MASTER DEVELOPMENT DRAINAGE PLAN &
FINAL DRAINAGE REPORT for

Broadview Business Park Filing No. 5 2570 Zeppelin Road Colorado Springs, CO

Prepared for:

Scannell Properties #298, LLC.
800 E. 96th Street, Suite 175
Indianapolis, Indiana 46240

Prepared by:

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Kimley»Horn

Project #: 09441002

Prepared: May 1, 2017
Revised: May 31, 2017
Revised: June 22, 2017

CERTIFICATION

ENGINEERS STATEMENT

This report and plan for the drainage design of 2570 Zeppelin Road was prepared by me (or under my direct supervision) and is correct to the best of my knowledge and belief. Said report and plan has been prepared in accordance with the City of Colorado Springs Drainage Criteria Manual and is in conformity with the master plan of the drainage basin. I understand that the City of Colorado Springs does not and will not assume liability for drainage facilities designed by others. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparing this report.

SIGNATURE (Affix Seal): Eric Gunderson
Colorado P.E. No. 49487 6/22/17 Date

COLORADO REGISTERED PROFESSIONAL ENGINEER ERIC SON GUNDERSON 49487 6/22/17

DEVELOPER'S STATEMENT

Scannell Properties #298 hereby certifies that the drainage facilities for 2570 Zeppelin Road shall be constructed according to the design presented in this report. I understand that the City of Colorado Springs does not and will not assume liability for the drainage facilities designed and/or certified by my engineer and that are submitted to the City of Colorado Springs pursuant to section 7.7.906 of the City Code; and cannot, on behalf of 2570 Zeppelin Road, guarantee that final drainage design review will absolve Scannell Properties #298 and/or their successors and/or assigns of future liability for improper design. I further understand that approval of the final plat does not imply approval of my engineer's drainage design.

Scannell Properties #298, LLC
Name of Developer

6/2/17
Authorized Signature Date

Marc D. Pfleging
Printed Name

Manager
Title

8801 River Crossing Blvd., Suite 300
Indianapolis, IN 46240
Address:

CITY OF COLORADO SPRINGS STATEMENT

Filed in accordance with Section 7.7.906 of the Code of the City of Colorado Springs, 2001, as amended.

Jonathan B. Scheve
For City Engineer

06/30/2017
Date

Conditions:

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Kimley»Horn

INTRODUCTION

PURPOSE AND SCOPE OF STUDY

The purpose of this report is to outline the master development drainage plan ("MDDP") for the Broadview Business Park Filing No. 5 located west of the intersection of Aviation Way and Zeppelin Road (the "Property"), City of Colorado Springs, Colorado (the "City"). This MDDP identifies on-site and offsite drainage patterns, storm sewer and inlet locations, areas tributary to the site and proposes to safely route developed storm water to adequate outfalls. The Property is 17.86 acres in size. The Property consists of Lot 1, Block 1 of the Broadview Business Park Filing No. 2 and is being subdivided into two separate lots. Lot 1 is the northern lot and is 9.66 acres in size. Lot 2 will consist of the southern lot that is 8.2 acres in size.

The Property is located within the Peterson Air Field Drainage Basin and is part of the subject area of the Drainage Basin Planning Study ("DBPS") titled "Peterson Field Drainage Basin Master Plan Update, dated August 1984 prepared by URS Company. Amendments to the approved DBPS are not included with the study.

GENERAL PROJECT DESCRIPTION

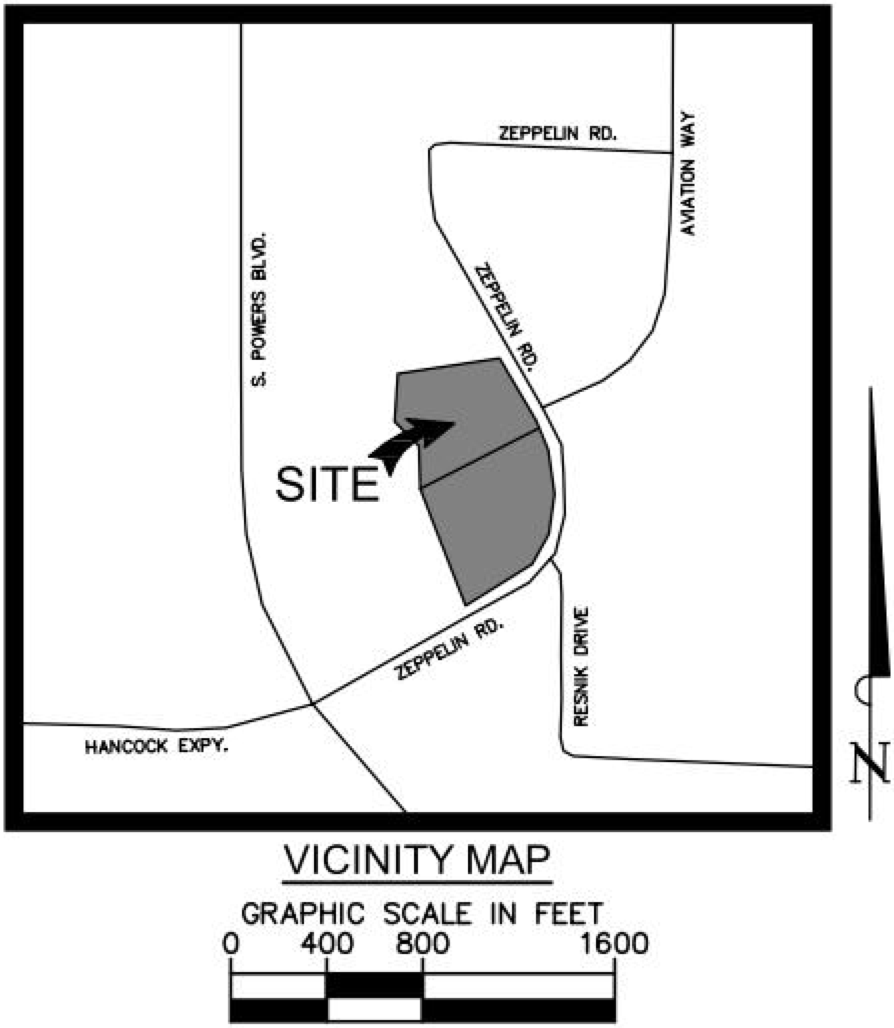
The proposed improvements consist of the construction of an approximately 131,000-gross square-foot, industrial warehouse/distribution building and parking lot (the "Project") within Lot 1 of the Property (the "Site"). Development of Lot 2 is not part of the Project. The Project will be processed through the City of Colorado Springs. Additional outside agency review or processing is not anticipated as part of the Project.

The Project is located within Township 14 South, Range 66 West of the Sixth Principal Meridian, City of Colorado Springs, County of El Paso, State of Colorado (see Vicinity Map). The Property is bounded by a regional detention pond to the west, undeveloped and unplatted land owned by Marla Nanninga to the north, and Zeppelin Road to the south and east. The Property is currently undeveloped and consists of a vacant field. The Property generally slopes down from the east to west with the anticipated stormwater outfall for both Lot 1 and Lot 2 being the existing Powers Boulevard Detention Facility (herein the "regional detention pond") to the west of the Property.

An ALTA and topographic field survey was completed for the Project by Clark Land Surveying Inc. dated March 30th 2017 and is the basis for design for the drainage improvements.

DEVELOPMENT DESIGN CRITERIA REFERENCE AND CONSTRAINTS

The proposed storm facilities are designed to be in compliance with the City of Colorado Springs Storm Drainage Criteria (the "CRITERIA") and the Urban Storm Drainage Criteria Manual (the "MANUAL"). Site drainage is not significantly impacted by such constraints as utilities or existing development.



PROJECT CHARACTERISTICS

The Property is centrally located along the southern boundary of the Peterson Field Drainage Basin. An approximate location of the Property within the major drainage basin is located in the Appendix. A minimal amount of off-site flows reaches the Site from the north and there is an existing 35' wide concrete channel along the southern boundary of the Property that conveys off-site flows east of Zeppelin Rd. The Project is in compliance with the approved DBPS.

Along the project frontage, Zeppelin Road slopes down from north to south at approximately 0.6%, the northern project boundary slopes from east to west at approximately 1.4%, the western project boundary slopes from north to south at approximately 0.3%, and the southern project boundary slopes from east to west at approximately 0.9%. This historic runoff pattern will be maintained and unaffected with the proposed Project. An existing conditions map is provided in the Appendix.

NRCS soil data is available for this Site and it has been noted that soils onsite are generally USCS Type A. There are no major drainage ways or irrigation facilities within the Site. The Site does not currently provide water quality or detention for the Project area. The existing land use is undeveloped vacant land the proposed land use is warehouse/distribution facility.

The proposed building, parking lot, paved drives, and other impervious surfaces comprise 62.8 percent (264,192 square feet) of the overall Project. Landscape areas internal to the site consist of landscape islands within the parking lot, and landscape zones within the building and landscape setback areas. The proposed internal landscaping areas make up 37.2 percent (156,730 square feet) of the Project. Landscape improvements (grass, tree lawns, etc.) are proposed along the project perimeter within the existing right-of-way.

The proposed drainage facilities for the Site are designed to follow the historic flow patterns of the Property as well as the intent of the original storm water design for the overall development. Please refer to the *Final Drainage Report for Broadview Business Park Filing No. 2 and No. 3*, dated May 28, 1986, (the "REPORT") for a full discussion of the original design for the subdivision. This report has been included in the Appendix for reference. As documented within the Final Drainage Plan in the REPORT, this proposed Project lies within Basin B-1 (the Property) of the original development. Drainage within Basin B-1 was designed to flow overland westward, with outfall into basin B-3 (the regional detention pond). Developed flows within this Project will collected via a proposed storm sewer system that will convey flows to the two proposed extended detention basins which will outfall directly to the regional detention pond.

The onsite flows were accounted for in the design of the regional detention pond as noted within the *Powers Boulevard Detention Facility Final Drainage Report*, dated April 13, 1990, (the "DETENTION REPORT"). The report has been included in the Appendix for reference. Per the DETENTION REPORT, the Site lies within sub-basin 3 and 5 which are included in the detention calculations for the regional detention pond. The DETENTION REPORT states that both water quality capture volume ("WQCV") and 100-year detention are provided within the regional detention pond. However, DETENTION REPORT states that a drain time of 24 hours was used for the release of the WQCV.

Additionally, the design plans for the regional detention pond have been included in the Appendix for reference. Sheet D4 of 15 depicts the 10-year and 100-year storage volumes and associated water surface elevations within the regional detention pond. This sheet also shows the location of the top of bank of the 100-year storage area and water quality pond. Sheet D8 of 15 shows the structural details of the outlet structure for the regional detention pond. The multi-stage outlet structure has three openings as follows:

- 18" RCP inlet pipe for water quality event
- 3'Wx10'Wx3.5'H trapezoidal opening for minor (10-year) event
- 10'x6' rectangular opening for major (100-year) event

The water quality capture volume (per current standards) will be provided for the Project by means of two extended water quality detention basins each with a water quality outlet structure. The water quality detention basins will be constructed along the western boundary of the Site and will only detain the proposed water quality capture volume. Detention for the minor (10-year) and major (100-year) storm events will be provided in the regional detention pond. The controlled 100-year release from the water quality detention basin will be piped to the regional detention pond west of the Site.

HYDROLOGIC ANALYSIS

MAJOR DRAINAGE BASIN DESCRIPTION

The Project is within the Peterson Field Drainage Basin. The major drainage basin is mostly developed. The Property is ultimately tributary to Sand Creek. Drainage facilities immediately downstream of this Site are in place including an existing City owned detention pond to the west of the Site. There are no known major irrigation facilities within 100 feet of the property.

EXISTING CONDITIONS SUB-BASIN DESCRIPTION

The existing run-off within the Property generally drains from east to west to the regional detention pond. Drainage analysis for the Property in it's existing condition was completed as part of the REPORT. Below is a description of the existing sub-basins and an existing conditions drainage plan is included in the Appendix.

Sub-Basin E1

Sub-basin E1 consists of the northern 9.66 acres of the property and is currently undeveloped vacant land. Drainage flows overland from east to west at approximately 2% to the existing regional detention pond. Runoff during the 5-year and 100-year events are 0.71 cfs and 10.96 cfs respectively.

Sub-Basin E2.

Sub-basin E2 consists of the northern 4.26 acres of the southern portion of the property and is currently undeveloped vacant land. Drainage flows overland from east to west at approximately 0.8% to the existing regional detention pond. Runoff during the 5-year and 100-year events are 0.31 cfs and 4.83 cfs respectively.

Sub-Basin E3

Sub-basin E3 consists of the southern 3.98 acres of the southern portion of the property and is currently undeveloped vacant land. Drainage flows overland from northeast to southwest at approximately 0.8% to an existing 35'W x 9'H concrete drainage channel that conveys offsite flow through the property along the southern boundary. Runoff during the 5-year and 100-year events are 2.82 cfs and 8.20 cfs respectively. The 100 year flood plain is contained within this channel and the approximate 100 year flow is 1750 cfs per the REPORT.

PROPOSED CONDITIONS SUB-BASIN DESCRIPTION

The developed runoff from the Project will generally be collected by means of private roof drains and storm sewer inlets located in the paved driveways within each delineated basin area. The runoff collected from each basin and the roof system of the proposed building will be conveyed to either of the two-proposed private water quality detention basins at the western edge of the Site. The controlled stormwater release from the water quality structure will be conveyed through a private 30" reinforced concrete storm sewer pipe (for the south water quality detention basin) and a 24" reinforced concrete storm sewer pipe (for the north water quality detention basin) to outfall into the existing City owned regional detention pond to the west of the Site. The regional detention pond is part of the existing public storm drainage system which conveys the released flows to the southwest, with ultimate outfall into Sand Creek.

The Property has been divided into eighteen sub-basins, A1-A10, R1-R4, and OS1-4. The runoff generated on the building roof area, sub-basins R1-R4, is collected and conveyed via a private roof drain system which outfalls to the proposed detention basins. Sub-basins A1-A10 are all internal areas within the parking lot and basin area. Each of the sub-basins drains to an inlet within the parking lot. A proposed conditions map is provided in the Appendix.

Sub-Basin R1-R4

Sub-basins R1-R4 consist of four equally divided sections (0.75 acres) of the rooftop of the proposed building. The runoff developed within these sub-basins is collected via building roof drains. These roof drains discharge to the underground storm sewer within the drive aisles. Developed runoff during the 5-year and 100-year events are each 3.20 cfs and 5.68 cfs respectively.

Sub-Basin A1

Sub-basin A1 is located at the northern boundary of the Site and consists of 1.70 acres of landscape area with a basin impervious value of 2%. Developed runoff for the 5-year and 100-year storm events are 0.13 and 1.93 cfs respectively and flows overland from east to west to the proposed northern water quality detention pond.

Sub-Basin A2

Sub-basin A2 is located west of the northern entrance to the Site and consists of 0.49 acres of mostly pavement area with a basin impervious value of 58.4%. Developed runoff for the 5-year and 100-year storm events are 1.28 and 2.48 cfs respectively and flows from east to west across the parking area to a 5' Type R inlet located at design point A2. Flows are conveyed via a private storm line to the proposed northern water quality detention pond.

Sub-Basin A3

Sub-basin A3 is located along the northern building elevation and consists of 0.64 acres of mostly pavement area with a basin impervious value of 87.8%. Developed runoff for the 5-year and 100-year storm events are 2.67 and 4.75 cfs respectively and flows from east to west across the parking area to a 5' Type R inlet located at design point A3. Flows are conveyed via a private storm line to the proposed northern water quality detention pond.

Sub-Basin A4

Sub-basin A4 is located along the western building elevation and consists of 0.51 acres of mostly pavement area with a basin impervious value of 65.4%. Developed runoff for the 5-year and 100-year storm events are 1.38 and 2.62 cfs respectively and flows from north to south across the emergency access drive aisle to a 5' Type R inlet located at design point A4. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basin A5

Sub-basin A5 is located at the southwest corner of the Site and consists of 0.20 acres of detention area with a basin impervious value of 2%. Developed runoff for the 5-year and 100-year storm events are 0.02 and 0.26 cfs respectively and flows from east to west to proposed CDOT Type C outlet structure at design point A7. Flows are conveyed via a private storm line to the existing regional detention pond to the west of the Site.

Sub-Basin A6

Sub-basin A6 is located along the southern building elevation on the west end of the building and consists of 0.75 acres of mostly pavement area with a basin impervious value of 68.9%.

Developed runoff for the 5-year and 100-year storm events are 2.39 and 4.46 cfs respectively and flows from the north and the south to center of the truck court to a CDOT Type C inlet located at design point A6. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basin A7

Sub-basin A7 is located along the southern building elevation at the center of the building and consists of 0.71 acres of mostly pavement area with a basin impervious value of 67.6%. Developed runoff for the 5-year and 100-year storm events are 1.85 and 3.47 cfs respectively and flows from the north and the south to center of the truck court to a CDOT Type C inlet located at design point A7. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basin A8

Sub-basin A8 is located along the southern building elevation at the east end of the building and consists of 1.19 acres of mostly pavement area with a basin impervious value of 89.7%. Developed runoff for the 5-year and 100-year storm events are 4.35 and 7.78 cfs respectively and flows from the north and the south to center of the truck court to a CDOT Type C inlet located at design point A8. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basin A9

Sub-basin A9 is located at the southeast corner of the building and consists of 0.25 acres of mostly landscape area with a basin impervious value of 9.7%. Developed runoff for the 5-year and 100-year storm events are 0.09 and 0.41 cfs respectively and flows from the northeast to the southwest to a 12”x12” landscape inlet located at design point A9. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basin A10

Sub-basin A10 is located along the eastern building elevation and consists of 0.15 acres of mostly landscape area with a basin impervious value of 2%. Developed runoff for the 5-year and 100-year storm events are 0.01 and 0.20 cfs respectively and flows from the north to the south and is collected by a series of 12”x12” landscape inlets. Flows are conveyed via a private storm line to the proposed southern water quality detention pond.

Sub-Basins OS1 and OS2

Sub-basins OS1 and OS2 are 0.25 and 0.05 acres respectively, and are located along the Site perimeter, between the edge of the parking lot and edge of the property. These two basins have negligible offsite flows due to their proximity to the Site perimeter and due to grades within these sub-basins.

Sub-Basins OS3 and OS4

Sub-basins OS3 and OS4 are 4.26 and 3.98 acres respectively, and are located in Lot 2 of the Broadview Business Park Filing No. 5 or the southern proposed Lot within the Property. Each of these basins will remain un-disturbed as part of the project and will follow historic drainage patterns. Sub-basin OS3 drains east to west and outfalls directly to the regional detention pond. Sub-basin OS4 drains southward to the existing 35’ wide concrete drainage channel. In the future, when Lot 2 is developed, both OS3 and OS4 will be detained in future water quality detention basins before they are releasing 100 year flows into the existing regional detention pond to the west of the Site. Conceptual water quality detention volumes have been calculated for Lot 2 which consists of 8.24 acres and an estimated proposed impervious value of 70%. Reference the appendices for they UD-Detention Basin calculations based on a conceptual design of Lot 2.

METHODOLOGY

The 5-year and 100-year design storm events were used in determining rainfall and runoff for the proposed drainage system per section 6 of the CRITERIA. Table 6-2 of the CRITERIA is the source for rainfall data for the 5-year and 100-year design storm events. Design runoff was calculated using the Rational Method for developed conditions as established in the CRITERIA and MANUAL. Runoff coefficients for the proposed development were determined using Table 6-6 of the MANUAL by calculating weighted impervious values for each specific Site basin. The water quality capture volume storage requirement was calculated using Full Spectrum Detention methods as specified in the CRITERIA and MANUAL. The water quality detention basin outlet structure was designed to release the Water Quality Capture Volume (WQCV) in 40 hours. Based upon this approach, the drainage design provided for the Site is conservative and in keeping with the zoning and historic drainage concept for the area. There are no additional provisions selected or deviations from the City of Colorado Springs Drainage Criteria, dated May 2014, for the proposed development.

HYDRAULIC ANALYSIS

MAJOR DRAINAGEWAYS

There is an existing 35' wide concrete drainage channel that runs along the southern boundary of the property. This channel conveys flows from areas east of Zeppelin Road, westward to the public storm system within Powers Boulevard. No changes or impacts to this channel are proposed with the Project.

METHODOLOGY

The proposed drainage facilities are designed in accordance with the CRITERIA and MANUAL. Floodplain identification was determined using FIRM panels by FEMA and information provided in the CRITERIA. Hydraulic calculations were computed using STORMCAD, which makes use of the Standard Step method to compute the hydraulic profile. Results of the hydraulic calculations are summarized in the Appendix. There are no additional provisions selected or deviations from the City of Colorado Springs Drainage Criteria, dated May 2014, for the proposed development.

Inlet capacity calculations have been provided in the Appendix for one 5' Type R Inlet and one CDOT Type C inlet. The capacity of each type of inlet is adequate for the 100 year developed flows for each sub-basin. Therefore, inlet calculations have not been provided for each individual inlet on Site.

The Project will consist of the removal of the onsite vegetation of native weeds, brush, grasses, and trees. The Project consists of an approximately 131,000 gross square foot, one-story industrial warehouse/distribution building and a surrounding parking lot. The Project will also provide associated utilities and landscaping.

In the pre-application meetings held with the City, it is understood that the water quality capture volume is required to be detained on-site at a minimum. As previously stated, review of the DETENTION REPORT reveals that detention for the proposed major and minor events is provided within the existing regional detention pond to the west of the Site, see Appendix. Water quality treatment will be provided by means of two (2) private extended water quality detention basins with water quality outlet structures. The water quality detention basins will be constructed along the western boundary of the Site. Each water quality detention pond is designed with an outlet structure that is fitted with a restrictor plate to release the WQCV in a 40 hour time period. The elevation of the top of each outlet structure is set at the WQCV water surface elevation. Therefore, any volume greater than the WQCV will flow into the outlet structure and will be piped directly to the regional detention pond. The outlet pipes are sized to be equal in diameter to the inflow pipes that enter the pond, thereby passing the developed 100 year flows through pond, directly to the regional detention pond to the west of the Site. The proposed onsite water quality detention ponds are designed to detain for the required WQCV only. The regional detention pond that each water quality detention pond outfalls into provides additional detention for the minor and major events.

The Site was designed in accordance with the four-step process to minimize adverse impacts of urbanization, as outlined in Chapter 1 Section 4.0 of the CRITERIA. Following step 1 of the process, the Site was designed to conserve as much of the existing vegetation as possible and to minimize the extent of paved areas. Wherever possible, impervious areas such as sidewalks and pavement, were designed to drain to pervious areas. Following step 2 of the process, the Site was designed to make use of an extended detention basin that would capture and slowly release the water quality capture volume. Following step 3 of the process, the extended water quality detention basin makes use of an engineered channel to prevent short circuiting and ensure proper settlement. The extended detention basin is naturally stabilized using vegetation and provides detained release of on-site flows. To ensure compliance with step 4 of the process, the erosion control features for both the initial and final stages of the Project were designed to reduce contamination. Source control BMPs include the use of vehicle tracking control, inlet protection, silt fences, concrete washout areas, stockpile management, and stabilized staging areas.

STRUCTURE CHARACTERISTICS

Detention Storage Required

Calculations included in the Appendix provide details regarding the private water quality detention basins design. The calculations include determination of the storage volumes required for full spectrum detention for the WQCV only, and allowable release rates. Overall, 0.075 acre-feet of water quality detention storage volume is required for the northern detention pond and the proposed basin provides 0.282 acre-feet of storage. Sub-basins A1-A3 and R1-R2 have a total area of 4.34 acres (51.6% imperviousness) contributing flow to the northern detention pond. Overall, 0.116 acre-feet of water quality detention storage volume is required for the southern detention pond and the proposed basin provides 0.505 acre-feet of storage. Sub-basins A4-A10 and R3-R4 have a total area of 5.02 acres (70.6% imperviousness) contributing flow to the southern detention pond. The required 5 year and 100 year detention volumes are 0.235 acre-feet and 0.544 acre-feet respectively for the north pond and 0.407 acre-feet and 0.814 acre-feet respectively for the south pond and will be detained within the regional detention pond as described throughout this report.

Outlet Requirements

The water quality standards established by the CRITERIA in section 13.5.10 are met by the proposed detention basin. The water quality outlet structures were designed per the specifications in section 13.5.10 of the CRITERIA. The structures meet the micro-pool requirement that it be integrated into the design of the structure with an additional initial surcharge volume. The orifice plates of the structures was designed based on section 13.4.2.2 of the CRITERIA. The orifice plates will allow the Water Quality Capture Volume to be drained from the structure in 40 hours. The calculations for the design of the water quality outlet structure are presented in the Appendix. Preliminary pond detail plans are provided for reference in the Appendix.

Storm Sewer Requirements

Calculations which determine the storm sewer capacity, type of flow, pipe losses, and hydraulic grade line calculations are included in the Appendix along with calculations which show outlet conditions and the protection design for the proposed system. The calculations meet City of Colorado Springs requirements as outlined in the CRITERIA.

Channel Design and Soil Erodibility

A proposed concrete lined trickle channel within the basin was designed per the CRITERIA. A forebay structure is located at the upstream entrance to the basin. This forebay structure was designed per the CRITERIA. The surrounding protection is designed as Type M riprap. Calculations detailing the design and dimensions of the trickle channel and forebay structure are included in the Appendix.

FLOODPLAINS

The Flood Insurance Rate Maps (FIRM) 08041C0761F effective date March 17, 1997, by FEMA, indicates that the Site is located in Zone X (outside of the 500-year flood plain) and Zone X shaded (within the 500-year flood plain). Revised by Letter of Map Revision (LOMR), Case No. 98-08-372P, dated December 14, 1999. This LOMR changes the 100-year flood plain in this area, containing it to the existing engineered concrete channel along the southern boundary of the adjacent property to the south. This panel and LOMR is included in the Appendix.

ENVIRONMENTAL EVALUATIONS

A Phase I Environmental Site Assessment was performed by Midwest Testing as part of the Project in which the assessment revealed “no evidence of recognized environmental conditions, controlled recognized environmental conditions, or historical recognized environmental conditions in connection with the subject property.”

EROSION CONTROL PLAN

An initial and final erosion control plan was developed for this site per local requirements. The construction drawings have been submitted as a separate stand along set. Below is a brief description of some of the BMPs proposed in those plans.

For the initial erosion control plan, two (2) sediment basins have been provided in the same proposed locations as the final detention ponds. Because the site drains from east to west, a diversion swale has been proposed along the west property line to direct the flows to either of these basins. Both of these basins have been designed with an emergency spillway that would direct flow to the regional detention pond to the west. The design for each pond also includes an outfall pipe that directs flow from the ponds to the regional detention pond to the west. There is also a proposed vehicle traction control located on the northeast corner of the site along Zeppelin Road. There is also a soil stockpile, concrete washout, and stabilized staging area in the northeast section of the site near the construction entrance. Silt fence has been proposed along the west edge of the property and portions of the north and south edges to protect adjacent land.

The final erosion control plan utilizes the same silt fence as from the initial design as the drainage patterns on the edges of the site are not proposed to change with final design. Permanent stabilization is proposed along all edges of the property where there is proposed seeding and mulching. Poa Pratensis (Kentucky bluegrass) is the primary ground cover proposed onsite. The area north of the parking on the north side of the building, south side of the truck court, and all landscape islands will be permanently stabilized with Kentucky bluegrass. The slopes and bottoms of both sediment basins will be stabilized with a detention basin mix by Applewood seed. Reference landscape plans for complete permanent stabilization details.

FEES DEVELOPMENT

DRAINAGE AND BRIDGE FEES

The drainage basin and bridge fees were waived with the platting of the Broadview Business Park Filing No. 2 per the REPORT. See Appendix.

CONSTRUCTION COST OPINION

An opinion of probable construction cost for the construction of the private drainage facilities for the Project has been included in the Appendix. There are no public drainage facilities proposed as part of the Project.

MAINTENANCE AND OPERATIONS

It is our recommendation that the detention basins maintenance cycles consist of twice per year inspections (spring and fall), evaluation of sedimentation within the basins, and removal of sediment if levels exceed two inches deep or if discharge is otherwise deemed insufficient. This satisfies the maintenance and access requirement set by the CRITERIA.

GROUNDWATER CONSIDERATIONS

During Site exploration, groundwater was not encountered. The proposed Project excavation consists of excavation for foundations at a depth of no more than 5 feet below existing grade with excavations for the detention basin at depths of no more than 15 feet below existing grade. Groundwater is not anticipated to be an issue.

A perimeter drain system will not be provided for this Project.

CONCLUSIONS

COMPLIANCE WITH STANDARDS

The drainage design presented within this report for Broadview Business Park Filing No. 5, conforms to the City of Colorado Springs Storm Drainage Criteria and the Urban Drainage and Flood Control District Manual. Additionally, the Site runoff and storm drain facilities will not adversely affect the downstream and surrounding developments. This report and its findings are consistent with the drainage requirements documented in the Broadview Business Park Filing

No. 2 and 3 drainage report and in general conformance with the DBPS.

REFERENCES

1. City of Colorado Springs Drainage Criteria Manual, May 2014.
2. Urban Drainage and Flood Control District Drainage Criteria Manual (UDFCDCM), Vol. 1, prepared by Wright-McLaughlin Engineers, June 2001, with latest revisions.
3. Flood Insurance Rate Map, El Paso County, Colorado and Incorporated Areas, Map Number 08041C0509F, Effective Date March 17, 1997, prepared by the Federal Emergency Management Agency (FEMA).
4. Flood Insurance Rate Map, El Paso County, Colorado and Incorporated Areas, Map Number 08041C0517F, Effective Date March 17, 1997, prepared by the Federal Emergency Management Agency (FEMA).

APPENDIX

SITE DRAINAGE CALCULATIONS

$$I = \frac{28.5\,P_1}{(10+T_D)^{0.786}}$$

Where:

I = rainfall intensity (inches per hour)

P_1 = one-hour rainfall depth (inches) from Table 6-2 One-hour Point Rainfall D
City of Colorado Springs Drainage Design

T_C = storm duration (minutes)

	2-yr	5-yr	10-yr	100-yr
P_1 =	1.19	1.50	1.75	2.52

Time Intensity Frequency Tabulation

TIME	2 YR	5 YR	10 YR	100 YR
5	4.04	5.09	5.94	8.55
10	3.22	4.06	4.73	6.82
15	2.70	3.41	3.97	5.72
30	1.87	2.35	2.75	3.95
60	1.20	1.52	1.77	2.55
120	0.74	0.93	1.09	1.57

Weighted Imperviousness Calculations

SUB-	AREA	AREA	ROOF	ROOF	ROOF				LANDSCAPE	LANDSCAPE	LANDSCAPE				PAVEMENT	PAVEMENT	PAVEMENT				WEIGHTED	WEIGHTED COEFFICIENTS			
BASIN	(SF)	(Acres)	AREA	PERVIOUSNE	C2	C5	C10	C100	AREA	IMPERVIOUSNESS	C2	C5	C10	C100	AREA	IMPERVIOUSNESS	C2	C5	C10	C100	IMPERVIOUSNESS	C2	C5	C10	C100
E1	420,923	9.66	0	90%	0.80	0.84	0.85	0.88	420,923	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
E2	185,423	4.26	0	90%	0.80	0.84	0.85	0.88	185,423	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
E3	173,296	3.98	0	90%	0.80	0.84	0.85	0.88	142,897	2%	0.02	0.02	0.02	0.17	30,399	100%	0.89	0.93	0.94	0.96	19.2%	0.17	0.18	0.18	0.31
TOTAL	853,679	19.60	0	90%	0.80	0.84	0.85	0.88	798,280	2%	0.02	0.02	0.02	0.17	55,399	100%	0.89	0.93	0.94	0.96	8.4%	0.07	0.08	0.08	0.22

2570 Zeppelin Road - Drainage Report
Existing Runoff Calculations
Time of Concentration

Watercourse Coefficient					
Forest & Meadow	2.50	Short Grass Pasture & Lawns	7.00	Grassed Waterway	15.00
Fallow or Cultivation	5.00	Nearly Bare Ground	10.00	Paved Area & Shallow Gutter	20.00

	SUB-BASIN DATA				INITIAL / OVERLAND TIME			TRAVEL TIME T(t)					T(c) CHECK (URBANIZED BASINS)			FINAL T(c)
DESIGN POINT	DRAIN BASIN	AREA sq. ft.	AREA ac.	C(5)	Length ft.	Slope %	T(i) min	Length ft.	Slope %	Coeff.	Velocity fps	T(t) min.	COMP. T(c)	TOTAL LENGTH	L/180+10	min.
E1	E1	420,923	9.66	0.02	100	2.0%	15.7	0	0.0%	37.00	0.0	0.0	15.7	100	10.6	10.6
E2	E2	185,423	4.26	0.02	100	0.8%	21.8	0	0.0%	42.00	0.0	0.0	21.8	100	10.6	10.6
E3	E3	173,296	3.98	0.18	100	0.8%	18.6	0	0.0%	43.00	0.0	0.0	18.6	100	10.6	10.6

2570 Zeppelin Road - Drainage Report
Existing Runoff Calculations
(Rational Method Procedure)

Design Storm *5 Year*

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	
E1	E1	9.66	0.02	10.6	0.18	3.96	0.71					Lot 1
E2	E2	4.26	0.02	10.6	0.08	3.96	0.31					north portion of Lot 2
E3	E3	3.98	0.18	10.6	0.71	3.96	2.82					south portion of Lot 2

2570 Zeppelin Road - Drainage Report

Existing Runoff Calculations

Design Storm 100 Year

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	
E1	E1	9.66	0.17	10.6	1.64	6.66	10.96					Lot 1
E2	E2	4.26	0.17	10.6	0.72	6.66	4.83					north portion of Lot 2
E3	E3	3.98	0.31	10.6	1.23	6.66	8.20					south portion of Lot 2

2570 Zeppelin Road - Drainage Report

Existing Runoff Calculations *Design Storm* *2 Year*

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	
E1	E1	9.663	0.02	10.6	0.17	3.15	0.5					Lot 1
E2	E2	4.257	0.02	10.6	0.08	3.15	0.2					north portion of Lot 2
E3	E3	3.978	0.17	10.6	0.68	3.15	2.1					south portion of Lot 2

SUMMARY - EXISTING RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)	CUMULATIVE 5-YR RUNOFF (CFS)	CUMULATIVE 100-YR RUNOFF (CFS)
E1	E1	9.66	0.71	10.96	0.71	10.96
E2	E2	4.26	0.31	4.83	0.31	4.83
E3	E3	3.98	2.82	8.20	2.82	8.20

Table 6-6. Runoff coefficient equations based on NRCS soil group and storm return period

NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	$C_A = 0.89i$	$C_A = 0.93i$	$C_A = 0.94i$	$C_A = 0.944i$	$C_A = 0.95i$	$C_A = 0.81i + 0.154$
B	$C_B = 0.89i$	$C_B = 0.93i$	$C_B = 0.81i + 0.125$	$C_B = 0.70i + 0.23$	$C_B = 0.59i + 0.364$	$C_B = 0.49i + 0.454$
C/D	$C_{C/D} = 0.89i$	$C_{C/D} = 0.87i + 0.052$	$C_{C/D} = 0.74i + 0.2$	$C_{C/D} = 0.64i + 0.31$	$C_{C/D} = 0.54i + 0.418$	$C_{C/D} = 0.45i + 0.508$

ROOF						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.80	0.84	0.85	0.85	0.86	0.88
B						
C/D						

I (%)	
ROOF	90.00%
LANDSCAPE	2.00%
PAVEMENT	100.00%

Soil Type
A
B
C/D

LANDSCAPE						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.02	0.02	0.02	0.02	0.02	0.17
B						
C/D						

PAVEMENT						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.89	0.93	0.94	0.94	0.95	0.96
B						
C/D						

$$I = \frac{28.5\, P_1}{(10+T_D)^{0.786}}$$

Where:

I = rainfall intensity (inches per hour)

P_1 = one-hour rainfall depth (inches) from Table 6-2 One-hour Point Rainfall D
City of Colorado Springs Drainage Design

T_C = storm duration (minutes)

	2-yr	5-yr	10-yr	100-yr
P_1 =	1.19	1.50	1.75	2.52

Time Intensity Frequency Tabulation

TIME	2 YR	5 YR	10 YR	100 YR
5	4.04	5.09	5.94	8.55
10	3.22	4.06	4.73	6.82
15	2.70	3.41	3.97	5.72
30	1.87	2.35	2.75	3.95
60	1.20	1.52	1.77	2.55
120	0.74	0.93	1.09	1.57

Weighted Imperviousness Calculations

SUB-BASIN	AREA (SF)	AREA (Acres)	ROOF AREA	ROOF PERVIOUSNE	ROOF C2	ROOF C5	ROOF C10	ROOF C100	LANDSCAPE AREA	LANDSCAPE IMPERVIOUSNESS	LANDSCAPE C2	LANDSCAPE C5	LANDSCAPE C10	LANDSCAPE C100	PAVEMENT AREA	PAVEMENT IMPERVIOUSNESS	PAVEMENT C2	PAVEMENT C5	PAVEMENT C10	PAVEMENT C100	WEIGHTED IMPERVIOUSNESS	WEIGHTED COEFFICIENTS C2	WEIGHTED COEFFICIENTS C5	WEIGHTED COEFFICIENTS C10	WEIGHTED COEFFICIENTS C100
A1	74,037	1.70	0	90%	0.80	0.84	0.85	0.88	74,037	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
A2	21,445	0.49	0	90%	0.80	0.84	0.85	0.88	9,097	2%	0.02	0.02	0.02	0.17	12,348	100%	0.89	0.93	0.94	0.96	58.4%	0.52	0.54	0.55	0.63
A3	27,974	0.64	0	90%	0.80	0.84	0.85	0.88	3,495	2%	0.02	0.02	0.02	0.17	24,479	100%	0.89	0.93	0.94	0.96	87.8%	0.78	0.82	0.82	0.86
A4	22,124	0.51	0	90%	0.80	0.84	0.85	0.88	7,808	2%	0.02	0.02	0.02	0.17	14,316	100%	0.89	0.93	0.94	0.96	65.4%	0.58	0.61	0.61	0.68
A5	8,855	0.20	0	90%	0.80	0.84	0.85	0.88	8,855	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
A6	32,597	0.75	0	90%	0.80	0.84	0.85	0.88	10,337	2%	0.02	0.02	0.02	0.17	22,260	100%	0.89	0.93	0.94	0.96	68.9%	0.61	0.64	0.65	0.71
A7	25,222	0.58	0	90%	0.80	0.84	0.85	0.88	8,352	2%	0.02	0.02	0.02	0.17	16,870	100%	0.89	0.93	0.94	0.96	67.5%	0.60	0.63	0.63	0.70
A8	47,060	1.08	0	90%	0.80	0.84	0.85	0.88	7,187	2%	0.02	0.02	0.02	0.17	39,873	100%	0.89	0.93	0.94	0.96	85.0%	0.76	0.79	0.80	0.84
A9	10,681	0.25	0	90%	0.80	0.84	0.85	0.88	9,844	2%	0.02	0.02	0.02	0.17	837	100%	0.89	0.93	0.94	0.96	9.7%	0.09	0.09	0.09	0.23
A10	6,538	0.15	0	90%	0.80	0.84	0.85	0.88	6,538	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
R1	32,760	0.75	32,760	90%	0.80	0.84	0.85	0.88	0	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	90.0%	0.80	0.84	0.85	0.88
R2	32,760	0.75	32,760	90%	0.80	0.84	0.85	0.88	0	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	90.0%	0.80	0.84	0.85	0.88
R3	32,760	0.75	32,760	90%	0.80	0.84	0.85	0.88	0	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	90.0%	0.80	0.84	0.85	0.88
R4	32,760	0.75	32,760	90%	0.80	0.84	0.85	0.88	0	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	90.0%	0.80	0.84	0.85	0.88
OS1	10,972	0.25	0	90%	0.80	0.84	0.85	0.88	10,972	2%	0.02	0.02	0.02	0.17	0	100%	0.89	0.93	0.94	0.96	2.0%	0.02	0.02	0.02	0.17
OS2	2,377	0.05	0	90%	0.80	0.84	0.85	0.88	0	2%	0.02	0.02	0.02	0.17	2,377	100%	0.89	0.93	0.94	0.96	100.0%	0.89	0.93	0.94	0.96
OS3	185,423	4.26	85,000	90%	0.80	0.84	0.85	0.88	48,000	2%	0.02	0.02	0.02	0.17	52,423	100%	0.89	0.93	0.94	0.96	70.0%	0.62	0.65	0.66	0.72
OS4	173,296	3.98	75,000	90%	0.80	0.84	0.85	0.88	45,400	2%	0.02	0.02	0.02	0.17	52,896	100%	0.89	0.93	0.94	0.96	70.0%	0.62	0.65	0.66	0.72
TOTAL	853,679	19.60	291,040	90%	0.80	0.84	0.85	0.88	298,959	2%	0.02	0.02	0.02	0.17	263,679	100%	0.89	0.93	0.94	0.96	62.3%	0.55	0.58	0.59	0.66
POND A (A1-A3,R1-R2)	188,976	4.34	65,520	90%	0.80	0.84	0.85	0.88	86,628	2%	0.02	0.02	0.02	0.17	36,827	100%	0.89	0.93	0.94	0.96	51.6%	0.46	0.48	0.49	0.57
POND B (A4-A10,R3-R4)	218,598	5.02	65,520	90%	0.80	0.84	0.85	0.88	58,922	2%	0.02	0.02	0.02	0.17	94,156	100%	0.89	0.93	0.94	0.96	70.6%	0.63	0.66	0.66	0.73

2570 Zeppelin Road - Drainage Report

Proposed Runoff Calculations

Time of Concentration

Watercourse Coefficient

Forest & Meadow	2.50	Fallow or Cultivation	5.00
Short Grass Pasture & Lawns	7.00	Nearly Bare Ground	10.00
Grassed Waterway	15.00	Paved Area & Shallow Gutter	20.00

	SUB-BASIN DATA				INITIAL / OVERLAND TIME			TRAVEL TIME T(t)					T(c) CHECK (URBANIZED BASINS)			FINAL T(c)
DESIGN POINT	DRAIN BASIN	AREA sq. ft.	AREA ac.	C(5)	Length ft.	Slope %	T(i) min	Length ft.	Slope %	Coeff.	Velocity fps	T(t) min.	COMP. T(c)	TOTAL LENGTH	L/180+10	min.
A1	A1	74,037	1.70	0.02	100	3.0%	13.7	0	1.0%	20.00	2.0	0.0	13.7	100	10.6	10.6
A2	A2	21,445	0.49	0.54	60	4.2%	4.9	207	1.5%	21.00	2.6	1.3	6.2	266.8	11.5	6.2
A3	A3	27,974	0.64	0.82	77	3.2%	3.1	80	0.5%	277.74	20.4	0.1	5.0	156.69	10.9	5.0
A4	A4	22,124	0.51	0.61	58	2.8%	4.8	390	1.0%	23.00	2.3	2.8	7.6	447.99	12.5	7.6
A5	A5	8,855	0.20	0.02	73	11.6%	7.5	35	0.8%	24.00	2.1	0.3	7.8	108	10.6	7.8
A6	A6	32,597	0.75	0.64	81	3.1%	5.2	48	2.6%	25.00	4.0	0.2	5.4	129.1	10.7	5.4
A7	A7	25,222	0.58	0.63	75	4.8%	4.4	58	2.2%	26.00	3.8	0.2	5.0	132.51	10.7	5.0
A8	A8	47,060	1.08	0.79	53	4.9%	2.4	145	3.8%	27.00	5.3	0.5	5.0	198.2	11.1	5.0
A9	A9	10,681	0.25	0.09	68	6.0%	8.4	67	1.3%	28.00	3.2	0.3	8.7	135.09	10.8	8.7
A10	A10	6,538	0.15	0.02	15	1.9%	6.2	39	2.0%	29.00	4.1	0.2	6.4	54	10.3	6.4
R1	R1	32,760	0.75	0.84	100	2.0%	3.8	0	1.0%	31.00	3.1	0.0	5.0	100	10.6	5.0
R2	R2	32,760	0.75	0.84	100	2.0%	3.8	0	1.0%	32.00	3.2	0.0	5.0	100	10.6	5.0
R3	R3	32,760	0.75	0.84	100	2.0%	3.8	0	1.0%	33.00	3.3	0.0	5.0	100	10.6	5.0
R4	R4	32,760	0.75	0.84	100	2.0%	3.8	0	1.0%	34.00	3.4	0.0	5.0	100	10.6	5.0
OS1	OS1	10,972	0.25	0.02	42	11.2%	5.7	0	0.0%	35.00	0.0	0.0	5.7	42	10.2	5.7
OS2	OS2	2,377	0.05	0.93	59	2.4%	1.8	0	0.0%	37.00	0.0	0.0	5.0	59.38	10.3	5.0
OS3	OS3	185,423	4.26	0.65	100	0.8%	9.0	0	0.0%	42.00	0.0	0.0	9.0	100	10.6	9.0
OS4	OS4	173,296	3.98	0.65	100	0.8%	9.0	0	0.0%	43.00	0.0	0.0	9.0	100	10.6	9.0

2570 Zeppelin Road - Drainage Report

Proposed Runoff Calculations — *Design Storm 5 Year*

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	1.70	0.02	10.6	0.03	3.96	0.13					Landscape area north of parking, outfall at north detention pond.
A2	A2	0.49	0.54	6.2	0.27	4.78	1.28	5.0	0.90	5.09	4.56	A2, R2
A3	A3	0.64	0.82	5.0	0.52	5.09	2.67	5.0	2.05	5.09	10.43	A2,A3,R1,R2
A4	A4	0.51	0.61	7.6	0.31	4.48	1.38	5.0	3.29	5.09	16.74	A4,A6,A7,A8,A9,A10, R3,R4
A5	A5	0.20	0.02	7.8	0.00	4.45	0.02					
A6	A6	0.75	0.64	5.4	0.48	4.98	2.39	5.0	2.98	5.09	15.17	A6,A7,A8,A9,A10, R3, R4
A7	A7	0.58	0.63	5.0	0.36	5.09	1.85	5.0	1.87	5.09	9.53	A7,A8,A9,A10,R4
A8	A8	1.08	0.79	5.0	0.85	5.09	4.35	5.0	1.51	5.09	7.68	A8,A9,A10,R4
A9	A9	0.25	0.09	8.7	0.02	4.27	0.09	6.4	0.02	4.75	0.12	A9,A10
A10	A10	0.15	0.02	6.4	0.00	4.75	0.01					
R1	R1	0.75	0.84	5.0	0.63	5.09	3.20					1/4 of Roof
R2	R2	0.75	0.84	5.0	0.63	5.09	3.20					1/4 of Roof
R3	R3	0.75	0.84	5.0	0.63	5.09	3.20					1/4 of Roof
R4	R4	0.75	0.84	5.0	0.63	5.09	3.20					1/4 of Roof
OS1	OS1	0.25	0.02	5.7	0.00	4.91	0.02					Section west of fire acess road drains towards drainage channel off-site
OS2	OS2	0.05	0.93	5.0	0.05	5.09	0.26					Section east of the building and drains towards the street
OS3	OS3	4.26	0.65	9.0	2.77	4.23	11.72					Runoff on parcel south of the proposed site
OS4	OS4	3.98	0.65	9.0	2.59	4.23	10.94					Runoff on parcel south of the proposed site

