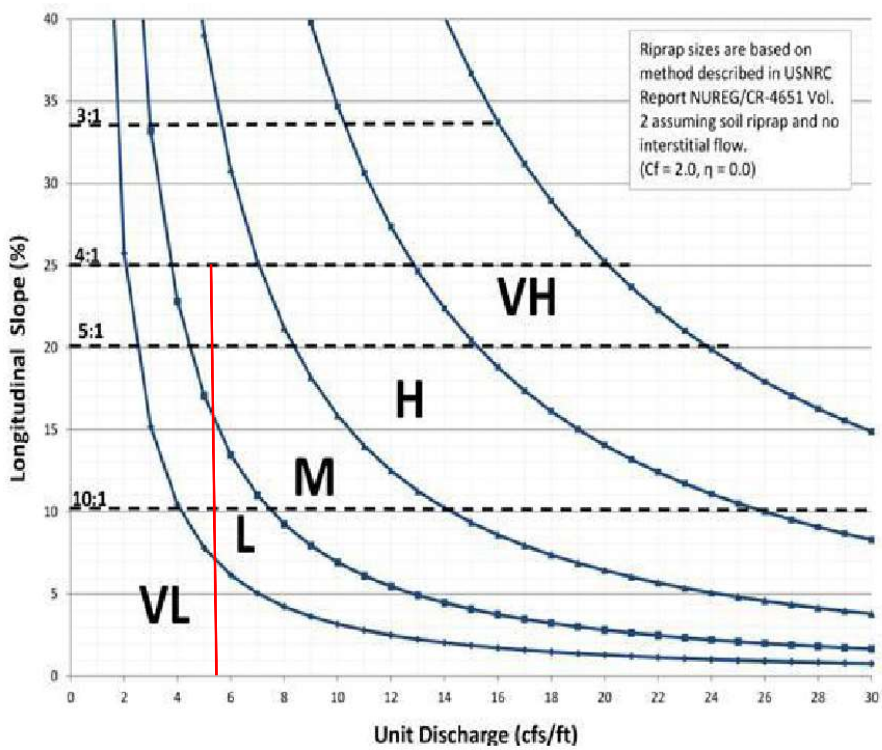


Figure 13-12d. Riprap Types for Emergency Spillway Protection



Spillway Q100 = 21.8 cfs (Undetained DP-1)
Unit Discharge = (21.8 cfs / 4 ft) = 5.5

APPENDIX E

RAIN GARDEN COST ESTIMATE

CATHEDRAL ROCK CHURCH
TRACT A, STRUTHERS RANCH FILING NO. 2
ENGINEER'S COST ESTIMATE
DRAINAGE IMPROVEMENTS - WATER QUALITY RAIN GARDEN

Item No.	Description	Quantity	Unit	Unit Cost ($$$)	Total Cost ($$$)
	PRIVATE DRAINAGE FACILITIES (NON-REIMBURSABLE)				
	Earthwork	150	CY	$5	$750
	Aggregate Base Course (Access Ramp)	15	CY	$66	$990
	Rain Garden Infiltration Media	355	CY	$20	$7,100
	Concrete Forebay	1	LS	$1,800	$1,800
	24" RCP Outlet Pipe	5	LF	$98	$490
	Outlet Structure	1	LS	$5,000	$5,000
	Buried Soil Riprap Spillway	6	TN	$104	$624
	SUBTOTAL				**$16,754**
	Engineering @ 10%				$1,675
	Contingency @ 5%				$838
	TOTAL (NON-REIMBURSABLE)				**$19,267**
	Note: This estimate does not include costs for street improvements and general civil costs (curb & gutter, crosspans, retaining walls, etc.)				

The cost estimate submitted herein is based on time-honored practices within the construction industry. As such the engineer does not control the cost of labor, materials, equipment or a contractor's method of determining prices and competitive bidding practices or market conditions. The estimate represents our best judgement as design professionals using current information available at the time of the preparation. The engineer cannot guarantee that proposals, bids and/or construction costs will not vary from this cost estimate.