2570 Zeppelin Road - Drainage Report
Proposed Runoff Calculations
(Rational Method Procedure)

Design Storm *100 Year*

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	NOTES
A1	A1	1.70	0.17	10.6	0.29	6.66	1.93					Landscape area north of parking, outfall at north detention pond.
A2	A2	0.49	0.63	6.2	0.31	8.03	2.48	5.0	0.97	8.55	8.32	A2, R2
A3	A3	0.64	0.86	5.0	0.56	8.55	4.75	5.0	2.19	8.55	18.74	A2,A3,R1,R2
A4	A4	0.51	0.68	7.6	0.35	7.53	2.62	5.0	3.61	8.55	30.83	A4,A6,A7,A8,A9,A10, R3,R4
A5	A5	0.20	0.17	7.8	0.03	7.48	0.26					
A6	A6	0.75	0.71	5.4	0.53	8.37	4.46	5.0	3.26	8.55	27.87	A6,A7,A8,A9,A10, R3, R4
A7	A7	0.58	0.70	5.0	0.41	8.55	3.47	5.0	2.06	8.55	17.63	A7,A8,A9,A10,R4
A8	A8	1.08	0.84	5.0	0.91	8.55	7.78	5.0	1.66	8.55	14.16	A8,A9,A10,R4
A9	A9	0.25	0.23	8.7	0.06	7.17	0.41	6.4	0.08	7.98	0.66	A9,A10
A10	A10	0.15	0.17	6.4	0.03	7.98	0.20					0.00
R1	R1	0.75	0.88	5.0	0.66	8.55	5.68					1/4 of Roof
R2	R2	0.75	0.88	5.0	0.66	8.55	5.68					1/4 of Roof
R3	R3	0.75	0.88	5.0	0.66	8.55	5.68					1/4 of Roof
R4	R4	0.75	0.88	5.0	0.66	8.55	5.68					1/4 of Roof
OS1	OS1	0.25	0.17	5.7	0.04	8.25	0.35					Section west of fire acess road drains towards drainage channel off-site
OS2	OS2	0.05	0.96	5.0	0.05	8.55	0.45					Section east of the building and drains towards the street
OS3	OS3	4.26	0.72	9.0	3.07	7.10	21.80					Runoff on parcel south of the proposed site
OS4	OS4	3.98	0.72	9.0	2.87	7.10	20.36					Runoff on parcel south of the proposed site

2570 Zeppelin Road - Drainage Report

Proposed Runoff Calculations *Design Storm* *10 Year*

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				CUMMULATIVE RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	1.7	0.02	10.6	0.03	4.63	0.15					Landscape area north of parking, outfall at north detention pond.
A2	A2	0.492	0.55	6.2	0.27	5.58	1.51	5.0	0.91	4.04	3.66	A2, R2
A3	A3	0.642	0.82	5.0	0.53	5.94	3.14	5.0	2.07	4.04	8.37	A2,A3,R1,R2
A4	A4	0.508	0.61	7.6	0.31	5.23	1.63	5.0	3.33	4.04	13.42	A4,A6,A7,A8,A9,A10, R3,R4
A5	A5	0.203	0.02	7.8	0.00	5.19	0.02					
A6	A6	0.748	0.65	5.4	0.48	5.81	2.82	5.0	3.01	4.04	12.16	A6,A7,A8,A9,A10, R3, R4
A7	A7	0.579	0.63	5.0	0.37	5.94	2.18	5.0	1.89	4.04	7.64	A7,A8,A9,A10,R4
A8	A8	1.08	0.80	5.0	0.86	5.94	5.13	5.0	1.52	4.04	6.16	A8,A9,A10,R4
A9	A9	0.245	0.09	8.7	0.02	4.98	0.11	6.4	0.03	3.77	0.09	A9,A10
A10	A10	0.15	0.02	6.4	0.00	5.54	0.02					0.00
R1	R1	0.752	0.85	5.0	0.64	5.94	3.78					1/4 of Roof
R2	R2	0.752	0.85	5.0	0.64	5.94	3.78					1/4 of Roof
R3	R3	0.752	0.85	5.0	0.64	5.94	3.78					1/4 of Roof
R4	R4	0.752	0.85	5.0	0.64	5.94	3.78					1/4 of Roof
OS1	OS1	0.252	0.02	5.7	0.00	5.73	0.03					Section west of fire acess road drains towards drainage channel off-site
OS2	OS2	0.055	0.94	5.0	0.05	5.94	0.30					Section east of the building and drains towards the street
OS3	OS3	4.257	0.66	9.0	2.80	4.93	13.82					Runoff on parcel south of the proposed site
OS4	OS4	3.978	0.66	9.0	2.62	4.93	12.90					Runoff on parcel south of the proposed site

SUMMARY - PROPOSED RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)	CUMULATIVE 5-YR RUNOFF (CFS)	CUMULATIVE 100-YR RUNOFF (CFS)
A1	A1	1.70	0.13	1.93	0.13	1.93
A2	A2	0.49	1.28	2.48	4.56	8.32
A3	A3	0.64	2.67	4.75	10.43	18.74
A4	A4	0.51	1.38	2.62	16.74	30.83
A5	A5	0.20	0.02	0.26	0.02	0.26
A6	A6	0.75	2.39	4.46	15.17	27.87
A7	A7	0.58	1.85	3.47	9.53	17.63
A8	A8	1.08	4.35	7.78	7.68	14.16
A9	A9	0.25	0.09	0.41	0.12	0.66
A10	A10	0.15	0.01	0.20	0.01	0.20
R1	R1	0.75	3.20	5.68	3.20	5.68
R2	R2	0.75	3.20	5.68	3.20	5.68
R3	R3	0.75	3.20	5.68	3.20	5.68
R4	R4	0.75	3.20	5.68	3.20	5.68
OS1	OS1	0.25	0.02	0.35	0.02	0.35
OS2	OS2	0.05	0.26	0.45	0.26	0.45
OS3	OS3	4.26	11.72	21.80	11.72	21.80
OS4	OS4	3.98	10.94	20.36	10.94	20.36

Table 6-6. Runoff coefficient equations based on NRCS soil group and storm return period

NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	$C_A = 0.89i$	$C_A = 0.93i$	$C_A = 0.94i$	$C_A = 0.944i$	$C_A = 0.95i$	$C_A = 0.81i + 0.154$
B	$C_B = 0.89i$	$C_B = 0.93i$	$C_B = 0.81i + 0.125$	$C_B = 0.70i + 0.23$	$C_B = 0.59i + 0.364$	$C_B = 0.49i + 0.454$
C/D	$C_{C/D} = 0.89i$	$C_{C/D} = 0.87i + 0.052$	$C_{C/D} = 0.74i + 0.2$	$C_{C/D} = 0.64i + 0.31$	$C_{C/D} = 0.54i + 0.418$	$C_{C/D} = 0.45i + 0.508$

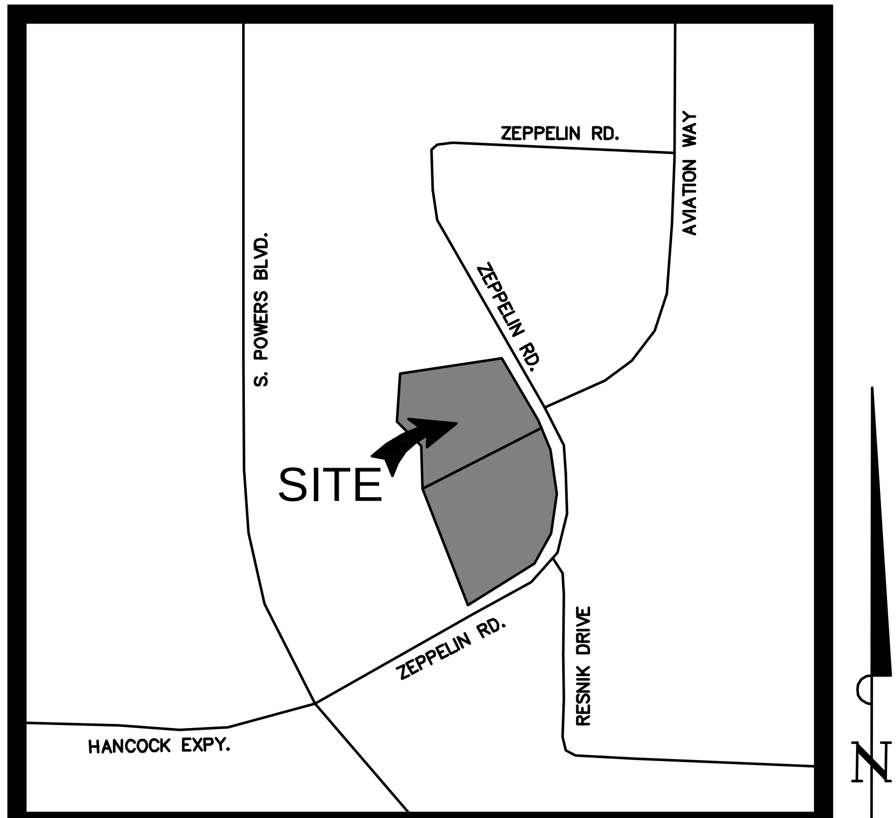
ROOF						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.80	0.84	0.85	0.85	0.86	0.88
B						
C/D						

LANDSCAPE						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.02	0.02	0.02	0.02	0.02	0.17
B						
C/D						

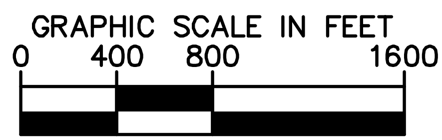
PAVEMENT						
NRCS Soil Group	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	0.89	0.93	0.94	0.94	0.95	0.96
B						
C/D						

I (%)	
ROOF	90.00%
LANDSCAPE	2.00%
PAVEMENT	100.00%

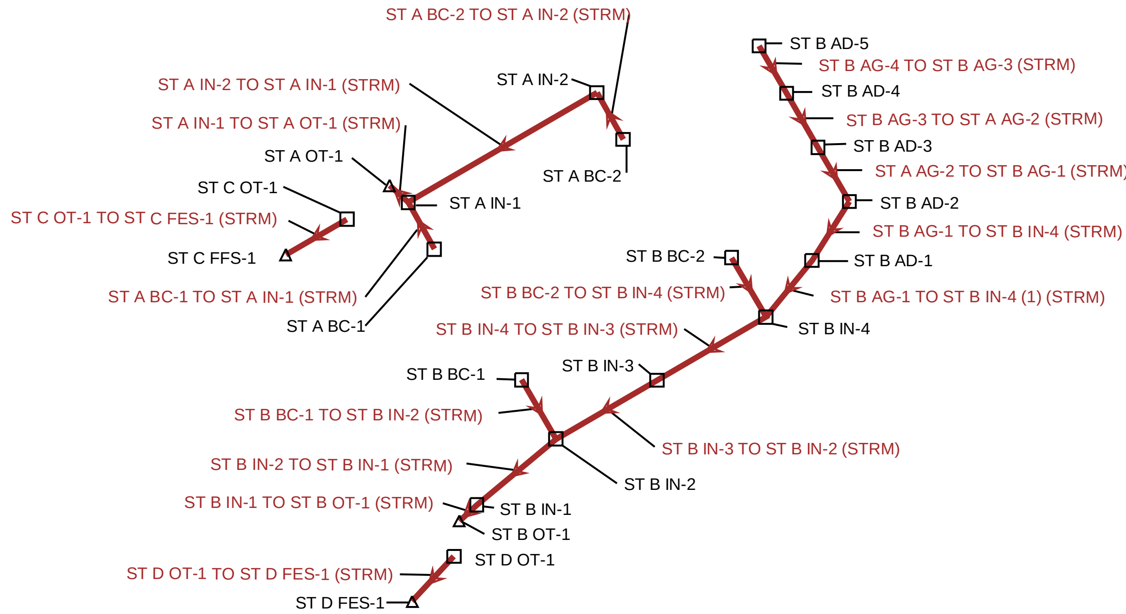
Soil Type
A
B
C/D



VICINITY MAP



Scenario: 100-YR



FlexTable: Catch Basin Table 5-YR

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Flow (Total Out) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Method	Headloss Coefficient (Standard)	Capture Efficiency (%)
ST A BC-2	6,000.25	5,999.01	3.20	6,000.36	5,999.78	Standard	1.52	100
ST A IN-2	6,002.18	5,997.44	4.48	5,998.55	5,998.18	Standard	1.32	90
ST A IN-1	6,000.55	5,994.29	10.35	5,996.28	5,995.44	Standard	1.77	90
ST A BC-1	6,000.25	5,999.01	3.20	6,000.36	5,999.78	Standard	1.52	100
ST B IN-1	5,999.41	5,991.67	16.47	5,993.07	5,993.04	Standard	0.05	90
ST B IN-2	5,998.05	5,992.37	15.09	5,994.60	5,993.68	Standard	1.77	50
ST B IN-3	5,998.15	5,993.65	9.50	5,994.80	5,994.78	Standard	0.05	50
ST B IN-4	5,998.45	5,994.71	7.65	5,996.38	5,995.69	Standard	1.77	50
ST B AD-1	6,000.62	5,996.71	0.10	5,996.84	5,996.84	Standard	0.05	50
ST B AD-2	6,002.12	5,997.62	0.01	5,997.68	5,997.67	Standard	0.64	50
ST B AD-3	6,003.42	5,998.34	0.01	5,998.38	5,998.38	Standard	0.05	50
ST B AD-4	6,003.42	5,999.19	0.01	5,999.23	5,999.23	Standard	0.05	50
ST B AD-5	6,004.00	5,999.95	0.01	5,999.98	5,999.98	Standard	0.00	50
ST C OT-1	5,995.29	5,991.01	10.35	5,994.16	5,994.16	Standard	0.00	100
ST D OT-1	5,992.80	5,988.01	16.46	5,991.38	5,991.38	Standard	0.00	100
ST B BC-1	6,000.25	5,999.01	3.20	6,000.36	5,999.78	Standard	1.52	100
ST B BC-2	5,998.56	5,997.32	3.20	5,998.67	5,998.09	Standard	1.52	100

FlexTable: Conduit Table 5-YR

Label	Invert (Start) (ft)	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Capacity (Full Flow) (cfs)	Flow / Capacity (Design) (%)	Upstream Structure Headloss Coefficient	Notes
ST B AG-4 TO ST B AG-3 (STRM)	5,999.95	5,999.19	87	0.009	12.0	0.010	0.01	0.95	4.33	0	0.00	12" RCP
ST B AG-3 TO ST A AG-2 (STRM)	5,999.19	5,998.34	85	0.010	12.0	0.010	0.01	1.13	4.63	0	0.05	12" RCP
ST A AG-2 TO ST B AG-1 (STRM)	5,998.34	5,997.62	72	0.010	12.0	0.010	0.01	1.24	4.63	0	0.05	12" RCP
ST A IN-2 TO ST A IN-1 (STRM)	5,997.44	5,994.48	297	0.010	24.0	0.013	4.48	5.60	22.58	20	1.32	24" RCP
ST A BC-2 TO ST A IN-2 (STRM)	5,999.01	5,998.01	73	0.014	12.0	0.010	3.20	7.19	5.42	59	1.52	12" RCP
ST B AG-1 TO ST B IN-4 (STRM)	5,997.62	5,996.71	92	0.010	12.0	0.010	0.01	1.32	4.61	0	0.64	12" RCP
ST B AG-1 TO ST B IN-4 (1) (STRM)	5,996.71	5,995.71	99	0.010	12.0	0.010	0.10	2.43	4.64	2	0.05	12" RCP
ST A IN-1 TO ST A OT-1 (STRM)	5,994.29	5,994.01	34	0.008	24.0	0.013	10.35	6.50	20.30	51	1.77	24" RCP
ST A BC-1 TO ST A IN-1 (STRM)	5,999.01	5,994.99	73	0.055	12.0	0.010	3.20	12.04	10.86	29	1.52	12" RCP
ST B BC-1 TO ST B IN-2 (STRM)	5,999.01	5,993.87	93	0.055	12.0	0.010	3.20	12.04	10.87	29	1.52	12" RCP
ST B IN-2 TO ST B IN-1 (STRM)	5,992.37	5,991.67	140	0.005	30.0	0.013	15.09	5.96	28.95	52	1.77	30" RCP
ST B IN-1 TO ST B OT-1 (STRM)	5,991.67	5,991.51	32	0.005	30.0	0.013	16.47	6.09	28.95	57	0.05	30" RCP
ST B BC-2 TO ST B IN-4 (STRM)	5,997.32	5,994.52	93	0.030	12.0	0.010	3.20	9.64	8.03	40	1.52	12" RCP
ST B IN-4 TO ST B IN-3 (STRM)	5,994.71	5,993.85	171	0.005	24.0	0.013	7.65	5.04	16.03	48	1.77	24" RCP
ST B IN-3 TO ST B IN-2 (STRM)	5,993.65	5,992.86	159	0.005	24.0	0.013	9.50	5.30	15.94	60	0.05	24" RCP
ST C OT-1 TO ST C FES-1 (STRM)	5,993.01	5,991.58	87	0.016	24.0	0.013	10.35	8.15	29.02	36	0.00	24" RCP
ST D OT-1 TO ST D FES-1 (STRM)	5,990.01	5,989.59	76	0.006	30.0	0.013	16.46	6.12	30.54	54	0.00	30" RCP

FlexTable: Catch Basin Table 10-YR

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Flow (Total Out) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Method	Headloss Coefficient (Standard)	Capture Efficiency (%)
ST A BC-2	6,000.25	5,999.01	3.78	6,000.54	5,999.84	Standard	1.52	100
ST A IN-2	6,002.18	5,997.44	5.29	5,998.65	5,998.25	Standard	1.32	90
ST A IN-1	6,000.55	5,994.29	12.21	5,996.49	5,995.54	Standard	1.77	90
ST A BC-1	6,000.25	5,999.01	3.78	6,000.54	5,999.84	Standard	1.52	100
ST B IN-1	5,999.41	5,991.67	19.47	5,993.20	5,993.17	Standard	0.05	90
ST B IN-2	5,998.05	5,992.37	17.84	5,994.84	5,993.80	Standard	1.77	50
ST B IN-3	5,998.15	5,993.65	11.24	5,995.12	5,995.10	Standard	0.05	50
ST B IN-4	5,998.45	5,994.71	9.06	5,996.55	5,995.79	Standard	1.77	50
ST B AD-1	6,000.62	5,996.71	0.15	5,996.87	5,996.87	Standard	0.05	50
ST B AD-2	6,002.12	5,997.62	0.04	5,997.72	5,997.70	Standard	0.64	50
ST B AD-3	6,003.42	5,998.34	0.03	5,998.41	5,998.41	Standard	0.05	50
ST B AD-4	6,003.42	5,999.19	0.02	5,999.25	5,999.25	Standard	0.05	50
ST B AD-5	6,004.00	5,999.95	0.01	5,999.99	5,999.99	Standard	0.00	50
ST C OT-1	5,995.29	5,991.01	12.21	5,994.27	5,994.27	Standard	0.00	100
ST D OT-1	5,992.80	5,988.01	19.47	5,991.51	5,991.51	Standard	0.00	100
ST B BC-1	6,000.25	5,999.01	3.78	6,000.54	5,999.84	Standard	1.52	100
ST B BC-2	5,998.56	5,997.32	3.78	5,998.85	5,998.15	Standard	1.52	100

FlexTable: Conduit Table 10-YR

Label	Invert (Start) (ft)	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Capacity (Full Flow) (cfs)	Flow / Capacity (Design) (%)	Upstream Structure Headloss Coefficient	Notes
ST B AG-4 TO ST B AG-3 (STRM)	5,999.95	5,999.19	87	0.009	12.0	0.010	0.01	1.13	4.33	0	0.00	12" RCP
ST B AG-3 TO ST A AG-2 (STRM)	5,999.19	5,998.34	85	0.010	12.0	0.010	0.02	1.47	4.63	0	0.05	12" RCP
ST A AG-2 TO ST B AG-1 (STRM)	5,998.34	5,997.62	72	0.010	12.0	0.010	0.03	1.65	4.63	1	0.05	12" RCP
ST A IN-2 TO ST A IN-1 (STRM)	5,997.44	5,994.48	297	0.010	24.0	0.013	5.29	5.87	22.58	23	1.32	24" RCP
ST A BC-2 TO ST A IN-2 (STRM)	5,999.01	5,998.01	73	0.014	12.0	0.010	3.78	7.46	5.42	70	1.52	12" RCP
ST B AG-1 TO ST B IN-4 (STRM)	5,997.62	5,996.71	92	0.010	12.0	0.010	0.04	1.81	4.61	1	0.64	12" RCP
ST B AG-1 TO ST B IN-4 (1) (STRM)	5,996.71	5,995.71	99	0.010	12.0	0.010	0.15	2.71	4.64	3	0.05	12" RCP
ST A IN-1 TO ST A OT-1 (STRM)	5,994.29	5,994.01	34	0.008	24.0	0.013	12.21	6.76	20.30	60	1.77	24" RCP
ST A BC-1 TO ST A IN-1 (STRM)	5,999.01	5,994.99	73	0.055	12.0	0.010	3.78	12.58	10.86	35	1.52	12" RCP
ST B BC-1 TO ST B IN-2 (STRM)	5,999.01	5,993.87	93	0.055	12.0	0.010	3.78	12.58	10.87	35	1.52	12" RCP
ST B IN-2 TO ST B IN-1 (STRM)	5,992.37	5,991.67	140	0.005	30.0	0.013	17.84	6.20	28.95	62	1.77	30" RCP
ST B IN-1 TO ST B OT-1 (STRM)	5,991.67	5,991.51	32	0.005	30.0	0.013	19.47	6.32	28.95	67	0.05	30" RCP
ST B BC-2 TO ST B IN-4 (STRM)	5,997.32	5,994.52	93	0.030	12.0	0.010	3.78	10.07	8.03	47	1.52	12" RCP
ST B IN-4 TO ST B IN-3 (STRM)	5,994.71	5,993.85	171	0.005	24.0	0.013	9.06	5.26	16.03	57	1.77	24" RCP
ST B IN-3 TO ST B IN-2 (STRM)	5,993.65	5,992.86	159	0.005	24.0	0.013	11.24	5.50	15.94	71	0.05	24" RCP
ST C OT-1 TO ST C FES-1 (STRM)	5,993.01	5,991.58	87	0.016	24.0	0.013	12.21	8.84	29.02	42	0.00	24" RCP
ST D OT-1 TO ST D FES-1 (STRM)	5,990.01	5,989.59	76	0.006	30.0	0.013	19.47	6.59	30.54	64	0.00	30" RCP

FlexTable: Catch Basin Table 100-YR

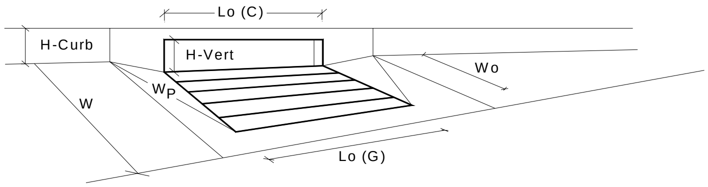
Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Flow (Total Out) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Method	Headloss Coefficient (Standard)	Capture Efficiency (%)
ST A BC-2	6,000.25	5,999.01	5.68	6,001.25	5,999.96	Standard	1.52	100
ST A IN-2	6,002.18	5,997.44	8.16	5,998.99	5,998.46	Standard	1.32	90
ST A IN-1	6,000.55	5,994.29	18.59	5,997.23	5,995.84	Standard	1.77	90
ST A BC-1	6,000.25	5,999.01	5.68	6,001.25	5,999.96	Standard	1.52	100
ST B IN-1	5,999.41	5,991.67	30.30	5,993.75	5,993.71	Standard	0.05	90
ST B IN-2	5,998.05	5,992.37	27.68	5,995.56	5,994.35	Standard	1.77	50
ST B IN-3	5,998.15	5,993.65	17.54	5,996.54	5,996.52	Standard	0.05	50
ST B IN-4	5,998.45	5,994.71	14.07	5,997.76	5,997.21	Standard	1.77	50
ST B AD-1	6,000.62	5,996.71	0.61	5,997.78	5,997.78	Standard	0.05	50
ST B AD-2	6,002.12	5,997.62	0.20	5,997.78	5,997.78	Standard	0.64	50
ST B AD-3	6,003.42	5,998.34	0.15	5,998.11	5,998.11	Standard	0.05	50
ST B AD-4	6,003.42	5,999.19	0.10	5,998.76	5,998.76	Standard	0.05	50
ST B AD-5	6,004.00	5,999.95	0.05	5,999.31	5,999.31	Standard	0.00	50
ST C OT-1	5,995.29	5,991.01	18.59	5,994.56	5,994.56	Standard	0.00	100
ST D OT-1	5,992.80	5,988.01	30.30	5,992.10	5,992.10	Standard	0.00	100
ST B BC-1	6,000.25	5,999.01	5.68	6,001.25	5,999.96	Standard	1.52	100
ST B BC-2	5,998.56	5,997.32	5.68	5,999.79	5,998.56	Standard	1.52	100

FlexTable: Conduit Table 100-YR

Label	Invert (Start) (ft)	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Capacity (Full Flow) (cfs)	Flow / Capacity (Design) (%)	Upstream Structure Headloss Coefficient	Notes
ST B AG-4 TO ST B AG-3 (STRM)	5,999.95	5,999.19	87	0.009	12.0	0.010	0.05	1.78	4.33	1	0.00	12" RCP
ST B AG-3 TO ST A AG-2 (STRM)	5,999.19	5,998.34	85	0.010	12.0	0.010	0.10	2.21	4.63	2	0.05	12" RCP
ST A AG-2 TO ST B AG-1 (STRM)	5,998.34	5,997.62	72	0.010	12.0	0.010	0.15	2.51	4.63	3	0.05	12" RCP
ST A IN-2 TO ST A IN-1 (STRM)	5,997.44	5,994.48	297	0.010	24.0	0.013	8.16	6.61	22.58	36	1.32	24" RCP
ST A BC-2 TO ST A IN-2 (STRM)	5,999.01	5,998.01	73	0.014	12.0	0.010	5.68	7.82	5.42	105	1.52	12" RCP
ST B AG-1 TO ST B IN-4 (STRM)	5,997.62	5,996.71	92	0.010	12.0	0.010	0.20	2.71	4.61	4	0.64	12" RCP
ST B AG-1 TO ST B IN-4 (1) (STRM)	5,996.71	5,995.71	99	0.010	12.0	0.010	0.61	0.78	4.64	13	0.05	12" RCP
ST A IN-1 TO ST A OT-1 (STRM)	5,994.29	5,994.01	34	0.008	24.0	0.013	18.59	7.33	20.30	92	1.77	24" RCP
ST A BC-1 TO ST A IN-1 (STRM)	5,999.01	5,994.99	73	0.055	12.0	0.010	5.68	13.99	10.86	52	1.52	12" RCP
ST B BC-1 TO ST B IN-2 (STRM)	5,999.01	5,993.87	93	0.055	12.0	0.010	5.68	13.99	10.87	52	1.52	12" RCP
ST B IN-2 TO ST B IN-1 (STRM)	5,992.37	5,991.67	140	0.005	30.0	0.013	27.68	6.71	28.95	96	1.77	30" RCP
ST B IN-1 TO ST B OT-1 (STRM)	5,991.67	5,991.51	32	0.005	30.0	0.013	30.30	6.68	28.95	105	0.05	30" RCP
ST B BC-2 TO ST B IN-4 (STRM)	5,997.32	5,994.52	93	0.030	12.0	0.010	5.68	7.23	8.03	71	1.52	12" RCP
ST B IN-4 TO ST B IN-3 (STRM)	5,994.71	5,993.85	171	0.005	24.0	0.013	14.07	4.48	16.03	88	1.77	24" RCP
ST B IN-3 TO ST B IN-2 (STRM)	5,993.65	5,992.86	159	0.005	24.0	0.013	17.54	5.58	15.94	110	0.05	24" RCP
ST C OT-1 TO ST C FES-1 (STRM)	5,993.01	5,991.58	87	0.016	24.0	0.013	18.59	9.42	29.02	64	0.00	24" RCP
ST D OT-1 TO ST D FES-1 (STRM)	5,990.01	5,989.59	76	0.006	30.0	0.013	30.30	6.75	30.54	99	0.00	30" RCP

INLET IN A SUMP OR SAG LOCATION

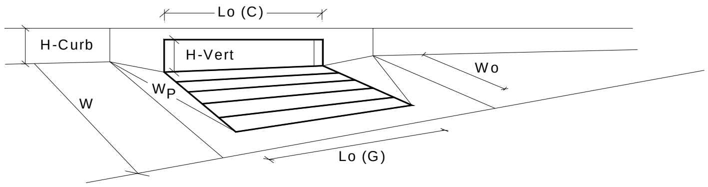
Version 4.05 Released March 2017



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' 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 =	7.0	8.0	inches
Grate Information		MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	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 =	5.00	5.00	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	$RF_{Combination}$ =	0.90	1.00	
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 =	**3.1**	**5.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.7	4.8	cfs

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017

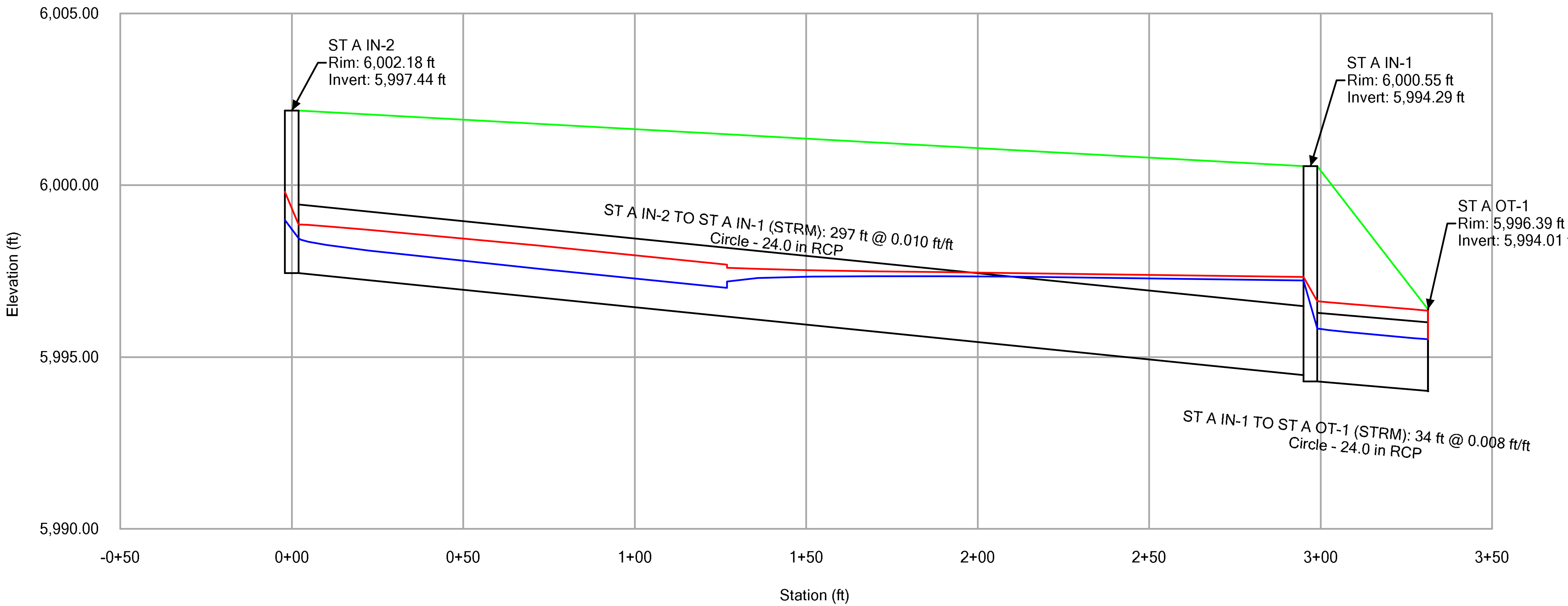


Design Information (Input)	CDOT Type C Grate		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type C Grate		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	6.00	6.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	9.0	inches
Grate Information			MINOR	MAJOR	☑ Override Depths
Length of a Unit Grate		L_o (G) =	2.92	2.92	feet
Width of a Unit Grate		W_o =	2.92	2.92	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)		A_{ratio} =	0.70	0.70	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)		C_f (G) =	0.50	0.50	
Grate Weir Coefficient (typical value 2.15 - 3.60)		C_w (G) =	2.41	2.41	
Grate Orifice Coefficient (typical value 0.60 - 0.80)		C_o (G) =	0.67	0.67	
Curb Opening Information			MINOR	MAJOR	
Length of a Unit Curb Opening		L_o (C) =	N/A	N/A	feet
Height of Vertical Curb Opening in Inches		H_{vert} =	N/A	N/A	inches
Height of Curb Orifice Throat in Inches		H_{throat} =	N/A	N/A	inches
Angle of Throat (see USDCM Figure ST-5)		Theta =	N/A	N/A	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)		W_p =	N/A	N/A	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)		C_f (C) =	N/A	N/A	
Curb Opening Weir Coefficient (typical value 2.3-3.7)		C_w (C) =	N/A	N/A	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)		C_o (C) =	N/A	N/A	

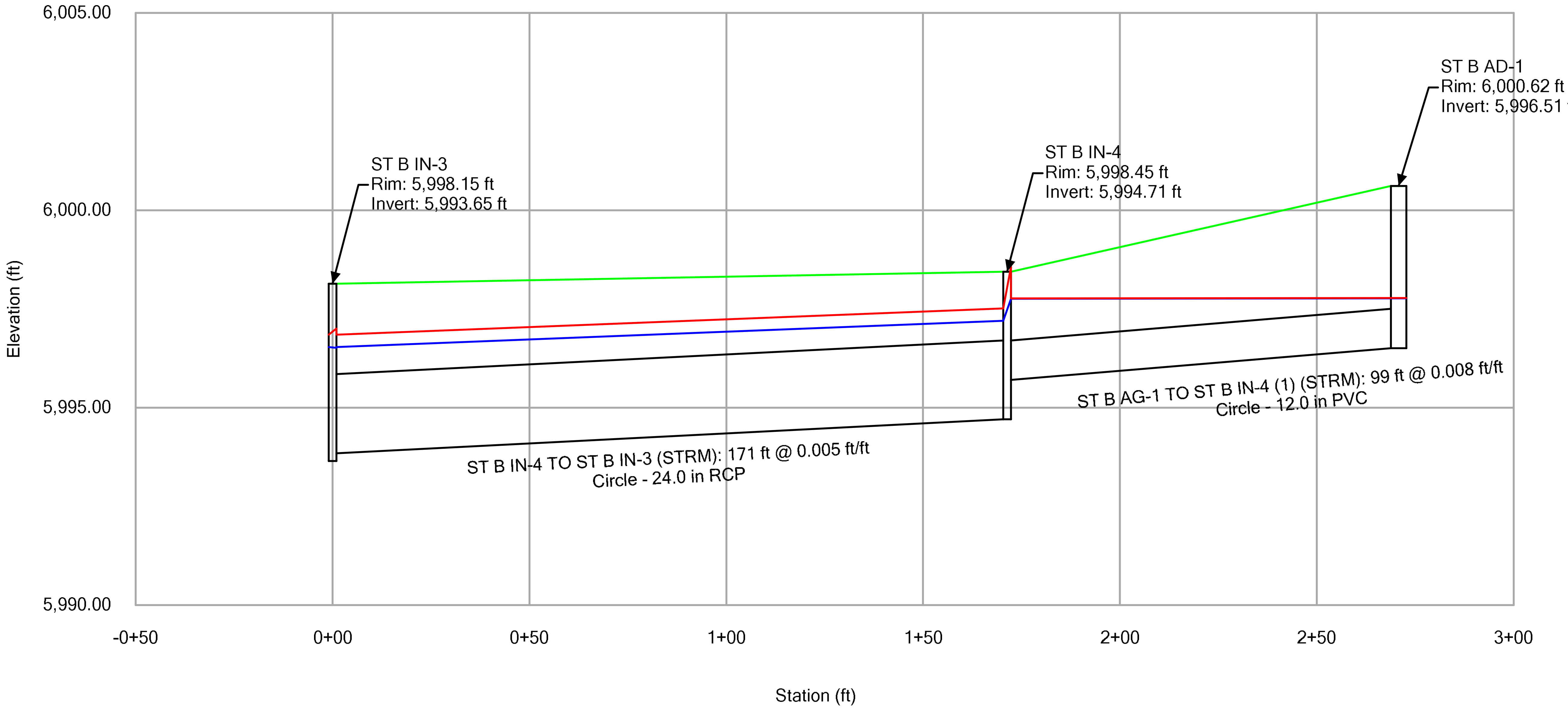
Low Head Performance Reduction (Calculated)		MINOR	MAJOR	
Depth for Grate Midwidth	d_{Grate} =	0.733	0.983	ft
Depth for Curb Opening Weir Equation	d_{Curb} =	N/A	N/A	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	N/A	N/A	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	0.95	1.00	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**5.3**	**8.6**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	4.4	7.8	cfs

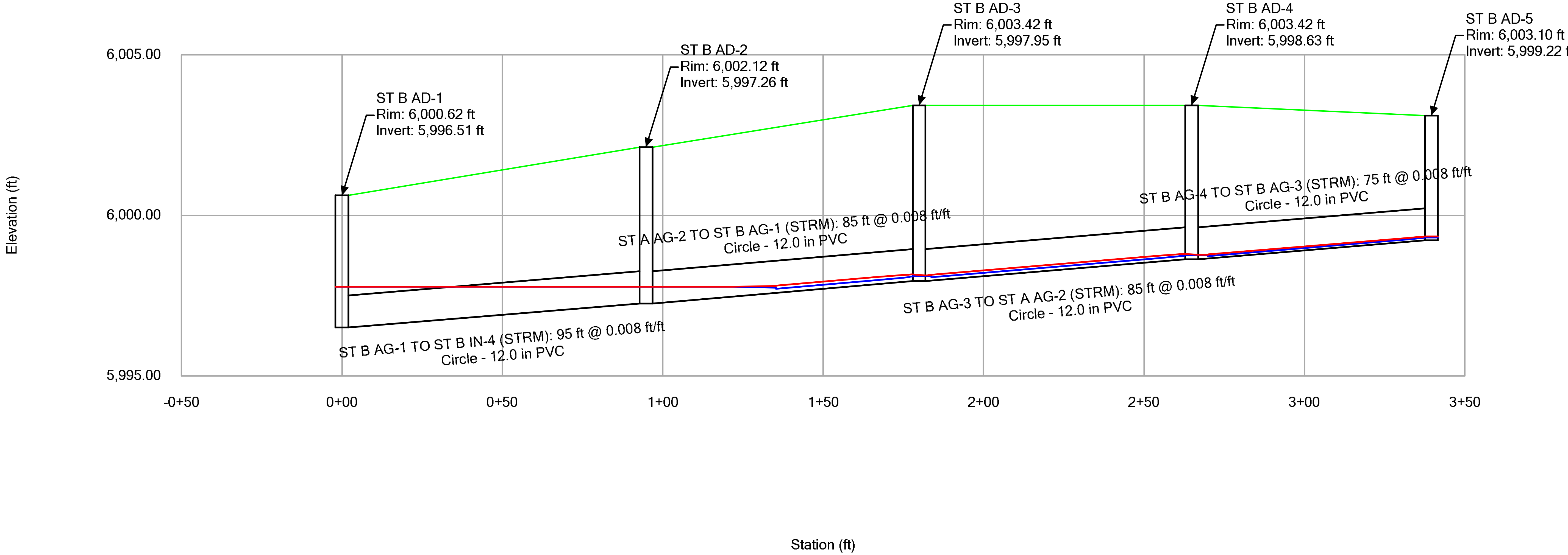
100-YR
Profile Report
Engineering Profile - STORM LINE A PROFILE (US AutoForce StormCAD Model.stsw)



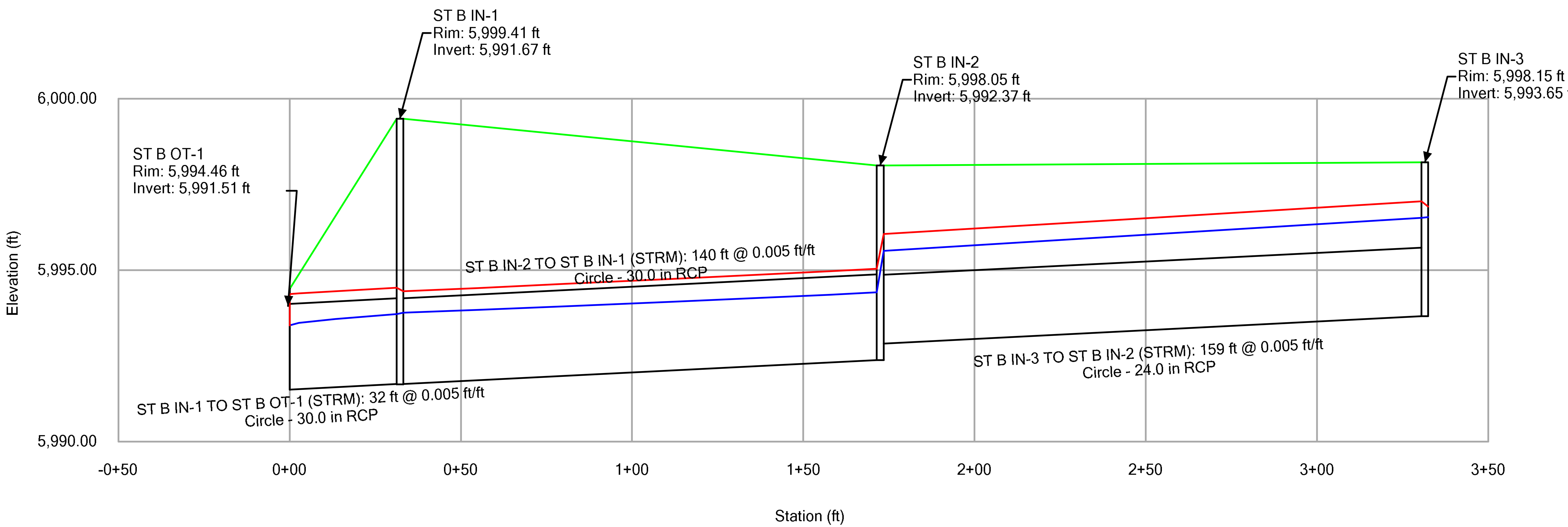
100-YR
Profile Report
Engineering Profile - STORM LINE B PROFILE PART 2 (US AutoForce StormCAD Model.stsw)



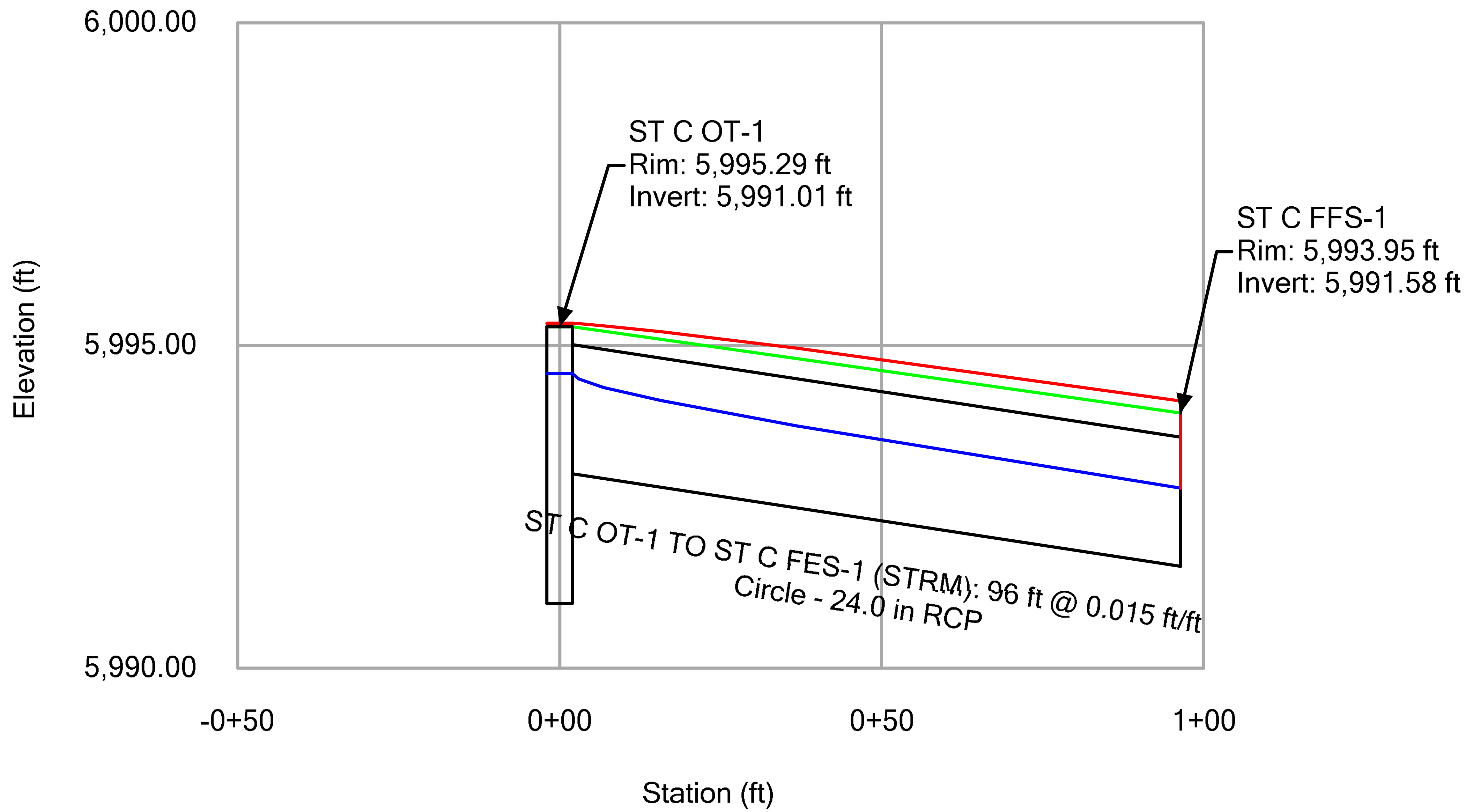
100-YR
Profile Report
Engineering Profile - STORM LINE B PROFILE PART 3 (US AutoForce StormCAD Model.stsw)



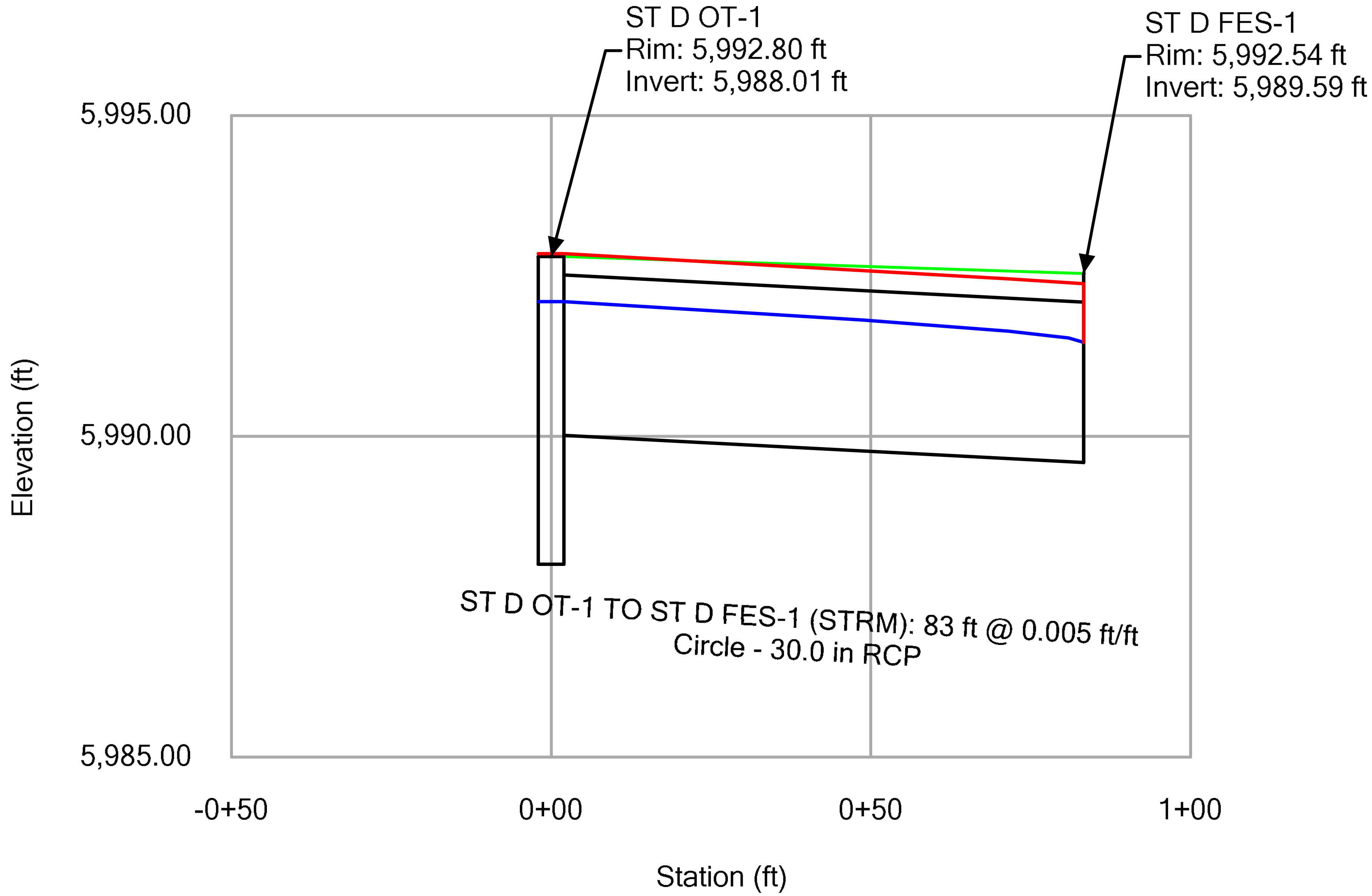
100-YR
Profile Report
Engineering Profile - STORM LINE B PROFILE PART 1 (US AutoForce StormCAD Model.stsw)



100-YR
Profile Report
Engineering Profile - STORM LINE C PROFILE (US AutoForce StormCAD Model.stsw)



100-YR
Profile Report
Engineering Profile - STORM LINE D PROFILE (US AutoForce StormCAD Model.stsw)

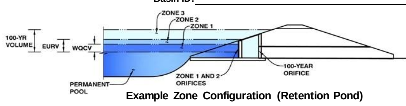


DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: 2570 Zeppelin Rd. Pond A (North Pond)

Basin ID:



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	4.34	acres		
Watershed Length =	780	ft		
Watershed Slope =	0.025	ft/ft		
Watershed Imperviousness =	51.60%	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		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =	Castle Pines - City Office			
Water Quality Capture Volume (WQCV) =	0.076	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.261	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.178	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.51 in.) =	0.235	acre-feet	1.51	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.287	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 1.72 in.) =	0.307	acre-feet		inches
50-yr Runoff Volume (P1 = 2.01 in.) =	0.396	acre-feet		inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.544	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.07 in.) =	0.760	acre-feet		inches
Approximate 2-yr Detention Volume =	0.168	acre-feet		
Approximate 5-yr Detention Volume =	0.222	acre-feet		
Approximate 10-yr Detention Volume =	0.269	acre-feet		
Approximate 25-yr Detention Volume =	0.282	acre-feet		
Approximate 50-yr Detention Volume =	0.327	acre-feet		
Approximate 100-yr Detention Volume =	0.411	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.076	acre-feet	
Zone 2 Volume (User Defined - Zone 1) =		acre-feet	**Total detention volume is less than 100-year volume.**
Zone 3 Volume (User Defined - Zones 1 & 2) =		acre-feet	
Total Detention Basin Volume =	0.076	acre-feet	
Initial Surcharge Volume (ISV) =	user	ft^3	
Initial Surcharge Depth (ISD) =	user	ft	
Total Available Detention Depth (H_{total}) =	user	ft	
Depth of Trickle Channel (H_{TC}) =	user	ft	
Slope of Trickle Channel (S_{TC}) =	user	ft/ft	
Slopes of Main Basin Sides (S_{main}) =	user	H:V	
Basin Length-to-Width Ratio ($R_{L/W}$) =	user		
Initial Surcharge Area (A_{ISV}) =	user	ft^2	
Surcharge Volume Length (L_{ISV}) =	user	ft	
Surcharge Volume Width (W_{ISV}) =	user	ft	
Depth of Basin Floor (H_{FLOOR}) =	user	ft	
Length of Basin Floor (L_{FLOOR}) =	user	ft	
Width of Basin Floor (W_{FLOOR}) =	user	ft	
Area of Basin Floor (A_{FLOOR}) =	user	ft^2	
Volume of Basin Floor (V_{FLOOR}) =	user	ft^3	
Depth of Main Basin (H_{MAIN}) =	user	ft	
Length of Main Basin (L_{MAIN}) =	user	ft	
Width of Main Basin (W_{MAIN}) =	user	ft	
Area of Main Basin (A_{MAIN}) =	user	ft^2	
Volume of Main Basin (V_{MAIN}) =	user	ft^3	
Calculated Total Basin Volume (V_{total}) =	user	acre-feet	

Depth Increment = 0.1 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	--	--	--	16	0.000		
	--	0.10	--	--	--	17	0.000	2	0.000
	--	0.20	--	--	--	18	0.000	3	0.000
	--	0.30	--	--	--	30	0.001	5	0.000
	--	0.40	--	--	--	115	0.003	12	0.000
	--	0.50	--	--	--	281	0.006	30	0.001
	--	0.60	--	--	--	353	0.008	61	0.001
	--	0.70	--	--	--	372	0.009	97	0.002
	--	0.80	--	--	--	393	0.009	135	0.003
	--	0.90	--	--	--	436	0.010	176	0.004
	--	1.00	--	--	--	629	0.014	227	0.005
	--	1.10	--	--	--	758	0.017	296	0.007
	--	1.20	--	--	--	902	0.021	377	0.009
	--	1.30	--	--	--	1,035	0.024	473	0.011
	--	1.40	--	--	--	1,167	0.027	581	0.013
	--	1.50	--	--	--	1,298	0.030	703	0.016
	--	1.60	--	--	--	1,430	0.033	838	0.019
	--	1.70	--	--	--	1,562	0.036	987	0.023
	--	1.80	--	--	--	1,693	0.039	1,148	0.026
	--	1.90	--	--	--	1,825	0.042	1,323	0.030
	--	2.00	--	--	--	1,956	0.045	1,510	0.035
	--	2.10	--	--	--	2,087	0.048	1,732	0.040
	--	2.20	--	--	--	2,217	0.051	1,947	0.045
	--	2.30	--	--	--	2,348	0.054	2,175	0.050
	--	2.40	--	--	--	2,478	0.057	2,417	0.055
	--	2.50	--	--	--	2,609	0.060	2,671	0.061
	--	2.60	--	--	--	2,739	0.063	2,939	0.067
	--	2.70	--	--	--	2,878	0.066	3,219	0.074
	--	2.80	--	--	--	2,979	0.068	3,512	0.081
	--	2.90	--	--	--	3,081	0.071	3,815	0.088
	--	3.00	--	--	--	3,184	0.073	4,128	0.095
	--	3.10	--	--	--	3,288	0.075	4,452	0.102
	--	3.20	--	--	--	3,393	0.078	4,786	0.110
	--	3.30	--	--	--	3,499	0.080	5,131	0.118
	--	3.40	--	--	--	3,607	0.083	5,486	0.126
	--	3.50	--	--	--	3,715	0.085	5,852	0.134
	--	3.60	--	--	--	4,485	0.103	6,262	0.144
	--	3.70	--	--	--	4,592	0.105	6,716	0.154
	--	3.80	--	--	--	4,694	0.108	7,180	0.165
	--	3.90	--	--	--	4,801	0.110	7,655	0.176
	--	4.00	--	--	--	4,906	0.113	8,140	0.187
	--	4.10	--	--	--	5,012	0.115	8,636	0.198
	--	4.20	--	--	--	5,117	0.117	9,143	0.210
	--	4.30	--	--	--	5,222	0.120	9,660	0.222
	--	4.40	--	--	--	5,326	0.122	10,187	0.234
	--	4.50	--	--	--	5,431	0.125	10,725	0.246
	--	4.60	--	--	--	4,969	0.114	11,245	0.258
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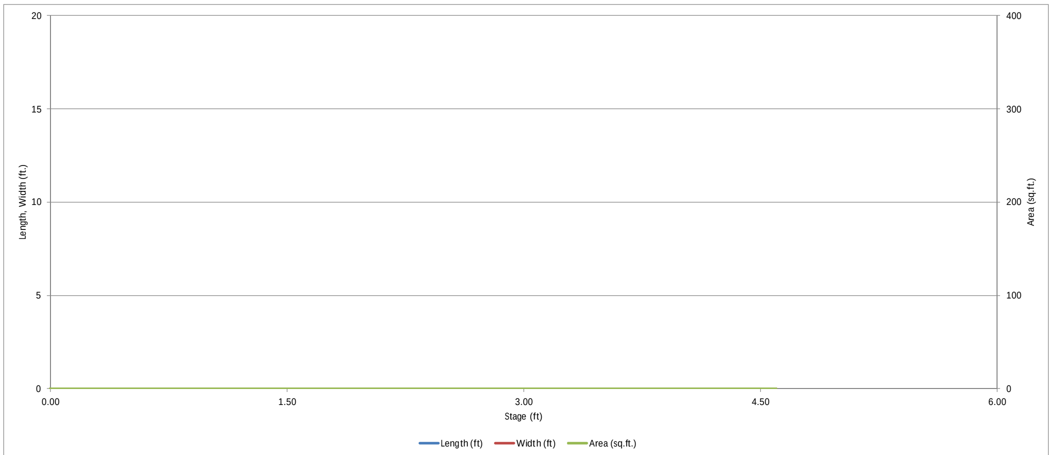
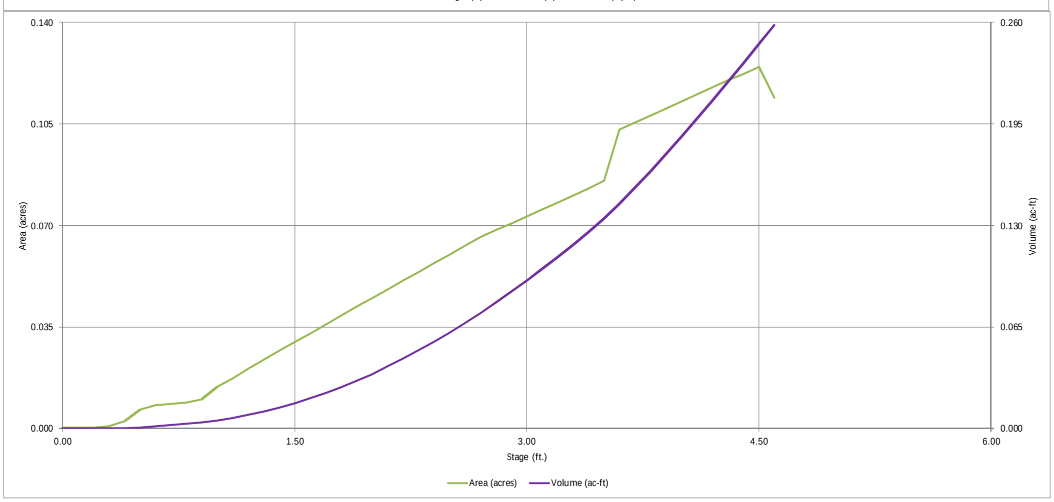
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)



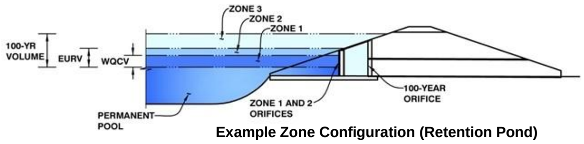


Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: 2570 Zeppelin Rd. Pond A (North Pond)

Basin ID:



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.74	0.076	Orifice Plate
Zone 2 (User)			Weir&Pipe (Circular)
Zone 3 (User)			Not Utilized
		0.076	Total

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

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

Calculated Parameters for Underdrain

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

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

Invert of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	2.74	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	11.00	inches
Orifice Plate: Orifice Area per Row =	0.29	sq. inches (diameter = 5/8 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	2.014E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	0.91	1.83					
Orifice Area (sq. inches)	0.29	0.29	0.29					

	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) and Grate (Flat or Sloped)

	Zone 2 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	2.74	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 2 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	2.74	N/A	feet
Over Flow Weir Slope Length =	4.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	3.57	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	11.20	N/A	ft^2
Overflow Grate Open Area w/ Debris =	5.60	N/A	ft^2

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

	Zone 2 Circular	Not Selected	
Depth to Invert of Outlet Pipe =	0.08	N/A	ft (distance below basin bottom at Stage = 0 ft)
Circular Orifice Diameter =	24.00	N/A	inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 2 Circular	Not Selected	
Outlet Orifice Area =	3.14	N/A	ft^2
Outlet Orifice Centroid =	1.00	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	N/A	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	3.92	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	3.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Calculated Parameters for Spillway

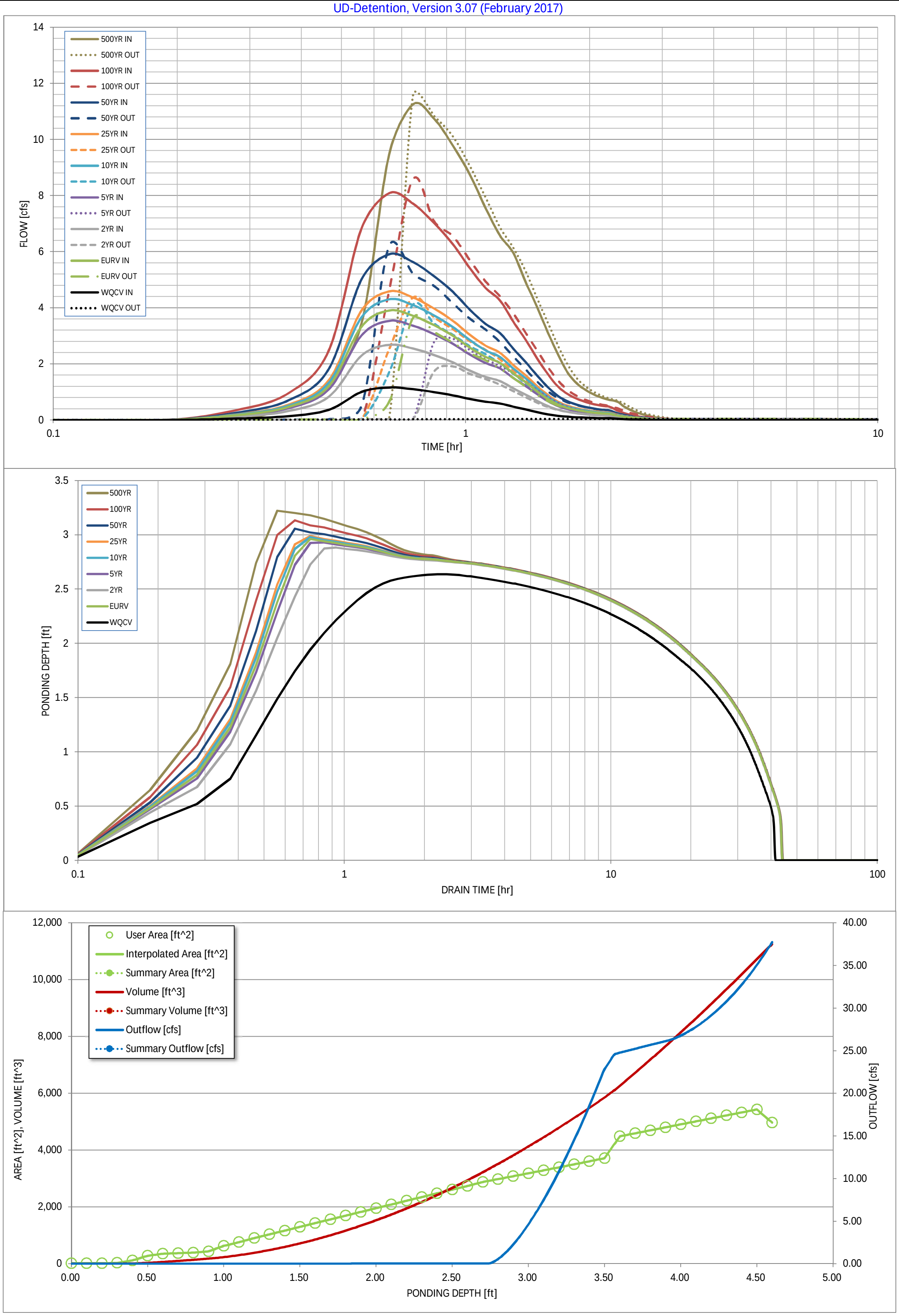
Spillway Design Flow Depth=	0.65	feet
Stage at Top of Freeboard =	5.57	feet
Basin Area at Top of Freeboard =	0.11	acres

Routed Hydrograph Results

Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.51	1.75	1.72	2.01	2.52	3.07
Calculated Runoff Volume (acre-ft) =	0.076	0.261	0.178	0.235	0.287	0.307	0.396	0.544	0.760
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.076	0.260	0.178	0.235	0.287	0.306	0.396	0.543	0.759
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.00	0.00	0.01	0.02	0.15	0.41	0.91
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.0	0.0	0.1	0.6	1.8	4.0
Peak Inflow Q (cfs) =	1.2	3.9	2.7	3.5	4.3	4.6	5.9	8.1	11.2
Peak Outflow Q (cfs) =	0.0	3.6	1.9	2.9	4.1	4.4	6.2	8.6	11.6
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	152.8	94.0	51.4	9.6	4.9	2.9
Structure Controlling Flow =	Plate	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1
Max Velocity through Grate 1 (fps) =	N/A	0.32	0.16	0.3	0.4	0.4	0.6	0.8	1.0
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	38	34	36	35	33	33	31	28	24
Time to Drain 99% of Inflow Volume (hours) =	40	40	41	40	39	39	38	36	34
Maximum Ponding Depth (ft) =	2.64	2.96	2.88	2.93	2.98	2.99	3.06	3.13	3.22
Area at Maximum Ponding Depth (acres) =	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08
Maximum Volume Stored (acre-ft) =	0.069	0.092	0.086	0.089	0.093	0.094	0.098	0.104	0.111

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)



S-A-V-D Chart Axis Override	X-axis	Left Y-Axis	Right Y-Axis
minimum bound			
maximum bound			

Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs UD-Detention, Version 3.07 (February 2017)

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

	SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
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]
5.59 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:05:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Hydrograph	0:11:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Constant	0:16:46	0.05	0.18	0.12	0.16	0.19	0.21	0.26	0.36	0.50
0.894	0:22:22	0.14	0.47	0.32	0.43	0.52	0.55	0.71	0.97	1.34
	0:27:57	0.36	1.21	0.83	1.10	1.33	1.42	1.82	2.48	3.44
	0:33:32	1.00	3.33	2.30	3.02	3.66	3.90	5.01	6.83	9.46
	0:39:08	1.16	3.90	2.68	3.53	4.30	4.59	5.91	8.09	11.25
	0:44:43	1.10	3.72	2.55	3.36	4.10	4.37	5.63	7.71	10.74
	0:50:19	1.00	3.38	2.32	3.06	3.73	3.97	5.12	7.02	9.78
	0:55:54	0.88	3.00	2.06	2.72	3.31	3.53	4.56	6.26	8.74
	1:01:29	0.75	2.58	1.76	2.33	2.85	3.04	3.93	5.40	7.55
	1:07:05	0.66	2.25	1.54	2.04	2.48	2.65	3.43	4.71	6.58
	1:12:40	0.59	2.04	1.39	1.84	2.25	2.40	3.10	4.27	5.96
	1:18:16	0.48	1.67	1.13	1.50	1.84	1.96	2.55	3.51	4.93
	1:23:51	0.38	1.35	0.91	1.22	1.49	1.59	2.07	2.87	4.03
	1:29:26	0.28	1.02	0.69	0.92	1.13	1.21	1.58	2.20	3.11
	1:35:02	0.20	0.75	0.50	0.67	0.83	0.89	1.17	1.63	2.32
	1:40:37	0.15	0.55	0.37	0.49	0.61	0.65	0.85	1.19	1.68
	1:46:13	0.12	0.43	0.29	0.39	0.47	0.51	0.66	0.92	1.30
	1:51:48	0.10	0.35	0.24	0.32	0.39	0.42	0.55	0.76	1.07
	1:57:23	0.08	0.30	0.20	0.27	0.33	0.36	0.46	0.64	0.91
	2:02:59	0.07	0.27	0.18	0.24	0.29	0.31	0.41	0.57	0.80
	2:08:34	0.07	0.24	0.16	0.22	0.26	0.28	0.37	0.51	0.72
	2:14:10	0.06	0.22	0.15	0.20	0.25	0.26	0.34	0.47	0.66
	2:19:45	0.05	0.16	0.11	0.15	0.18	0.19	0.25	0.34	0.48
	2:25:20	0.03	0.12	0.08	0.11	0.13	0.14	0.18	0.25	0.36
	2:30:56	0.02	0.09	0.06	0.08	0.10	0.10	0.13	0.19	0.26
	2:36:31	0.02	0.06	0.04	0.06	0.07	0.08	0.10	0.14	0.19
	2:42:07	0.01	0.04	0.03	0.04	0.05	0.05	0.07	0.10	0.14
	2:47:42	0.01	0.03	0.02	0.03	0.04	0.04	0.05	0.07	0.10
	2:53:17	0.01	0.02	0.01	0.02	0.02	0.03	0.03	0.05	0.07
	2:58:53	0.00	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.05
	3:04:28	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03
	3:10:04	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01
	3:15:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:21:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:26:50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:32:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:38:01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:43:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:49:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3:54:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:00:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:05:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:11:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:17:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:22:44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:28:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:33:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:39:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:45:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:50:41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4:56:16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:01:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:07:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:13:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:18:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:24:13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:29:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:35:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:40:59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:46:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:52:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5:57:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:03:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:08:56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:14:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:20:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:25:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:31:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:36:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6:42:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Site-Level Low Impact Development (LID) Design Effective Impervious Calculator

LID Credit by Impervious Reduction Factor (IRF) Method

UD-BMP (Version 3.06, November 2016)

User Input

Calculated cells

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	10-Year Event	1.75	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	CUHP		
(CUHP) NOAA 1 Hour Rainfall Depth and Frequency for User Defined Storm	100-Year Event	2.52	
Max Intensity for Optional User Defined Storm	2.51496		

Designer: Eric Gunderson, P.E.
Company: Kimley-Horn and Associates
Date: June 19, 2017
Project: US Autoforce
Location: 2570 Zeppelin Rd. North Pond (NP)

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	NP													
Receiving Pervious Area Soil Type	Sand													
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	4.340													
Directly Connected Impervious Area (DCIA, acres)	2.239													
Unconnected Impervious Area (UIA, acres)	0.000													
Receiving Pervious Area (RPA, acres)	0.000													
Separate Pervious Area (SPA, acres)	2.101													
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	V													

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	4.340													
Directly Connected Impervious Area (DCIA, %)	51.6%													
Unconnected Impervious Area (UIA, %)	0.0%													
Receiving Pervious Area (RPA, %)	0.0%													
Separate Pervious Area (SPA, %)	48.4%													
A_R (RPA / UIA)	0.000													
I_a Check	1.000													
f / I for WQCV Event:	9.8													
f / I for 10-Year Event:	0.6													
f / I for 100-Year Event:	0.6													
f / I for Optional User Defined Storm CUHP:	0.57													
IRF for WQCV Event:	0.00													
IRF for 10-Year Event:	1.00													
IRF for 100-Year Event:	1.00													
IRF for Optional User Defined Storm CUHP:	1.00													
Total Site Imperviousness: I_{total}	51.6%													
Effective Imperviousness for WQCV Event:	51.6%													
Effective Imperviousness for 10-Year Event:	51.6%													
Effective Imperviousness for 100-Year Event:	51.6%													
Effective Imperviousness for Optional User Defined Storm CUHP:	51.6%													

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV Event CREDIT: Reduce Detention By:	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10-Year Event CREDIT**: Reduce Detention By:	0.0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
100-Year Event CREDIT**: Reduce Detention By:	0.0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
User Defined CUHP CREDIT: Reduce Detention By:	0.0%													

Total Site Imperviousness:	51.6%
Total Site Effective Imperviousness for WQCV Event:	51.6%
Total Site Effective Imperviousness for 10-Year Event:	51.6%
Total Site Effective Imperviousness for 100-Year Event:	51.6%
Total Site Effective Imperviousness for Optional User Defined Storm CUHP:	51.6%

Notes:

* Use Green-Ampt average infiltration rate values from Table 3-3.

** Flood control detention volume credits based on empirical equations from Storage Chapter of USDCM.

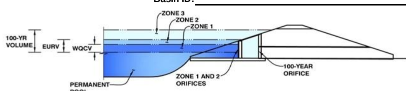
*** Method assumes that 1-hour rainfall depth is equivalent to 1-hour intensity for calculation purposed

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: **2570 Zeppelin Rd. South Pond (Pond B)**

Basin ID:



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	5.02	acres		
Watershed Length =	780	ft		
Watershed Slope =	0.025	ft/ft		
Watershed Imperviousness =	70.60%	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		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =				
Water Quality Capture Volume (WQCV) =	0.116	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.450	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.310	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.51 in.) =	0.407	acre-feet	1.51	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.490	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 1.72 in.) =	0.506	acre-feet		inches
50-yr Runoff Volume (P1 = 2.01 in.) =	0.619	acre-feet		inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.814	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.07 in.) =	1.066	acre-feet		inches
Approximate 2-yr Detention Volume =	0.294	acre-feet		
Approximate 5-yr Detention Volume =	0.386	acre-feet		
Approximate 10-yr Detention Volume =	0.460	acre-feet		
Approximate 25-yr Detention Volume =	0.474	acre-feet		
Approximate 50-yr Detention Volume =	0.540	acre-feet		
Approximate 100-yr Detention Volume =	0.659	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.116	acre-feet	
Zone 2 Volume (User Defined - Zone 1) =	0.000	acre-feet	**Total detention volume is less than 100-year volume.**
Zone 3 Volume (User Defined - Zones 1 & 2) =	0.000	acre-feet	
Total Detention Basin Volume =	0.116	acre-feet	
Initial Surcharge Volume (ISV) =	N/A	ft^3	
Initial Surcharge Depth (ISD) =	N/A	ft	
Total Available Detention Depth (H_{total}) =	user	ft	
Depth of Trickle Channel (H_{TC}) =	N/A	ft	
Slope of Trickle Channel (S_{TC}) =	N/A	ft/ft	
Slopes of Main Basin Sides (S_{main}) =	user	H:V	
Basin Length-to-Width Ratio ($R_{L/W}$) =	user		
Initial Surcharge Area (A_{ISV}) =	user	ft^2	
Surcharge Volume Length (L_{ISV}) =	user	ft	
Surcharge Volume Width (W_{ISV}) =	user	ft	
Depth of Basin Floor (H_{FLOOR}) =	user	ft	
Length of Basin Floor (L_{FLOOR}) =	user	ft	
Width of Basin Floor (W_{FLOOR}) =	user	ft	
Area of Basin Floor (A_{FLOOR}) =	user	ft^2	
Volume of Basin Floor (V_{FLOOR}) =	user	ft^3	
Depth of Main Basin (H_{MAIN}) =	user	ft	
Length of Main Basin (L_{MAIN}) =	user	ft	
Width of Main Basin (W_{MAIN}) =	user	ft	
Area of Main Basin (A_{MAIN}) =	user	ft^2	
Volume of Main Basin (V_{MAIN}) =	user	ft^3	
Calculated Total Basin Volume (V_{total}) =	user	acre-feet	

Depth Increment = 0.1 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	--	--	--	16	0.000		
	--	0.10	--	--	--	16	0.000	2	0.000
	--	0.20	--	--	--	40	0.001	4	0.000
	--	0.30	--	--	--	158	0.004	13	0.000
	--	0.40	--	--	--	169	0.004	29	0.001
	--	0.50	--	--	--	249	0.006	49	0.001
	--	0.60	--	--	--	313	0.007	76	0.002
	--	0.70	--	--	--	479	0.011	114	0.003
	--	0.80	--	--	--	593	0.014	167	0.004
	--	0.90	--	--	--	716	0.016	231	0.005
	--	1.00	--	--	--	834	0.019	307	0.007
	--	1.10	--	--	--	947	0.022	395	0.009
	--	1.20	--	--	--	1,065	0.024	495	0.011
	--	1.30	--	--	--	1,167	0.027	605	0.014
	--	1.40	--	--	--	1,264	0.029	726	0.017
	--	1.50	--	--	--	1,355	0.031	856	0.020
	--	1.60	--	--	--	1,441	0.033	995	0.023
	--	1.70	--	--	--	1,508	0.035	1,142	0.026
	--	1.80	--	--	--	1,573	0.036	1,295	0.030
	--	1.90	--	--	--	1,639	0.038	1,455	0.033
	--	2.00	--	--	--	1,705	0.039	1,622	0.037
	--	2.10	--	--	--	1,773	0.041	1,813	0.042
	--	2.20	--	--	--	1,841	0.042	1,993	0.046
	--	2.30	--	--	--	1,910	0.044	2,181	0.050
	--	2.40	--	--	--	1,980	0.045	2,375	0.055
	--	2.50	--	--	--	2,051	0.047	2,577	0.059
	--	2.60	--	--	--	2,122	0.049	2,786	0.064
	--	2.70	--	--	--	2,195	0.050	3,001	0.069
	--	2.80	--	--	--	2,268	0.052	3,225	0.074
	--	2.90	--	--	--	2,342	0.054	3,455	0.079
	--	3.00	--	--	--	2,417	0.055	3,693	0.085
	--	3.10	--	--	--	2,493	0.057	3,939	0.090
	--	3.20	--	--	--	2,569	0.059	4,192	0.096
	--	3.30	--	--	--	2,647	0.061	4,452	0.102
	--	3.40	--	--	--	2,725	0.063	4,721	0.108
	--	3.50	--	--	--	2,804	0.064	4,998	0.115
	--	3.60	--	--	--	2,884	0.066	5,282	0.121
	--	3.70	--	--	--	2,979	0.068	5,575	0.128
	--	3.80	--	--	--	3,075	0.071	5,878	0.135
	--	3.90	--	--	--	3,173	0.073	6,190	0.142
	--	4.00	--	--	--	3,272	0.075	6,512	0.150
	--	4.10	--	--	--	3,373	0.077	6,845	0.157
	--	4.20	--	--	--	3,476	0.080	7,187	0.165
	--	4.30	--	--	--	3,580	0.082	7,540	0.173
	--	4.40	--	--	--	3,686	0.085	7,903	0.181
	--	4.50	--	--	--	3,794	0.087	8,277	0.190
	--	4.60	--	--	--	3,903	0.090	8,662	0.199
	--	4.70	--	--	--	4,013	0.092	9,058	0.208
	--	4.80	--	--	--	4,126	0.095	9,465	0.217
	--	4.90	--	--	--	4,240	0.097	9,883	0.227
	--	5.00	--	--	--	4,355	0.100	10,313	0.237
	--	5.10	--	--	--	4,472	0.103	10,754	0.247
	--	5.20	--	--	--	4,591	0.105	11,207	0.257
	--	5.30	--	--	--	4,711	0.108	11,672	0.268
	--	5.40	--	--	--	4,833	0.111	12,150	0.279
	--	5.50	--	--	--	4,957	0.114	12,639	0.290
	--	5.60	--	--	--	5,550	0.127	13,164	0.302
	--	5.70	--	--	--	5,723	0.131	13,728	0.315
	--	5.80	--	--	--	5,861	0.135	14,307	0.328
	--	5.90	--	--	--	5,994	0.138	14,900	0.342
	--	6.00	--	--	--	6,128	0.141	15,506	0.356
	--	6.10	--	--	--	6,260	0.144	16,126	0.370
	--	6.20	--	--	--	6,391	0.147	16,758	0.385
	--	6.30	--	--	--	6,522	0.150	17,404	0.400
	--	6.40	--	--	--	6,654	0.153	18,063	0.415
	--	6.50	--	--	--	6,787	0.156	18,735	0.430
	--	6.60	--	--	--	6,917	0.159	19,420	0.446
	--	6.70	--	--	--	7,044	0.162	20,118	0.462
	--	6.80	--	--	--	7,171	0.165	20,829	0.478
	--	6.90	--	--	--	7,177	0.165	21,546	0.495
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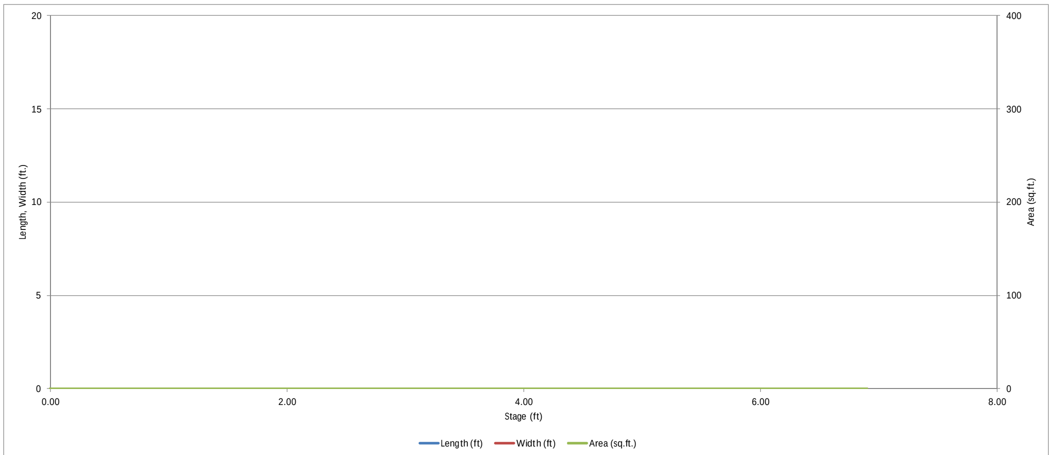
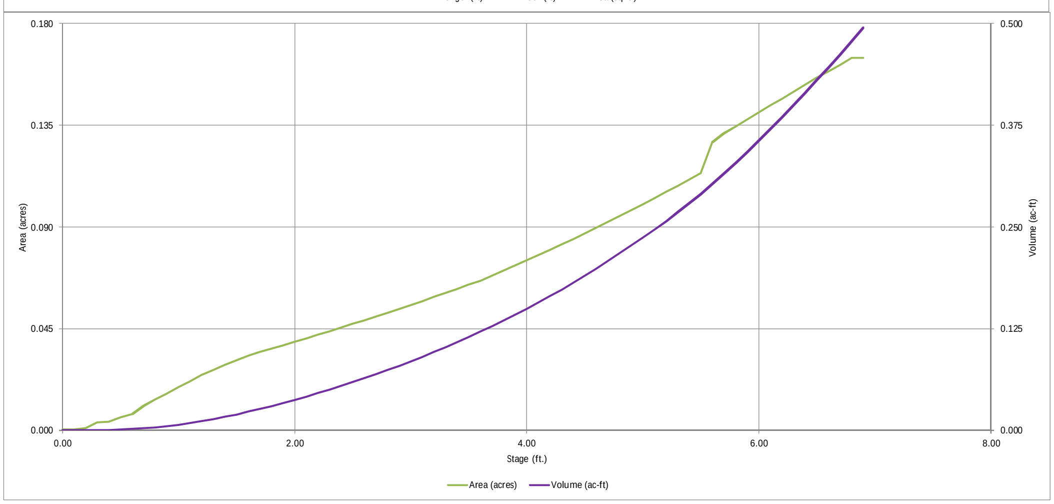
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)



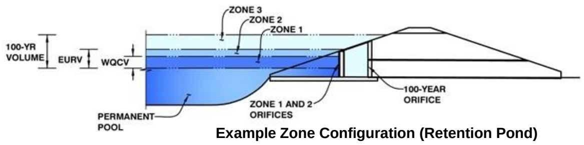


Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: 2570 Zeppelin Rd. South Pond (Pond B)

Basin ID:



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	3.53	0.116	Orifice Plate
Zone 2 (User)	3.53	0.000	Weir&Pipe (Circular)
Zone 3 (User)	3.53	0.000	Not Utilized
		0.116	Total

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

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

Calculated Parameters for Underdrain

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

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

Invert of Lowest Orifice =	0.00	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	3.53	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	14.10	inches
Orifice Plate: Orifice Area per Row =	0.44	sq. inches (diameter = 3/4 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	3.056E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	1.18	2.35					
Orifice Area (sq. inches)	0.44	0.44	0.44					

	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) and Grate (Flat or Sloped)

	Zone 2 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	3.53	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 2 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	3.53	N/A	feet
Over Flow Weir Slope Length =	4.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	2.28	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	11.20	N/A	ft^2
Overflow Grate Open Area w/ Debris =	5.60	N/A	ft^2

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

	Zone 2 Circular	Not Selected	
Depth to Invert of Outlet Pipe =	0.28	N/A	ft (distance below basin bottom at Stage = 0 ft)
Circular Orifice Diameter =	30.00	N/A	inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 2 Circular	Not Selected	
Outlet Orifice Area =	4.91	N/A	ft^2
Outlet Orifice Centroid =	1.25	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	N/A	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	6.25	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	4.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Calculated Parameters for Spillway

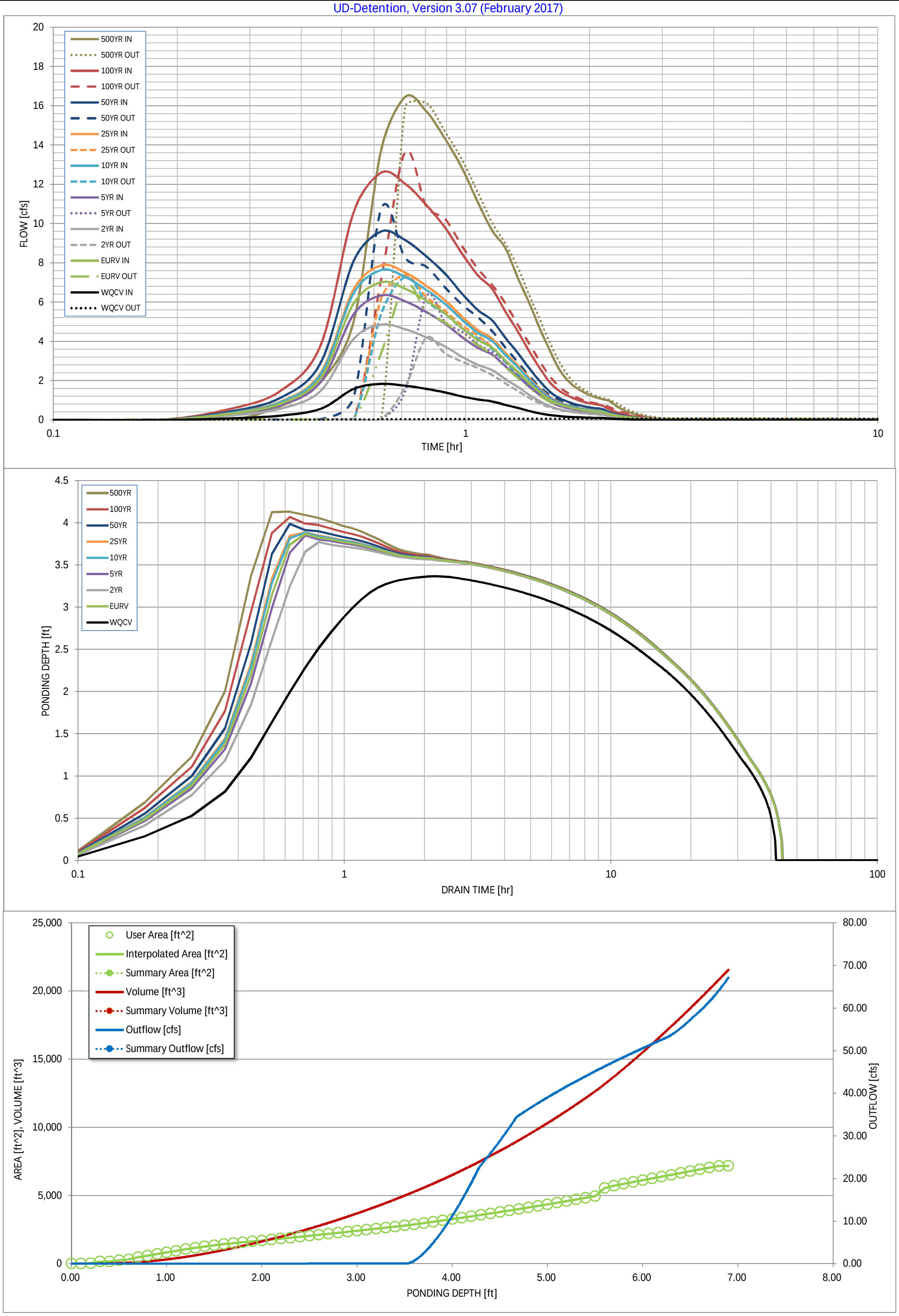
Spillway Design Flow Depth=	0.75	feet
Stage at Top of Freeboard =	8.00	feet
Basin Area at Top of Freeboard =	0.16	acres

Routed Hydrograph Results

Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.51	1.75	1.72	2.01	2.52	3.07
Calculated Runoff Volume (acre-ft) =	0.116	0.450	0.310	0.407	0.490	0.506	0.619	0.814	1.066
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.116	0.450	0.310	0.406	0.490	0.506	0.618	0.814	1.066
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.00	0.00	0.01	0.02	0.16	0.43	0.97
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.0	0.1	0.1	0.8	2.2	4.8
Peak Inflow Q (cfs) =	1.8	7.0	4.9	6.3	7.6	7.9	9.6	12.6	16.5
Peak Outflow Q (cfs) =	0.1	6.9	4.2	6.3	7.3	7.4	10.7	13.6	16.1
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	268.8	134.6	70.3	13.4	6.3	3.3
Structure Controlling Flow =	Plate	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1
Max Velocity through Grate 1 (fps) =	N/A	0.61	0.36	0.6	0.6	0.7	1.0	1.2	1.4
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	38	32	35	33	32	31	30	27	24
Time to Drain 99% of Inflow Volume (hours) =	40	39	40	39	39	39	38	36	34
Maximum Ponding Depth (ft) =	3.37	3.87	3.77	3.85	3.88	3.89	3.99	4.07	4.13
Area at Maximum Ponding Depth (acres) =	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08
Maximum Volume Stored (acre-ft) =	0.106	0.139	0.133	0.138	0.141	0.141	0.148	0.154	0.159

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)



S-A-V-D Chart Axis Override	X-axis	Left Y-Axis	Right Y-Axis
minimum bound			
maximum bound			

Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs   UD-Detention, Version 3.07 (February 2017)

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

Time Interval: 5.35 min

Hydrograph Constant: 0.935

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
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]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:05:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:10:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:16:03	0.08	0.31	0.22	0.28	0.34	0.35	0.43	0.56	0.72
0:21:24	0.22	0.84	0.58	0.76	0.91	0.94	1.15	1.50	1.95
0:26:45	0.57	2.16	1.50	1.96	2.35	2.42	2.94	3.85	5.01
0:32:06	1.58	5.93	4.13	5.37	6.46	6.65	8.09	10.58	13.76
0:37:27	1.84	7.01	4.85	6.34	7.64	7.87	9.60	12.60	16.45
0:42:48	1.74	6.68	4.62	6.04	7.28	7.51	9.16	12.03	15.72
0:48:09	1.58	6.08	4.21	5.50	6.63	6.83	8.34	10.95	14.31
0:53:30	1.40	5.42	3.74	4.90	5.91	6.10	7.45	9.79	12.82
0:58:51	1.20	4.67	3.21	4.22	5.09	5.25	6.43	8.47	11.10
1:04:12	1.05	4.07	2.81	3.68	4.44	4.58	5.60	7.37	9.66
1:09:33	0.95	3.69	2.54	3.33	4.02	4.15	5.07	6.68	8.75
1:14:54	0.77	3.03	2.08	2.74	3.31	3.42	4.19	5.53	7.26
1:20:15	0.61	2.47	1.69	2.23	2.70	2.78	3.42	4.52	5.95
1:25:36	0.46	1.89	1.28	1.70	2.07	2.14	2.63	3.50	4.62
1:30:57	0.33	1.40	0.94	1.26	1.53	1.58	1.96	2.61	3.48
1:36:18	0.24	1.02	0.69	0.91	1.11	1.15	1.42	1.89	2.52
1:41:39	0.19	0.79	0.54	0.71	0.86	0.89	1.10	1.46	1.94
1:47:00	0.16	0.65	0.44	0.59	0.71	0.74	0.90	1.20	1.59
1:52:21	0.14	0.55	0.38	0.50	0.61	0.62	0.77	1.02	1.35
1:57:42	0.12	0.49	0.33	0.44	0.53	0.55	0.67	0.89	1.18
2:03:03	0.11	0.44	0.30	0.40	0.48	0.49	0.61	0.80	1.06
2:08:24	0.10	0.41	0.28	0.37	0.44	0.46	0.56	0.74	0.98
2:13:45	0.07	0.30	0.20	0.27	0.32	0.33	0.41	0.54	0.72
2:19:06	0.05	0.22	0.15	0.20	0.24	0.25	0.30	0.40	0.53
2:24:27	0.04	0.16	0.11	0.14	0.17	0.18	0.22	0.29	0.39
2:29:48	0.03	0.12	0.08	0.11	0.13	0.13	0.16	0.22	0.28
2:35:09	0.02	0.08	0.06	0.07	0.09	0.09	0.12	0.15	0.20
2:40:30	0.01	0.06	0.04	0.05	0.06	0.07	0.08	0.11	0.15
2:45:51	0.01	0.04	0.03	0.04	0.05	0.05	0.06	0.08	0.10
2:51:12	0.01	0.03	0.02	0.02	0.03	0.03	0.04	0.05	0.07
2:56:33	0.00	0.02	0.01	0.01	0.02	0.02	0.02	0.03	0.04
3:01:54	0.00	0.01	0.00	0.01	0.01	0.01	0.01	0.02	0.02
3:07:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3:12:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:17:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:23:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:28:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:34:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:39:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:44:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:50:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:55:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:00:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:06:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:11:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:16:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:22:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:27:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:32:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:38:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:43:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:48:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:54:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:59:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:04:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:10:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:15:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:21:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:26:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:31:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:37:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:42:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:47:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:53:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:58:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:03:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:09:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:14:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:19:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:25:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Worksheet Protected

Site-Level Low Impact Development (LID) Design Effective Impervious Calculator

LID Credit by Impervious Reduction Factor (IRF) Method

UD-BMP (Version 3.06, November 2016)

User Input

Calculated cells

Designer: Eric Gunderson, P.E.
Company: Kimley-Horn and Associates
Date: June 19, 2017
Project: US Autoforce
Location: 2570 Zeppelin Rd. South Pond (SP)

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	10-Year Event	1.75	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	CUHP		
(CUHP) NOAA 1 Hour Rainfall Depth and Frequency for User Defined Storm	100-Year Event	2.52	
Max Intensity for Optional User Defined Storm	2.51496		

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	SP													
Receiving Pervious Area Soil Type	Sand													
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	5.020													
Directly Connected Impervious Area (DCIA, acres)	3.544													
Unconnected Impervious Area (UIA, acres)	0.000													
Receiving Pervious Area (RPA, acres)	0.000													
Separate Pervious Area (SPA, acres)	1.476													
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	V													

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	5.020													
Directly Connected Impervious Area (DCIA, %)	70.6%													
Unconnected Impervious Area (UIA, %)	0.0%													
Receiving Pervious Area (RPA, %)	0.0%													
Separate Pervious Area (SPA, %)	29.4%													
A_R (RPA / UIA)	0.000													
I_a Check	1.000													
f / I for WQCV Event:	9.8													
f / I for 10-Year Event:	0.6													
f / I for 100-Year Event:	0.6													
f / I for Optional User Defined Storm CUHP:	0.57													
IRF for WQCV Event:	0.00													
IRF for 10-Year Event:	1.00													
IRF for 100-Year Event:	1.00													
IRF for Optional User Defined Storm CUHP:	1.00													
Total Site Imperviousness: I_{total}	70.6%													
Effective Imperviousness for WQCV Event:	70.6%													
Effective Imperviousness for 10-Year Event:	70.6%													
Effective Imperviousness for 100-Year Event:	70.6%													
Effective Imperviousness for Optional User Defined Storm CUHP:	70.6%													

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV Event CREDIT: Reduce Detention By:	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10-Year Event CREDIT**: Reduce Detention By:	0.0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
100-Year Event CREDIT**: Reduce Detention By:	0.0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
User Defined CUHP CREDIT: Reduce Detention By:	0.0%													

Total Site Imperviousness:	70.6%
Total Site Effective Imperviousness for WQCV Event:	70.6%
Total Site Effective Imperviousness for 10-Year Event:	70.6%
Total Site Effective Imperviousness for 100-Year Event:	70.6%
Total Site Effective Imperviousness for Optional User Defined Storm CUHP:	70.6%

Notes:

* Use Green-Ampt average infiltration rate values from Table 3-3.