APPENDIX F

FIGURES

National Flood Hazard Layer FIRMette

FEMA

104°50'52"W 39°2'56"N

104°50'15"W 39°2'28"N

Feet

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

1:6,000

Basemap Imagery Source: USGS National Map 2023

Legend

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

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

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

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

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

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

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

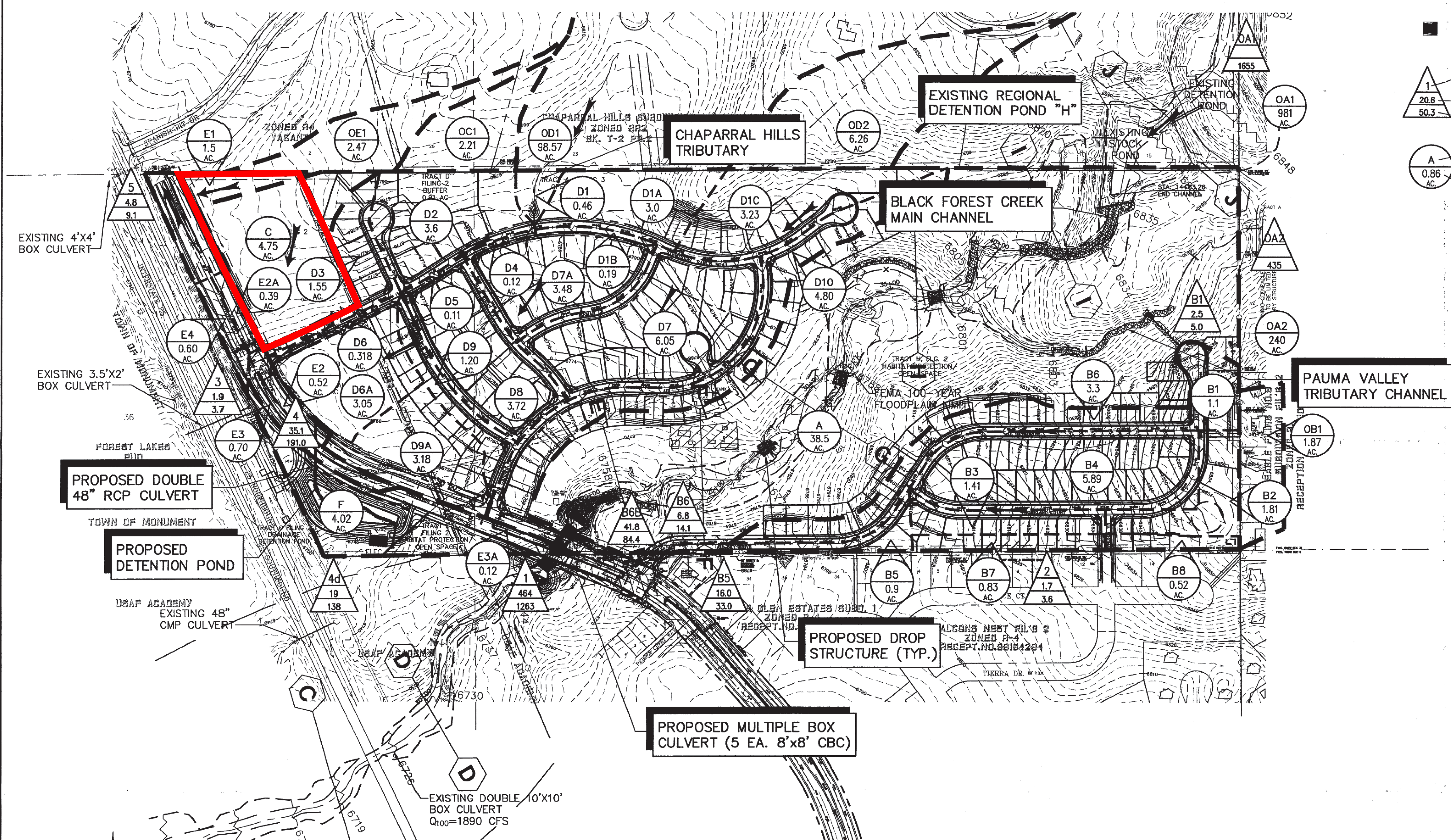
N

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

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 9/20/2024 at 2:03 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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