** Flood control detention volume credits based on empirical equations from Storage Chapter of USDCM.

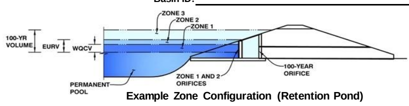
*** Method assumes that 1-hour rainfall depth is equivalent to 1-hour intensity for calculation purposed

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: 2570 Zeppelin Road Conceptual Lot 2 Design

Basin ID:



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	EDB			
Watershed Area =	8.24	acres		
Watershed Length =	780	ft		
Watershed Slope =	0.008	ft/ft		
Watershed Imperviousness =	70.00%	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		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =	Castle Pines - City Office			
Water Quality Capture Volume (WQCV) =	0.189	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.731	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.504	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.51 in.) =	0.661	acre-feet	1.51	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.797	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 1.72 in.) =	0.823	acre-feet		inches
50-yr Runoff Volume (P1 = 2.01 in.) =	1.007	acre-feet		inches
100-yr Runoff Volume (P1 = 2.52 in.) =	1.327	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.07 in.) =	1.740	acre-feet		inches
Approximate 2-yr Detention Volume =	0.477	acre-feet		
Approximate 5-yr Detention Volume =	0.627	acre-feet		
Approximate 10-yr Detention Volume =	0.748	acre-feet		
Approximate 25-yr Detention Volume =	0.770	acre-feet		
Approximate 50-yr Detention Volume =	0.879	acre-feet		
Approximate 100-yr Detention Volume =	1.071	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.189	acre-feet	
Select Zone 2 Storage Volume (Optional) =		acre-feet	**Total detention volume is less than 100-year volume.**
Select Zone 3 Storage Volume (Optional) =		acre-feet	
Total Detention Basin Volume =	0.189	acre-feet	
Initial Surcharge Volume (ISV) =	user	ft^3	
Initial Surcharge Depth (ISD) =	user	ft	
Total Available Detention Depth (H_{total}) =	user	ft	
Depth of Trickle Channel (H_{TC}) =	user	ft	
Slope of Trickle Channel (S_{TC}) =	user	ft/ft	
Slopes of Main Basin Sides (S_{main}) =	user	H:V	
Basin Length-to-Width Ratio ($R_{L/W}$) =	user		
Initial Surcharge Area (A_{ISV}) =	user	ft^2	
Surcharge Volume Length (L_{ISV}) =	user	ft	
Surcharge Volume Width (W_{ISV}) =	user	ft	
Depth of Basin Floor (H_{FLOOR}) =	user	ft	
Length of Basin Floor (L_{FLOOR}) =	user	ft	
Width of Basin Floor (W_{FLOOR}) =	user	ft	
Area of Basin Floor (A_{FLOOR}) =	user	ft^2	
Volume of Basin Floor (V_{FLOOR}) =	user	ft^3	
Depth of Main Basin (H_{MAIN}) =	user	ft	
Length of Main Basin (L_{MAIN}) =	user	ft	
Width of Main Basin (W_{MAIN}) =	user	ft	
Area of Main Basin (A_{MAIN}) =	user	ft^2	
Volume of Main Basin (V_{MAIN}) =	user	ft^3	
Calculated Total Basin Volume (V_{total}) =	user	acre-feet	

Depth Increment = 0.1 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	--	--	--	14	0.000		
	--	0.10	--	--	--	15	0.000	1	0.000
	--	0.20	--	--	--	15	0.000	3	0.000
	--	0.30	--	--	--	15	0.000	4	0.000
	--	0.40	--	--	--	15	0.000	6	0.000
	--	0.50	--	--	--	15	0.000	7	0.000
	--	0.60	--	--	--	15	0.000	9	0.000
	--	0.70	--	--	--	15	0.000	10	0.000
	--	0.80	--	--	--	15	0.000	12	0.000
	--	0.90	--	--	--	15	0.000	13	0.000
	--	1.00	--	--	--	15	0.000	15	0.000
	--	1.10	--	--	--	15	0.000	16	0.000
	--	1.20	--	--	--	15	0.000	18	0.000
	--	1.30	--	--	--	15	0.000	19	0.000
	--	1.40	--	--	--	15	0.000	21	0.000
	--	1.50	--	--	--	15	0.000	22	0.001
	--	1.60	--	--	--	15	0.000	24	0.001
	--	1.70	--	--	--	15	0.000	25	0.001
	--	1.80	--	--	--	16	0.000	27	0.001
	--	1.90	--	--	--	16	0.000	28	0.001
	--	2.00	--	--	--	16	0.000	30	0.001
	--	2.10	--	--	--	16	0.000	31	0.001
	--	2.20	--	--	--	16	0.000	33	0.001
	--	2.30	--	--	--	16	0.000	35	0.001
	--	2.40	--	--	--	16	0.000	36	0.001
	--	2.50	--	--	--	16	0.000	38	0.001
	--	2.60	--	--	--	16	0.000	39	0.001
	--	2.80	--	--	--	40	0.001	45	0.001
	--	2.90	--	--	--	158	0.004	53	0.001
	--	3.00	--	--	--	169	0.004	71	0.002
	--	3.10	--	--	--	249	0.006	90	0.002
	--	3.20	--	--	--	313	0.007	117	0.003
	--	3.30	--	--	--	479	0.011	160	0.004
	--	3.40	--	--	--	593	0.014	208	0.005
	--	3.50	--	--	--	716	0.016	279	0.006
	--	3.60	--	--	--	834	0.019	348	0.008
	--	3.70	--	--	--	947	0.022	436	0.010
	--	3.80	--	--	--	1,065	0.024	546	0.013
	--	3.90	--	--	--	1,167	0.027	646	0.015
	--	4.00	--	--	--	1,264	0.029	779	0.018
	--	4.10	--	--	--	1,355	0.031	897	0.021
	--	4.20	--	--	--	1,441	0.033	1,036	0.024
	--	4.30	--	--	--	1,508	0.035	1,198	0.027
	--	4.40	--	--	--	1,573	0.036	1,336	0.031
	--	4.50	--	--	--	1,639	0.038	1,512	0.035
	--	4.60	--	--	--	1,705	0.039	1,662	0.038
	--	4.70	--	--	--	1,773	0.041	1,836	0.042
	--	4.80	--	--	--	1,841	0.042	2,034	0.047
	--	4.90	--	--	--	1,910	0.044	2,203	0.051
	--	5.00	--	--	--	1,980	0.045	2,416	0.055
	--	5.10	--	--	--	2,051	0.047	2,597	0.060
	--	5.20	--	--	--	2,122	0.049	2,805	0.064
	--	5.30	--	--	--	2,195	0.050	3,042	0.070
	--	5.40	--	--	--	2,268	0.052	3,243	0.074
	--	5.50	--	--	--	2,342	0.054	3,496	0.080
	--	5.60	--	--	--	2,417	0.055	3,710	0.085
	--	5.70	--	--	--	2,493	0.057	3,954	0.091
	--	5.80	--	--	--	2,569	0.059	4,232	0.097
	--	5.90	--	--	--	2,647	0.061	4,467	0.103
	--	6.00	--	--	--	2,725	0.063	4,762	0.109
	--	6.10	--	--	--	2,804	0.064	5,010	0.115
	--	6.20	--	--	--	2,884	0.066	5,294	0.122
	--	6.30	--	--	--	2,979	0.068	5,616	0.129
	--	6.40	--	--	--	3,075	0.071	5,888	0.135
	--	6.50	--	--	--	3,173	0.073	6,231	0.143
	--	6.60	--	--	--	3,272	0.075	6,520	0.150
	--	6.70	--	--	--	3,373	0.077	6,852	0.157
	--	6.80	--	--	--	3,476	0.080	7,228	0.166
	--	6.90	--	--	--	3,580	0.082	7,545	0.173
	--	7.00	--	--	--	3,686	0.085	7,944	0.182
	--	7.10	--	--	--	3,794	0.087	8,280	0.190
	--	7.20	--	--	--	3,903	0.090	8,664	0.199
	--	7.30	--	--	--	4,013	0.092	9,099	0.209
	--	7.40	--	--	--	4,126	0.095	9,464	0.217
	--	7.50	--	--	--	4,240	0.097	9,924	0.228
	--	7.60	--	--	--	4,355	0.100	10,310	0.237
	--	7.70	--	--	--	4,472	0.103	10,750	0.247
	--	7.80	--	--	--	4,591	0.105	11,248	0.258
	--	7.90	--	--	--	4,711	0.108	11,666	0.268
	--	8.00	--	--	--	4,833	0.111	12,190	0.280
	--	8.10	--	--	--	4,957	0.114	12,630	0.290
	--	8.20	--	--	--	5,550	0.127	13,150	0.302
	--	8.30	--	--	--	5,723	0.131	13,769	0.316
	--	8.40	--	--	--	5,861	0.135	14,289	0.328
	--	8.50	--	--	--	5,994	0.138	14,941	0.343
	--	8.60	--	--	--	6,128	0.141	15,486	0.356
	--	8.70	--	--	--	6,260	0.144	16,104	0.370
	--	8.80	--	--	--	6,391	0.147	16,799	0.386
	--	8.90	--	--	--	6,522	0.150	17,379	0.399
	--	9.00	--	--	--	6,654	0.153	18,103	0.416
	--	9.10	--	--	--	6,787	0.156	18,708	0.429
	--	9.20	--	--	--	6,917	0.159	19,391	0.445
	--	9.30	--	--	--	7,044	0.162	20,159	0.463
	--	9.40	--	--	--	7,171	0.165	20,798	0.477
	--	9.50	--	--	--	7,177	0.165	21,587	0.496
	--		--	--	--				
	--		--	--	--				
	--		--	--	--				
	--		--	--	--				

Rip-Rap Calculation

North Pond - Emergency Overflow

.

Applicable Equations:

$L_p = (1/2\tan\Theta)(A_t/Y_t\text{-}D)$	Equation 9-11 per USCDM
$A_t = Q/V$	Equation 9-12 per USDCM
$\Theta = \tan^{-1}(1/(2*\text{ExpansionFactor}))$	Equation 9-13 per USDCM
$W = 2(L_p\tan\Theta)+D$	Equation 9-14 per USDCM
$T = 2D_{50}$	Equation 9-15 per USDCM

Assumptions

Maximum Major Event Velocity is 7fps for FES outletting into trickle channels

Input parameters:

Description		Variable	Input	Unit
Width of the conduit (use diameter for circular conduits),		D:	3.00	ft
HGL Elevation			97.65	ft
Invert Elevation			97.00	ft
Tailwater depth (ft),		Y_t:	0.65	ft
Expansion angle of the culvert flow		Θ:	0.62	radians
Design discharge (cfs)*		Q:	25.11	cfs
Froude Number		F_r	0.78	Subcritical
Unitless Variables for Tables:				
	For Figure 9-35	$Q/D^{2.5}$	1.61	
	For Figure 9-35	Y_t/D	0.22	
	For Figure 9-38	$Q/D^{1.5}$	4.83	
	For Figure 9-38	Y_t/D	0.22	
Allowable non-eroding velocity in the downstream channel (ft/sec)		V:	5	ft/sec
Expansion Factor (Figure 9-35), 1/(2tan(θ))			0.7	

Solve for:

Description	Variable	Output	Unit
1. Required area of flow at allowable velocity (ft^2)	A_t:	5.02	ft^2
2. Length of Protection	L_p:	3.31	ft
	L_p < 3D?	Yes	
	L_{pmin}:	9.00	ft
3. Width of downstream riprap protection	W:	16.00	ft
4. Rip Rap Type (Figure 9-38)	-	L	
5. Rip Rap Size (Figure 8-34)	D_{50}:	9	inches

Rip Rap Summary

Length	L_p	9.00	ft
Width	W	16.00	ft
Size	D_{50}	9	inches
Type	-	L	-
Thickness	T	18	inches

Rip-Rap Calculation

South Pond - Emergency Overflow

.

Applicable Equations:

$L_p = (1/2\tan\Theta)(A_t/Y_t\text{-}D)$	Equation 9-11 per USCDM
$A_t = Q/V$	Equation 9-12 per USDCM
$\Theta = \tan^{-1}(1/(2*\text{ExpansionFactor}))$	Equation 9-13 per USDCM
$W = 2(L_p\tan\Theta)+D$	Equation 9-14 per USDCM
$T = 2D_{50}$	Equation 9-15 per USDCM

Assumptions

Maximum Major Event Velocity is 7fps for FES outletting into trickle channels

Input parameters:

Description	Variable	Input	Unit
Width of the conduit (use diameter for circular conduits),	D:	4.00	ft
HGL Elevation		96.75	ft
Invert Elevation		96.00	ft
Tailwater depth (ft),	Y_t:	0.75	ft
Expansion angle of the culvert flow	Θ:	0.62	radians
Design discharge (cfs)*	Q:	30.83	cfs
Froude Number	F_r	0.50	Subcritical
Unitless Variables for Tables:			
For Figure 9-35	$Q/D^{2.5}$	0.96	
For Figure 9-35	Y_t/D	0.19	
For Figure 9-38	$Q/D^{1.5}$	3.85	
For Figure 9-38	Y_t/D	0.19	
Allowable non-eroding velocity in the downstream channel (ft/sec)	V:	5	ft/sec
Expansion Factor (Figure 9-35), 1/(2tan(θ))		0.7	

Solve for:

Description	Variable	Output	Unit
1. Required area of flow at allowable velocity (ft^2)	A_t:	6.17	ft^2
2. Length of Protection	L_p:	2.95	ft
	L_p < 3D?	Yes	
	L_{pmin}:	12.00	ft
3. Width of downstream riprap protection	W:	18.00	ft
4. Rip Rap Type (Figure 9-38)	-	L	
5. Rip Rap Size (Figure 8-34)	D_{50}:	9	inches

Rip Rap Summary

Length	L_p	12.00	ft
Width	W	18.00	ft
Size	D_{50}	9	inches
Type	-	L	-
Thickness	T	18	inches

Rip-Rap Calculation

ST C-FES 1

.

Applicable Equations:

$L_p = (1/2\tan\Theta)(A_t/Y_t\text{-}D)$	Equation 9-11 per USCDM
$A_t = Q/V$	Equation 9-12 per USDCM
$\Theta = \tan^{-1}(1/(2*\text{ExpansionFactor}))$	Equation 9-13 per USDCM
$W = 2(L_p\tan\Theta)+D$	Equation 9-14 per USDCM
$T = 2D_{50}$	Equation 9-15 per USDCM

Assumptions

Maximum Major Event Velocity is 7fps for FES outletting into trickle channels

Input parameters:

Description	Variable	Input	Unit
Width of the conduit (use diameter for circular conduits),	D:	2.00	ft
HGL Elevation		92.79	ft
Invert Elevation		91.58	ft
Tailwater depth (ft),	Y_t:	1.21	ft
Expansion angle of the culvert flow	Θ:	0.62	radians
Design discharge (cfs)*	Q:	18.60	cfs
Froude Number	F_r	0.95	Subcritical
Unitless Variables for Tables:			
For Figure 9-35	$Q/D^{2.5}$	3.29	
For Figure 9-35	Y_t/D	0.61	
For Figure 9-38	$Q/D^{1.5}$	6.58	
For Figure 9-38	Y_t/D	0.61	
Allowable non-eroding velocity in the downstream channel (ft/sec)	V:	5	ft/sec
Expansion Factor (Figure 9-35), 1/(2tan(θ))		0.7	

Solve for:

Description	Variable	Output	Unit
1. Required area of flow at allowable velocity (ft^2)	A_t:	3.72	ft^2
2. Length of Protection	L_p:	0.75	ft
	L_p < 3D?	Yes	
	L_{pmin}:	6.00	ft
3. Width of downstream riprap protection	W:	11.00	ft
4. Rip Rap Type (Figure 9-38)	-	L	
5. Rip Rap Size (Figure 8-34)	D_{50}:	9	inches

Rip Rap Summary

Length	L_p	6.00	ft
Width	W	11.00	ft
Size	D_{50}	9	inches
Type	-	L	-
Thickness	T	18	inches

Rip-Rap Calculation

ST D FES-1

.

Applicable Equations:

$L_p = (1/2\tan\Theta)(A_t/Y_t\text{-}D)$	Equation 9-11 per USCDM
$A_t = Q/V$	Equation 9-12 per USDCM
$\Theta = \tan^{-1}(1/(2*\text{ExpansionFactor}))$	Equation 9-13 per USDCM
$W = 2(L_p\tan\Theta)+D$	Equation 9-14 per USDCM
$T = 2D_{50}$	Equation 9-15 per USDCM

Assumptions

Maximum Major Event Velocity is 7fps for FES outletting into trickle channels

Input parameters:

Description		Variable	Input	Unit
Width of the conduit (use diameter for circular conduits),		D:	2.50	ft
HGL Elevation			91.47	ft
Invert Elevation			89.59	ft
Tailwater depth (ft),		Y_t:	1.88	ft
Expansion angle of the culvert flow		Θ:	0.62	radians
Design discharge (cfs)*		Q:	30.30	cfs
Froude Number		F_r	0.79	Subcritical
Unitless Variables for Tables:				
	For Figure 9-35	$Q/D^{2.5}$	3.07	
	For Figure 9-35	Y_t/D	0.75	
	For Figure 9-38	$Q/D^{1.5}$	7.67	
	For Figure 9-38	Y_t/D	0.75	
Allowable non-eroding velocity in the downstream channel (ft/sec)		V:	5	ft/sec
Expansion Factor (Figure 9-35), 1/(2tan(θ))			0.7	

Solve for:

Description	Variable	Output	Unit
1. Required area of flow at allowable velocity (ft^2)	A_t:	6.06	ft^2
2. Length of Protection	L_p:	0.51	ft
	L_p < 3D?	Yes	
	L_{pmin}:	7.50	ft
3. Width of downstream riprap protection	W:	13.00	ft
4. Rip Rap Type (Figure 9-38)	-	L	
5. Rip Rap Size (Figure 8-34)	D_{50}:	9	inches

Rip Rap Summary

Length	L_p	8.00	ft
Width	W	13.00	ft
Size	D_{50}	9	inches
Type	-	L	-
Thickness	T	18	inches

Kimley»Horn

Kimley-Horn & Associates, Inc. **Opinion of Probable Construction Cost**

Client:	**SCANNELL PROPERTIES, LLC**	**Date:**	**6/22/2017**
Project:	**US Auto Force, 2570 Zeppelin Road**	**Prepared By:**	**JJM**
KHA No.:	**096441002**	**Checked By:**	**EJG**

Sheet: **1 of 1**

This OPC is not intended for basing financial decisions, or securing funding. Review all notes and assumptions. Since Kimley-Horn & Associates, Inc. has no control over the cost of labor, materials, equipment, or services furnished by others, or over methods of determining price, or over competitive bidding or market conditions, any and all opinions as to the cost herein, including but not limited to opinions as to the costs of construction materials, shall be made on the basis of experience and best available data. Kimley-Horn & Associates, Inc. cannot and does not guarantee that proposals, bids, or actual costs will not vary from the opinions on costs shown herein. The total costs and other numbers in this Opinion of Probable Cost have been rounded.

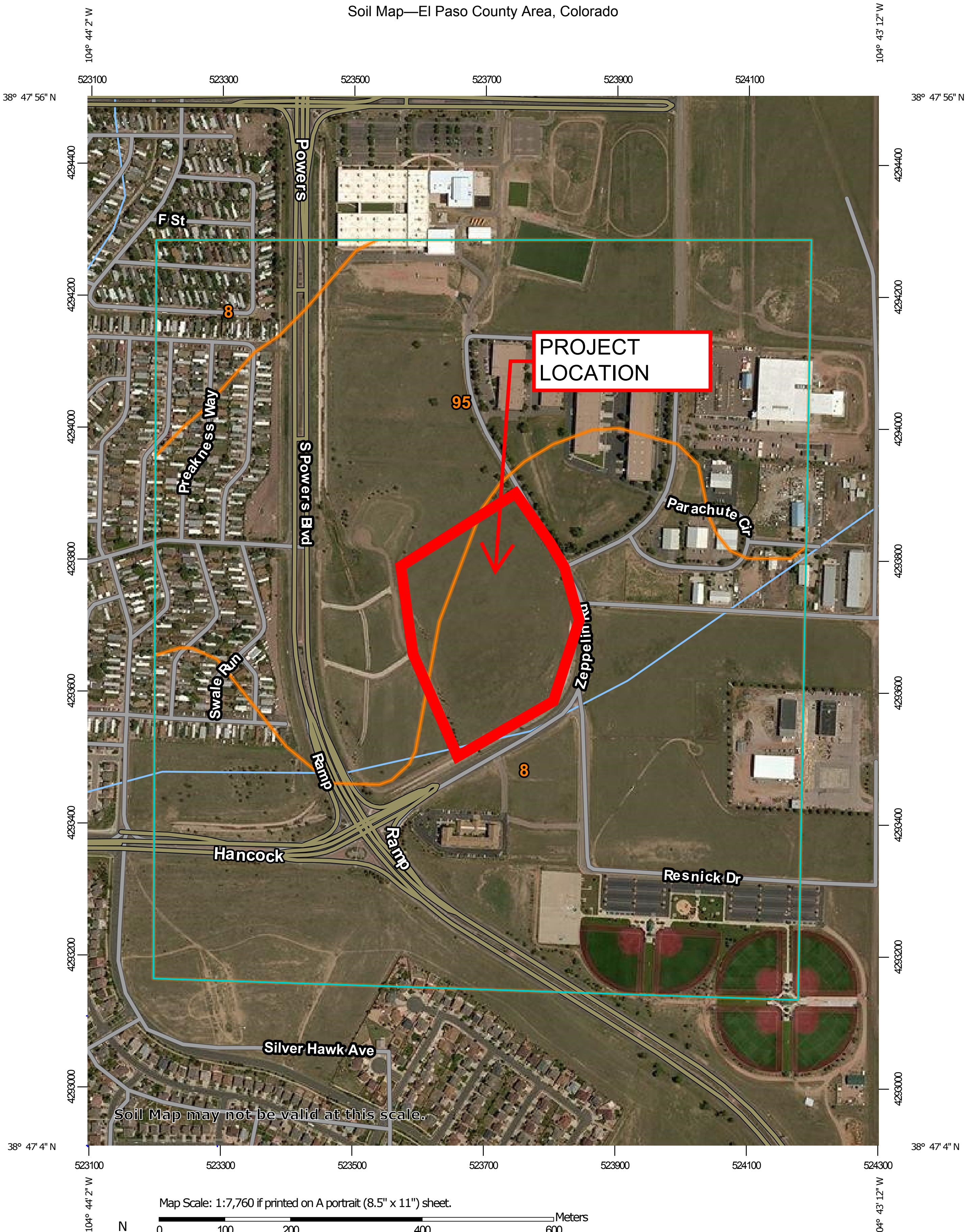
Item No.	Item Description	Quantity	Unit	Unit Price	Item Cost
	Private Storm Sewer				
1	24" RCP	747	LF	$100.00	$74,700
2	30" RCP	248	LF	$125.00	$31,000
	12" PVC Pipe	11,611	LF	$75.00	$870,825
3	CDOT Type C Inlet	3	EA	$1,500.00	$4,500
4	COS Type R 5' Inlet	3	EA	$2,500.00	$7,500
5	12"x12" Nyloplast Area Drain Inlet	5	EA	$200.00	$1,000
6	Concrete Forebay	2	EA	$7,500.00	$15,000
7	Concrete Outlet Structure	2	EA	$10,000.00	$20,000
8	24" Concrete Flared End Section	1	EA	$225.00	$225
9	30" Concrete Flared End Section	1	EA	$250.00	$250
10	Concrete Trickle Channel	100	LF	$10.00	$1,000
		Subtotal:			$1,026,000
		Contingency (%,+/-)		10%	$102,600
		Project Total:			**$1,128,600**

Basis for Cost Projection:

- ☐ No Design Completed
- ☐ Preliminary Design
- ☑ Final Design

Design Engineer:

Eric Gunderson
Registered Professional Engineer, State of Colorado No. 49487



Map Scale: 1:7,760 if printed on A portrait (8.5" x 11") sheet.

Meters: 0, 100, 200, 400, 600

Feet: 0, 350, 700, 1400, 2100

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

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

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

Background
- Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 14, Sep 23, 2016

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

Date(s) aerial images were photographed: Apr 15, 2011—Jun 17, 2014

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

Map Unit Legend

El Paso County Area, Colorado (CO625)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
8	Blakeland loamy sand, 1 to 9 percent slopes	160.3	57.6%
95	Truckton loamy sand, 1 to 9 percent slopes	118.2	42.4%
Totals for Area of Interest		**278.5**	**100.0%**

El Paso County Area, Colorado

95—Truckton loamy sand, 1 to 9 percent slopes

Map Unit Setting

National map unit symbol: 36bd
Elevation: 6,000 to 7,000 feet
Mean annual precipitation: 14 to 16 inches
Mean annual air temperature: 46 to 50 degrees F
Frost-free period: 125 to 145 days
Farmland classification: Not prime farmland

Map Unit Composition

Truckton and similar soils: 85 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Truckton

Setting

Landform: Flats, hills
Landform position (three-dimensional): Side slope, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Arkosic alluvium derived from sedimentary rock and/or arkosic residuum weathered from sedimentary rock

Typical profile

A - 0 to 8 inches: loamy sand
Bt - 8 to 24 inches: sandy loam
C - 24 to 60 inches: coarse sandy loam

Properties and qualities

Slope: 1 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 6.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Low (about 5.4 inches)

Interpretive groups

Land capability classification (irrigated): 4e
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: A
Ecological site: Sandy Foothill (R049BY210CO)
Hydric soil rating: No

Minor Components

Other soils

Percent of map unit:
Hydric soil rating: No

Pleasant

Percent of map unit:
Landform: Depressions
Hydric soil rating: Yes

Data Source Information

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 14, Sep 23, 2016

El Paso County Area, Colorado

8—Blakeland loamy sand, 1 to 9 percent slopes

Map Unit Setting

National map unit symbol: 369v
Elevation: 4,600 to 5,800 feet
Mean annual precipitation: 14 to 16 inches
Mean annual air temperature: 46 to 48 degrees F
Frost-free period: 125 to 145 days
Farmland classification: Not prime farmland

Map Unit Composition

Blakeland and similar soils: 85 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Blakeland

Setting

Landform: Flats, hills
Landform position (three-dimensional): Side slope, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium derived from sedimentary rock and/or eolian deposits derived from sedimentary rock

Typical profile

A - 0 to 11 inches: loamy sand
AC - 11 to 27 inches: loamy sand
C - 27 to 60 inches: sand

Properties and qualities

Slope: 1 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Somewhat excessively drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 5 percent
Available water storage in profile: Low (about 4.5 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: A
Ecological site: Sandy Foothill (R049BY210CO)
Hydric soil rating: No

Minor Components

Other soils

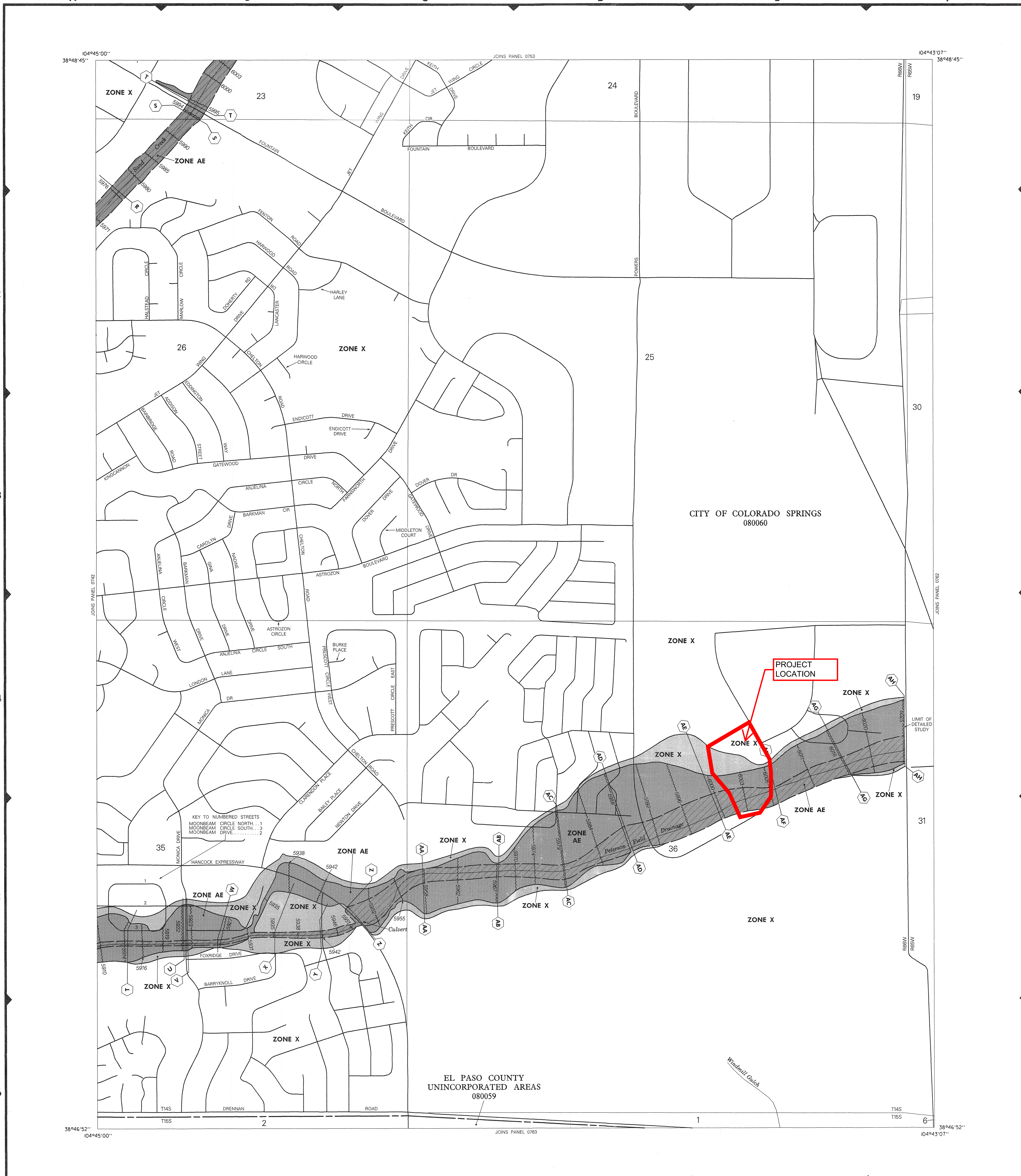
Percent of map unit:
Hydric soil rating: No

Pleasant

Percent of map unit:
Landform: Depressions
Hydric soil rating: Yes

Data Source Information

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 14, Sep 23, 2016



LEGEND

SPECIAL FLOOD HAZARD AREAS INUNDATED BY 100-YEAR FLOOD

ZONE A No base flood elevations determined.

ZONE AE Base flood elevations determined.

ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); base flood elevations determined.

ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.

ZONE A99 To be protected from 100-year flood by Federal flood protection system under construction; no base elevations determined.

ZONE V Coastal flood with velocity hazard (wave action); no base flood elevations determined.

ZONE VE Coastal flood with velocity hazard (wave action); base flood elevations determined.

FLOODWAY AREAS IN ZONE AE

OTHER FLOOD AREAS

ZONE X Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.

OTHER AREAS

ZONE X Areas determined to be outside 500-year floodplain.

ZONE D Areas in which flood hazards are undetermined.

UNDEVELOPED COASTAL BARRIERS

Identified 1983 | Identified 1990 | Otherwise Protected Areas

Coastal barrier areas are normally located within or adjacent to Special Flood Hazard Areas.

Flood Boundary

Floodway Boundary

Zone D Boundary

Boundary Dividing Special Flood Hazard Zones, and Boundary Dividing Areas of Different Coastal Base Flood Elevations Within Special Flood Hazard Zones.

513 Base Flood Elevation Line; Elevation in Feet. See Map Index for Elevation Datum.

D — D Cross Section Line

(EL 987) Base Flood Elevation in Feet Where Uniform Within Zone. See Map Index for Elevation Datum.

RM7 Elevation Reference Mark

M2 River Mile

97°07'30", 32°22'30" Horizontal Coordinates Based on North American Datum of 1927 (NAD 27) Projection.

NOTES

This map is for use in administering the National Flood Insurance Program; it does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size, or all planimetric features outside Special Flood Hazard Areas.

Coastal base flood elevations apply only landward of 0.0 NGVD, and include the effects of wave action; these elevations may also differ significantly from those developed by the National Weather Service for hurricane evacuation planning.

Areas of Special Flood Hazard (100-year flood) include Zones A, AE, AH, AO, A99, V, and VE.

Certain areas not in Special Flood Hazard Areas may be protected by flood control structures.

Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the Federal Emergency Management Agency.

Floodway widths in some areas may be too narrow to show to scale. Floodway widths are provided in the Flood Insurance Study Report.

This map may incorporate approximate boundaries of Coastal Barrier Resource System Units and /or Otherwise Protected Areas established under the Coastal Barrier Improvement Act of 1990 (PL 101-591).

Corporate limits shown are current as of the date of this map. The user should contact appropriate community officials to determine if corporate limits have changed subsequent to the issuance of this map.

For community map revision history prior to countywide mapping, see Section 6.0 of the Flood Insurance Study Report.

For adjoining map panels and base map source see separately printed Map Index.

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:

Refer to the FLOOD INSURANCE RATE MAP EFFECTIVE DATE shown on this map to determine when actuarial rates apply to structures in zones where elevations or depths have been established.

To determine if flood insurance is available, contact an insurance agent or call the National Flood Insurance Program at (800) 638-6620.

APPROXIMATE SCALE IN FEET
500 0 500

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

EL PASO COUNTY, COLORADO AND INCORPORATED AREAS

PANEL 761 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)

CONTAINS: COMMUNITY	NUMBER	PANEL	SUFFIX
COLORADO SPRINGS, CITY OF	080060	0761	F
EL PASO COUNTY, UNINCORPORATED AREAS	080059	0761	F

MAP NUMBER 08041C0761 F

EFFECTIVE DATE: MARCH 17, 1997

Federal Emergency Management Agency

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

IN REPLY REFER TO:
Case No.: 98-08-372P

The Honorable Mary Lou Makepeace
Mayor, City of Colorado Springs
P.O. Box 1575
Colorado Springs, CO 80901-1575

Community: City of Colorado Springs, Colorado
Community No.: 080060
Panel Affected: 08041C0761 F
Effective Date of This Revision: DEC 14 1999

102-D-A

Dear Mayor Makepeace:

This responds to a request that the Federal Emergency Management Agency (FEMA) revise the effective Flood Insurance Rate Map (FIRM) and Flood Insurance Study (FIS) report for El Paso County, Colorado and Incorporated Areas (the effective FIRM and FIS report for your community), in accordance with Part 65 of the National Flood Insurance Program (NFIP) regulations. In a letter dated July 6, 1998, Mr. Dan Bunting, Regional Floodplain Administrator, Pikes Peak Regional Building Department, requested that FEMA revise the FIRM and FIS report to show the effects of channelization of Peterson Field Drainage from approximately 900 feet upstream to approximately 6,500 feet upstream of Hancock Expressway and construction of two 8-foot by 9-foot concrete box culverts under Powers Avenue approximately 3,300 feet upstream of Hancock Expressway and two 8-foot by 9-foot concrete box culverts under Zeppelin Drive approximately 5,200 feet upstream of Hancock Expressway.

All data required to complete our review of this request were submitted with letters from Mr. Michael A. Chaves, Project Manager, Engineering Division, City of Colorado Springs; [redacted], P.E., Project Engineer, URS Greiner Woodward Clyde, Inc.; and Mr. Bunting.

We have completed our review of the submitted data and the flood data shown on the effective FIRM and FIS report. We have revised the FIRM and FIS report to modify the elevations and floodplain and floodway boundary delineations of the flood having a 1-percent chance of being equaled or exceeded in any given year (base flood) along the revised reach of Peterson Field Drainage from just upstream to approximately 6,500 feet upstream of Hancock Expressway. As a result of the modifications, the base flood elevations (BFEs) for the revised reach and the widths of the Special Flood Hazard Area (SFHA), the area that would be inundated by the base flood, and the regulatory floodway decreased. As a result of the channelization, the base flood and the regulatory floodway are contained in the engineered channel from approximately 2,000 feet upstream to approximately 6,500 feet upstream of Hancock Expressway.

The modifications are shown on the enclosed annotated copies of FIRM Panel(s) 08041C0761 F, Profile Panel(s) 177P and 178P, and affected portions of the Floodway Data Table. This Letter of Map Revision (LOMR) hereby revises the above-referenced panel(s) of the effective FIRM and the affected portions of the FIS report, both dated March 17, 1997.

The modifications are effective as of the date shown above. The map panel(s) as listed above and as modified by this letter will be used for all flood insurance policies and renewals issued for your community.

The following table is a partial listing of existing and modified BFEs:

Location	Existing BFE (feet)*	Modified BFE (feet)*
Approximately 1,300 feet upstream of Hancock Expressway	5,962	5,960
Approximately 1,700 feet upstream of Hancock Expressway	5,967	5,963
Approximately 2,000 feet upstream of Hancock Expressway	5,972	5,965

*Referenced to the National Geodetic Vertical Datum, rounded to the nearest whole foot

Public notification of the modified BFEs will be given in the *Gazette Telegraph* on or about January 4 and January 11, 2000. A copy of this notification is enclosed. In addition, a notice of changes will be published in the *Federal Register*. Within 90 days of the second publication in the *Gazette Telegraph*, a citizen may request that FEMA reconsider the determination made by this LOMR. Any request for reconsideration must be based on scientific or technical data. All interested parties are on notice that, until the 90-day period elapses, the determination to modify the BFEs presented in this LOMR may itself be modified.

Because this LOMR will not be printed and distributed to primary users, such as local insurance agents and mortgage lenders, your community will serve as a repository for these new data. We encourage you to disseminate the information reflected by this LOMR throughout the community, so that interested persons, such as property owners, local insurance agents, and mortgage lenders, may benefit from the information. We also encourage you to prepare a related article for publication in your community's local newspaper. This article should describe the assistance that officials of your community will give to interested persons by providing these data and interpreting the NFIP maps.

We will not physically revise and republish the FIRM and FIS report for your community to reflect the modifications made by this LOMR at this time. When changes to the previously cited FIRM panel(s) and FIS report warrant physical revision and republication in the future, we will incorporate the modifications made by this LOMR at that time.

The floodway is provided to your community as a tool to regulate floodplain development. Therefore, the floodway modifications described in this LOMR, while acceptable to FEMA, must also be acceptable to your community and adopted by appropriate community action, as specified in Paragraph 60.3(d) of the NFIP regulations.

This LOMR is based on minimum floodplain management criteria established under the NFIP. Your community is responsible for approving all floodplain development, and for ensuring all necessary permits required by Federal or State law have been received. State, county, and community officials, based on knowledge of local conditions and in the interest of safety, may set higher standards for construction in the SFHA. If the State, county, or community has adopted more restrictive or comprehensive floodplain management criteria, these criteria take precedence over the minimum NFIP criteria.

The basis of this LOMR is, in whole or in part, a channel-modification/culvert project. NFIP regulations, as cited in Paragraph 60.3(b)(7), require that communities ensure that the flood-carrying capacity within the altered or relocated portion of any watercourse is maintained. This provision is incorporated into your community's existing floodplain management regulations. Consequently, the ultimate responsibility for maintenance of the modified channel and culverts rests with your community.

This determination has been made pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (Public Law 93-234) and is in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, Public Law 90-448), 42 U.S.C. 4001-4128, and 44 CFR Part 65. Pursuant to Section 1361 of the National Flood Insurance Act of 1968, as amended, communities participating in the NFIP are required to adopt and enforce floodplain management regulations that meet or exceed NFIP criteria. These criteria are the minimum requirements and do not supersede any State or local requirements of a more stringent nature. This includes adoption of the effective FIRM and FIS report to which the regulations apply and the modifications described in this LOMR.

FEMA makes flood insurance available in participating communities; in addition, we encourage communities to develop their own loss reduction and prevention programs. Our Project Impact initiative, developed by FEMA Director James Lee Witt, seeks to focus the energy of businesses, citizens, and communities in the United States on the importance of reducing their susceptibility to the impact of all natural disasters, including floods, hurricanes, severe storms, earthquakes, and wildfires. Natural hazard mitigation is most effective when it is planned for and implemented at the local level, by the entities who are most knowledgeable of local conditions and whose economic stability and safety are at stake. For your information, we are enclosing a Project Impact Fact Sheet. For additional information on Project Impact, please visit our Web site at **www.fema.gov**.

If you have any questions regarding floodplain management regulations for your community or the NFIP in general, please contact the Consultation Coordination Officer (CCO) for your community. Information on the CCO for your community may be obtained by contacting the Director, Mitigation Division of FEMA in Denver, Colorado, at (303) 235-4830. If you have any technical questions regarding this LOMR, please contact Ms. Sally Magee of our staff in Washington, DC, either by telephone at (202) 646-8242 or by facsimile at (202) 646-4596.

Sincerely,

Sally Magee

Sally P. Magee, Project Engineer
Hazards Study Branch
Mitigation Directorate

For: Matthew B. Miller, P.E., Chief
Hazards Study Branch
Mitigation Directorate

Enclosure(s)

cc: Mr. Dan Bunting
Regional Floodplain Administrator
Pikes Peak Regional
Building Department

Mr. Brian Hyde
Senior Water Resources Specialist
Colorado Water Conservation Board
Department of Conservation

Mr. Michael A. Chaves
Project Manager
Engineering Division
City of Colorado Springs

[redacted], P.E.
Project Engineer
URS Greiner Woodward Clyde, Inc.

CHANGES ARE MADE IN DETERMINATIONS OF BASE FLOOD ELEVATIONS FOR THE CITY OF COLORADO SPRINGS, EL PASO COUNTY, COLORADO, UNDER THE NATIONAL FLOOD INSURANCE PROGRAM

On March 17, 1997, the Federal Emergency Management Agency identified Special Flood Hazard Areas (SFHAs) in the City of Colorado Springs, El Paso County, Colorado, through issuance of a Flood Insurance Rate Map (FIRM). The Mitigation Directorate has determined that modification of the elevations of the flood having a 1-percent chance of being equaled or exceeded in any given year (base flood) for certain locations in this community is appropriate. The modified base flood elevations (BFEs) revise the FIRM for the community.

The changes are being made pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (Public Law 93-234) and are in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, Public Law 90-448), 42 U.S.C. 4001-4128, and 44 CFR Part 65.

A hydraulic analysis was performed to incorporate channelization of Peterson Field Drainage from approximately 900 feet upstream to approximately 6,500 feet upstream of Hancock Expressway and construction of two 8-foot by 9-foot concrete box culverts under Powers Avenue approximately 3,300 feet upstream of Hancock Expressway and two 8-foot by 9-foot concrete box culverts under Zeppelin Drive approximately 5,200 feet upstream of Hancock Expressway. This has resulted in a revised delineation of the regulatory floodway, a decrease in SFHA width, and decreased BFEs for the revised reach of Peterson Field Drainage from just upstream to approximately 6,500 feet upstream of Hancock Expressway. As a result of channelization, the base flood and floodway are contained in the engineered channel from approximately 2,000 feet upstream to approximately 6,500 feet upstream of Hancock Expressway. The table below indicates existing and modified BFEs for selected locations along the affected lengths of the flooding source(s) cited above.

Location	Existing BFE (feet)*	Modified BFE (feet)*
Approximately 1,300 feet upstream of Hancock Expressway	5,962	5,960
Approximately 1,700 feet upstream of Hancock Expressway	5,967	5,963
Approximately 2,000 feet upstream of Hancock Expressway	5,972	5,965

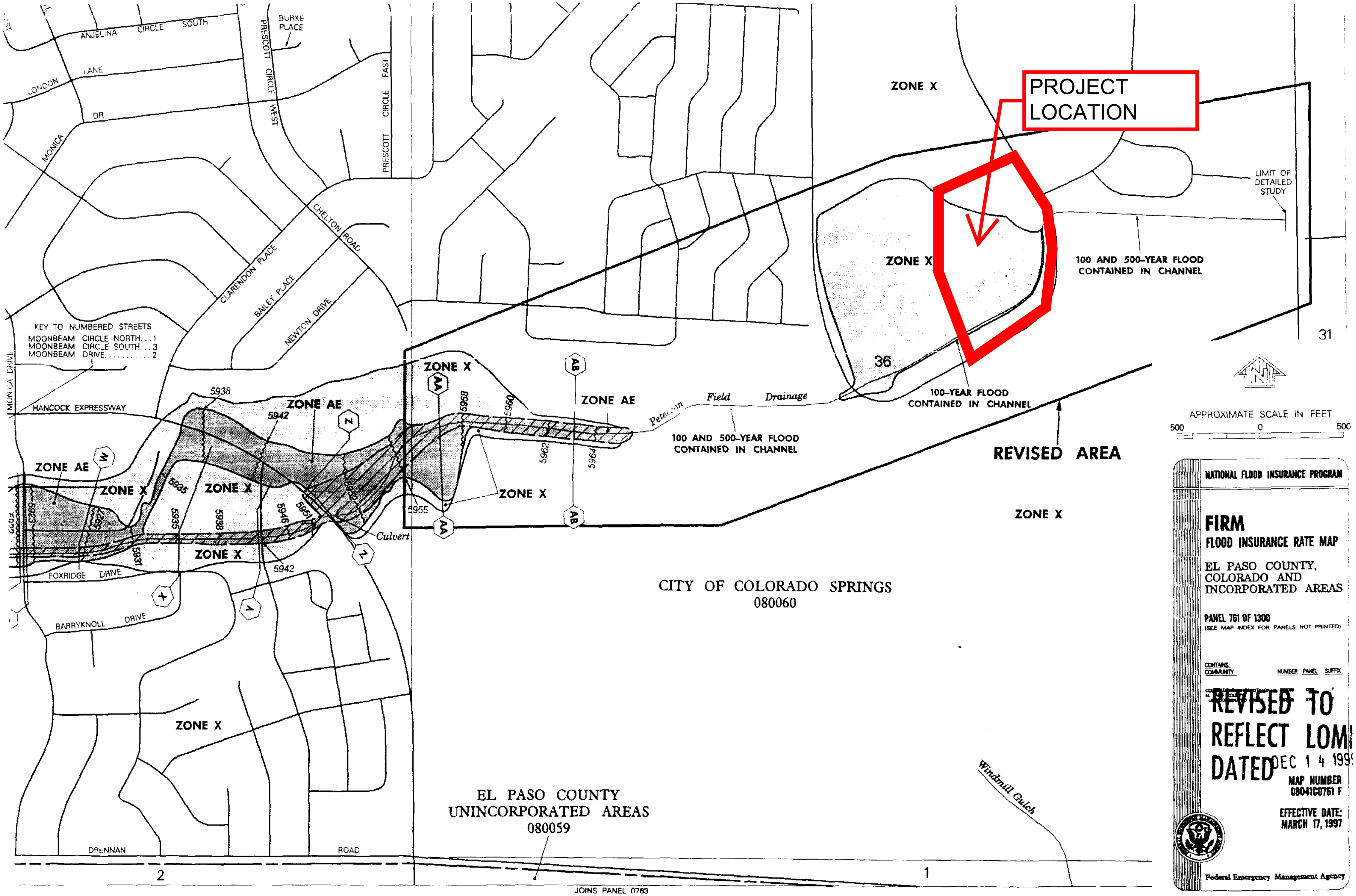
*National Geodetic Vertical Datum, rounded to nearest whole foot

Under the above-mentioned Acts of 1968 and 1973, the Mitigation Directorate must develop criteria for floodplain management. To participate in the National Flood Insurance Program (NFIP), the community must use the modified BFEs to administer the floodplain management measures of the NFIP. These modified BFEs will also be used to calculate the appropriate flood insurance premium rates for new buildings and their contents and for the second layer of insurance on existing buildings and contents.

Upon the second publication of notice of these changes in this newspaper, any person has 90 days in which he or she can request, through the Chief Executive Officer of the community, that the Mitigation Directorate reconsider the determination. Any request for reconsideration must be based on knowledge of changed conditions or new scientific or technical data. All interested parties are on notice that until the 90-day period elapses, the Mitigation Directorate's determination to modify the BFEs may itself be changed.

Any person having knowledge or wishing to comment on these changes should immediately notify:

The Honorable Mary Lou Makepeace
Mayor, City of Colorado Springs
P.O. Box 1575
Colorado Springs, Colorado 80901-1575



PROJECT LOCATION
ZONE X
ZONE AE
LIMIT OF DETAILED STUDY
100 AND 500-YEAR FLOOD CONTAINED IN CHANNEL
100-YEAR FLOOD CONTAINED IN CHANNEL
REVISED AREA
Peterson Field Drainage
Culvert
KEY TO NUMBERED STREETS
MOONBEAM CIRCLE NORTH...1
MOONBEAM CIRCLE SOUTH...3
MOONBEAM DRIVE...2
HANCOCK EXPRESSWAY
FOXRIDGE DRIVE
BARRYKNOLL DRIVE
DRENNAN ROAD
ANJELINA CIRCLE SOUTH
LONDON LANE DR
MONICA
PRESCOTT CIRCLE WEST
PRESCOTT CIRCLE EAST
BURKE PLACE
CHELTON ROAD
CLARENDON PLACE
BAILEY PLACE
NEWTON DRIVE
Windmill Gulch
CITY OF COLORADO SPRINGS
080060
EL PASO COUNTY
UNINCORPORATED AREAS
080059
JOINS PANEL 0763
36
31
APPROXIMATE SCALE IN FEET
500 0 500
NATIONAL FLOOD INSURANCE PROGRAM
FIRM
FLOOD INSURANCE RATE MAP
EL PASO COUNTY, COLORADO AND INCORPORATED AREAS
PANEL 761 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)
REVISED TO REFLECT LOMR DATED DEC 14 1999
MAP NUMBER 08041C0761 F
EFFECTIVE DATE: MARCH 17, 1997
Federal Emergency Management Agency

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY (FEET NGVD)	WITHOUT FLOODWAY (FEET NGVD)	WITH FLOODWAY (FEET NGVD)	INCREASE (FEET NGVD)
Peterson Field Drainage (Cont'd)								
				Revised Data				
AA	11,858	60	324	7.6	5,957.0	5,957.0	5,957.0	0.0
AB	12,655	70	301	8.2	5,963.5	5,963.5	5,963.5	0.0

[1]Feet Above Confluence With Sand Creek

REVISED TO REFLECT LOMR DATED DEC 1 4 1999

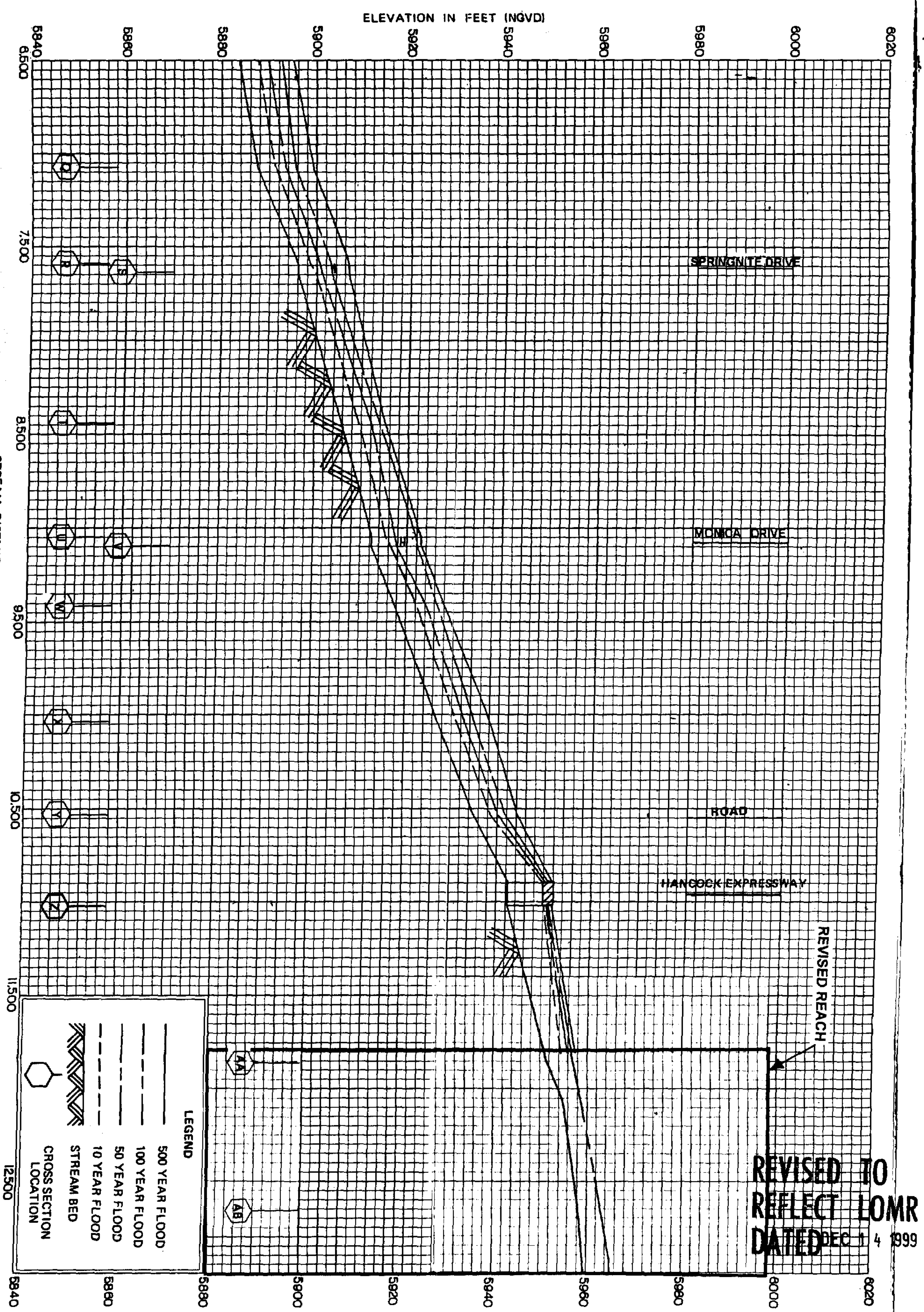
TABLE 5

FEDERAL EMERGENCY MANAGEMENT AGENCY

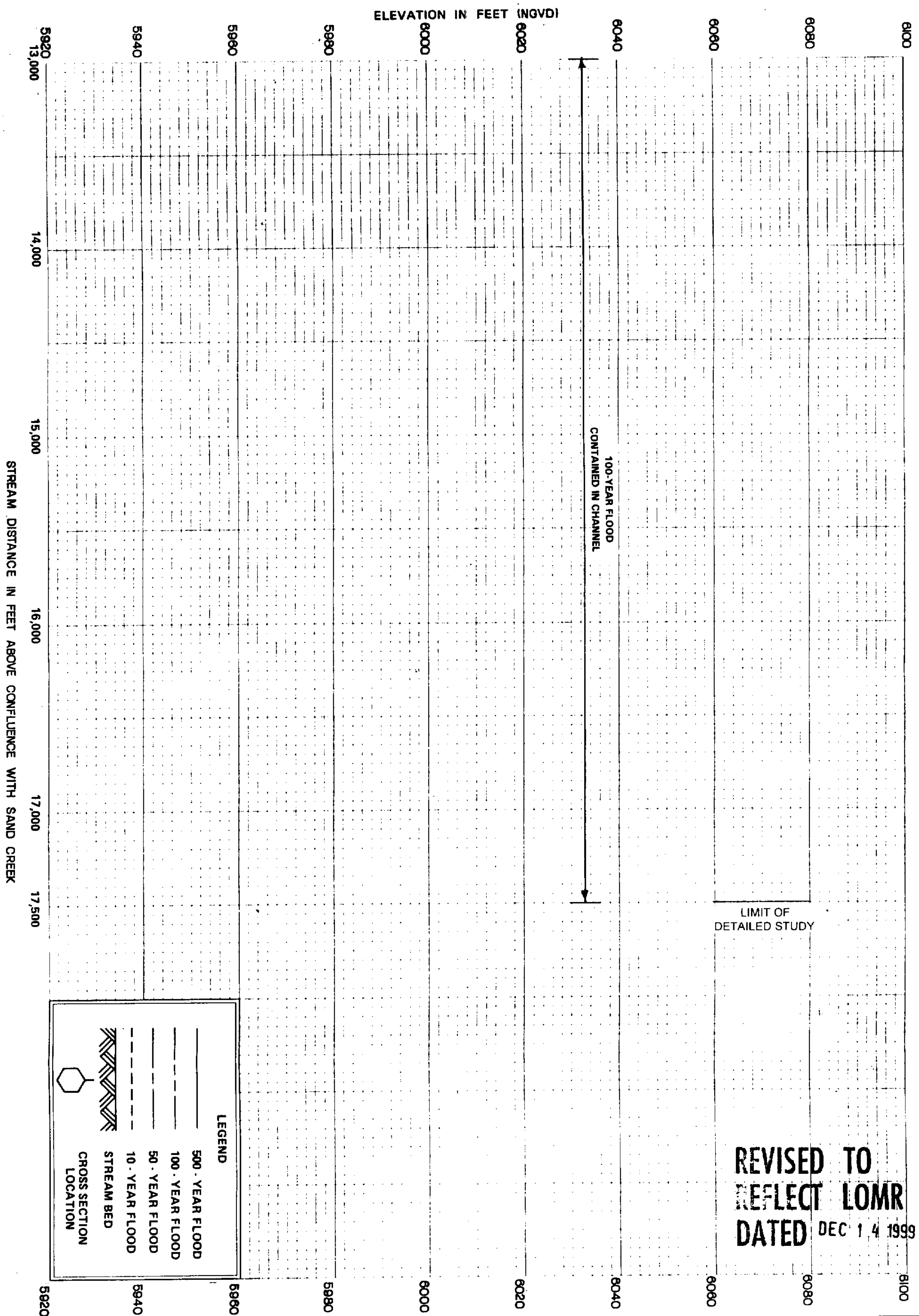
EL PASO COUNTY, CO
AND INCORPORATED AREAS

FLOODWAY DATA

PETERSON FIELD DRAINAGE



177P	FEDERAL EMERGENCY MANAGEMENT AGENCY EL PASO COUNTY,CO AND INCORPORATED AREAS	FLOOD PROFILES PETERSON FIELD DRAINAGE



REVISED TO REFLECT LOMR DATED DEC 14 1999

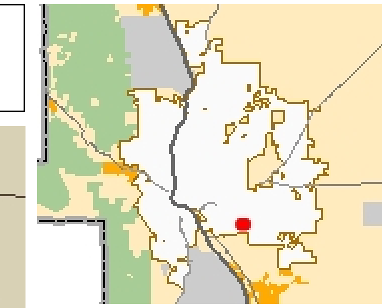
FEDERAL EMERGENCY MANAGEMENT AGENCY

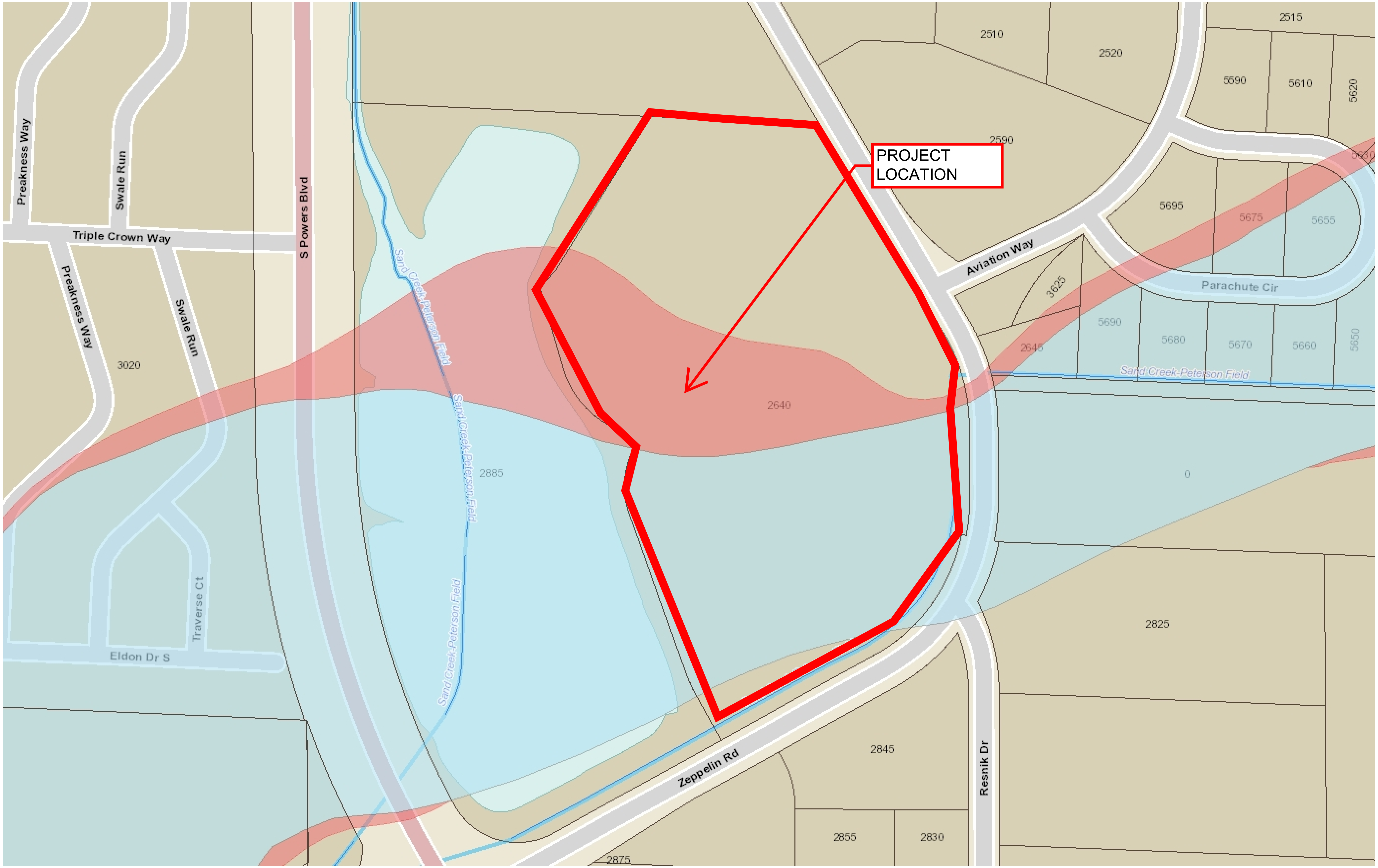
EL PASO COUNTY,CO
AND INCORPORATED AREAS

FLOOD PROFILES

PETERSON FIELD DRAINAGE

City Of Colorado Springs





Legend

- Parcels
- Open Drainage
- Flood (100 yr)
- Flood (500 yr)
- Towns
- County Boundary
- Bus Stops
- Hospitals
- Schools
- Streets
  - Interstate
  - Major Roads
  - Local Roads
  - Ramps
- Trails_Parks
- Trails_Urban_Paved
- Trails_Urban_Gravel
- Trails_Urban_Natural
- Creeks
- Railroads
- Parcels
- Golf Fairways
- Streams
- Parks
- Lakes
- Military
- National Forest

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Notes

SITE DRAINAGE MAP AND PLANS

LEGEND

A = BASIN DESIGNATION
B = AREA (ACRES)
C = BASIN IMPERVIOUSNESS
D = 100YR DESIGN STORM RUNOFF (CFS)

= DESIGN POINT

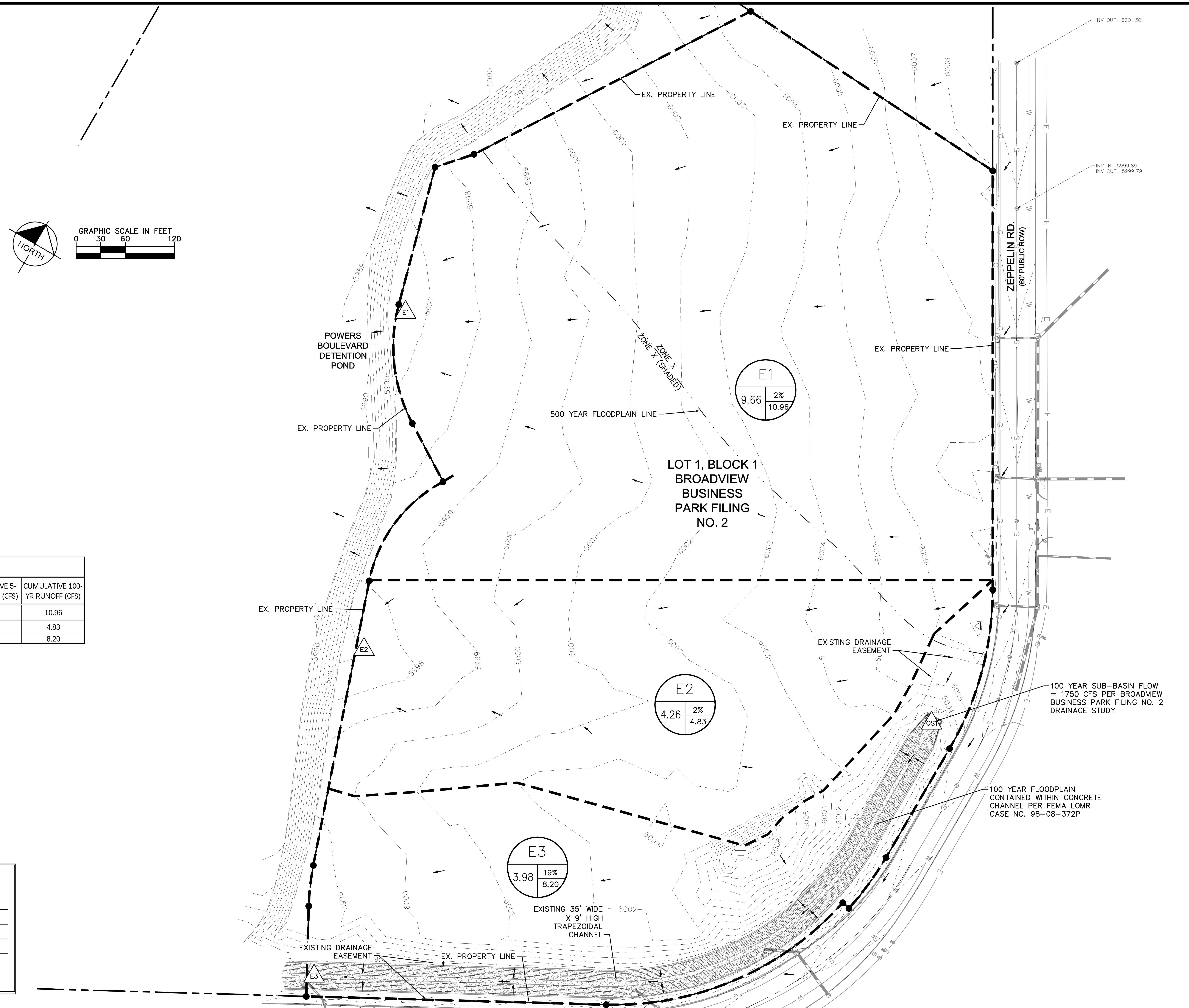
FLOW DIRECTION

DRAINAGE BASIN BOUNDARY

EMERGENCY OVERFLOW PATH

XXXX EXISTING MAJOR CONTOUR

XXXX EXISTING MINOR CONTOUR



COLORADO SPRINGS GENERAL NOTE

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SUMMARY - EXISTING RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)	CUMULATIVE 5-YR RUNOFF (CFS)	CUMULATIVE 100-YR RUNOFF (CFS)
E1	E1	9.66	0.71	10.96	0.71	10.96
E2	E2	4.26	0.31	4.83	0.31	4.83
E3	E3	3.98	2.82	8.20	2.82	8.20

COLORADO SPRINGS REVIEW

STREET DESIGN FOR CITY ENGINEERING:

UTILITY GRADE REVIEW: ____________ DATE: ______

CURB & GUTTER REVIEW: ____________ DATE: ______

FINAL REVIEW: ____________ DATE: ______

DRAINAGE DESIGN: ____________ DATE: ______

THIS IS FILED IN ACCORDANCE WITH SECTION 7.7.906 (DRAINAGE ORDINANCE) OF THE CODE OF THE CITY OF COLORADO SPRINGS, 2001 AS AMENDED.

NO.	REVISION	BY	DATE	APPR

Kimley»Horn
2017 KIMLEY-HORN AND ASSOCIATES, INC.
2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0182

DESIGNED BY: MRT
DRAWN BY: MRT
CHECKED BY: EJG
DATE: 05/10/17

2570 ZEPPELIN ROAD
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS
EXISTING DRAINAGE PLAN

COLORADO REGISTERED PROFESSIONAL ENGINEER 48487 6/27/17

PROJECT NO.
096441002

SHEET
C1.9

LEGEND

A = BASIN DESIGNATION
B = AREA (ACRES)
C = BASIN IMPERVIOUSNESS
D = 100YR DESIGN STORM RUNOFF (CFS)

= DESIGN POINT

FLOW DIRECTION

DRAINAGE BASIN BOUNDARY

EMERGENCY OVERFLOW PATH

XXXX PROPOSED MAJOR CONTOUR

XXXX PROPOSED MINOR CONTOUR

XXXX EXISTING MAJOR CONTOUR

XXXX EXISTING MINOR CONTOUR

COLORADO SPRINGS GENERAL NOTE

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SUMMARY - PROPOSED RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)	CUMULATIVE 5-YR RUNOFF (CFS)	CUMULATIVE 100-YR RUNOFF (CFS)
A1	A1	1.70	0.13	1.93	0.13	1.93
A2	A2	0.49	1.28	2.48	4.56	8.32
A3	A3	0.64	2.67	4.75	10.43	18.74
A4	A4	0.51	1.38	2.62	16.74	30.83
A5	A5	0.20	0.02	0.26	0.02	0.26
A6	A6	0.75	2.39	4.46	15.17	27.87
A7	A7	0.58	1.85	3.47	9.53	17.63
A8	A8	1.08	4.35	7.78	7.68	14.16
A9	A9	0.25	0.09	0.41	0.12	0.66
A10	A10	0.15	0.01	0.20	0.01	0.20
R1	R1	0.75	3.20	5.68	3.20	5.68
R2	R2	0.75	3.20	5.68	3.20	5.68
R3	R3	0.75	3.20	5.68	3.20	5.68
R4	R4	0.75	3.20	5.68	3.20	5.68
OS1	OS1	0.25	0.02	0.35	0.02	0.35
OS2	OS2	0.05	0.26	0.45	0.26	0.45
OS3	OS3	4.26	11.72	21.80	11.72	21.80
OS4	OS4	3.98	10.94	20.36	10.94	20.36

COLORADO SPRINGS REVIEW

STREET DESIGN FOR CITY ENGINEERING:
UTILITY GRADE REVIEW: ____________ DATE: ______
CURB & GUTTER REVIEW: ____________ DATE: ______
FINAL REVIEW: ____________ DATE: ______
DRAINAGE DESIGN: ____________ DATE: ______

THIS IS FILED IN ACCORDANCE WITH SECTION 7.7.906 (DRAINAGE ORDINANCE) OF THE CODE OF THE CITY OF COLORADO SPRINGS, 2001 AS AMENDED.

GRAPHIC SCALE IN FEET
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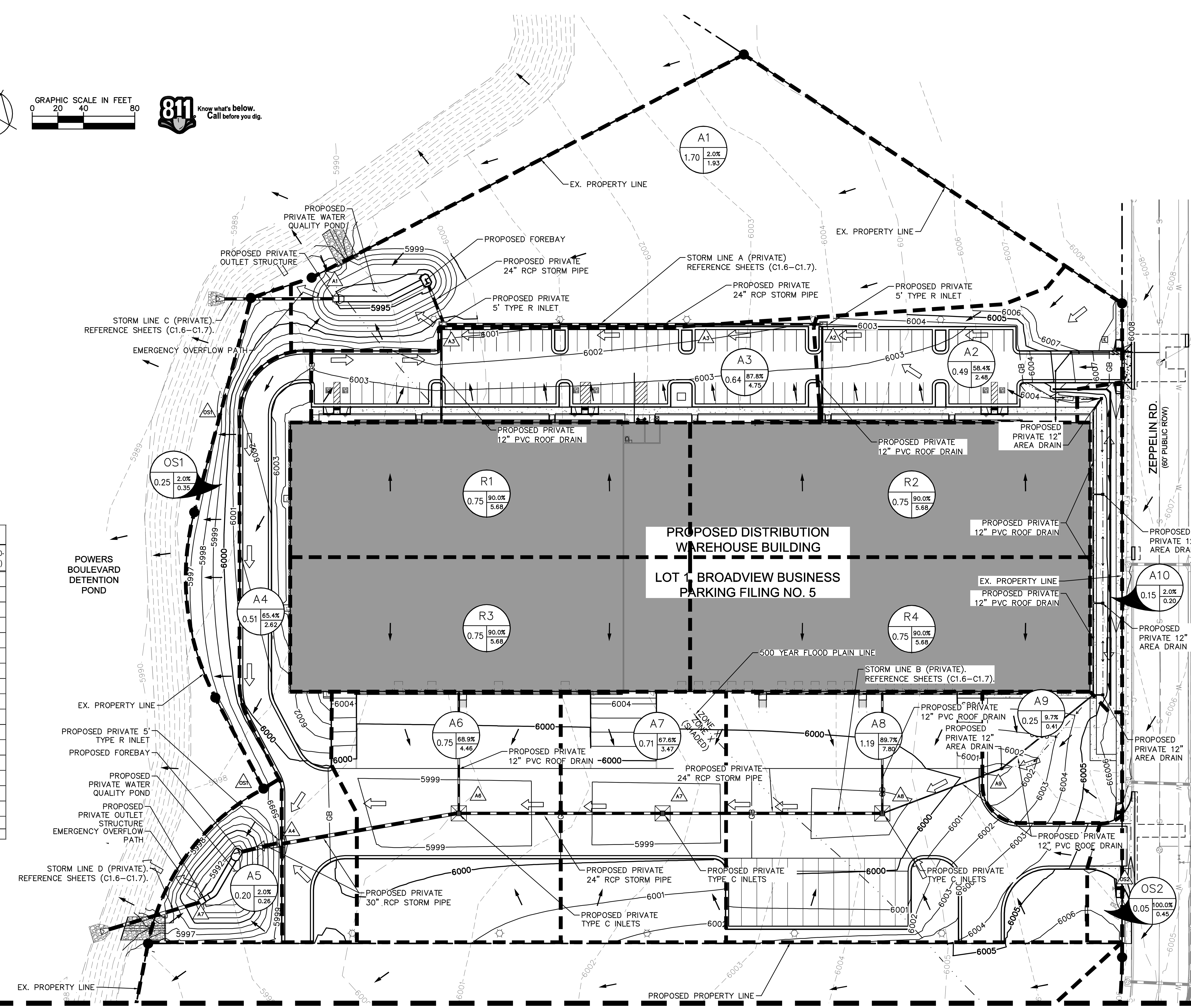
811 Know what's below. Call before you dig.



Kimley»Horn
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2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0182

NO.	REVISION	BY	DATE	APPR.

DESIGNED BY: MRT
DRAWN BY: MRT
CHECKED BY: EJG
DATE: 05/10/17

2570 ZEPPELIN ROAD
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS
PROPOSED DRAINAGE PLAN NORTH

COLORADO REGISTERED PROFESSIONAL ENGINEER 49487 6/27/17

PROJECT NO. 096441002

SHEET C1.10

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LEGEND

A = BASIN DESIGNATION
B = AREA (ACRES)
C = BASIN IMPERVIOUSNESS
D = 100YR DESIGN STORM RUNOFF (CFS)

= DESIGN POINT

FLOW DIRECTION

DRAINAGE BASIN BOUNDARY

EMERGENCY OVERFLOW PATH

XXXX PROPOSED MAJOR CONTOUR

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XXXX EXISTING MAJOR CONTOUR

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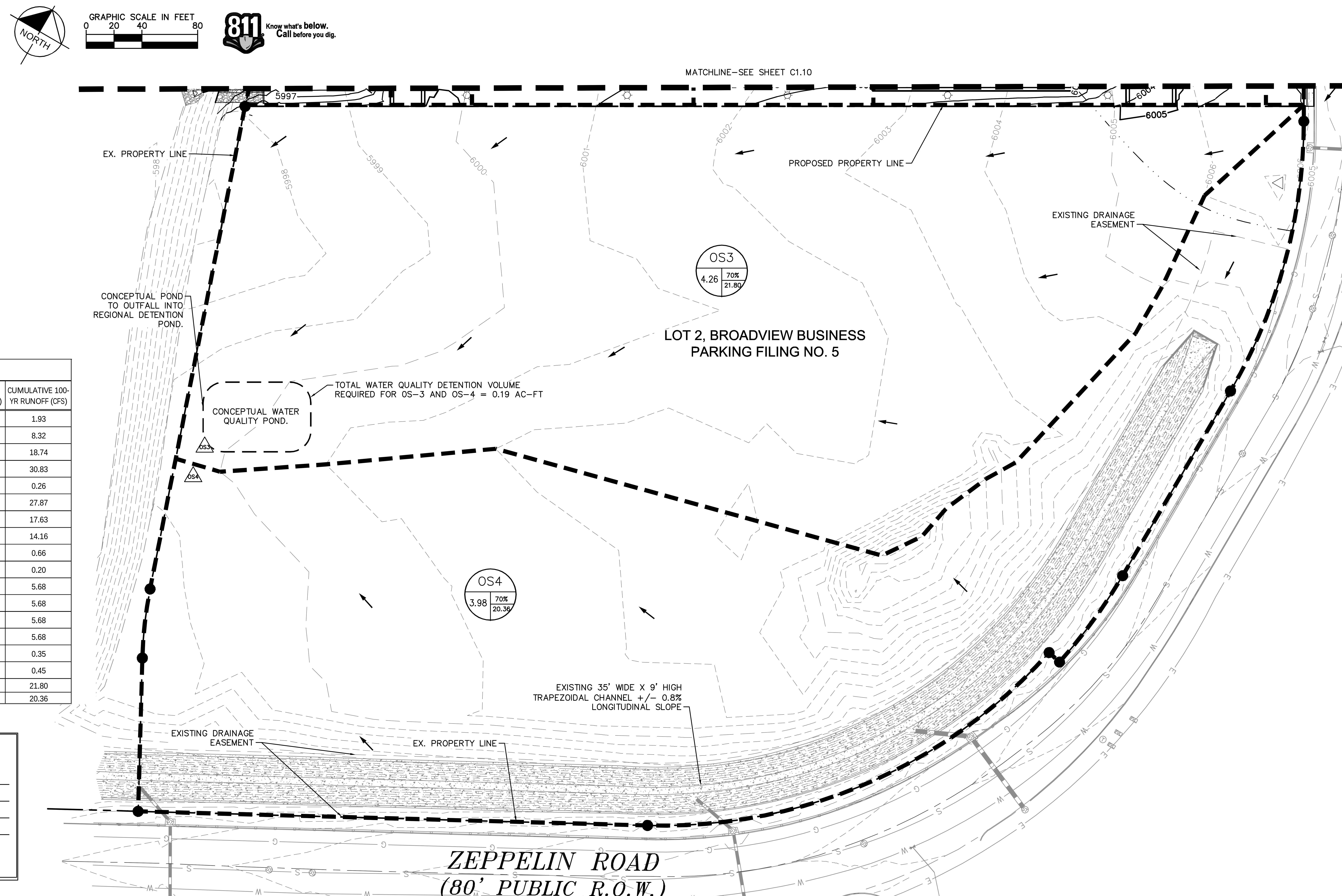

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COLORADO SPRINGS REVIEW

STREET DESIGN FOR CITY ENGINEERING:

UTILITY GRADE REVIEW: ______________________ DATE: ________

CURB & GUTTER REVIEW: ______________________ DATE: ________

FINAL REVIEW: ______________________ DATE: ________

DRAINAGE DESIGN: ______________________ DATE: ________

THIS IS FILED IN ACCORDANCE WITH SECTION 7.7.906 (DRAINAGE ORDINANCE) OF THE CODE OF THE CITY OF COLORADO SPRINGS, 2001 AS AMENDED.

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NO.	REVISION	BY	DATE	APPR.

Kimley»Horn

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2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0182

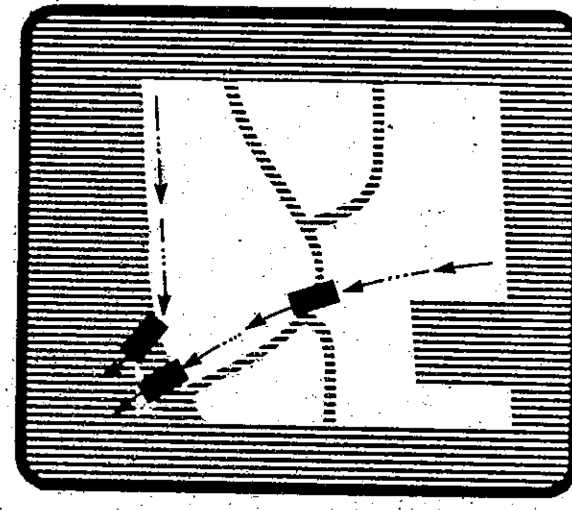
DESIGNED BY: MRT
DRAWN BY: MRT
CHECKED BY: EJG
DATE: 05/10/17

2570 ZEPPELIN ROAD
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS
PROPOSED DRAINAGE PLAN SOUTH

COLORADO REGISTERED PROFESSIONAL ENGINEER JON GUNDERSON 49487 6/27/17

PROJECT NO. 096441002

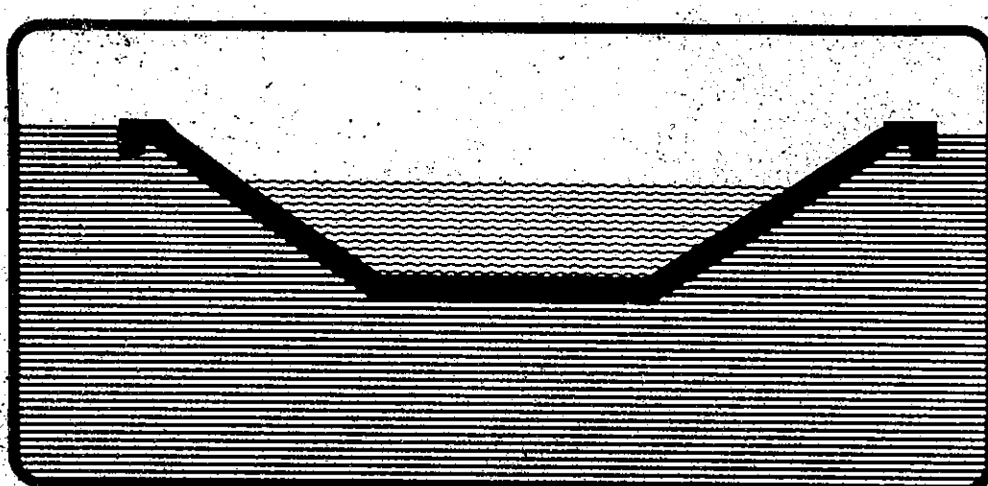
SHEET C1.11

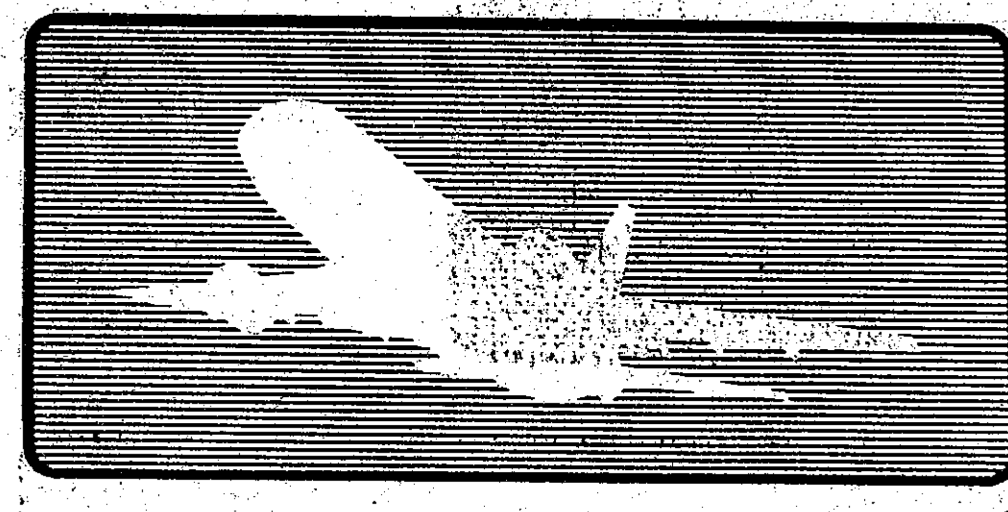


Peterson Field Drainage Basin Master Plan Update

City of Colorado Springs, Co.

August 1984





URS

Approved by City Council
December 11, 1984

PETERSON FIELD DRAINAGE MASTER PLAN
COLORADO SPRINGS, COLORADO
SEPTEMBER 28, 1984

PREPARED BY:

URS/NES 911 South 8th Street
Colorado Springs, Colorado 80906
(303) 471-0073

C E R T I F I C A T I O N

I, Stephen C. Behrens, a Registered Engineer in the State of Colorado, hereby certify that the attached Drainage Study for the Peterson Field Drainage Basin was prepared under my direction and supervision and is correct to the best of my knowledge and belief. I further certify that said Drainage Study is in accordance with all City of Colorado Springs Ordinances, Specifications, and Criteria.

STEPHEN CHARLES BEHRENS
REGISTERED
11447
PROFESSIONAL ENGINEER
STATE OF COLORADO

Stephen C. Behrens

Stephen C. Behrens, P.E.

A P P R O V A L

The City of Colorado Springs City Council and Department of Public Works do hereby approve the contents of the attached Peterson Field Drainage Study. The Study shall be used as a guide for development of all drainage facilities within the study area.

____
Department of Public Works
(SEE ALSO ATTACHED MINUTES OF THE CITY OF COLORADO SPRINGS DRAINAGE BOARD)

(SEE ATTACHED RESOLUTION)
City Council

Haynes
Raider
Havck

CITY OF COLORADO SPRINGS

December 13, 1984

TO:
Bob Gordon
DeWitt Miller
Jim Phillips
Jim Ringe
Larry Schenk
Chief Smith
Chief Stratton
Jim Wilson
Jim Colvin
Bob Parker
Johnnie Rogers
Larry Allison
Sterling Campbell
Ann Altier
Pauline Knopp
Bud Owsley
Dick Zickefoose
Bob Wilder
Jim Alice Scott
Rolf Philipsen
Dave Nickerson

FROM: City Manager

SUBJECT: Council Actions of December 11, 1984

At its regular meeting of December 11, 1984, City Council took the following actions with regard to contracts, agreements, ordinances and other fiscal matters.

PARK AND RECREATION

1) Approved a resolution accepting gifts to the Park and Recreation Department and expressing gratitude to the donors for their generous gifts.

2) Approved 1985 Budgeted and approved annual Contracts for the Park and Recreation Department sundry services.

RECEIVED
PUBLIC WORKS
COLORADO SPRINGS, COLO

DEC 17 1984
AM PM
7 8 9 10 11 12 1 2 3 4 5 6

UTILITIES (Cont'd.)

10) Tabled until the first meeting in January a request for water and wastewater service to Lots 1 - 6, Block 2 and Lot 23, Park Vista Addition by John R. Manus on behalf of Jon R. Staples.

PUBLIC WORKS

1) Tabled approval of Dry Creek Drainage Basin Master Study and establishment of a new drainage fee for the Dry Creek Drainage Basin equal to $6,364.00 per acre.

2) Approved Peterson Field Drainage Basin Master Plan Update and establishment of a new drainage fee in the amount of $3,612.00 per acre for a new bridge fee in the amount of $209.00 per acre.

3) See Park and Recreation No. 4.

4) Approved award of contract in the amount of $2,353,974.00 to Schmidt-Tiago Construction Company for 1985 asphaltic materials, with permission to extend the contract amount to the budgeted amount of $2,505,000.00.

5) See Utilities No. 10.

6) Authorized the proper City officials to enter into contracts with MRC and the Health Association of the Pikes Peak Region for transportation of the handicapped for 1985.

7) See Attorney No. 1 and 2.

8) Approved expenditure of $90,000.00 from Projects to be Determined Fund for engineering services for Centennial Boulevard - Fillmore to Fontanero.

POLICE

1) Approved Ordinance No. 84-310 on second reading amending the Code of the City of Colorado Springs 1980, as amended, relating to contributions to the Police and Fire Pension Funds.

2) Approved request by Silver Key Senior Services of donating the van frequently used by Silver Key as an extension of its contract for services.

CITY OF COLORADO SPRINGS
The "America the Beautiful" City
DEPARTMENT OF PUBLIC WORKS CITY ENGINEERING DIVISION (303) 578-6606
30 S. NEVADA SUITE 403 P.O. BOX 1575
COLORADO SPRINGS, COLORADO 80901

MINUTES

COLORADO SPRINGS/EL PASO COUNTY DRAINAGE BOARD

of November 15, 1984

The Colorado Springs/El Paso County Drainage Board met at 2:00 P.M. on Thursday, November 15, 1984 in the City Council Chambers, City Administration Building, 30 S. Nevada Avenue.