LEGEND

- FILING LIMITS
- MAJOR BASIN BOUNDARY
- MINOR BASIN BOUNDARY
- 6520 EXISTING CONTOUR
- FLOWLINE
- PROPOSED FLOW DIRECTION ARROW
- PROPOSED DROP STRUCTURE
- DESIGN POINT (1; 20.6 = Q_5 (cfs); 50.3 = Q_{100}(cfs))
- BASIN DESIGNATION (A)
- BASIN AREA (ACRES) (0.86 AC.)

GRAPHIC SCALE

200 0 100 200 400

ORIGINAL SCALE: 1"=200'
CONTOUR INTERVAL=2'

NOTES:

1. DEVELOPMENT SHALL FOLLOW ALL REQUIREMENTS OF THE APPROVED HABITAT CONSERVATION PLAN (HCP) AND ASSOCIATED ENVIRONMENTAL PERMITS.

2. PERIMETER DRAINS SHALL BE REQUIRED FOR ALL HOMES WITH BASEMENTS.

JPS ENGINEERING
19 E. Willamette Ave.
Colorado Springs, CO 80903
PH: 719-477-9429
FAX: 719-471-0766

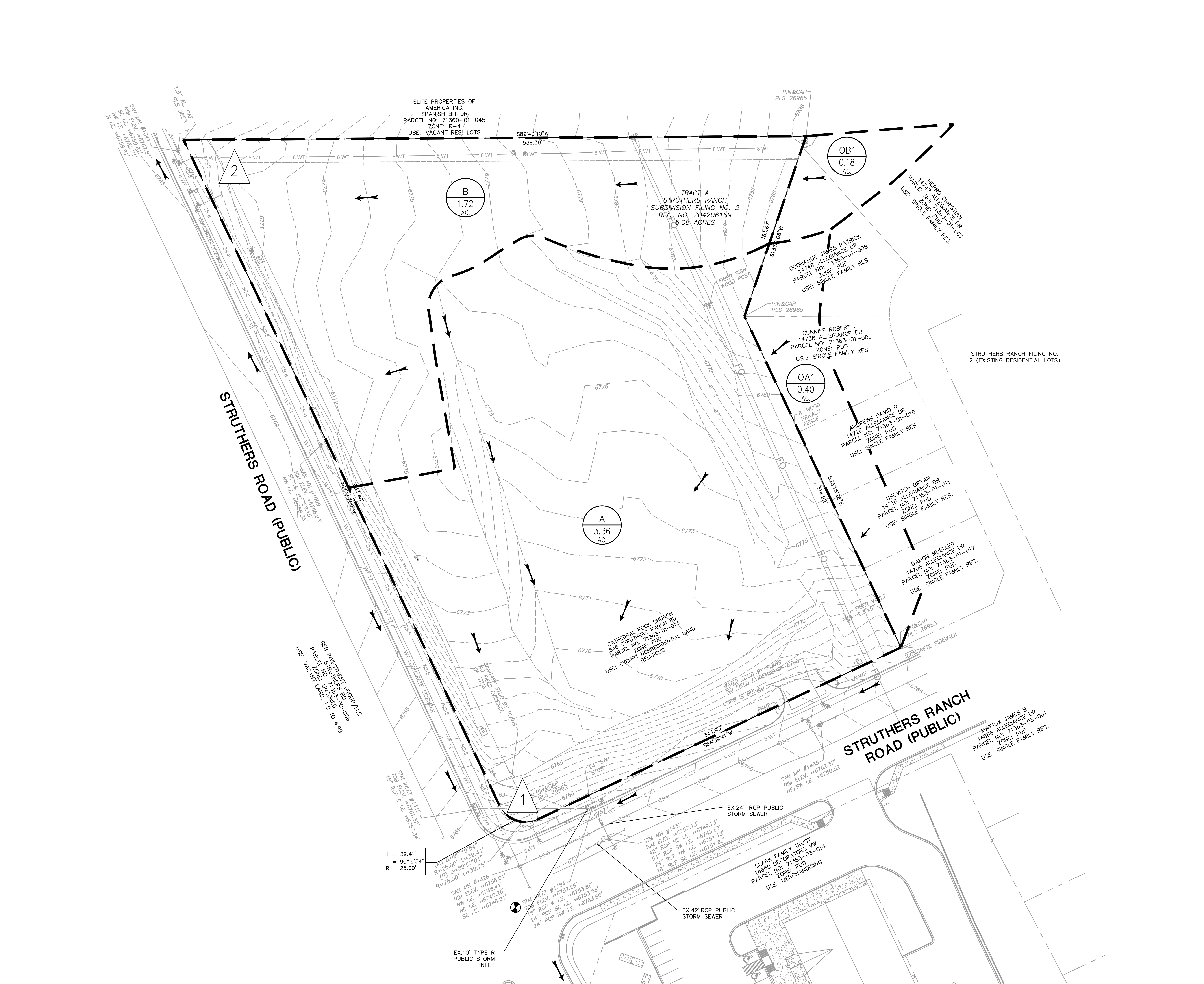
STRUTHERS RANCH SUBDIVISION

DEVELOPED DRAINAGE PLAN

No.	REVISION	BY	DATE
E	EPC COMMENTS	JPS	4/8/04
F	EPC COMMENTS	JPS	5/7/04
G	EPC COMMENTS	JPS	5/25/04
H	EPC COMMENTS	JPS	9/2/04
I	RE-SUBMITTAL TO EPC	JPS	9/30/04

HORZ. SCALE: 1"=200'	DRAWN: MJP
VERT. SCALE: N/A	DESIGNED: JPS
SURVEYED: PINNACLE	CHECKED: JPS
CREATED: 9/11/00	LAST MODIFIED: 10/15/04
PROJECT NO: 080006	MODIFIED BY: MJP
SHEET: D1	

J:\jpsprojects\080006.struthers\dwg\civil\D1.dwg Oct 15, 2004 - 12:12pm



BENCHMARK:
STORM INLET NEAR SW CORNER
OF SITE TOB EL=6757.26
(DATUM: NGVD 1929)

LEGEND

- PROPERTY LINE
- DRAINAGE BASIN BOUNDARY
- PROPOSED CONTOUR
- EXISTING CONTOUR
- FLOWLINE
- FLOW DIRECTION ARROW
- DESIGN POINT
- BASIN DESIGNATION
- BASIN AREA (ACRES)