Members Present	Members Absent	Others Present
William Weber, Chairman	Rick Brown	DeWitt Miller, Dir Public Works
Leigh Whitehead	Fred Gibson	Gary Haynes, City Engineer
Richard Dailey		Jack Smith, Asst City Attorney
George Jury		Chris Smith, Subdivision Admin
Mike Mallon		Ken Jorgensen
		Roger Sams
		Laurence Schenk
		Others

The meeting was called to order at 2:00 P.M.

Item 1

Approval of the minutes of the October 18, 1984 Board Meeting. (The minutes were previously mailed.) The motion to accept the minutes was made by Mr. Jury. Mr. Whitehead seconded the motion and the motion was passed with a unanimous vote.

Items 2, 3 and 4

Items 2, 3 and 4 were acted upon by the Board with one motion. The items were treated as Consent Items.

A motion was made by Mr. Jury to accept the City Engineer's recommendations on Items 2, 3 and 4 (see Drainage Board Agenda, November 15th). The motion was seconded by Mr. Dailey. The motion passed with a unanimous vote.

Item 5

Request for credits for construction of drainage facilities within the Spring Creek Drainage Basin, Greystone Subdivision, Fountain and Academy Associates, Developer.

After review of the item by the City Engineer, the Board heard a motion by Mr. Whitehead to approve the staff's recommendation (see Drainage Board Agenda, November 15th). Mr. Mallon seconded the motion. The vote was unanimous in favor of the motion.

Item 6

Request for cash reimbursement for construction of drainage facilities within the Cottonwood Creek Drainage Basin, Dublin Business Park Subdivision Filing No. 1, Gibralter Development Corporation, Developer.

The item was reviewed by the City Engineer. The Board heard a motion by Mr. Dailey to accept the staff's recommendation (see Drainage Board Agenda, November 15th). The motion received a second by Mr. Whitehead. The motion passed with a unanimous vote.

Item 7

Establishment of drainage and bridge fees for the Peterson Field Drainage Basin.

The City Engineer presented the Board with the revised proposed basin fees. The proposed fee included the Basin Fund Balance as of September 1984, as well as the basin deficit per the Board's motion of October 18, 1984 (see Drainage Board Agenda, November 15th).

Mr. Miller stated that it was his opinion that the Board should rescind their previous action of the October 18, 1984 meeting. The Board was in agreement and heard a motion by Mr. Whitehead to rescind the Board action of October 18, 1984. The motion was seconded by Mr. Dailey. The vote was unanimous in favor of the motion.

During discussion of this item, Mr. Jury stated that he was in opposition to the new fee. Mr. Jury expressed concern that the new fee would have a negative impact on the potential for development of the unplatted acreage in the basin.

Mr. Whitehead also expressed Mr. Jury's concern but felt that the new fees established in conjunction with a basin restudy must address fund deficits to make the basin fund balance out at build out.

The Board heard a motion by Mr. Whitehead to approve the staff's recommendation that a drainage fee of $3,612.00 per acre and a bridge fee of $209.00 per acre be established for the Peterson Field Basin. The motion was seconded by Mr. Dailey. The vote was 4 - 1 in favor of the motion with Mr. Jury voting in opposition to the motion.

Item 8

Request by City Engineer to revise the cash reimbursement for construction of drainage facilities for Columbine Indust-Rail Center, Miscellaneous Drainage Basin, Columbine Industrail Development, Mr. Kenneth B. Jorgensen, Developer.

Mr. Whitehead excused himself for this item.

URS
AN INTERNATIONAL PROFESSIONAL SERVICES ORGANIZATIC

URS COMPANY
3955 EAST EXPOSITION AVENUE
DENVER, COLORADO 80209
TEL: (303) 744-1861

ANCHORAGE
ARLINGTON
ATLANTA
BUFFALO
CLEVELAND
COLOMBIA
COLORADO SPRINGS
DALLAS
DENVER
JEDDAH
KANSAS CITY
LAS VEGAS
MONTVALE
NEW ORLEANS
NEW YORK
PARIS
SALT LAKE CITY
SAN BERNARDINO
SAN FRANCISCO
SAN MATEO
SANTA BARBARA
SANTA FE
SEATTLE
TAMPA
WASHINGTON, D.C.

October 10, 1984

Mr. Gary Haynes, City Engineer
City of Colorado Springs, Colorado
30 South Nevada, Suite 402
P.O. Box 1575
Colorado Springs, Colorado 80901

Re: Peterson Field Drainage Basin
Master Plan Update

Dear Mr. Haynes:

As you are aware, URS has been retained by the Crestone Development Corporation of Colorado Springs to prepare update recommendations to the 1976 Peterson Field Drainage Masterplan to reflect existing and planned changes which have developed over the last several years.

On August 23, 1984 URS met with the Airport Advisory Commission and received the Commission's approval to abandon the 1976 masterplanned storm water detention area proposed immediately east of planned Powers Boulevard. The Commission's approval was granted based on the following information:

a) The existing two large storm water detention ponds within Peterson Field reduce the future fully developed peak 100-year storm runoff west of Powers Boulevard to a level below that proposed in the 1976 Masterplan.

b) The masterplanned storm drainage facilities identified in the 1984 update are adequate to convey future fully developed 100-year peak flood flows without having to provide additional storm water detention within Peterson Field proper.

c) Airport operators are solely responsible for the construction of any and all drainage storm drainage improvements required within Peterson Field proper.

The report includes a basin description, hydrology, hydraulics, design criteria, and a cost estimate for the remaining improvements for the basin. The report utilizes information obtained from previous studies for the Peterson Field drainage basin. A map has been prepared as a Master Drainage Plan showing existing and proposed improvements for the basin.

AN INTERNATIONAL PROFESSIONAL SERVICES ORGANIZATION

The study has been prepared as a Master Plan guide for coordinated drainage facility construction as development occurs in the study area. The recommended improvements are often general in nature as to size and location. The intent of the preliminary facility design has been to include enough construction costs in the basin fee to insure a fund for reimbursement that will theoretically "zero out" after all facilities are in place. The recommendations included herein should therefore be used as a guide in planning future development in Peterson Field Drainage Basin.

Very truly yours,

URS COMPANY

Stephen C. Behrens

Stephen C. Behrens, P.E.
Vice President

SCB/pk

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

PETERSON FIELD DRAINAGE MASTER PLAN
SEPTEMBER 28, 1984

1. PURPOSE AND SCOPE

URS was retained by the Crestone Development Corporation of Colorado Springs, Colorado to update recommendations to the 1976 Peterson Field Drainage Master Plan to reflect existing and planned changes which have occurred over the last several years.

These existing and planned changes include the following:

- Relocated Fountain Boulevard
- Planned Powers Boulevard
- Existing Peterson Field storm water detention ponds #1 and #2
- Local storm drainage improvements within Peterson Field
- Projected land use changes.

The purpose of this study is to define the general nature and location of improvements required to meet present (1984) City drainage design criteria. The scope of this study excludes establishing the exact design of required drainage improvements.

This study specifically examines the following two drainage concerns within the Peterson Field Basin:

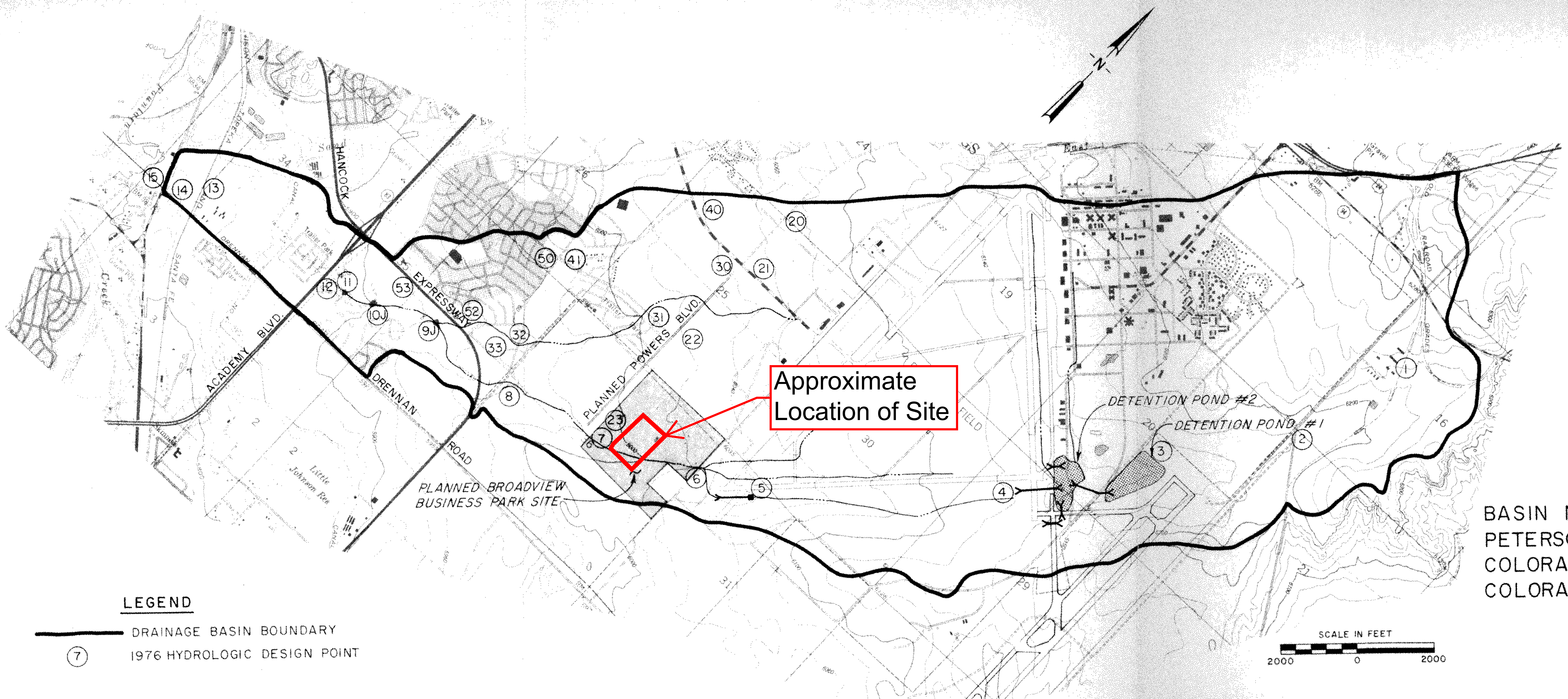
(1) the hydrologic impact of existing Peterson Field storm water detention ponds #1 and #2 on future fully developed 100-year flood flows and;

(2) the potential benefits and drawbacks associated with locating additional storm water detention facilities within Peterson Field proper.

2. BASIN DESCRIPTION

The Project Study Area encompasses that portion of Peterson Field Drainage Basin located east of planned Powers Boulevard as shown on Figure 1. Features of interest within the Study Area include planned Powers Boulevard, planned Hancock Expressway, Fountain Boulevard, Peterson Field, Colorado Highway 94, and U.S. Highway 24. The central portion of the Study Area is within the City of Colorado Springs, Colorado. The eastern and western portions of the Study Area are within unincorporated El Paso County.

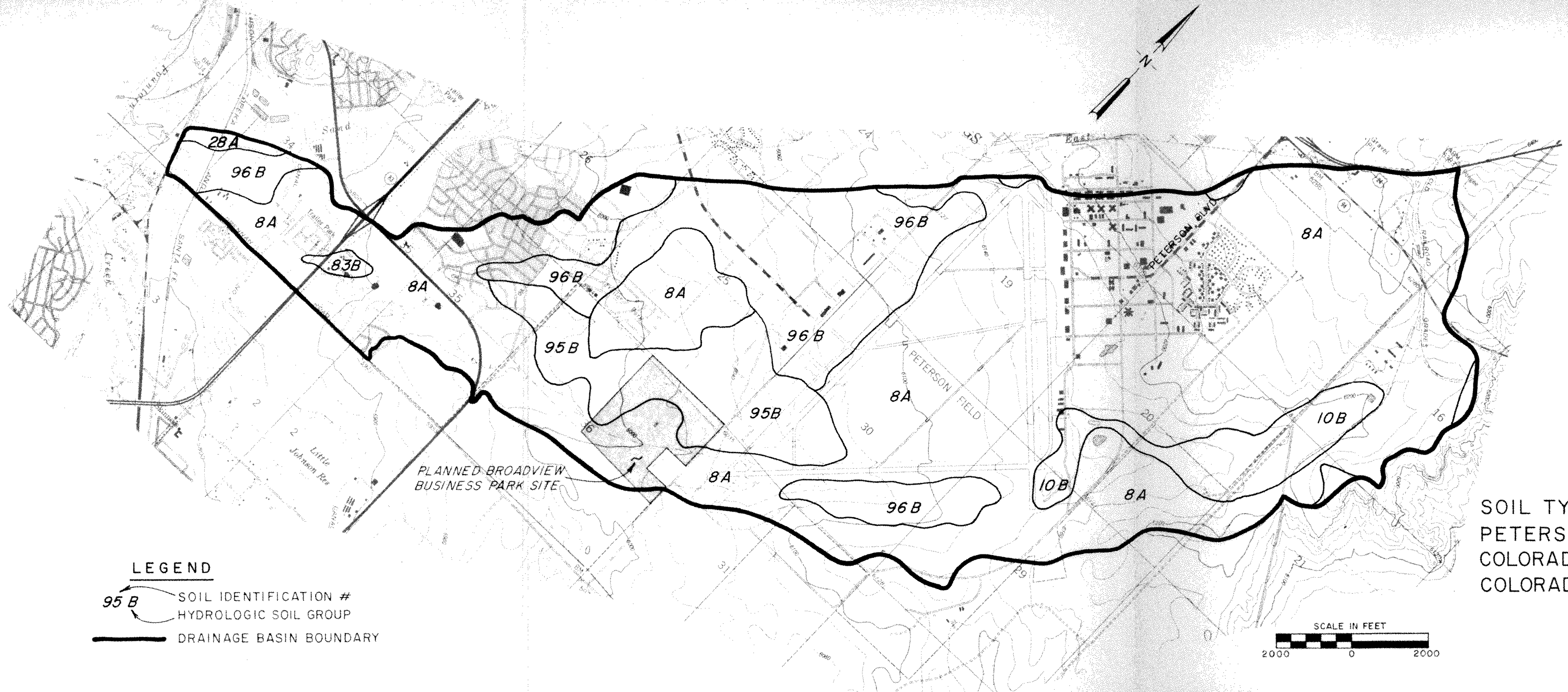
Peterson Field Basin outfalls to Sand Creek which in turn outfalls to Fountain Creek. Sand Creek Basin is a major drainage planning basin located north of the Peterson Field Basin. Chandelle and Windmill Gulch basins are major drainageway planning basins located south of the Peterson Field Basin. Peterson Field Basin encompasses a total of approximately 8.6 square miles above Fountain Creek of which the Project Study Area encompasses a total of approximately 7.2 square miles. Peterson Field proper occupies approximately 3.9 square miles of the Project Study Area. Peterson Field Basin has a total length of approximately nine miles of which approximately six miles are within the Project Study Area. Elevations within



BASIN MAP
PETERSON FIELD BASIN
COLORADO SPRINGS,
COLORADO

BROADVIEW BUSINESS PARK FILING NO. 1

FIGURE 1



SOIL TYPES
PETERSON FIELD BASIN
COLORADO SPRINGS,
COLORADO

BROADVIEW BUSINESS PARK FILING NO. 1

SOURCE: SOIL SURVEY OF EL PASO COUNTY AREA, COLORADO
SOIL CONSERVATION SERVICE JUNE, 1981

FIGURE 2

Peterson Field Basin are approximately 5750 at Fountain Creek, 5990 at planned Powers Boulevard, and 6440 at the upper end of the Basin.

Basin soil and land use characteristics directly affect the relationship between rainfall and runoff within a basin. The U.S. Soil Conservation Service classifies soils into four hydrologic groups (A, B, C and D) according to a soil's runoff potential. Group A soils exhibit high infiltration rates when thoroughly wetted and are considered to have low runoff potential. Group B soils exhibit moderate infiltration rates when thoroughly wetted. Group C soils exhibit slow infiltration rates when throughly wetted. Group D soils exhibit very slow infiltration rates when throughly wetted and are considered to have high runoff potential.

Soil types within the Peterson Field Basin are listed in Table 1 and delineated in Figure 2. The Peterson Field Basin encompasses approximately 2.5 square miles of group 'B' hydrologic soils and the remainder are group 'A' soils. Most of the soils in the Peterson Field Basin have a high infiltration rate, are excessively drained, and are easily erodible. Reservoir embankments, dikes and levees constructed of Peterson Field Basin soils may be subject to piping and seepage. Water storage reservoirs constructed in Peterson Field Basin soils may experience excessive seepage. Group 'A' hydrologic soils in the Peterson Field Basin are expected to have relatively low potential for frost action. Group 'B' hydrologic soils in the Peterson Field Basin are expected to have moderate potential for frost action.

RETURN WITHIN 2 WEEKS TO:
CITY OF COLORADO SPRINGS
STORM WATER & SUBDIVISION
W. COSTILLA, SUITE 113
COLORADO SPRINGS, CO 80903
9) 385-5979

RECEIVED
PUBLIC WORKS/ENGINEERING
COLORADO SPRINGS, COLO.
JUN 1 0 1986
AM 7 8 9 10 11 12 1 2 3 4 5 6 PM

FINAL DRAINAGE REPORT

FOR

BROADVIEW BUSINESS PARK
FILING NO. 2
AND
FILING NO. 3

RECEIVED
PUBLIC WORKS/ENGINEERING
COLORADO SPRINGS, COLO.
MAY 2 8 1986
AM 7 8 9 10 11 12 1 2 3 4 5 6 PM

PREPARED FOR:

BROADVIEW PARTNERS
25 N. Cascade
Colorado Springs, CO 80903

PREPARED BY:

K L H ENGINEERING CONSULTANTS, INC.
206-208 Sutton Lane
Colorado Springs, CO 80907
J.N.: KLH 85 629 01
85 629 02

Conditions of Approval: *

No Building Permit will be issued for any portion of the platted area until:

1. A suitable public outfall system is constructed at Powers Blvd. and Hancock Ave. per the basin Master Study January, 1986 or,
2. Detailed plans and cost estimates of the required temporary (non-reimbursible) detention facilities are reviewed and on file with the City Engineering Division. A letter of credit will be posted with the City for the detention facilities. These facilities must be constructed at the time the parcel is developed if the outfall system is not in place.

PURPOSE AND SCOPE

The purpose of this report is to examine the existing and proposed drainage conditions for Filing No. 2 and Filing No. 3 of Broadview Business Park, in the city of Colorado Springs. Since these subdivisions lie adjacent to each other, both sites are discussed in this report so that the reader will have a better understanding of the overall drainage for this portion of Broadview Business Park.

GENERAL

Broadview Business Park Filing No. 2 is a 17.90 acre subdivision containing 1 lot zoned PIP-2. This subdivision is bounded by Zeppelin Road on the South and East, by unplatted land on the North, and by the proposed Broadview Business Park Filing No. 3 on the West.

Broadview Business Park Filing No. 3 is a 22.06 acre subdivision containing one 18.1 acre lot zoned OC, with the remaining land in the Westerly portion of the subdivision being dedicated as Powers Boulevard Right-of-Way. Filing No. 3 is bounded on the South by Zeppelin Road and on the East by the proposed Filing No. 2.

Broadview Business Park Filing No. 2 and Filing No. 3 are located within the Peterson Field Drainage Basin, and are a part of the subject area of the "Peterson Field Drainage Basin Master Plan Update" prepared by URS Company in August, 1984. This drainage report for Filing No. 2 and Filing No. 3 is in compliance with the Master Drainage Plan Update. The reader is also referred to the "Broadview Business Park Filing No. 1 Subdivision Drainage Report", prepared by URS/NES, and filed by the City of Colorado Springs in December, 1984. This Filing No. 1 drainage report discusses the major drainage improvements required in Broadview Business Park.

Portions of both Broadview Business Park Filing No. 2 and Filing No. 3 lie within the 100-year floodplain for the Peterson Field Drainage Basin, according to the F.E.M.A. maps. The extent of the area within the floodplain varies, depending on whether the City F.E.M.A. maps or the County F.E.M.A. maps are used to determine the floodplain limits. However, in the time since these maps were prepared, a concrete channel has been constructed through Broadview Business Park. This channel was designed to carry the 100-year flows, according to the "Peterson Field Drainage Basin Master Plan Update" and the "Broadview Business Park Filing No. 1 Subdivision Drainage Report", therefore, the floodplain as shown on the F.E.M.A. maps is no longer a true representation of the actual conditions which exist at the site.

The soils within these subdivisions consist of the following:

1. Blakeland loamy sand, S.C.S. Soils Number 8 and Hydrologic Group A;
2. Truckton loamy sand, S.C.S. Soils Number 95 and Hydrologic Group B.

The S.C.S. Soils Map Numbers are shown on the accompanying drainage plans. This site slopes gently from East to West, as shown on the drainage plans.

METHOD OF COMPUTATIONS:

Runoff quantities are calculated using the Modified S.C.S. Methodology as approved by the City of Colorado Springs Engineering Division and outlined in the manual, "Determination of Storm Runoff Criteria" by the City of Colorado Springs, March 1977. A weighted curve number was calculated using the respective areas of rangeland, industrial area, office area and soils types.

Peak runoff flows were calculated for both 5-year and 100-year, 6 hour frequency storm events, and are shown on the drainage plan. Hydrologic calculations are included at the end of this report. Per City of Colorado Springs criteria, all drainage structures have been sized for the 5-year storm for peak 100-year flows less than 500 c.f.s., and for the 100-year storm for peak flows in excess of 500 c.f.s.

EXISTING DRAINAGE

At the present time, runoff enters Filing No. 2 and Filing No. 3 as overland flow from a small area to the North, and as channel flow from the main Peterson Field drainage channel. This concrete channel (which was discussed in the "Broadview Business Park Filing No. 1 Subdivision Drainage Report") is designed to carry the 100-year storm runoff for the Peterson Field Basin. This channel continues in a drainage easement along the Easterly and Southerly portions of Filing No. 2 and Filing No. 3, adjacent to Zeppelin Road. At the intersection of Zeppelin Road and Powers Boulevard, the channel transitions to a double reinforced concrete box culvert for the Powers Boulevard crossing. A second RCB crossing exists just to the North of the first crossing, to carry the 100-year flow from the future channel proposed on the East side of Powers Boulevard (see "Peterson Field Drainage Basin Master Plan Update).

Runoff presently enters Filing No. 3 as overland flow from the proposed Filing No. 2 area. Refer to the accompanying drainage plans for existing topography.

PROPOSED DRAINAGE

As previously stated, a concrete channel will be required along the East side of Powers Boulevard (within the Right-of-Way), according to the "Peterson Field Drainage Basin Master Plan Update". Inlets in Powers Boulevard will be required to prevent the street flow from exceeding the allowable street capacity. At this time, the design of Powers is uncertain. For this report, a street slope of 1% was assumed, with a low point in Powers near the Southerly line of this subdivision. The resulting facilities required are shown on the drainage plan. When the design of Powers Boulevard is finalized, the drainage will need to be re-examined to determine the adequacy of the facilities proposed in this drainage report.

A detailed site plan is not available at this time for either Filing No. 2 or Filing No. 3. When the location of the private interior streets becomes known, any necessary private drainage facilities will be determined. The flows calculated for the developed conditions are shown on the accompanying drainage plans. These flows will be routed through the sites in the private streets, and then discharged into the channel along Zeppelin Drive and the channel along Powers Boulevard. The owner of Broadview Business Park Filing No. 3 will accept the developed flows from Filing No. 2.

At the present time, the main channel and crossings end just West of the future Powers Boulevard/Hancock Road intersection. The developer of Broadview Business Park will construct the permanent public outfall from these facilities to the existing channel downstream. If development of either Filing No. 2 or Filing No. 3 occurs before this public outfall channel is in place, then private onsite detention facilities will be required. At that time, detailed plans for the detention facilities shall be submitted to the City Engineer prior to issuance of building permits.

DRAINAGE FACILITIES COST ESTIMATE:

Broadview Business Park Filing No. 2:

No public facilities are proposed for this subdivision.

Broadview Business Park Filing No. 3:

Public and Reimburseable				
1280 L.F. Conc. Channel (b=10', d=5', z=1.5)		@ $ 90.00/L.F.	=	$ 115,200.00
130 L.F. (8-8)x 6 RCB		@ $ 400.00/L.F.	=	$ 52,000.00
6'D-10R	3 Each	@ $2,000.00/Each	=	$ 6,000.00
10'D-10R	1 Each	@ $2,500.00/Each	=	$ 2,500.00
24"RCP	280 L.F.	@ $ 37.00/L.F.	=	$ 10,360.00
30"RCP	90 L.F.	@ $ 42.00/L.F.	=	$ 3,780.00
				$ 189,840.00
	15% Engineering & Contingency			28,476.00
	TOTAL			$ 218,316.00

Permanent Public Outfall Facility (estimates from Table 4, Peterson Field Drainage Basin Master Plan Update, August 1984 by URS):

Public and Reimburseable		
2700 L.F. Conc. Channel (b=14', d=8', z=1.5)		$ 355,500.00
1000 L.F. Conc. Channel (b=18', d=8', z=1.5)		142,900.00
130 L.F. (12-12)x 8 RCB		175,500.00
140 L.F. (12-12)x 8 RCB		175,500.00
		$ 849,400.00
	15% Engineering & Contingency	127,410.00
	TOTAL	$ 976,810.00

for all of Broadview filings will be constructed as each plat is processed

DRAINAGE AND BRIDGE FEES

Broadview Business Park Filing No. 2
Peterson Field Basin

1986 Drainage Fee: 17.90 Ac. @ $3793.00/Ac.	=	$67,894.70
1986 Bridge Fee : 17.90 Ac. @ $ 219.00/Ac.	=	$ 3,920.10

Broadview Business Park Filing No. 3
Peterson Field Basin

1986 Drainage Fee: 22.06 Ac. @ $3793.00/Ac.	=	$83,673.58
1986 Bridge Fee : 22.06 Ac. @ $ 219.00/Ac.	=	$ 4,831.14

Since the cost of the facilities previously built in Broadview Business Park exceeds the drainage fees for Filing Nos. 1, 2 and 3, no cash drainage fees will be required with the platting of either Broadview Business Park Filing No. 2 or Broadview Business Park Filing No. 3.

DRAINAGE REPORT STATEMENTS

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 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 the negligent acts, errors or omissions on my part in preparing this report.

Name ______________________ Seal

Developer's Statement:

The developer has read and will comply with all of the requirements specified in this drainage report.

BROADVIEW PARTNERS
Business Name
By: ______________________
Title: General Partner
Address: 25 N. Cascade
Colorado Springs, CO 80903

City of Colorado Springs:

Filed in accordance with Section 15-3-906 of the Code of the City of Colorado Springs, 1980, as amended.

______________________ City Engineer   8/8/86 Date

Conditions:

BASIN

BVIEW - 1

ACREAGE	SQ.MI.	LAND USE	SOIL	CN	%	% x CN
.5		RANGE	A	54	2.8	150.8
.4		RANGE	B	70	2.2	156.4
15.2		INDUSTRIAL/OFFICE	A	81	84.9	6878.2
1.8		INDUSTRIAL/OFFICE	B	88	10.1	884.9
17.9	.028				100.0	8070.4

WEIGHTED CN = 80.7

FLOW TYPE	L(ft)	H(ft)	Tc(hrs)	RUNOFF(in)	qp(CSM/in)	Q (cfs)
OVERLAND	400	3	.171			
STREET	1100	6	.113			
	1500	9	.284	.66	940	17.2 (5yr FLOW)
				1.69		44.4 (100yr FLOW)

BASIN

BVIEW - 2

ACREAGE	SQ.MI.	LAND USE	SOIL	CN	%	% x CN
5.2		RANGE	B	70	22.3	1562.2
5.3		INDUSTRIAL/OFFICE	A	89	22.7	2024.5
12.8		INDUSTRIAL/OFFICE	B	92	54.9	5054.1
23.3	.036				100.0	8640.8

WEIGHTED CN = 86.4

FLOW TYPE	L(ft)	H(ft)	Tc(hrs)	RUNOFF(in)	qp(CSM/in)	Q (cfs)
OVERLAND	650	8	.175			
STREET	600	8	.073			
CHANNEL	1000	5	.028			
	2250	21	.276	.95	950	32.8 (5yr FLOW)
				2.13		73.7 (100yr FLOW)

ESTIMATED STREET FLOWS IN POWERS BLVD (FUTURE)

Assumed $s = 1.0\%$

street capacity for Arterial w/ 8" vert. curb. @ 1.0% = 30 cfs

½ street capacity = 15 cfs

Assume Powers Boulevard is at capacity at Northerly boundary

Provide 6' D-10R to maximize pickup / minimize size/cost

Capacity of 6' D-10R @ 1.0% = 5.3 cfs (60%)

Calculate area of street so that ½ street capacity is reached

$A = 105' \times L$

CN = 98 ⟹ 5-yr : 1.87 in. runoff depth

Use $t_c < 0.10$ hr ⟹ 1300 csm/inch

$$q = A q_p Q$$

$$A = \frac{q}{q_p Q} = \frac{(5.3\ \cancel{cfs})\ 640\ Ac/sq.mi}{(1300\ \cancel{csm/in})(1.87\ \cancel{in})} = 1.40\ Ac$$

$$A = wL \Rightarrow L = \frac{A}{w} = \left(\frac{1.40\ Ac}{105\ ft}\right)\left(\frac{43560\ sq.ft}{Ac}\right) \Rightarrow L = 580\ ft$$

∴ Maximum spacing of 6' D-10R's = 580 ft.

Distance between Northerly boundary & assumed Powers low.pt. ≈ 1350 ft

Using 2 - 6' D-10R's, spacing = 675' ⟹ too far apart

∴ Use 6' D-10R's @ 450 ft

(ESTIMATE ONLY)

Check capacity of proposed channel along Powers Blvd.
@ assumed slope s = 1.0%
b = 10', d = 5', z = 1.5 – from Filing No. 1 Drainage Report (URS)

```
TRAPIZOIDAL CHANNEL
SOLUTION FOR NORMAL DEPTH USING
MANNINGS EQUATION
SECTION  1
------------------------------
Q=  1090.0000  cfs
S=      .0100  ft/ft
B=    10.0000  ft
Z=     1.5000
N=      .0150
V=    18.3541  fps
Tw=   21.3613  ft   Tt=   24.3613  ft
Y=     3.7871  ft   D =    4.7871  ft

FROUDE NUMBER = 1.940
MIN FREEBOARD = 1.00 ft
X-sect LENGTH =27.26 ft
```

D < 5' ∴ OK @ 1.0%

Find minimum slope for channel:

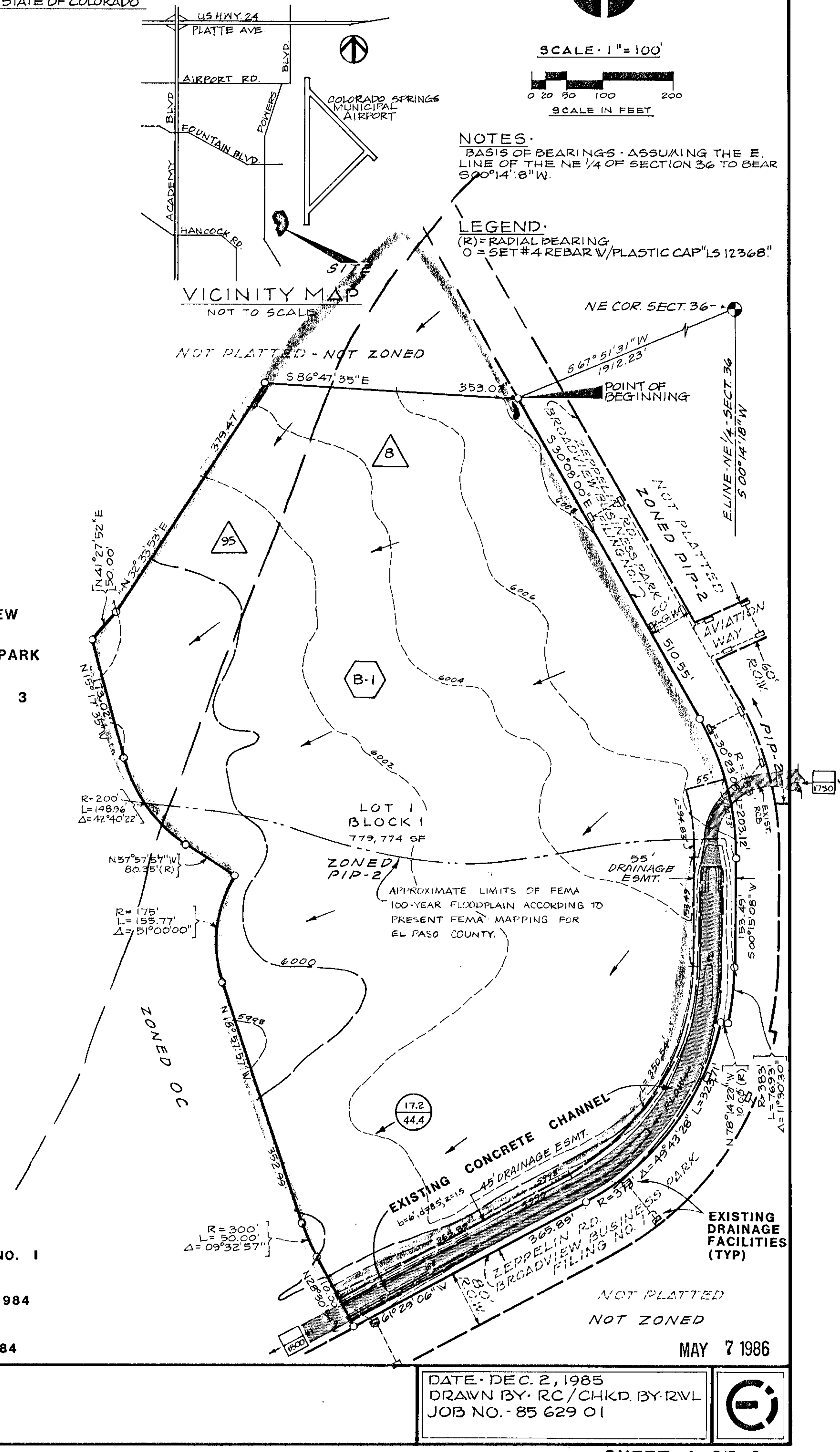
```
SECTION  3
------------------------------
Q=  1090.0000  cfs
S=      .0080  ft/ft
B=    10.0000  ft
Z=     1.5000
N=      .0150
V=    16.9308  fps
Tw=   22.0513  ft   Tt=   25.0641  ft
Y=     4.0171  ft   D =    5.0214  ft

FROUDE NUMBER = 1.746
MIN FREEBOARD = 1.00 ft
X-sect LENGTH =28.10 ft
```

Minimum slope = 0.80%

PRELIMINARY PLAT
BROADVIEW BUSINESS PARK
FILING NO. 2

A SUBDIVISION OF A PORTION OF THE NE 1/4 OF SECTION 36, T 14 S, R 66 W. OF THE 6TH P.M.,
CITY OF COLORADO SPRINGS, COUNTY OF EL PASO, STATE OF COLORADO



NOTES·
BASIS OF BEARINGS · ASSUMING THE E. LINE OF THE NE 1/4 OF SECTION 36 TO BEAR S00°14'18"W.

LEGEND·
(R) = RADIAL BEARING
O = SET #4 REBAR W/PLASTIC CAP "LS 12368".

BROADVIEW
BUSINESS PARK
FILING NO. 3

NOTE: FF ELEVATIONS TO BE ABOVE 100-YEAR FLOODPLAIN ELEVATION.

DRAINAGE PLAN

JANUARY, 1986

LEGEND:

- 17.2 / 44.4 — 5 YEAR SUB-BASIN FLOW, cfs; 100 YEAR SUB-BASIN FLOW, cfs
- 1750 — 5 YEAR ACCUMULATIVE FLOW, cfs; 100 YEAR ACCUMULATIVE FLOW, cfs
- B-1 — SUB-BASIN DESIGNATION
- 8 — S.C.S. SOILS MAP NUMBER

*1 - FROM "BROADVIEW BUSINESS PARK FILING NO. 1 SUBDIVISION DRAINAGE REPORT" prepared by: URS/NES, DECEMBER, 1984

*2 - FROM "PETERSON FIELD DRAINAGE BASIN MASTER PLAN UPDATE" AUGUST, 1984

OWNER:

Broadview Partners
James F. Morley, General Partner
25 North Cascade Avenue
Colorado Springs, Colorado

MAY 7 1986

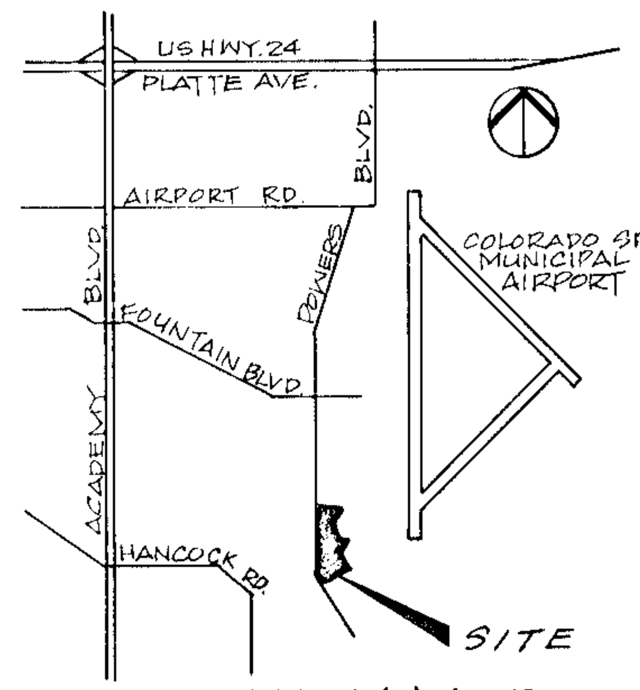
KLH ENGINEERING CONSULTANTS, INC.
ENGINEERING•SURVEYING•PLANNING•CONSTRUCTION MANAGEMENT
206-208 Sutton Lane • Colorado Springs, Colorado 80907 • (303) 594-4200

DATE · DEC. 2, 1985
DRAWN BY · RC / CHKD. BY · RWL
JOB NO. - 85 629 01

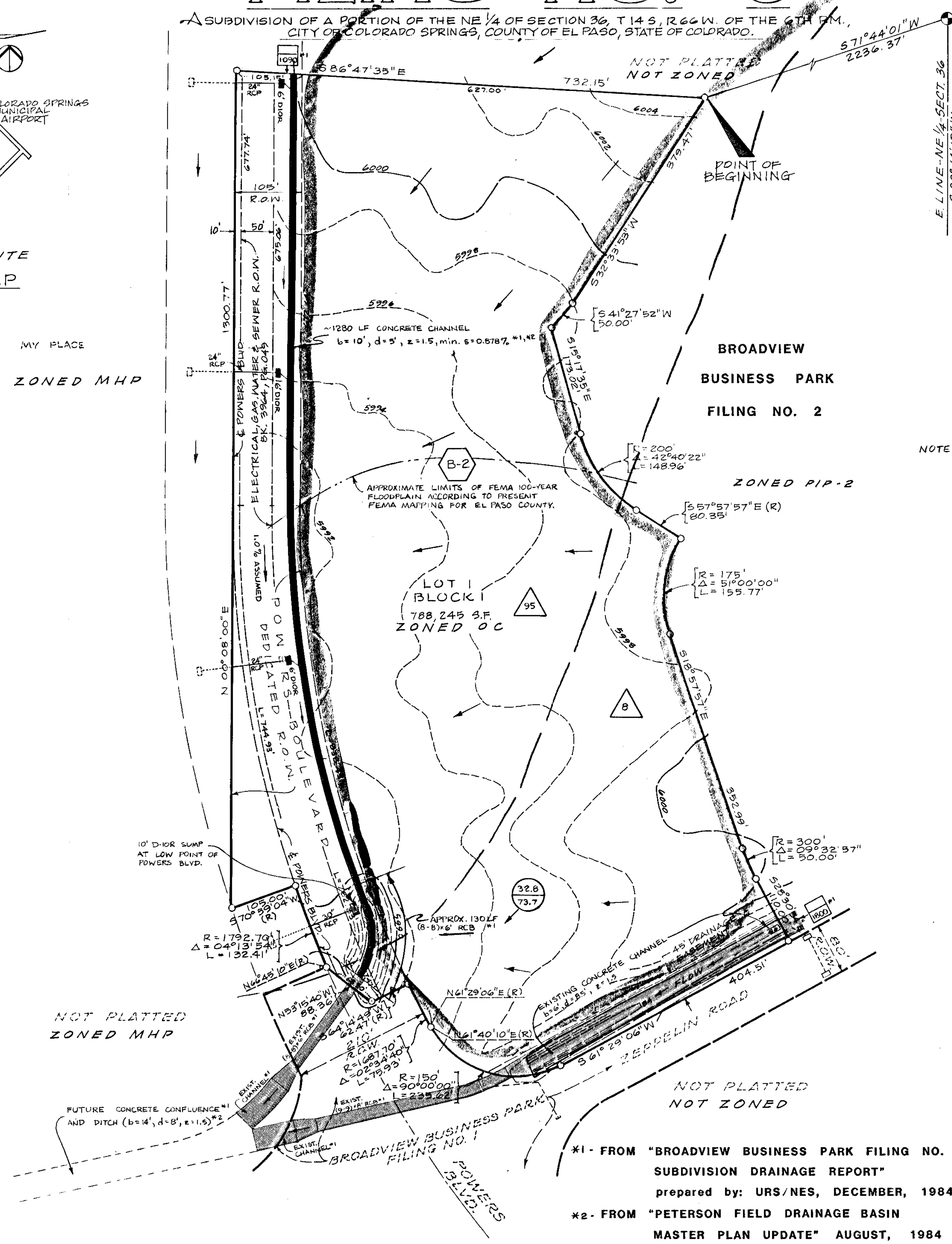
PRELIMINARY PLAT

BROADVIEW BUSINESS PARK FILING NO. 3

A SUBDIVISION OF A PORTION OF THE NE 1/4 OF SECTION 36, T 14 S, R 66 W. OF THE 6TH P.M., CITY OF COLORADO SPRINGS, COUNTY OF EL PASO, STATE OF COLORADO.



VICINITY MAP
NOT TO SCALE



E LINE-NE 1/4-SECT. 36
S 00°14'18" W

SCALE: 1" = 100'

0 20 50 100 200
SCALE IN FEET

NOTES:
BASIS OF BEARINGS: ASSUMING THE E. LINE OF THE NE 1/4 OF SECTION 36 TO BEAR S00°14'18"W.

NOTE: FF ELEVATIONS TO BE ABOVE 100-YEAR FLOODPLAIN ELEVATION.

LEGEND:
(R) = RADIAL BEARING
O = SET #4 REBAR W/PLASTIC CAP "LS 12368"

DRAINAGE PLAN

JANUARY, 1986

LEGEND:

- 5 YEAR SUB-BASIN FLOW, cfs / 100 YEAR SUB-BASIN FLOW, cfs
- 5 YEAR ACCUMULATIVE FLOW, cfs / 100 YEAR ACCUMULATIVE FLOW, cfs
- B-2 SUB-BASIN DESIGNATION
- 95 S.C.S. SOILS MAP NUMBER

OWNER:

Broadview Partners
James F. Morley, General Partner
25 North Cascade Avenue
Colorado Springs, Colorado

*1 - FROM "BROADVIEW BUSINESS PARK FILING NO. 1 SUBDIVISION DRAINAGE REPORT" prepared by: URS/NES, DECEMBER, 1984

*2 - FROM "PETERSON FIELD DRAINAGE BASIN MASTER PLAN UPDATE" AUGUST, 1984

MAY 7 1986

KLH ENGINEERING CONSULTANTS, INC.
ENGINEERING • SURVEYING • PLANNING • CONSTRUCTION MANAGEMENT
206-208 Sutton Lane • Colorado Springs, Colorado 80907 • (303) 594-4200

DATE: DEC. 2, 1985
DRAWN BY: RC / CHKD. BY: RWL
JOB NO. - 85 629 02

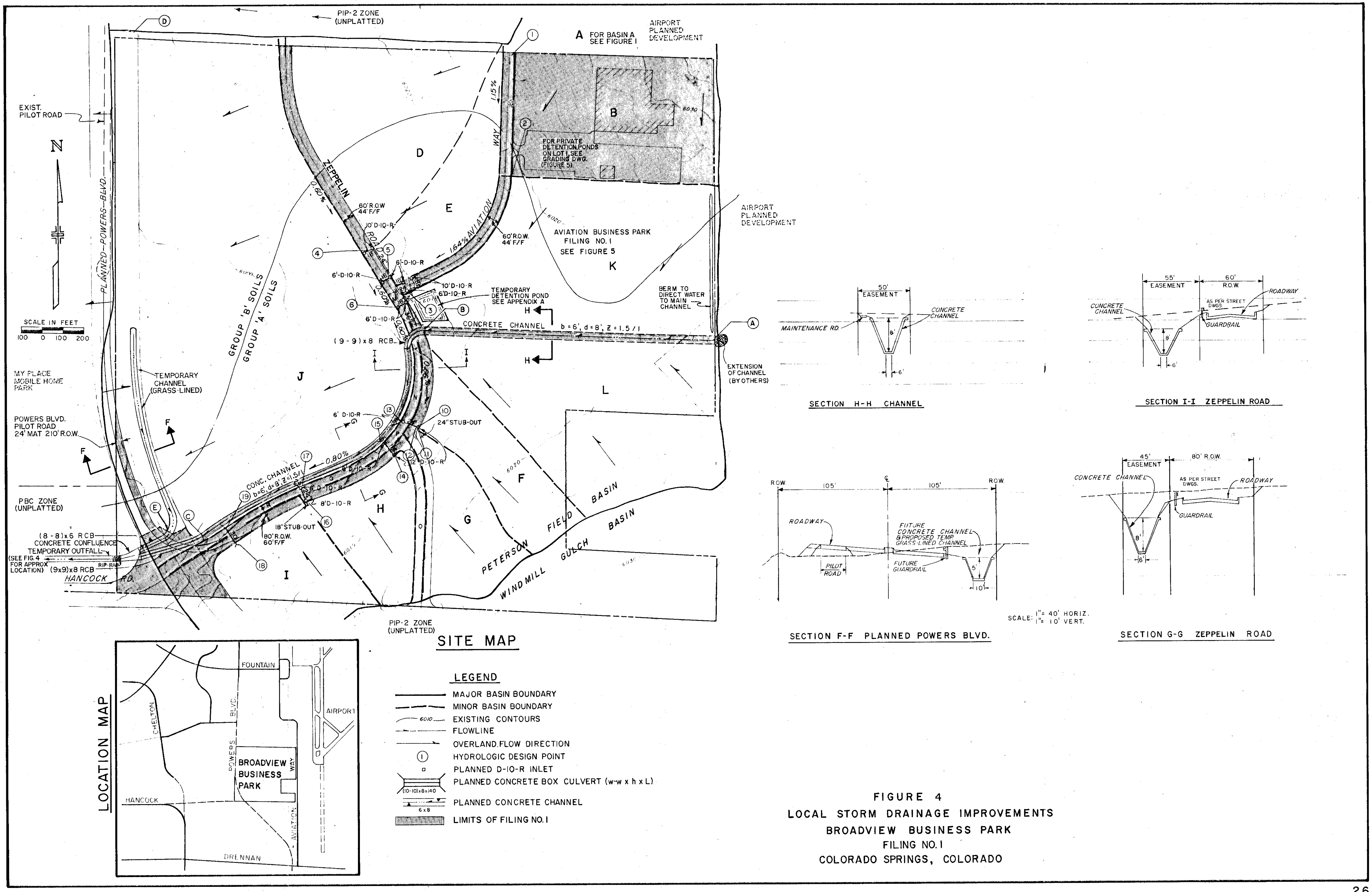


FIGURE 4
LOCAL STORM DRAINAGE IMPROVEMENTS
BROADVIEW BUSINESS PARK
FILING NO. I
COLORADO SPRINGS, COLORADO

POWERS BOULEVARD DETENTION FACILITY

FINAL DRAINAGE REPORT

Prepared for:

City of Colorado Springs
Department of Public Works
30 South Nevada
Colorado Springs, Colorado 80903

Prepared by:

Kiowa Engineering Corporation
419 West Bijou Street
Colorado Springs, Colorado 80905-1308

Kiowa Project No. 89.08.16
D12/R61

January, 1990

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.