BASIN SUMMARY TABLE

BASIN	Q_5 (CFS)	Q_{100} (CFS)
OA1	0.5	1.3
A	1.0	7.6
OB1	0.2	0.6
B	0.7	5.2

SUMMARY HYDROLOGY TABLE

DESIGN POINT	Q_5 (CFS)	Q_{100} (CFS)
1	1.4	8.5
2	0.7	4.1

CATHEDRAL ROCK CHURCH
TRACT A, STRUTHERS RANCH FILING NO. 2

EXISTING CONDITIONS DRAINAGE PLAN

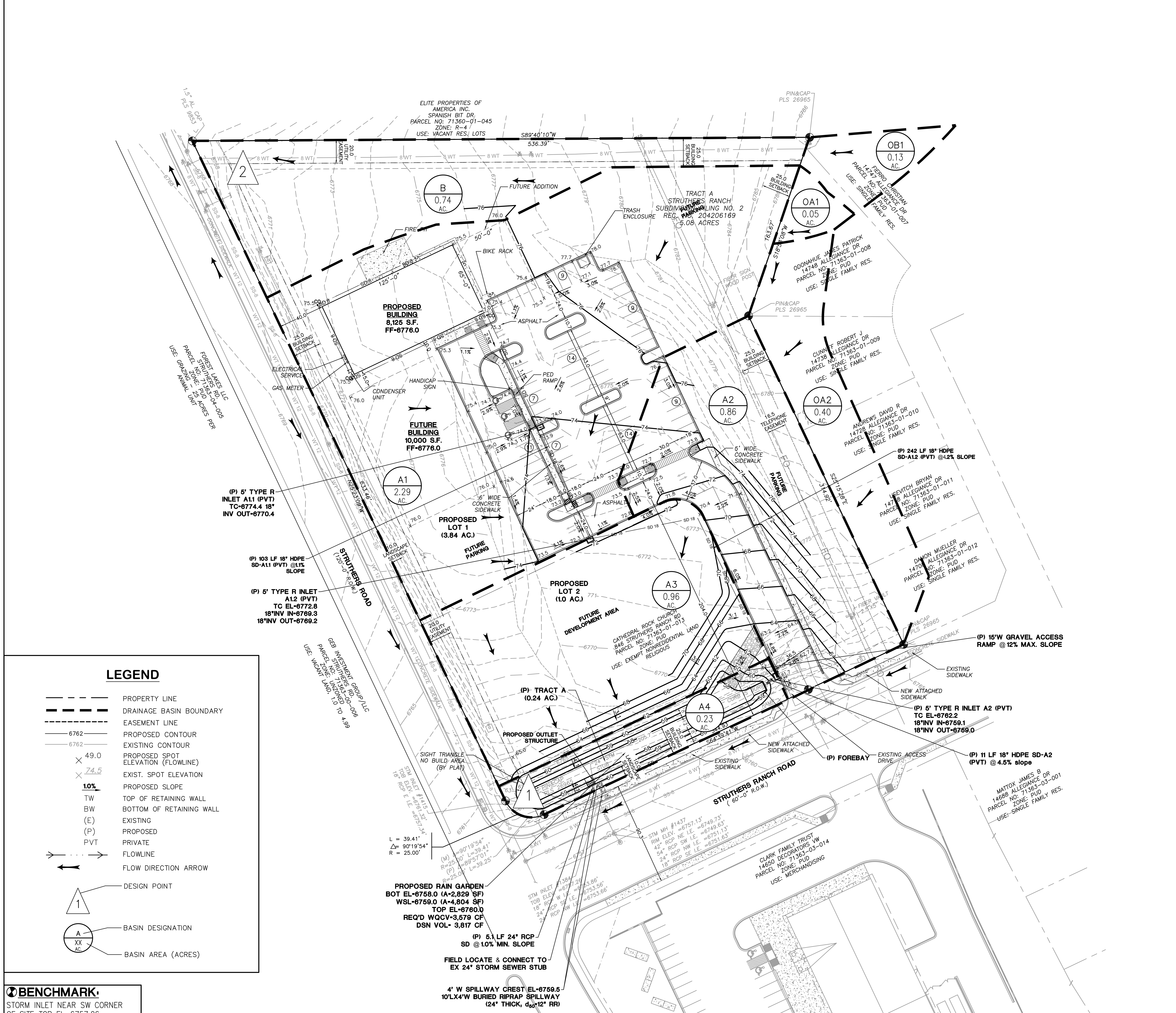
JPS ENGINEERING
19 E. Willamette Ave.
Colorado Springs, CO
80903
PH: 719-477-9429
FAX: 719-471-0766
www.jpsengr.com

CALL UTILITY NOTIFICATION CENTER OF COLORADO
1-800-922-1987
CALL 2-BUSINESS DAYS IN ADVANCE BEFORE YOU DIG, GRADE, OR EXCAVATE FOR THE MARKING OF UNDERGROUND MEMBER UTILITIES.