Kiowa Engineering Corporation, 419 W. Bijou, Colorado Springs, Colorado 80905-1308

4-12-90
Date

19310

I. INTRODUCTION

Authorization

The preparation of this final drainage report was authorized under the terms of the agreement between the City of Colorado Springs and Kiowa Engineering Corporation dated August 14, 1989.

Purpose and Scope

The purpose of the final drainage report for the Powers Boulevard Detention Facility is to refine the preliminary hydrologic and hydraulic analyses summarized in the Preliminary Design Report. Specifically, the scope of this report is as follows:

1. Address review comments related to the hydrologic analysis contained within the Preliminary Design Report.
2. Refine the hydrologic model used to determine the stage, storage, and discharge relationships for the detention facility.
3. Analyze the hydrologic characteristics related to the sizing of water quality features within the detention facility, based upon climatological data for the Colorado Springs area.
4. Prepare final recommendations for the layout of the detention facility and the various appurtenant structures.

Review comments were received from City utility departments, and from CH2M-Hill, Inc., regarding the design of the detention facility. The assumptions made during the preliminary design report preparation regarding the surface area draining to the facility have been specifically readdressed (reference CH2M-Hill, Inc., letter of October 6, 1989).

II. HYDROLOGIC ANALYSIS

Shown on Figure 1 is the sub-basin map used to develop the hydrologic model for the sizing of the detention facility. The "Powers Boulevard" drainage area, shown as the shaded area on Figure 1, has been reevaluated. Field visits and further review of the Powers Boulevard Design Plans prepared by CH2M Hill, Inc., were used to confirm the areas to be directly routed to the detention facility. In the Preliminary Design Report, it was assumed that sub-basins 1 through 6 would be tributary to the detention facility (Reference Figure 8, Sub-basin delineation, Powers Boulevard Drainage Report, prepared by CH2M Hill, Inc.). It was confirmed that sub-basins 4 and 6 drain to the existing concrete swale along Zeppelin Road, and it is not practical to route these two basins through the detention facility. Summarized on Table 1 is peak flow data for the revised hydrologic analysis, which eliminated basins 4 and 6. The TR-20 computer output is contained within Appendix A. The peak flow data shown on Table 1 will be used in sizing the detention facility storage area and outlet structure(s).

Water Quality Hydrology

Contained within Appendix B is a description of the analysis which will be used to size the water quality features of the Powers Boulevard Detention Facility. The analysis is based upon climatological data for the Colorado Springs area and provides for a methodology to size water quality pond volumes of an optimum size to store and treat urban runoff.

Based upon the methodology summarized in Appendix B, it has been determined that a water quality storage volume of 32 acre feet should be provided within the Powers Boulevard Detention Facility. This is based upon the precipitation and runoff parameters for a 24-hour storm separation time, and 24-hour release time for the water quality storage area. The depth of the water quality pool will be 3.5 to 4-feet. A 24-hour release time will be used to control the retention time. The water quality pond will be drained by a culvert controlled by an orifice (or other flow control device), and will outfall to the existing box culvert under Powers Boulevard. A final TR-20 run will be compiled for the detention facility, which will account for the water quality pool volume. For the purposes of this analysis, the water quality storage area has been assumed to be empty at the time of a 100-year storm event.

III. HYDRAULICS

The control of the developed inflow to the proposed detention facility will be achieved by extending the existing twin, 6-foot by 10-foot box culvert under Powers Boulevard into the detention area, and constructing a drop inlet structure. The inlet structure will be sized to convey the 100-year peak discharge from the detention basin to the flow shown on Table 1. The drop inlet will be protected with a trash rack, and will discharge into one or both of the bays of the existing box culvert. Presented on Figure 1 is a detail of the drop inlet structure. Control of the water quality pond level will be accomplished through a separate drop inlet structure, with a peak flow capacity equal to the discharge required to drain the pool in no more than 24 hours. This inlet will discharge into the 100-year drop inlet. The estimated rate of discharge is 16 cubic feet per second, based upon the volume obtained using the methodology presented in Appendix B.

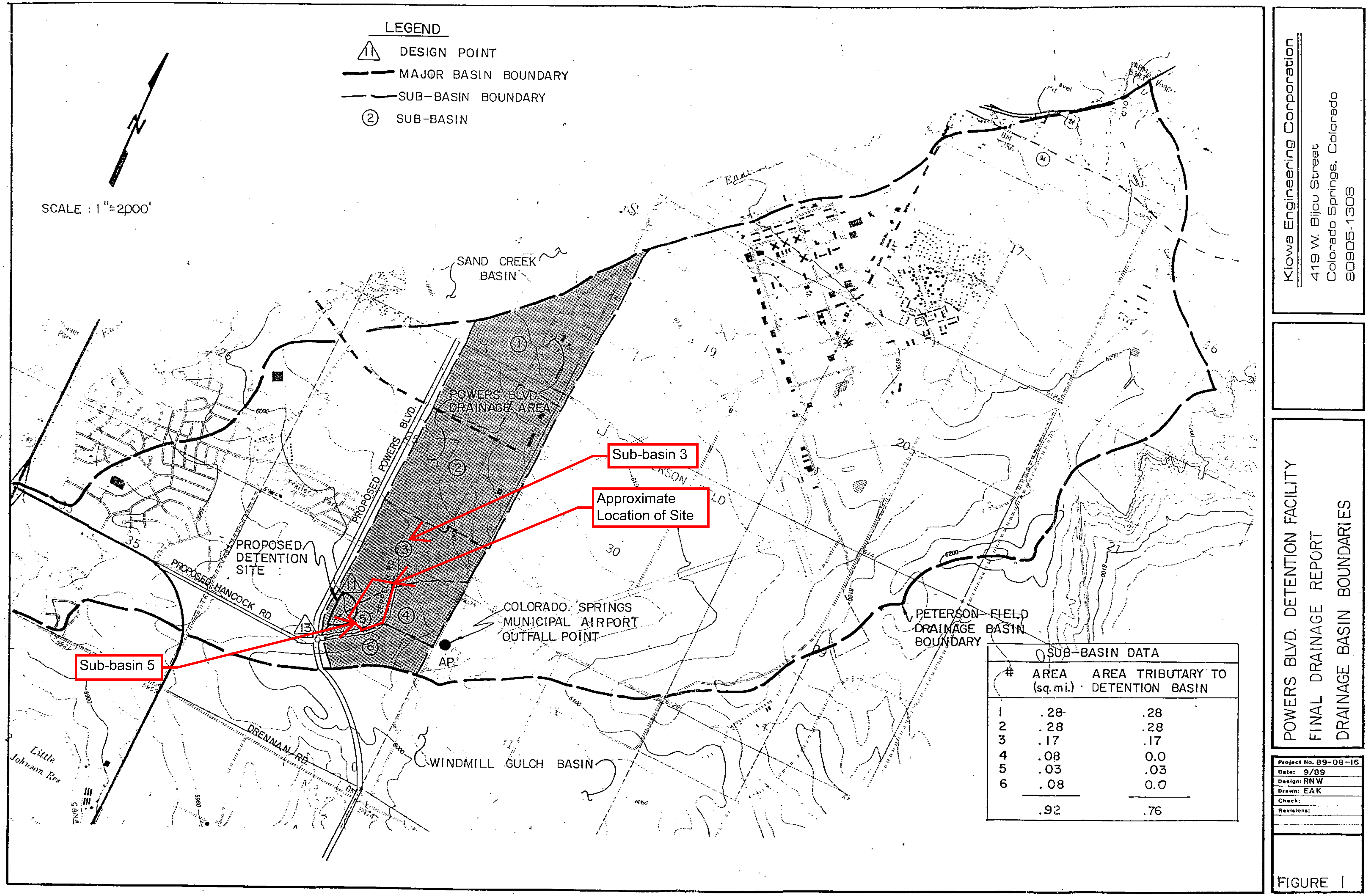
The emergency spillway has been sized to convey the developed 100-year peak flow out of the pond, assuming that the principal outlet is blocked. A riprap weir, of approximately 400 feet in length and a 100-year depth of 1.5 feet, has been sized for the detention basin. The crest elevation has been set at 92.5, which is approximately 1.8 feet higher than the low point of Powers Boulevard adjacent to the detention basin (i.e., Powers Boulevard Station 345+11.75). The crest of the emergency overflow weir will be centered at the low point of proposed Powers Boulevard.

Because of the elevation of the low point of the proposed roadway, the embankment/excavation alternative presented in the Preliminary Design Report is recommended for further design. An embankment of approximately 2000-feet in length, with a maximum elevation of 94.0 will be required for the detention facility. The embankment will form the emergency overflow crest, and can be constructed from materials excavated from the active storage area of the detention facility. A 15-foot crest width will be used. A maintenance trail will follow the crest.

A concrete channel will convey the majority of the developed runoff to the detention basin (Reference CH2M-Hill, Inc., Powers Boulevard, Phase I Design Plans, Sheets 26 and 27). Flow from this channel will pass through an energy dissipation/debris collection structure and then spread into the water quality pool area with a channel transition structure. A trickle channel will be required within the water quality pool to convey very low flows to the water quality outlet structure. Cross slopes within the water quality area will be no more than 0.5 percent.

It is recommended that a forebay be constructed within the water quality pond. The forebay will be formed by constructing a berm across the mid-section of the water quality storage area. The forebay will act to further limit the area where routine (annual) maintenance must be conducted. The forebay will be drained by culverts passing under the berm which form the two bays, or all of the water quality pool. A hard surface maintenance trail will be constructed on top of this berm, which will be capable of withstanding an overtopping event. The forebay will primarily catch the more frequent rainfall events which are not of sufficient volume to entirely fill the water quality pool.

Presented on Figure 1 is the conceptual layout of the detention facility, and the various structures which will be required to operate and maintain the detention basin. Quantity cost and estimates for the facility depicted will be prepared during the later preliminary design phases.



SUB-BASIN DATA

#	AREA (sq. mi.)	AREA TRIBUTARY TO DETENTION BASIN
1	.28	.28
2	.28	.28
3	.17	.17
4	.08	0.0
5	.03	.03
6	.08	0.0
	.92	.76

A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	4.60	24.00	3.29	---	6.07	1896.60	2495.5
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	4.60	24.00	3.29	90.93	6.45	536.42	705.8
STRUCTURE 13	ADDHYD	6.66	7	2	.10	.0	4.60	24.00	1.48	---	6.10	2226.44	334.4
ALTERNATE 1 STORM 2													
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	3.00	24.00	1.82	---	6.08	1038.10	1365.9
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	3.00	24.00	1.82	88.60	6.61	184.01	242.1
STRUCTURE 13	ADDHYD	.92	7	2	.10	.0	3.00	24.00	1.82	---	6.10	342.62	372.4
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	2.70	24.00	1.55	---	6.09	883.00	1161.8
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	2.70	24.00	1.55	88.04	6.61	161.78	212.9
STRUCTURE 13	ADDHYD	.92	7	2	.10	.0	2.70	24.00	1.55	---	6.11	285.75	310.6

SUMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK
A QUESTION MARK(?) AFTER COEFF.(C) INDICATES PARAMETERS OUTSIDE ACCEPTABLE LIMITS, SEE PREVIOUS WARNINGS)

SEC ID	REACH LENGTH (FT)	INFLOW PEAK (CFS)	INFLOW TIME (HR)	OUTFLOW PEAK (CFS)	OUTFLOW TIME (HR)	-OUTFLOW+ INTERV.AREA PEAK (CFS)	-OUTFLOW+ INTERV.AREA TIME (HR)	BASE-FLOW (CFS)	VOLUME ABOVE BASE (IN)	MAIN TIME INCR (HR)	ITER-ATION #	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O/I (Q*)	S/Q @PEAK (K) (SEC)	ATT-KIN COEFF (C)	PEAK TRAVEL TIME STOR-AGE (HR)	PEAK TRAVEL TIME KINE-MATIC (HR)
ALTERNATE 1 STORM 1																			
2	2700	666	6.1	666	6.1	---	---	0	3.29	.10	0	3.81	1.41	.015	1.000	113	1.00?	.00	.00
3	3600	1386	6.1	1386	6.1	---	---	0	3.29	.10	0	3.30	1.42	.018	1.000	130	1.00?	.00	.00
4	1335	1584	6.1	1584	6.1	---	---	0	1.19	.10	0	3.42	1.36	.001	1.000	56	1.00?	.00	.00
5	1680	1584	6.1	1584	6.1	---	---	0	1.19	.10	0	3.79	1.36	.002	1.000	65	1.00?	.00	.00
5	1680	212	6.0	212	6.0	---	---	0	3.29	.10	0	3.20	1.42	.015	1.000	106	1.00?	.00	.00
ALTERNATE 1 STORM 2																			

2	2700	361	6.1	361	6.1	---	---	0	1.82	.10	0	3.59	1.43	.017	1.000	132	1.00?	.00	.00
+ 3	3600	764	6.1	764	6.1	---	---	0	1.82	.10	0	3.03	1.44	.022	1.000	151	1.00?	.00	.00
+ 5	1680	116	6.1	116	6.1	---	---	0	1.82	.10	0	3.18	1.43	.019	1.000	126	1.00?	.00	.00
+ 2	2700	306	6.1	306	6.1	---	---	0	1.55	.10	0	3.55	1.43	.018	1.000	138	1.00?	.00	.00
+ 3	3600	650	6.1	650	6.1	---	---	0	1.55	.10	0	2.97	1.45	.023	1.000	157	1.00?	.00	.00
+ 5	1680	100	6.1	100	6.1	---	---	0	1.55	.10	0	3.18	1.43	.020	1.000	132	1.00?	.00	.00

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

SECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS...24 HR DURATION 1 100YR	2 10YR	
0 STRUCTURE 13	.92			
ALTERNATE 1		2226.44	285.75	COMBINED OUTFLOW
STRUCTURE 12	.76			
ALTERNATE 1		536.42	161.78	OUTFLOW FROM BASIN
STRUCTURE 11	.76			
ALTERNATE 1		1896.60	883.00	INFLOW TO BASIN

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

..AIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

IIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

IIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

END OF 1 JOBS IN THIS RUN

Table 1. Summary of Discharges with Detention.

Design Point	Description	Area (sq.mi.)	24-hour (cfs) (2) 10-year	24-hour (cfs) (2) 100-year
AP	Airport Outfall	5.74	770	1630 (1)
11 in	Powers Boulevard Basin	.76	1040	1900
11 out		.76	370	540
13	Combined Powers Boulevard/Airport Basins	6.5	510	2440

(1) Assumes future Airport detention basins in-place.

(2) 24-hour storm duration controls peak flow and volume for Powers Boulevard Detention Facility design.

APPENDIX A

Hydrologic Analysis

Input File Powcz.DAT

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR-20                                              NOPLOTS
TITLE 001 "POWER DETENTION ALT-6"
TITLE     FUTURE CONDITION (NOT INCL. BASINS 4 & 6)
5 RAINFL 7            0.25
8         0.0         .0005      .0015      .0030      .0045
8         .006        .008       .010       .012       .0143
8         .0165       .0188      .021       .0233      .0255
8         .0278       .0320      .0390      .0460      .0530
8         .06         .075       .10        .400       .70
8         .725        .750       .765       .780       .790
8         .800        .810       .820       .825       .830
8         .835        .840       .845       .850       .855
8         .860        .8638      .8675      .8713      .8750
8         .8788       .8825      .8863      0.8900     0.8938
8         .8975       .9013      .9050      .9083      .9115
8         .9148       .9180      .9210      .9240      .9270
8         .9300       .9325      .9350      .9375      .9400
8         .9425       .9450      .9475      .9500      .9525
8         .9550       .9575      .9600      .9625      .9650
8         .9675       .9700      .9725      .9750      .9775
8         .9800       .9813      .9825      0.9838     0.9850
8         .9863       .9875      .9888      .9900      .9913
8         .9925       .9938      .9950      .9963      .9975
8         .9988       1.000      1.000      1.000      1.000
9 ENDTBL
3 STRUCT      12
8                     82.5       0.         0.
8                     83.0       6.         0.4
8                     84.0       16.        2.3
8                     85.0       21.        7.
8                     86.0       50.        12.5
8                     87.0       100.       19.5
8                     88.0       160.       30.
8                     89.0       200.       40.5
8                     90.0       350.       52.5
8                     91.0       550.       65.
8                     91.5       610.       71.0
8                     92.5       670.       90.
9 ENDTBL
2 XSECTN     2        1.0
8                     6020.      0.         0.
8                     6021.      76.2       8.5
8                     6022.      260.9      20.
8                     6023.      556.7      34.5
8                     6024.      975.3      52.0
8                     6025.      1529.5     72.5


********************80-80 LIST OF INPUT DATA (CONTINUED)**********************

9 ENDTBL
2 XSECTN     3        1.0
8                     6020.      0.         0.
8                     6021.      101.8      11.5
8                     6022.      338.9      26.
8                     6023.      704.1      43.5
8                     6024.      1204.4     64.
[illegible]
```

```
 9 ENDTBL
 2 XSECTN    4        1.0
 8                    6020.      0.         0.
 8                    6021.      50.4       7.5
 8                    6022.      175.5      18.
 8                    6023.      379.5      31.5
 8                    6024.      673.3      48.
 8                    6025.      1064.6     67.5
 8                    6026.      1569.2     90.
 8                    6027.      2187.7     115.5
 8                    6028.      2944.4     144.
 8                    6028.5     3359.8     159.4
 9 ENDTBL
 2 XSECTN    5        1.0
 8                    6020.      0.         0.
 8                    6021.      56.3       7.5
 8                    6022.      196.2      18.
 8                    6023.      424.2      31.5
 8                    6024.      752.8      48.
 8                    6025.      1190.2     67.5
 8                    6026.      1754.4     90.
 8                    6027.      2445.9     115.5
 8                    6028.      3291.9     144.
 8                    6028.5     3756.4     159.4
 9 ENDTBL
 6 RUNOFF 1    01     5 0.282    88.0       0.43
 6 REACH  3  02   5   6 2700.
 6 RUNOFF 1    02     7 0.279    88.0       0.32
 6 ADDHYD 4    02 6 7 5
 6 REACH  3  03   5   6 3600.
 6 RUNOFF 1    03     7 0.169    88.0       0.39
 6 ADDHYD 4    03 6 7 5
 6 SAVMOV 5    03 5   1
 6 REACH  3  04   2   5 1335.
 6 REACH  3  05   5   4 1680.
 6 RUNOFF 1    04     7 0.08     88.0       0.3
 6 REACH  3  05   7   2 1680.
 6 RUNOFF 1    05     3 0.03     88.0       .29
```

********************80-80 LIST OF INPUT DATA (CONTINUED)********************

```
 6 SAVMOV 5    03 1   5
 6 ADDHYD 4    11 5 3 6                              1 1 1 1  1
 6 RUNOFF 1    06     5 0.08     88.0       0.27
 6 RESVOR 2    12 6   3  82.5                        1 1 1 1  1
 6 ADDHYD 4    13 4 3 1
 6 ADDHYD 4    13 1 2 6
 6 ADDHYD 4    13 6 5 1                              1 1 1 1  1
 6 RUNOFF 1    07     6 0.045    88.0       0.25
 6 RUNOFF 1    08     7 0.045    88.0       0.28
 6 RUNOFF 1    09     5 0.41     49.0       1.1
 6 ADDHYD 4    10 5 7 6
   ENDATA
 7 READHD 8      2
 7 READHD 9 0.0         .0830      5.738      0.000
 8          0.0         .0         .0         .0         .0
 8          0.0         .0         .0         .0         .0
 8          0.0         .0         .0         .0         .0
 8          0.0         .0         .0         .0         .0
 8          0.0         .0         .0         .0         .02
 8          0.06        .12        .19        .29        .41
 8          0.54        .68        .82        .97        1.12
```

```
8          1.27      1.43      1.58      1.72      1.86
8          2.01      2.15      2.29      2.42      2.54
8          2.66      2.78      2.92      3.13      3.47
8          3.95      4.6       5.5       6.51      7.6
8          8.7       9.7       10.6      11.5      12.3
8          13.1      13.8      14.6      15.9      18.2
8          21.7      26.4      44.4      149.2     367.7
8          672.2     1027.6    1380.7    1614.5    1552.1
8          1355.7    1154.4    980.3     840.4     727.9
8          631.1     550.2     486.1     436.9     400.4
8          372.1     346.2     324.2     306.8     293.5
8          283.6     276.1     270.5     266.3     263.1
8          260.7     259.0     256.3     249.0     240.2
8          232.2     225.5     220.2     216.0     212.7
8          210.      207.9     206.2     204.8     203.9
8          203.2     202.7     202.4     202.0     201.8
8          201.5     201.3     201.2     201.0     200.8
8          200.7     200.3     198.7     196.6     194.6
8          192.8     191.3     190.1     189.3     188.7
8          188.2     187.7     187.2     186.8     186.6
8          186.4     186.3     186.1     185.8     185.6
8          185.5     185.4     185.4     185.2     185.
8          184.8     184.7     184.7     184.7     184.5
8          184.3     184.1     184.1     184.1     184.1
8          183.9     183.8     183.5     182.9     182.1
1
```

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********************80-80 LIST OF INPUT DATA (CONTINUED)*********************

8          181.3     180.5     179.8     179.2     178.8
8          178.6     178.4     178.2     177.9     177.6
8          177.2     176.7     176.2     176.8     175.5
8          175.2     174.9     174.8     174.6     174.4
8          174.3     174.2     173.6     172.6     171.8
8          171.0     170.4     169.9     169.5     169.2
8          168.9     168.7     168.5     168.4     168.2
8          168.1     168.1     168.0     167.9     167.9
8          167.9     167.8     167.8     167.8     167.8
8          167.8     167.8     167.8     167.8     167.8
8          167.8     167.8     167.8     167.9     167.9
8          167.9     168.0     168.0     168.1     168.1
8          168.2     168.2     168.3     168.3     168.4
8          168.4     168.5     168.6     168.6     168.7
8          168.8     168.9     168.9     169.0     169.1
8          169.2     169.3     169.3     169.4     169.5
8          169.6     169.7     168.9     167.0     165.1
8          163.3     161.9     160.7     159.9     159.3
8          159.0     158.9     158.4     158.1     158.0
8          158.0     158.1     158.1     158.0     157.9
8          157.9     158.0     158.2     158.2     158.2
8          158.1     158.1     158.2     158.4     158.4
8          158.4     158.3     158.3     158.4     158.6
8          158.7     158.6     158.5     158.8     158.6
8          158.8     158.9     158.8     158.7     158.7
8          158.8     158.9     159.0     159.0     158.9
8          158.2     156.3     154.2     152.3     150.8
8          149.6     148.7     148.0     148.5     147.1
9 ENDTBL
  LIST
  INCREM 6            .100
7 COMPUT 7    01    10 0.0       4.6       1.0       7 2   1   1
  ENDCMP 1
  COMPUT 7    01    10 0.0       3.0       1.0       7 2   1   2
  ENDCMP 1
```

```
 7 COMPUT 7     01 - 10 0.0        2.7         1.0        7 2  1  2
   ENDCMP 1
   ENDJOB 2



*******************************END OF 80-80 LIST*******************************
1


TR20 XEQ  2/ 1/90   17:53        "POWER DETENTION ALT-6"                                              JOB  1   PASS   1
    REV PC/09/83                 FUTURE CONDITION (NOT INCL. BASINS 4 & 6)                                     PAGE   1

FILE NO.  1
                              COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY   USER NOTES

          THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20.  CHANGES FROM THE 2/14/74 VERSION INCLUDE:

          REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD.  INPUT DATA PREPARED FOR
             PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

             THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH.  IT IS RECOMMENDED THAT
             THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED.
             THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M"
             VALUES USED IN THE ROUTING PROCEDURE.

             GUIDELINES FOR DETERMINING OR ANALYZING REACH LENGTHS AND COEFFICIENTS (X,M) ARE AVAILABLE IN THE USERS
             MANUAL.  SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

          HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT
             HYDROGRAPH HAVE BEEN IMPROVED.  PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION.  OUTPUT
             HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

          INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

          OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT.  USER
             OPTIONS HAVE BEEN MODIFIED AND AUGMENTED ON THE JOB RECORD, RAINTABLES ADDED, ERROR AND WARNING MESSAGES
             EXPANDED, AND THE SUMMARY TABLES COMPLETELY REVISED.  THE HOLDOUT OPTION IS NOT OPERATIONAL AT THIS TIME.

          PROGRAM QUESTIONS OR PROBLEMS SHOULD BE DIRECTED TO HYDRAULIC ENGINEERS AT THE SCS NATIONAL TECHNICAL CENTERS:

               CHESTER, PA (NORTHEAST)  -- 215-499-3933,       FORT WORTH, TX (SOUTH) -- 334-5242 (FTS)
               LINCOLN, NB (MIDWEST)    -- 541-5318 (FTS),     PORTLAND, OR (WEST)    -- 423-4099 (FTS)
               OR HYDROLOGY UNIT, ENGINEERING DIVISION, LANHAM, MD -- 436-7383 (FTS).


     PROGRAM CHANGES SINCE MAY 1982:

           12/17/82 - CORRECT PEAK RATE FACTOR FOR USER ENTERED DIMHYD
                      CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION
            5/02/83 - CORRECT COMPUTATIONS FOR ---
                              1. DIVISION OF BASEFLOW IN DIVERT OPERATION
                              2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
                              3. CROSS SECTION DATA PLOTTING POSITION
                              4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
                              5. STORAGE ROUTED REACH TRAVEL TIME FOR MULTIPEAK HYDROGRAPH
                              6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
                              7. BASEFLOW ENTERED WITH READHYD
                              8. LOW FLOW SPLIT DURING DIVERT PROCEDURE #2 WHEN SECTION RATINGS START AT DIFFERENT ELEVATIONS
                      ENHANCEMENTS ---
                              1. REPLACE USER MANUAL ERROR CODES (PAGE 4-9 TO 4-11) WITH MESSAGES
```

2. LABEL OUTPUT HYDROGRAPH FILES WITH CROSS SECTION/STRUCTURE, ALTERNATE AND STORM NO'S
09/01/83 - CORRECT INPUT AND OUTPUT ERRORS FOR INTERMEDIATE PEAKS
CORRECT COMBINATION OF RATING TABLES FOR DIVERT
CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS
ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE

EXECUTIVE CONTROL OPERATION READHD RECORD ID

DISCHARGE HYDROGRAPH, HYDROGRAPH LOCATION 2

STARTING TIME= .00 TIME INCREMENT= .08 DRAINAGE AREA= 5.74 BASE FLOW= .00

.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.02
.06	.12	.19	.29	.41
.54	.68	.82	.97	1.12
1.27	1.43	1.58	1.72	1.86
2.01	2.15	2.29	2.42	2.54
2.66	2.78	2.92	3.13	3.47
3.95	4.60	5.50	6.51	7.60
8.70	9.70	10.60	11.50	12.30
13.10	13.80	14.60	15.90	18.20
21.70	26.40	44.40	149.20	367.70
672.20	1027.60	1380.70	1614.50	1552.10
1355.70	1154.40	980.30	840.40	727.90
631.10	550.20	486.10	436.90	400.40
372.10	346.20	324.20	306.80	293.50
283.60	276.10	270.50	266.30	263.10
260.70	259.00	256.30	249.00	240.20
232.20	225.50	220.20	216.00	212.70
210.00	207.90	206.20	204.80	203.90
203.20	202.70	202.40	202.00	201.80
201.50	201.30	201.20	201.00	200.80
200.70	200.30	198.70	196.60	194.60
192.80	191.30	190.10	189.30	188.70
188.20	187.70	187.20	186.80	186.60
186.40	186.30	186.10	185.80	185.60
185.50	185.40	185.40	185.20	185.00
184.80	184.70	184.70	184.70	184.50
184.30	184.10	184.10	184.10	184.10
183.90	183.80	183.50	182.90	182.10
181.30	180.50	179.80	179.20	178.80
178.60	178.40	178.20	177.90	177.60
177.20	176.70	176.20	176.80	175.50
175.20	174.90	174.80	174.60	174.40
174.30	174.20	173.60	172.60	171.80
171.00	170.40	169.90	169.50	169.20
168.90	168.70	168.50	168.40	168.20
168.10	168.10	168.00	167.90	167.90
167.90	167.80	167.80	167.80	167.80
167.80	167.80	167.80	167.80	167.80
167.80	167.80	167.80	167.90	167.90

8	167.90	168.00	168.00	168.10	168.10
8	168.20	168.20	168.30	168.30	168.40
8	168.40	168.50	168.60	168.60	168.70
8	168.80	168.90	168.90	169.00	169.10
8	169.20	169.30	169.30	169.40	169.50
8	169.60	169.70	168.90	167.00	165.10
8	163.30	161.90	160.70	159.90	159.30
8	159.00	158.90	158.40	158.10	158.00
8	158.00	158.10	158.10	158.00	157.90
8	157.90	158.00	158.20	158.20	158.20
8	158.10	158.10	158.20	158.40	158.40
8	158.40	158.30	158.30	158.40	158.60
8	158.70	158.60	158.50	158.80	158.60
8	158.80	158.90	158.80	158.70	158.70
8	158.80	158.90	159.00	159.00	158.90
8	158.20	156.30	154.20	152.30	150.80
8	149.60	148.70	148.00	148.50	147.10

ENDTBL

XECUTIVE CONTROL OPERATION LIST RECORD ID

ISTING OF CURRENT DATA

	XSECTN NO.	DRAINAGE AREA
2 XSECTN	2	1.0000

	ELEVATION	DISCHARGE	END AREA
8	6020.00	.00	.00
	6021.00	76.20	8.50
	6022.00	260.90	20.00
8	6023.00	556.70	34.50
8	6024.00	975.30	52.00
	6025.00	1529.50	72.50

9 ENDTBL

	XSECTN NO.	DRAINAGE AREA
2 XSECTN	3	1.0000

	ELEVATION	DISCHARGE	END AREA
8	6020.00	.00	.00
8	6021.00	101.80	11.50
	6022.00	338.90	26.00
8	6023.00	704.10	43.50
8	6024.00	1204.40	64.00
	6025.00	1850.50	87.50

ENDTBL

```
            XSECTN NO.      DRAINAGE AREA
 2 XSECTN     4               1.0000

                             ELEVATION   DISCHARGE    END AREA
 8                            6020.00         .00         .00
 8                            6021.00       50.40        7.50
 8                            6022.00      175.50       18.00
 8                            6023.00      379.50       31.50
 8                            6024.00      673.30       48.00
 8                            6025.00     1064.60       67.50
 8                            6026.00     1569.20       90.00
 8                            6027.00     2187.70      115.50
 8                            6028.00     2944.40      144.00
1
```



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 8                            6028.50     3359.80      159.40
 9 ENDTBL

            XSECTN NO.      DRAINAGE AREA
 2 XSECTN     5               1.0000

                             ELEVATION   DISCHARGE    END AREA
 8                            6020.00         .00         .00
 8                            6021.00       56.30        7.50
 8                            6022.00      196.20       18.00
 8                            6023.00      424.20       31.50
 8                            6024.00      752.80       48.00
 8                            6025.00     1190.20       67.50
                              6026.00     1754.40       90.00
 8                            6027.00     2445.90      115.50
 8                            6028.00     3291.90      144.00
                              6028.50     3756.40      159.40
   ENDTBL

             STRUCT NO.      ELEVATION   DISCHARGE     STORAGE
   STRUCT       12

 8                              82.50         .00         .00
                                83.00        6.00         .40
                                84.00       16.00        2.30
 8                              85.00       21.00        7.00
 8                              86.00       50.00       12.50
                                87.00      100.00       19.50
 8                              88.00      160.00       30.00
 8                              89.00      200.00       40.50
                                90.00      350.00       52.50
                                91.00      550.00       65.00
 8                              91.50      610.00       71.00
                                92.50      670.00       90.00
   ENDTBL

                       TIME INCREMENT
   DIMHYD                  .0200

 8                 .0000       .0300       .1000       .1900       .3100
                   .4700       .6600       .8200       .9300       .9900
                  1.0000       .9900       .9300       .8600       .7800
 8                 .6800       .5600       .4600       .3900       .3300
```

```
6              .2800      .2410      .2070      .1740      .1470
8              .1260      .1070      .0910      .0770      .0660
8              .0550      .0470      .0400      .0340      .0290
8              .0250      .0210      .0180      .0150      .0130
1
```

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TR20 XEQ  2/ 1/90   17:53      "POWER DETENTION ALT-6"                              JOB  1   PASS  1
    REV PC/09/83               FUTURE CONDITION (NOT INCL. BASINS 4 & 6)                      PAGE  6
```

```
8              .0110      .0090      .0080      .0070      .0060
8              .0050      .0040      .0030      .0020      .0010
8              .0000      .0000      .0000      .0000      .0000
9 ENDTBL

  COMPUTED PEAK RATE FACTOR =  484.00


    TABLE NO.       TIME INCREMENT
 5 RAINFL 1                 .5000

8              .0000      .0080      .0170      .0260      .0350
8              .0450      .0550      .0650      .0760      .0870
8              .0990      .1120      .1260      .1400      .1560
8              .1740      .1940      .2190      .2540      .3030
8              .5150      .5830      .6240      .6550      .6820
8              .7060      .7280      .7480      .7660      .7830
8              .7990      .8150      .8300      .8440      .8570
8              .8700      .8820      .8930      .9050      .9160
8              .9260      .9360      .9460      .9560      .9650
8              .9740      .9830      .9920     1.0000     1.0000
  ENDTBL

    TABLE NO.       TIME INCREMENT
  RAINFL 2                  .2500

8              .0000      .0020      .0050      .0080      .0110
               .0140      .0170      .0200      .0230      .0260
               .0290      .0320      .0350      .0380      .0410
8              .0440      .0480      .0520      .0560      .0600
8              .0640      .0680      .0720      .0760      .0800
               .0850      .0900      .0950      .1000      .1050
8              .1100      .1150      .1200      .1260      .1330
8              .1400      .1470      .1550      .1630      .1720
               .1810      .1910      .2030      .2180      .2360
               .2570      .2830      .3870      .6630      .7070
8              .7350      .7580      .7760      .7910      .8040
8              .8150      .8250      .8340      .8420      .8490
               .8560      .8630      .8690      .8750      .8810
8              .8870      .8930      .8980      .9030      .9080
8              .9130      .9180      .9220      .9260      .9300
               .9340      .9380      .9420      .9460      .9500
               .9530      .9560      .9590      .9620      .9650
8              .9680      .9710      .9740      .9770      .9800
8              .9830      .9860      .9890      .9920      .9950
               .9980     1.0000     1.0000     1.0000     1.0000
9 ENDTBL
1
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```
TR20 XEQ  2/ 1/90   17:53      "POWER DETENTION ALT-6"                              JOB  1   PASS  1
    REV PC/09/83               FUTURE CONDITION (NOT INCL. BASINS 4 & 6)                      PAGE  7
```

TABLE NO.	TIME INCREMENT				
5 RAINFL 3	.5000				

8	.0000	.0100	.0220	.0360	.0510
8	.0670	.0830	.0990	.1160	.1350
8	.1560	.1790	.2040	.2330	.2680
8	.3100	.4250	.4800	.5200	.5500
8	.5770	.6010	.6230	.6440	.6640
8	.6830	.7010	.7190	.7360	.7530
8	.7690	.7850	.8000	.8150	.8300
8	.8440	.8580	.8710	.8840	.8960
8	.9080	.9200	.9320	.9440	.9560
8	.9670	.9780	.9890	1.0000	1.0000

ENDTBL

TABLE NO.	TIME INCREMENT				
5 RAINFL 4	.5000				

8	.0000	.0040	.0080	.0120	.0160
8	.0200	.0250	.0300	.0350	.0400
	.0450	.0500	.0550	.0600	.0650
	.0700	.0750	.0810	.0870	.0930
8	.0990	.1050	.1110	.1180	.1250
	.1320	.1400	.1480	.1560	.1650
	.1740	.1840	.1950	.2070	.2200
8	.2360	.2550	.2770	.3030	.4090
8	.5150	.5490	.5830	.6050	.6240
	.6400	.6550	.6690	.6820	.6940
	.7050	.7160	.7270	.7380	.7480
8	.7580	.7670	.7760	.7840	.7920
	.8000	.8080	.8160	.8230	.8300
	.8370	.8440	.8510	.8580	.8640
8	.8700	.8760	.8820	.8880	.8940
8	.9000	.9060	.9110	.9160	.9210
	.9260	.9310	.9360	.9410	.9460
	.9510	.9560	.9610	.9660	.9710
8	.9760	.9800	.9840	.9880	.9920
	.9960	1.0000	1.0000	1.0000	1.0000

ENDTBL

TABLE NO.	TIME INCREMENT				
RAINFL 5	.5000				

8	.0000	.0020	.0050	.0080	.0110
	.0140	.0170	.0200	.0230	.0260

8	.0290	.0320	.0350	.0380	.0410
	.0440	.0470	.0510	.0550	.0590
	.0630	.0670	.0710	.0750	.0790
8	.0840	.0890	.0940	.0990	.1040
8	.1090	.1140	.1200	.1260	.1330
	.1400	.1470	.1540	.1620	.1710
	.1810	.1920	.2040	.2170	.2330
8	.2520	.2770	.3180	.6380	.6980
	.7290	.7520	.7700	.7850	.7980
	.8090	.8190	.8290	.8380	.8460
8	.8540	.8610	.8680	.8740	.8800

.8880	.8920	.8970	.9020	.9070
.9120	.9170	.9210	.9250	.9290
.9330	.9370	.9410	.9450	.9490
.9530	.9570	.9600	.9630	.9660
.9690	.9720	.9750	.9780	.9810
.9840	.9870	.9900	.9930	.9960
.9980	1.0000	1.0000	1.0000	1.0000

9 ENDTBL

TABLE NO. TIME INCREMENT
5 RAINFL 6 .0200

.0000	.0080	.0162	.0246	.0333
.0425	.0524	.0630	.0743	.0863
.0990	.1124	.1265	.1420	.1595
.1800	.2050	.2550	.3450	.4370
.5300	.6030	.6330	.6600	.6840
.7050	.7240	.7420	.7590	.7750
.7900	.8043	.8180	.8312	.8439
.8561	.8678	.8790	.8898	.9002
.9103	.9201	.9297	.9391	.9483
.9573	.9661	.9747	.9832	.9916
1.0000	1.0000	1.0000	1.0000	1.0000

ENDTBL

TABLE NO. TIME INCREMENT
RAINFL 7 .2500

.0000	.0005	.0015	.0030	.0045
.0060	.0080	.0100	.0120	.0143
.0165	.0188	.0210	.0233	.0255
.0278	.0320	.0390	.0460	.0530
.0600	.0750	.1000	.4000	.7000
.7250	.7500	.7650	.7800	.7900
.8000	.8100	.8200	.8250	.8300
.8350	.8400	.8450	.8500	.8550

.8600	.8638	.8675	.8713	.8750
.8788	.8825	.8863	.8900	.8938
.8975	.9013	.9050	.9083	.9115
.9148	.9180	.9210	.9240	.9270
.9300	.9325	.9350	.9375	.9400
.9425	.9450	.9475	.9500	.9525
.9550	.9575	.9600	.9625	.9650
.9675	.9700	.9725	.9750	.9775
.9800	.9813	.9825	.9838	.9850
.9863	.9875	.9888	.9900	.9913
.9925	.9938	.9950	.9963	.9975
.9988	1.0000	1.0000	1.0000	1.0000

ENDTBL

STANDARD CONTROL INSTRUCTIONS

```
6 RUNOFF 1    1    5      .2820     88.0000      .43000 0 0 0 0 0
6 REACH  3  2  5   6  2700.0000      .0000      .00000 0 0 0 0 0
6 RUNOFF 1    2    7      .2790     88.0000      .32000 0 0 0 0 0
6 ADDHYD 4    2 6 7 5                                 0 0 0 0 0 0
6 REACH  3  3  5   6  3600.0000      .0000      .00000 0 0 0 0 0
6 RUNOFF 1    3    7      .1690     88.0000      .39000 0 0 0 0 0
6 ADDHYD 4    3 6 7 5                                 0 0 0 0 0 0
6 SAVMOV 5    3 5  1
6 REACH  3  4  2   5  1335.0000      .0000      .00000 0 0 0 0 0
6 REACH  3  5  5   4  1680.0000      .0000      .00000 0 0 0 0 0
6 RUNOFF 1    4    7      .0800     88.0000      .30000 0 0 0 0 0
6 REACH  3  5  7   2  1680.0000      .0000      .00000 0 0 0 0 0
6 RUNOFF 1    5    3      .0300     88.0000      .29000 0 0 0 0 0
6 SAVMOV 5    3 1  5
6 ADDHYD 4   11 5 3 6                                 1 1 1 1 0 1
6 RUNOFF 1    6    5      .0800     88.0000      .27000 0 0 0 0 0
6 RESVOR 2   12 6  3    82.5000                       1 1 1 1 0 1
6 ADDHYD 4   13 4 3 1                                 0 0 0 0 0 0
6 ADDHYD 4   13 1 2 6                                 0 0 0 0 0 0
6 ADDHYD 4   13 6 5 1                                 1 1 1 1 0 1
6 RUNOFF 1    7    6      .0450     88.0000      .25000 0 0 0 0 0
6 RUNOFF 1    8    7      .0450     88.0000      .28000 0 0 0 0 0
6 RUNOFF 1    9    5      .4100     49.0000     1.10000 0 0 0 0 0
  ADDHYD 4   10 5 7 6                                 0 0 0 0 0 0
  ENDATA

END OF LISTING
1
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TR20 XEQ  2/ 1/90   17:53      "POWER DETENTION ALT-6"                                   JOB  1   PASS  1
    REV PC/09/83               FUTURE CONDITION (NOT INCL. BASINS 4 & 6)                          PAGE  11


EXECUTIVE CONTROL OPERATION INCREM                                                       RECORD ID
                               MAIN TIME INCREMENT =   .10 HOURS

EXECUTIVE CONTROL OPERATION COMPUT                                                       RECORD ID
                               FROM STRUCTURE  1
                                             TO STRUCTURE 10
      STARTING TIME =   .00   RAIN DEPTH =  4.60   RAIN DURATION=  1.00   RAIN TABLE NO.= 7   ANT. MOIST. COND= 2
      ALTERNATE NO.= 1        STORM NO.= 1   MAIN TIME INCREMENT =   .10 HOURS

    ***  WARNING  REACH   2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   4 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***


OPERATION ADDHYD   STRUCTURE 11

         PEAK TIME(HRS)             PEAK DISCHARGE(CFS)         PEAK ELEVATION(FEET)
            6.07                        1896.60                      (NULL)
```

9.00	41.75	(NULL)
12.85	31.65	(NULL)
13.83	27.55	(NULL)
19.86	21.20	(NULL)
23.85	10.77	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.20	1.32	4.08	9.72	19.65	102.35	421.77	923.23	1411.73
6.00	DISCHG	1778.56	1878.35	1519.89	1002.83	631.70	430.47	319.22	243.67	191.29	159.30
7.00	DISCHG	141.64	129.75	115.04	101.13	92.24	87.71	85.37	84.15	83.51	83.22
8.00	DISCHG	82.96	79.87	70.04	58.57	50.67	46.37	44.12	42.95	42.32	41.98
9.00	DISCHG	41.81	41.73	41.69	41.68	41.68	41.70	41.71	41.72	41.73	41.74
10.00	DISCHG	41.72	41.00	38.63	35.86	33.82	32.55	31.88	31.68	31.68	31.60
11.00	DISCHG	31.39	31.27	31.37	31.53	31.54	31.38	31.28	31.39	31.56	31.57
12.00	DISCHG	31.41	31.32	31.42	31.59	31.60	31.44	31.35	31.45	31.63	31.63
13.00	DISCHG	31.46	31.07	30.17	29.17	28.37	27.78	27.46	27.44	27.54	27.51
14.00	DISCHG	27.33	27.04	26.53	26.02	25.67	25.49	25.39	25.34	25.32	25.31
15.00	DISCHG	25.29	24.98	23.98	22.82	22.02	21.58	21.35	21.24	21.17	21.14
16.00	DISCHG	21.12	21.12	21.11	21.11	21.11	21.12	21.12	21.12	21.12	21.13
17.00	DISCHG	21.13	21.13	21.13	21.14	21.14	21.14	21.14	21.15	21.15	21.15
18.00	DISCHG	21.15	21.16	21.16	21.16	21.16	21.16	21.17	21.17	21.17	21.17
19.00	DISCHG	21.18	21.18	21.18	21.18	21.18	21.19	21.19	21.19	21.19	21.20
20.00	DISCHG	21.17	20.43	18.01	15.17	13.10	11.82	11.13	10.91	10.91	10.82
21.00	DISCHG	10.61	10.48	10.57	10.73	10.73	10.56	10.46	10.56	10.73	10.73
22.00	DISCHG	10.57	10.46	10.57	10.73	10.73	10.57	10.47	10.57	10.73	10.73

23.00	DISCHG	10.57	10.47	10.57	10.74	10.74	10.57	10.47	10.57	10.74	10.74
24.00	DISCHG	10.54	9.62	7.11	4.25	2.27	1.20	.63	.34	.18	.09
25.00	DISCHG	.04	.02	.01	.00						

RUNOFF VOLUME ABOVE BASEFLOW = 3.29 WATERSHED INCHES, 1613.43 CFS-HRS, 133.33 ACRE-FEET; BASEFLOW = .00 CFS

PERATION RESVOR STRUCTURE 12

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.45	536.42	90.93

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.01	.10	.40	1.16	2.74	7.29	16.44	26.76	84.16
5.00	ELEV	82.50	82.50	82.51	82.53	82.60	82.73	83.13	84.09	85.20	86.68
6.00	DISCHG	156.97	230.97	381.80	490.89	531.37	531.34	511.93	483.34	450.37	416.25
6.00	ELEV	87.95	89.21	90.16	90.70	90.91	90.91	90.81	90.67	90.50	90.33
7.00	DISCHG	383.28	352.58	329.43	307.69	286.96	267.61	249.82	233.61	218.90	205.58
7.00	ELEV	90.17	90.01	89.86	89.72	89.58	89.45	89.33	89.22	89.13	89.04
8.00	DISCHG	197.96	194.35	190.65	186.73	182.64	178.48	174.35	170.30	166.34	162.49
8.00	ELEV	88.95	88.86	88.77	88.67	88.57	88.46	88.36	88.26	88.16	88.06
9.00	DISCHG	158.14	152.78	147.65	142.76	138.10	133.65	129.41	125.36	121.50	117.82
9.00	ELEV	87.97	87.88	87.79	87.71	87.63	87.56	87.49	87.42	87.36	87.30
10.00	DISCHG	114.31	110.95	107.67	104.42	101.21	97.60	93.85	90.29	86.93	83.76
10.00	ELEV	87.24	87.18	87.13	87.07	87.02	86.95	86.88	86.81	86.74	86.68
11.00	DISCHG	80.76	77.93	75.26	72.74	70.38	68.15	66.04	64.05	62.18	60.43
11.00	ELEV	86.62	86.56	86.51	86.45	86.41	86.36	86.32	86.28	86.24	86.21
12.00	DISCHG	58.77	57.20	55.71	54.33	53.02	51.79	50.62	49.64	48.87	48.13
12.00	ELEV	86.18	86.14	86.11	86.09	86.06	86.04	86.01	85.99	85.96	85.94
13.00	DISCHG	47.43	46.74	46.05	45.35	44.64	43.94	43.24	42.57	41.93	41.31
13.00	ELEV	85.91	85.89	85.86	85.84	85.82	85.79	85.77	85.74	85.72	85.70
14.00	DISCHG	40.72	40.14	39.57	39.01	38.44	37.90	37.36	36.85	36.36	35.89
14.00	ELEV	[illegible]	85.66	85.64	85.62	85.60	85.58	85.56	85.55	85.53	85.51

15.00	DISCHG	35.44	35.00	34.55	34.07	33.58	33.07	32.58	32.10	31.63	31.19
15.00	ELEV	85.50	85.48	85.47	85.45	85.43	85.42	85.40	85.38	85.37	85.35
16.00	DISCHG	30.76	30.35	29.95	29.58	29.22	28.87	28.54	28.22	27.92	27.63
16.00	ELEV	85.34	85.32	85.31	85.30	85.28	85.27	85.26	85.25	85.24	85.23
17.00	DISCHG	27.35	27.09	26.83	26.59	26.36	26.14	25.92	25.72	25.52	25.34
17.00	ELEV	85.22	85.21	85.20	85.19	85.18	85.18	85.17	85.16	85.16	85.15
18.00	DISCHG	25.16	24.99	24.82	24.67	24.52	24.38	24.24	24.11	23.98	23.86
18.00	ELEV	85.14	85.14	85.13	85.13	85.12	85.12	85.11	85.11	85.10	85.10
19.00	DISCHG	23.75	23.64	23.53	23.43	23.34	23.25	23.16	23.07	22.99	22.92
19.00	ELEV	85.09	85.09	85.09	85.08	85.08	85.08	85.07	85.07	85.07	85.07
20.00	DISCHG	22.84	22.76	22.61	22.35	22.00	21.59	21.16	20.94	20.86	20.77
20.00	ELEV	85.06	85.06	85.06	85.05	85.03	85.02	85.01	84.99	84.97	84.95
21.00	DISCHG	20.68	20.59	20.50	20.42	20.33	20.25	20.16	20.08	20.00	19.91
21.00	ELEV	84.94	84.92	84.90	84.88	84.87	84.85	84.83	84.82	84.80	84.78
22.00	DISCHG	19.83	19.75	19.67	19.59	19.51	19.44	19.36	19.28	19.21	19.13
22.00	ELEV	84.77	84.75	84.73	84.72	84.70	84.69	84.67	84.66	84.64	84.63

23.00	DISCHG	19.06	18.98	18.91	18.84	18.77	18.69	18.62	18.55	18.48	18.42
23.00	ELEV	84.61	84.60	84.58	84.57	84.55	84.54	84.52	84.51	84.50	84.48
24.00	DISCHG	18.35	18.28	18.19	18.08	17.95	17.81	17.66	17.51	17.36	17.21
24.00	ELEV	84.47	84.46	84.44	84.42	84.39	84.36	84.33	84.30	84.27	84.24
25.00	DISCHG	17.06	16.91	16.76	16.61	16.47	16.32	16.18	16.04	15.51	14.85
25.00	ELEV	84.21	84.18	84.15	84.12	84.09	84.06	84.04	84.01	83.95	83.88
26.00	DISCHG	14.22	13.61	13.03	12.48	11.95	11.44	10.95	10.48	10.04	9.61
26.00	ELEV	83.82	83.76	83.70	83.65	83.59	83.54	83.50	83.45	83.40	83.36
27.00	DISCHG	9.20	8.81	8.43	8.08	7.73	7.40	7.09	6.79	6.50	6.22
27.00	ELEV	83.32	83.28	83.24	83.21	83.17	83.14	83.11	83.08	83.05	83.02
28.00	DISCHG	5.88	5.19	4.59	4.05	3.58	3.16	2.79	2.47	2.18	1.92
28.00	ELEV	82.99	82.93	82.88	82.84	82.80	82.76	82.73	82.71	82.68	82.66
29.00	DISCHG	1.70	1.50	1.33	1.17	1.03	.91	.81	.71	.63	.56
29.00	ELEV	82.64	82.63	82.61	82.60	82.59	82.58	82.57	82.56	82.55	82.55

RUNOFF VOLUME ABOVE BASEFLOW = 3.29 WATERSHED INCHES, 1611.83 CFS-HRS, 133.20 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.10	2226.44	(NULL)
19.95	197.17	(NULL)
23.81	179.74	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = 6.66 SQ.MI.