No.	REVISION	BY	DATE

HORZ. SCALE:	1"=40'	DRAWN:	PV
VERT. SCALE:	N/A	DESIGNED:	JPS
SURVEYED:	RIDGELINE	CHECKED:	JPS
CREATED:	05/27/20	LAST MODIFIED:	09/23/24
PROJECT NO:	082401	MODIFIED BY:	PV

SHEET:
EX1



LEGEND

— — —	PROPERTY LINE
— — —	DRAINAGE BASIN BOUNDARY
- - - -	EASEMENT LINE
6762	PROPOSED CONTOUR
6762	EXISTING CONTOUR
× 49.0	PROPOSED SPOT ELEVATION (FLOWLINE)
× 74.5	EXIST. SPOT ELEVATION
1.0%	PROPOSED SLOPE
TW	TOP OF RETAINING WALL
BW	BOTTOM OF RETAINING WALL
(E)	EXISTING
(P)	PROPOSED
PVT	PRIVATE
→···→	FLOWLINE
←	FLOW DIRECTION ARROW
△1	DESIGN POINT
A / XX AC.	BASIN DESIGNATION / BASIN AREA (ACRES)

BENCHMARK:
STORM INLET NEAR SW CORNER OF SITE TOB EL=6757.26 (DATUM: NGVD 1929)

BASIN SUMMARY TABLE

DESIGN BASIN	Q5 (CFS)	Q100 (CFS)
OA1	0.1	0.2
OA2	0.5	1.3
A1	7.9	15.6
A2	2.6	5.3
A3	3.6	6.7
A4	0.1	0.7
OB1	0.2	0.4
B	0.3	2.3

DEVELOPED CONDITIONS SUMMARY HYDROLOGY TABLE

DESIGN POINT	Q5 (CFS)	Q100 (CFS)
1	10.6	21.8
2	0.4	1.9

IMPERVIOUS AREA CALCULATIONS:
BASIN A1 AREA = 2.29 AC.

SURFACE TYPE	AREA
CONC. SIDEWALK	5,071 SF
ASPHALT PAVEMENT	27,048 SF
BUILDING (PH.1)	8,125 SF
BUILDING (FUTURE)	13,250 SF
PAVEMENT (FUTURE)	17,584 SF
TOTAL IMPERVIOUS AREA	71,078 SF = 1.6 AC = 69.86%

IMPERVIOUS AREA CALCULATIONS:
BASIN A2 AREA = 0.86 AC.

SURFACE TYPE	AREA
CONC. SIDEWALK	1,691 SF
ASPHALT PAVEMENT	11,876 SF
PAVEMENT (FUTURE)	9,213 SF
TOTAL IMPERVIOUS AREA	22,780 SF = 0.5 AC = 58.13%

IMPERVIOUS AREA CALCULATIONS:
BASIN A3 AREA = 0.96 AC.

FUTURE COMMERCIAL DEVELOPMENT
ASSUMED IMPERVIOUS AREA = 85%

IMPERVIOUS AREA CALCULATIONS:
BASIN A4 AREA = 0.23 AC.

SURFACE TYPE	AREA
CONC. SIDEWALK	0 SF
ASPHALT PAVEMENT	0 SF
BUILDING (PH.)	0 SF
BUILDING (FUTURE)	0 SF
PAVEMENT (FUTURE)	0 SF
TOTAL IMPERVIOUS AREA	0 SF = 0 AC = 0%

IMPERVIOUS AREA CALCULATIONS:
BASIN B AREA = 0.74 AC.

SURFACE TYPE	AREA
CONC. SIDEWALK	0 SF
ASPHALT PAVEMENT	0 SF
BUILDING (PH.)	0 SF
BUILDING (FUTURE)	0 SF
PAVEMENT (FUTURE)	0 SF
TOTAL IMPERVIOUS AREA	0 SF = 0 AC = 0%

CATHEDRAL ROCK CHURCH
TRACT A, STRUTHERS RANCH FILING NO. 2

DEVELOPED DRAINAGE PLAN

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No.	REVISION	BY	DATE

HORZ. SCALE: 1"=40'	DRAWN: PV
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SURVEYED: RIDGELINE	CHECKED: JPS
CREATED: 05/27/20	LAST MODIFIED: 09/23/24
PROJECT NO: 082401	MODIFIED BY: PV
SHEET: D1.1	