2.00	DISCHG	.02	.08	.16	.26	.40	.56	.73	.90	1.08	1.26
3.00	DISCHG	1.45	1.63	1.80	1.97	2.14	2.31	2.46	2.61	2.75	2.92
4.00	DISCHG	3.20	3.66	4.34	5.33	6.52	7.84	9.12	10.26	11.35	12.33
5.00	DISCHG	13.27	14.24	15.99	19.53	26.02	39.63	134.01	450.43	930.24	1507.72
6.00	DISCHG	2035.53	2226.19	2062.21	1804.08	1580.43	1396.91	1237.74	1093.52	971.72	876.05
7.00	DISCHG	800.79	735.83	681.10	634.67	597.12	566.25	540.31	518.35	499.53	483.43
8.00	DISCHG	473.38	462.73	445.87	429.63	416.62	406.06	397.22	389.60	382.98	377.14
9.00	DISCHG	371.32	365.00	359.20	353.92	348.83	344.09	339.56	335.36	331.28	327.39
10.00	DISCHG	323.61	318.68	312.26	305.99	300.39	294.95	289.98	285.67	281.73	277.93
11.00	DISCHG	274.33	271.15	268.29	265.68	263.01	260.42	258.14	256.08	254.22	252.19
12.00	DISCHG	250.24	248.50	247.05	245.66	244.09	242.57	241.31	240.38	239.61	238.65
13.00	DISCHG	237.64	236.22	234.34	232.46	230.69	229.10	227.83	226.93	226.08	225.15
14.00	DISCHG	224.14	223.04	221.74	220.78	219.96	218.52	217.62	216.96	216.22	215.56
15.00	DISCHG	214.99	213.85	211.94	210.24	208.78	207.58	206.55	205.70	204.91	204.22
16.00	DISCHG	203.62	203.00	202.50	202.08	201.60	201.22	200.89	200.47	200.17	199.88
17.00	DISCHG	199.60	199.34	199.08	198.84	198.61	198.39	198.18	198.07	197.88	197.76

18.00	DISCHG	197.61	197.54	197.41	197.32	197.24	197.13	197.10	197.00	196.99	196.92
19.00	DISCHG	196.90	196.91	196.89	196.85	196.87	196.90	196.92	196.87	196.91	196.96
20.00	DISCHG	197.00	195.52	192.31	189.20	186.67	184.60	183.17	182.38	182.08	181.52
21.00	DISCHG	180.98	180.78	180.79	180.80	180.60	180.35	180.28	180.41	180.48	180.37
22.00	DISCHG	180.13	180.07	180.21	180.27	180.17	179.94	179.88	180.02	180.16	180.00
23.00	DISCHG	179.79	179.91	179.86	179.99	179.83	179.60	179.55	179.65	179.74	179.67
24.00	DISCHG	179.34	177.68	174.41	171.29	168.99	17.89	17.69	17.52	17.36	17.21

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25.00	DISCHG	17.06	16.91	16.76	16.61	16.47	16.32	16.18	16.04	15.51	14.85
26.00	DISCHG	14.22	13.61	13.03	12.48	11.95	11.44	10.95	10.48	10.04	9.61
27.00	DISCHG	9.20	8.81	8.43	8.08	7.73	7.40	7.09	6.79	6.50	6.22
28.00	DISCHG	5.88	5.19	4.59	4.05	3.58	3.16	2.79	2.47	2.18	1.92
29.00	DISCHG	1.70	1.50	1.33	1.17	1.03	.91	.81	.71	.63	.56

RUNOFF VOLUME ABOVE BASEFLOW = 1.48 WATERSHED INCHES, 6345.51 CFS-HRS, 524.39 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

COMPUTATIONS COMPLETED FOR PASS 1

EXECUTIVE CONTROL OPERATION COMPUT RECORD ID

+ FROM STRUCTURE 1

TO STRUCTURE 10

STARTING TIME = .00 RAIN DEPTH = 3.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 7 ANT. MOIST. COND= 2

ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

*** WARNING REACH 2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

PERATION ADDHYD STRUCTURE 11

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.08	1038.10	(NULL)
9.91	25.29	(NULL)
12.86	19.28	(NULL)
13.83	16.80	(NULL)
19.87	13.01	(NULL)
23.85	6.62	(NULL)

IME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.28	25.47	156.94	407.55	694.02
6.00	DISCHG	939.01	1034.86	854.39	569.21	361.18	248.33	185.95	143.04	112.98	94.60
7.00	DISCHG	84.48	77.60	68.92	60.66	55.37	52.69	51.32	50.61	50.26	50.11
8.00	DISCHG	49.98	48.15	42.23	35.33	30.58	27.99	26.64	25.94	25.57	25.37
9.00	DISCHG	25.27	25.23	25.22	25.21	25.22	25.24	25.25	25.27	25.28	25.29
10.00	DISCHG	25.28	24.85	23.42	21.74	20.51	19.75	19.34	19.22	19.23	19.18
11.00	DISCHG	19.06	18.99	19.05	19.16	19.16	19.07	19.02	19.08	19.19	19.20
12.00	DISCHG	19.11	19.05	19.12	19.22	19.23	19.14	19.08	19.15	19.26	19.27
13.00	DISCHG	19.17	18.93	18.38	17.78	17.30	16.94	16.74	16.73	16.80	16.78
14.00	DISCHG	16.67	16.50	16.19	15.88	15.67	15.56	15.50	15.48	15.46	15.46
15.00	DISCHG	15.45	15.26	14.65	13.94	13.45	13.19	13.05	12.98	12.94	12.93
16.00	DISCHG	12.92	12.91	12.91	12.91	12.92	12.92	12.92	12.93	12.93	12.93

17.00	DISCHG	12.93	12.94	12.94	12.94	12.94	12.95	12.95	12.95	12.95	12.96

18.00	DISCHG	12.96	12.96	12.97	12.97	12.97	12.97	12.98	12.98	12.98	12.98
19.00	DISCHG	12.99	12.99	12.99	12.99	13.00	13.00	13.00	13.00	13.01	13.01
20.00	DISCHG	13.00	12.54	11.06	9.31	8.04	7.26	6.83	6.70	6.70	6.65
21.00	DISCHG	6.52	6.44	6.49	6.59	6.59	6.49	6.43	6.49	6.59	6.59
22.00	DISCHG	6.49	6.43	6.49	6.60	6.60	6.50	6.43	6.50	6.60	6.60
23.00	DISCHG	6.50	6.43	6.50	6.60	6.60	6.50	6.44	6.50	6.60	6.61
24.00	DISCHG	6.48	5.91	4.37	2.61	1.40	.74	.39	.21	.11	.06
25.00	DISCHG	.03	.01	.00							

RUNOFF VOLUME ABOVE BASEFLOW = 1.82 WATERSHED INCHES, 891.53 CFS-HRS, 73.68 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 12

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.61	184.01	88.60

IME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.02	1.52	8.18	16.79	23.27
5.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.63	83.22	84.16	85.08
6.00	DISCHG	59.55	110.24	148.73	169.88	179.04	182.93	183.99	183.39	181.67	179.26
6.00	ELEV	86.19	87.17	87.81	88.25	88.48	88.57	88.60	88.58	88.54	88.48
7.00	DISCHG	176.48	173.52	170.41	167.14	163.76	160.35	155.53	150.70	146.08	141.65
7.00	ELEV	88.41	88.34	88.26	88.18	88.09	88.01	87.93	87.85	87.77	87.69
8.00	DISCHG	137.43	133.35	129.28	125.11	120.86	116.63	112.51	108.53	104.71	101.06
8.00	ELEV	87.62	87.56	87.49	87.42	87.35	87.28	87.21	87.14	87.08	87.02
9.00	DISCHG	96.97	92.86	88.98	85.32	81.88	78.63	75.57	72.68	69.97	67.40
9.00	ELEV	86.94	86.86	86.78	86.71	86.64	86.57	86.51	86.45	86.40	86.35
10.00	DISCHG	64.99	62.70	60.49	58.32	56.18	54.12	52.13	50.25	48.86	47.60
10.00	ELEV	86.30	86.25	86.21	86.17	86.12	86.08	86.04	86.00	85.96	85.92
11.00	DISCHG	46.38	45.22	44.10	43.03	42.02	41.04	40.10	39.20	38.35	37.53
11.00	ELEV	85.88	85.84	85.80	85.76	85.72	85.69	85.66	85.63	85.60	85.57
12.00	DISCHG	36.75	35.99	35.27	34.59	33.93	33.30	32.70	32.12	31.57	31.04
12.00	ELEV	85.54	85.52	85.49	85.47	85.45	85.42	85.40	85.38	85.36	85.35
13.00	DISCHG	30.54	30.05	29.56	29.07	28.58	28.09	27.61	27.15	26.71	26.28
13.00	ELEV	85.33	85.31	85.30	85.28	85.26	85.24	85.23	85.21	85.20	85.18
14.00	DISCHG	25.87	25.48	25.09	24.70	24.32	23.95	23.59	23.25	22.91	22.60
14.00	ELEV	85.17	85.15	85.14	85.13	85.11	85.10	85.09	85.08	85.07	85.06
15.00	DISCHG	22.29	22.00	21.70	21.38	21.05	20.94	20.87	20.81	20.74	20.67
15.00	ELEV	85.04	85.03	85.02	85.01	85.00	84.99	84.97	84.96	84.95	84.93
16.00	DISCHG	20.60	20.53	20.47	20.40	20.34	20.27	20.21	20.14	20.08	20.02
16.00	ELEV	84.92	84.91	84.89	84.88	84.87	84.85	84.84	84.83	84.82	84.80
17.00	DISCHG	19.95	19.89	19.83	19.77	19.71	19.65	19.59	19.54	19.48	19.42
17.00	ELEV	84.79	84.78	84.77	84.75	84.74	84.73	84.72	84.71	84.70	84.68
18.00	DISCHG	19.36	19.31	19.25	19.20	19.14	19.09	19.04	18.98	18.93	18.88
18.00	ELEV	84.67	84.66	84.65	84.64	84.63	84.62	84.61	84.60	84.59	84.58
19.00	DISCHG	18.83	18.78	18.73	18.67	18.63	18.58	18.53	18.48	18.43	18.38
19.00	ELEV	84.57	84.56	84.55	84.53	84.53	84.52	84.51	84.50	84.49	84.48
20.00	DISCHG	18.34	18.29	18.23	18.16	18.08	17.99	17.89	17.79	17.70	17.60

20.00	ELEV	84.47	84.46	84.45	84.43	84.42	84.40	84.38	84.36	84.34	84.32
21.00	DISCHG	17.50	17.41	17.31	17.22	17.12	17.03	16.94	16.85	16.76	16.67
21.00	ELEV	84.30	84.28	84.26	84.24	84.22	84.21	84.19	84.17	84.15	84.13
22.00	DISCHG	16.58	16.49	16.40	16.32	16.23	16.15	16.06	15.89	15.49	15.11
22.00	ELEV	84.12	84.10	84.08	84.06	84.05	84.03	84.01	83.99	83.95	83.91
23.00	DISCHG	14.75	14.40	14.06	13.74	13.43	13.14	12.86	12.59	12.33	12.09
23.00	ELEV	83.87	83.84	83.81	83.77	83.74	83.71	83.69	83.66	83.63	83.61
24.00	DISCHG	11.85	11.61	11.33	11.00	10.62	10.21	9.80	9.40	9.00	8.62
24.00	ELEV	83.58	83.56	83.53	83.50	83.46	83.42	83.38	83.34	83.30	83.26
25.00	DISCHG	8.26	7.91	7.57	7.25	6.94	6.64	6.36	6.09	5.54	4.89
25.00	ELEV	83.23	83.19	83.16	83.12	83.09	83.06	83.04	83.01	82.96	82.91
26.00	DISCHG	4.32	3.81	3.37	2.98	2.63	2.32	2.05	1.81	1.60	1.41
26.00	ELEV	82.86	82.82	82.78	82.75	82.72	82.69	82.67	82.65	82.63	82.62
27.00	DISCHG	1.25	1.10	.97	.86	.76	.67	.59	.52	.46	.41
27.00	ELEV	82.60	82.59	82.58	82.57	82.56	82.56	82.55	82.54	82.54	82.53
28.00	DISCHG	.36	.32	.28	.25	.22	.19	.17	.15	.13	.12
28.00	ELEV	82.53	82.53	82.52	82.52	82.52	82.52	82.51	82.51	82.51	82.51
29.00	DISCHG	.10	.09	.08	.07	.06	.06	.05	.04	.04	.03
29.00	ELEV	82.51	82.51	82.51	82.51	82.51	82.50	82.50	82.50	82.50	82.50

RUNOFF VOLUME ABOVE BASEFLOW = 1.82 WATERSHED INCHES, 891.03 CFS-HRS, 73.63 ACRE-FEET; BASEFLOW = .00 CFS

WARNING - NO HYDROGRAPH IN INPUT LOCATION 4 OR 3 IN ADDHYD OPERATION*
STRUCTURE 13

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.10	342.62	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .92 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.14	10.03	61.20	139.95	210.31
6.00	DISCHG	293.95	342.55	300.79	250.24	227.89	218.46	212.48	205.75	199.95	195.84
7.00	DISCHG	192.44	188.62	183.55	178.75	174.69	171.02	166.09	161.22	156.59	152.17
8.00	DISCHG	147.95	143.23	137.25	131.56	126.63	122.12	117.88	113.86	110.02	106.36
9.00	DISCHG	102.27	98.16	94.29	90.63	87.19	83.95	80.89	78.01	75.29	72.73
10.00	DISCHG	70.32	67.87	65.20	62.65	60.33	58.16	56.14	54.27	52.91	51.62
11.00	DISCHG	50.38	49.21	48.12	47.08	46.05	45.04	44.10	43.23	42.40	41.57
12.00	DISCHG	40.76	40.00	39.31	38.65	37.98	37.32	36.71	36.16	35.64	35.10
13.00	DISCHG	34.56	34.00	33.35	32.73	32.15	31.60	31.10	30.67	30.25	29.81
14.00	DISCHG	29.37	28.93	28.45	28.00	27.59	27.21	26.85	26.50	26.17	25.85
15.00	DISCHG	25.54	25.18	24.69	24.21	23.82	23.68	23.60	23.53	23.45	23.39
16.00	DISCHG	23.32	23.25	23.19	23.12	23.06	22.99	22.93	22.86	22.80	22.74
17.00	DISCHG	22.68	22.62	22.56	22.50	22.44	22.38	22.32	22.26	22.21	22.15
18.00	DISCHG	22.09	22.04	21.98	21.93	21.87	21.82	21.77	21.72	21.66	21.61
19.00	DISCHG	21.56	21.51	21.46	21.41	21.36	21.31	21.26	21.22	21.17	21.12
20.00	DISCHG	21.07	20.86	20.33	19.87	19.59	19.39	19.26	19.18	19.10	18.98
21.00	DISCHG	18.85	18.75	18.69	18.62	18.51	18.38	18.28	18.22	18.16	18.05
22.00	DISCHG	17.93	17.84	17.78	17.72	17.62	17.50	17.41	17.27	16.90	16.50
23.00	DISCHG	16.10	15.74	15.44	15.14	14.82	14.50	14.21	13.97	13.73	13.47
24.00	DISCHG	13.20	12.77	12.02	11.29	10.74	10.26	9.82	9.40	9.00	8.62

1

25.00	DISCHG	8.26	7.91	7.57	7.25	6.94	6.64	6.36	6.09	5.54	4.89
26.00	DISCHG	4.32	3.81	3.37	2.98	2.63	2.32	2.05	1.81	1.60	1.41
27.00	DISCHG	1.25	1.10	.97	.86	.76	.67	.59	.52	.46	.41
28.00	DISCHG	.36	.32	.28	.25	.22	.19	.17	.15	.13	.12
29.00	DISCHG	.10	.09	.08	.07	.06	.06	.05	.04	.04	.03

RUNOFF VOLUME ABOVE BASEFLOW = 1.82 WATERSHED INCHES, 1078.78 CFS-HRS, 89.15 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

COMPUTATIONS COMPLETED FOR PASS 2

XECUTIVE CONTROL OPERATION COMPUT RECORD ID

FROM STRUCTURE 1

TO STRUCTURE 10

STARTING TIME = .00 RAIN DEPTH = 2.70 RAIN DURATION= 1.00 RAIN TABLE NO.= 7 ANT. MOIST. COND= 2

ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

*** WARNING REACH 2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

PERATION ADDHYD STRUCTURE 11

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.09	883.00	(NULL)
9.92	22.18	(NULL)
12.86	16.94	(NULL)
13.83	14.77	(NULL)
19.87	11.46	(NULL)
23.85	5.84	(NULL)

IME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.04	16.93	117.12	322.79	570.01
6.00	DISCHG	789.12	881.32	732.16	489.24	311.15	214.53	161.11	124.22	98.31	82.45
7.00	DISCHG	73.72	67.78	60.23	53.03	48.42	46.08	44.89	44.28	43.99	43.86
8.00	DISCHG	43.76	42.16	36.98	30.94	26.78	24.52	23.34	22.73	22.41	22.24
9.00	DISCHG	22.15	22.12	22.10	22.11	22.12	22.13	22.14	22.16	22.17	22.18
10.00	DISCHG	22.18	21.80	20.55	19.08	18.00	17.33	16.97	16.87	16.88	16.84
11.00	DISCHG	16.73	16.67	16.73	16.82	16.82	16.74	16.70	16.76	16.85	16.86
12.00	DISCHG	16.78	16.73	16.79	16.89	16.89	16.81	16.77	16.83	16.92	16.93
13.00	DISCHG	16.84	16.63	16.16	15.62	15.20	14.89	14.72	14.71	14.77	14.75
14.00	DISCHG	14.66	14.51	14.24	13.96	13.78	13.68	13.63	13.61	13.60	13.59
15.00	DISCHG	13.59	13.42	12.89	12.26	11.83	11.60	11.48	11.42	11.39	11.37
16.00	DISCHG	11.36	11.36	11.36	11.36	11.37	11.37	11.37	11.37	11.38	11.38
17.00	DISCHG	11.38	11.39	11.39	11.39	11.39	11.40	11.40	11.40	11.40	11.41

18.00	DISCHG	11.41	11.41	11.41	11.42	11.42	11.42	11.43	11.43	11.43	11.43
19.00	DISCHG	11.44	11.44	11.44	11.44	11.45	11.45	11.45	11.45	11.46	11.46
20.00	DISCHG	11.45	11.05	9.74	8.21	7.08	6.39	6.02	5.90	5.90	5.85
21.00	DISCHG	5.74	5.67	5.72	5.81	5.81	5.72	5.66	5.72	5.81	5.81
22.00	DISCHG	5.72	5.67	5.72	5.81	5.81	5.73	5.67	5.72	5.81	5.81
23.00	DISCHG	5.73	5.67	5.73	5.82	5.82	5.73	5.67	5.73	5.82	5.82
24.00	DISCHG	5.71	5.21	3.85	2.30	1.23	.65	.34	.18	.10	.05
25.00	DISCHG	.02	.01	.00							

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 762.06 CFS-HRS, 62.98 ACRE-FEET; BASEFLOW = .00 CFS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.61	161.78	88.04

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.00	.99	6.98	16.01	19.78
5.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.58	83.10	84.00	84.76
6.00	DISCHG	43.18	86.25	122.18	144.72	156.50	160.95	161.78	161.19	159.46	156.28
6.00	ELEV	85.76	86.73	87.37	87.75	87.94	88.02	88.04	88.03	87.99	87.94
7.00	DISCHG	152.67	148.89	144.97	140.90	136.74	132.61	128.59	124.71	121.00	117.44
7.00	ELEV	87.88	87.81	87.75	87.68	87.61	87.54	87.48	87.41	87.35	87.29
8.00	DISCHG	114.04	110.76	107.48	104.09	100.62	96.47	92.31	88.34	84.57	81.00
8.00	ELEV	87.23	87.18	87.12	87.07	87.01	86.93	86.85	86.77	86.69	86.62
9.00	DISCHG	77.63	74.44	71.44	68.61	65.95	63.43	61.07	58.83	56.73	54.75
9.00	ELEV	86.55	86.49	86.43	86.37	86.32	86.27	86.22	86.18	86.13	86.10
10.00	DISCHG	52.88	51.11	49.55	48.28	47.01	45.76	44.54	43.36	42.23	41.15
10.00	ELEV	86.06	86.02	85.98	85.94	85.90	85.85	85.81	85.77	85.73	85.69
11.00	DISCHG	40.11	39.11	38.16	37.25	36.37	35.54	34.74	33.97	33.24	32.54
11.00	ELEV	85.66	85.62	85.59	85.56	85.53	85.50	85.47	85.45	85.42	85.40
12.00	DISCHG	31.87	31.22	30.61	30.02	29.46	28.92	28.40	27.91	27.44	26.99
12.00	ELEV	85.37	85.35	85.33	85.31	85.29	85.27	85.26	85.24	85.22	85.21
13.00	DISCHG	26.56	26.14	25.73	25.31	24.88	24.46	24.05	23.65	23.27	22.91
13.00	ELEV	85.19	85.18	85.16	85.15	85.13	85.12	85.11	85.09	85.08	85.07
14.00	DISCHG	22.56	22.22	21.89	21.55	21.23	20.98	20.92	20.85	20.79	20.73
14.00	ELEV	85.05	85.04	85.03	85.02	85.01	85.00	84.98	84.97	84.96	84.95
15.00	DISCHG	20.66	20.60	20.54	20.47	20.39	20.32	20.24	20.16	20.09	20.01
15.00	ELEV	84.93	84.92	84.91	84.89	84.88	84.86	84.85	84.83	84.82	84.80
16.00	DISCHG	19.93	19.86	19.79	19.71	19.64	19.57	19.49	19.42	19.35	19.28
16.00	ELEV	84.79	84.77	84.76	84.74	84.73	84.71	84.70	84.68	84.67	84.66
17.00	DISCHG	19.21	19.15	19.08	19.01	18.94	18.88	18.81	18.75	18.68	18.62
17.00	ELEV	84.64	84.63	84.62	84.60	84.59	84.58	84.56	84.55	84.54	84.52
18.00	DISCHG	18.56	18.49	18.43	18.37	18.31	18.25	18.19	18.13	18.07	18.01
18.00	ELEV	84.51	84.50	84.49	84.47	84.46	84.45	84.44	84.43	84.41	84.40
19.00	DISCHG	17.96	17.90	17.84	17.79	17.73	17.68	17.62	17.57	17.51	17.46
19.00	ELEV	84.39	84.38	84.37	84.36	84.35	84.34	84.32	84.31	84.30	84.29
20.00	DISCHG	17.41	17.35	17.29	17.22	17.14	17.05	16.95	16.85	16.76	16.66

20.00	ELEV	84.28	84.27	84.26	84.24	84.23	84.21	84.19	84.17	84.15	84.13
21.00	DISCHG	16.57	16.47	16.38	16.29	16.19	16.10	16.01	15.62	15.20	14.80
21.00	ELEV	84.11	84.09	84.08	84.06	84.04	84.02	84.00	83.96	83.92	83.88
22.00	DISCHG	14.41	14.04	13.69	13.35	13.03	12.72	12.42	12.13	11.86	11.61
22.00	ELEV	83.84	83.80	83.77	83.73	83.70	83.67	83.64	83.61	83.59	83.56
23.00	DISCHG	11.36	11.12	10.89	10.67	10.46	10.26	10.07	9.88	9.71	9.54
23.00	ELEV	83.54	83.51	83.49	83.47	83.45	83.43	83.41	83.39	83.37	83.35
24.00	DISCHG	9.38	9.21	9.01	8.76	8.46	8.14	7.82	7.50	7.18	6.88
24.00	ELEV	83.34	83.32	83.30	83.28	83.25	83.21	83.18	83.15	83.12	83.09
25.00	DISCHG	6.59	6.31	6.04	5.41	4.78	4.22	3.73	3.29	2.91	2.57
25.00	ELEV	83.06	83.03	83.00	82.95	82.90	82.85	82.81	82.77	82.74	82.71
26.00	DISCHG	2.27	2.00	1.77	1.56	1.38	1.22	1.08	.95	.84	.74
26.00	ELEV	82.69	82.67	82.65	82.63	82.62	82.60	82.59	82.58	82.57	82.56
27.00	DISCHG	.66	.58	.51	.45	.40	.35	.31	.27	.24	.21
27.00	ELEV	82.55	82.55	82.54	82.54	82.53	82.53	82.53	82.52	82.52	82.52
28.00	DISCHG	.19	.17	.15	.13	.12	.10	.09	.08	.07	.06
28.00	ELEV	82.52	82.51	82.51	82.51	82.51	82.51	82.51	82.51	82.51	82.51
29.00	DISCHG	.05	.05	.04	.04	.03	.03	.03	.02	.02	.02
29.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 761.68 CFS-HRS, 62.94 ACRE-FEET; BASEFLOW = .00 CFS

WARNING - NO HYDROGRAPH IN INPUT LOCATION 4 OR 3 IN ADDHYD OPERATION
STRUCTURE 13

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.11	285.75	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .92 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.03	6.51	46.86	114.79	175.22
6.00	DISCHG	242.04	285.54	253.20	214.11	198.81	191.82	186.59	180.69	175.42	170.76
7.00	DISCHG	166.61	162.09	156.46	151.05	146.30	141.94	137.83	133.92	130.20	126.65
8.00	DISCHG	123.25	119.42	114.46	109.74	105.67	101.28	97.02	93.00	89.21	85.64
9.00	DISCHG	82.27	79.09	76.09	73.27	70.61	68.10	65.73	63.50	61.40	59.42
10.00	DISCHG	57.56	55.65	53.68	52.09	50.65	49.31	48.06	46.89	45.78	44.69
11.00	DISCHG	43.62	42.62	41.69	40.80	39.92	39.05	38.25	37.51	36.80	36.09
12.00	DISCHG	35.39	34.74	34.15	33.59	33.02	32.45	31.93	31.46	31.01	30.55
13.00	DISCHG	30.09	29.61	29.05	28.52	28.02	27.55	27.12	26.75	26.39	26.01
14.00	DISCHG	25.63	25.25	24.84	24.45	24.10	23.85	23.78	23.71	23.65	23.59
15.00	DISCHG	23.52	23.40	23.17	22.96	22.82	22.72	22.64	22.56	22.48	22.40
16.00	DISCHG	22.33	22.25	22.18	22.10	22.03	21.96	21.89	21.82	21.75	21.68
17.00	DISCHG	21.61	21.54	21.48	21.41	21.34	21.28	21.21	21.15	21.08	21.02
18.00	DISCHG	20.96	20.90	20.83	20.77	20.71	20.65	20.59	20.54	20.48	20.42
19.00	DISCHG	20.36	20.31	20.25	20.20	20.14	20.09	20.03	19.98	19.93	19.87
20.00	DISCHG	19.82	19.62	19.14	18.73	18.47	18.28	18.15	18.07	17.99	17.88
21.00	DISCHG	17.76	17.66	17.59	17.52	17.41	17.29	17.20	16.83	16.43	16.02
22.00	DISCHG	15.60	15.23	14.90	14.58	14.25	13.91	13.61	13.35	13.10	12.83
23.00	DISCHG	12.55	12.30	12.10	11.91	11.68	11.46	11.26	11.10	10.95	10.77
24.00	DISCHG	10.57	10.24	9.62	9.02	8.57	8.19	7.84	7.50	7.19	6.88

1

25.00	DISCHG	6.59	6.31	6.04	5.41	4.78	4.22	3.73	3.29	2.91	2.57
26.00	DISCHG	2.27	2.00	1.77	1.56	1.38	1.22	1.08	.95	.84	.74
27.00	DISCHG	.66	.58	.51	.45	.40	.35	.31	.27	.24	.21
28.00	DISCHG	.19	.17	.15	.13	.12	.10	.09	.08	.07	.06
29.00	DISCHG	.05	.05	.04	.04	.03	.03	.03	.02	.02	.02

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 922.15 CFS-HRS, 76.21 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

COMPUTATIONS COMPLETED FOR PASS 3

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID

1

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH

APPENDIX B

Water Quality Analysis

DETERMINATION OF THE OPTIMAL DETENTION POND SIZE FOR THE CITY OF COLORADO SPRINGS, COLORADO

BY

JAMES C.Y. GUO, PH.D., P.E.

SUBMITTED TO

KIOWA ENGINEERING CORPORATION
DENVER, COLORADO

DECEMBER 27, 1989

CONTENTS

DETERMINATION OF
THE OPTIMAL DETENTION POND SIZE
FOR THE CITY OF COLORADO SPRINGS, COLORADO

Background

Detention pond is an effective tool for runoff water quality and quantity control. The storage of a detention pond reduces peak runoff rate. Therefore, the larger the pond is, the more attenuation on peak flow will result. As a common practice, when designing a flood control detention pond, pond size is determined by a design flood with a specified return period such as a 100 year flood. However, considering water quality control, runoff volume treatment on daily events is more important than peak flow rate attenuation on less frequent events. Using the concept of design flood may result in a hugh storage which may be excessive to daily runoffs.

To determine the proper size of a water quality control pond requires to understand local daily rainfall or runoff characteristics including the statistic spectrum of local rainfall and runoff patterns, precipitation distribution, average time interval between storms, and then a risk cost analysis can be performed. Since rainfall pattern varies from one place to another, in this study, the hourly precipitation data collected at the Station 051778 in the City of Colorado Springs by the National Weather Service was used to apply the methodology developed by the Denver Urban Drainage and Flood Control District to the determination of cost effective water quality pond size. It has found that drainage basin runoff coefficient, pond emptying time, and local mean precipitation are important factors.

Work Description

The computer model, PONDRISK, developed by the Department of Civil Engineering, University of Colorado at Denver was employed to analyze the hourly rainfall data collected in the City of Colorado Springs from 1974 to 1989. The model first computes rainfall statistics and then assesses the treatment capacities for a range of pond sizes. The optimal pond size is determined by its performance effectiveness among the pond sizes studied for each hydrologic cases. In the portion of rainfall statistics, the continuous hourly precipitation record is separated into individual storms using six, 12, 24 and 48 hours as separation time intervals. For instance, when using 12 hours as a separation time, any adjacent hourly precipitations occurred with a time interval less than 12 hours are accounted into one single storm. The computer model accumulates rainfall depth and duration for each storm and then computes statistics for average rainfall depth, duration, intensity and dry hours (time period between two adjacent storms.) among storms identified. The second portion of this study was to convert the point precipitations into runoff volumes using runoff coefficient, C. Namely, Runoff Volume = C *(Precipitation - Infiltration Loss)

The infiltration loss was determined to be 0.1 inch.

In the computation, it was assured that before the beginning of each storm, the pond is empty; in other words, the pond emptying hour is equal to the storm separation time. The corresponding average release rate from the pond is determined by the ratio of pond volume to pond emptying time. Whenever, the pond becomes full, the difference between the incoming runoff and the released runoff is considered untreated and overflown. For a selected pond size, the program computes the runoff capture rate which is defined as the ratio of treated runoff volume to the total runoff volume throughout the entire precipitation record.

Results

In this study, there were three runoff coefficients, 0.2, 0.5 and 0.9, used to determine the optimal detention pond sizes expressed in inches/square foot. The detailed explanation of the pond performance optimization methodology can be found in the Appendix A. Results of this study, as tabulated, the statistics of rainfall characteristics vary with respect to the storm separation time interval. The optimal runoff capture rates for different runoff coefficients are around 85% which means that 85% of runoff volume would be treated if the optimal pond size was used.

RAIN DURATION AND DEPTH STATISTICS FOR COLORADO SPRINGS

STORM SEPARATION TIME INTERVAL IN HOURS	DURATION MEAN HOURS	DURATION S.D. HOURS	DURATION SKEWNESS	PRECIPITATION MEAN INCH	PRECIPITATION S.D. INCH	PRECIPITATION SKEWNESS
6.000	5.400	6.860	2.760	0.450	0.470	3.180
12.000	7.530	9.820	2.340	0.460	0.480	3.000
24.000	16.260	20.380	2.220	0.572	0.617	2.828
48.000	32.790	44.420	2.570	0.684	0.751	2.600

RAIN INTENSITY AND DRY HOURS STATISTICS FOR COLORADO SPRINGS

STORM SEPARATION TIME INTERVAL IN HOURS	INTENSITY MEAN IN/HR	INTENSITY S.D. IN/HR	INTENSITY SKEWNESS	TIME INTERVAL MEAN HOURS	TIME INTERVAL S.D. HOURS	TIME INTERVAL SKEWNESS
6.000	1.850	4.480	3.990	92.600	116.900	2.640
12.000	0.078	0.154	11.490	105.900	120.500	2.510
24.000	0.045	0.077	4.480	136.600	126.200	2.320
48.000	0.026	0.047	6.044	168.900	129.200	2.250

NOTE: RAIN SEPARATION TIME= THE MINIMUM TIME INTERVAL BETWEEN TWO ADJACENT RAIN STORMS ON A CONTINEOUS RECORD.

TIME INTERVAL= DRY HOURS BETWEEN ADJACENT RAINSTORMS.

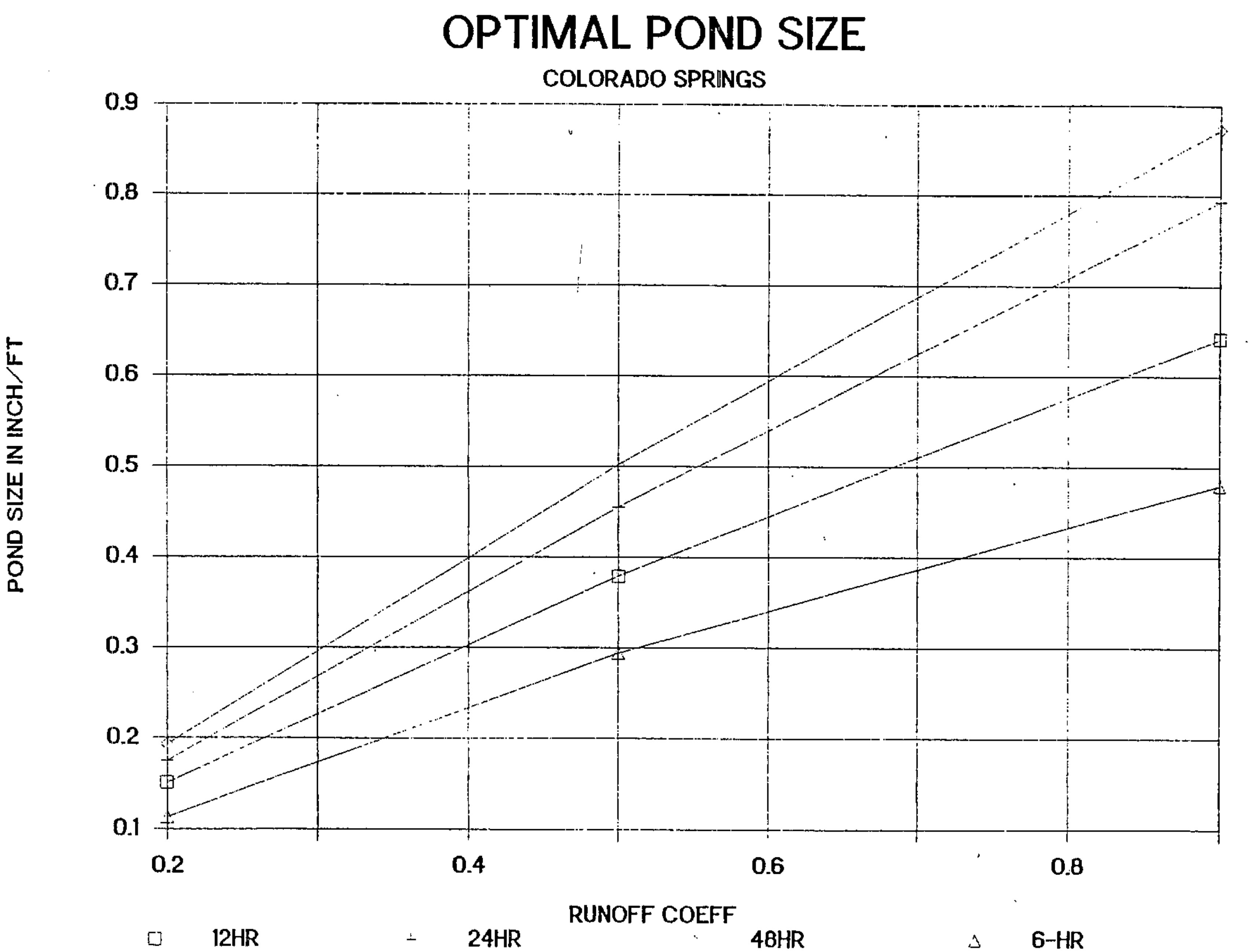
OPTIMAL POND SIZE AND RUNOFF CAPTURE RATE
FOR COLORADO SPRINGS

POND EMPTYING TIME HOURS	C=0.2 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %	C=0.5 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %	C=0.9 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %
6.000	0.257	82.79	0.652	83.57	1.060	82.39
12.000	0.325	86.10	0.816	86.19	1.380	84.97
24.000	0.305	85.36	0.795	86.30	1.390	85.60
48.000	0.277	81.67	0.718	82.84	1.250	87.27

NOTE: C= RUNOFF COEFFICIENT.
CAPTURE RATE= RUNOFF TREATED VOLUME/TOTAL RUNOFF VOLUME

OPTIMAL POND SIZE IN INCHES/SQ FOOT

RUNOFF COEFF	POND EMPTYING TIME IN HOURS 6.000	12.000	24.000	48.000
0.200	0.113	0.151	0.175	0.193
0.500	0.294	0.379	0.455	0.502
0.900	0.480	0.642	0.794	0.873


OPTIMAL POND SIZE
COLORADO SPRINGS
POND SIZE IN INCH/FT
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.2
0.4
0.6
0.8
RUNOFF COEFF
12HR
24HR
48HR
6-HR

Design Example

A detention pond, located in the City of Colorado Springs, is designed to have emptying hours of 24 hours for a drainage basin of 100 acres and runoff coefficient of 0.9. According to the results of this study, using 24 hours as storm separation time, the mean precipitation is 0.572 inch with an average duration of 20.4 hours and intensity of 0.045 inch/hour. The most effective pond size to the mean precipitation is 1.390 which is equivalent to 0.794 inch/square foot or 6.62 acre-foot, 100 acre * (0.794/12) foot, for this drainage basin. The average release rate from this pond is

Pond Volume/Emptying Time = 6.62 acre-ft/24 hour=3.34 cfs

According to the computed statistics, this pond shall have a runoff volume capture rate of 85.60%.

Summary

This study has been successfully performed for the Colorado Springs areas using the methodology developed by the University of Colorado at Denver and the Denver Urban Drainage and Flood Control District. The City of Colorado Springs is one of major metropolitan areas in the State of Colorado. Results from this study shall help engineers to further understand the local rainfall and runoff patterns and to optimize the use of detention pond facility. Living in this fast paced modern society, development of new understanding of our natural environment shall definitely help engineers make more proper decisions, especially for civil engineers who ought to work with the natural environment.

APPENDIX A.

TECHNICAL PAPER:

OPTIMIZATION OF STORMWATER QUALITY CAPTURE VOLUME

OPTIMIZATION OF STORMWATER QUALITY CAPTURE VOLUME

Ben Urbonas, P.E.[1], James C.Y. Guo, Ph.D., P.E.[2] and L. Scott Tucker, P.E.[3], all M.ASCE

ABSTRACT

There is a need for rational, scientifically based, methods to size urban stormwater runoff facilities for the purpose of water quality enhancement. This paper describes a procedure that utilizes hydrologic principles for optimizing the capture volume. This procedure takes recorded precipitation data and processes it using a quasi-continuous simulation method to determine the number of storm events and total of storm runoff volume being captured within the period being studied. The application of this procedure is illustrated using a 40 year hourly rainfall record at the Denver Raingauge.

INTRODUCTION

The practice of urban stormwater management has until recently focused primarily on quantity issues such as drainage and flood control. Flooding of streets, streams, and rivers has been the main concern. Local governments have constructed thousands of miles of curb, gutter, road side ditches, and storm sewers to convey stormwaters as quickly and efficiently as possible to the nearest stream. This practice along with the increase in impervious surfaces accompanied by urbanization increases the volume and peak flow of runoff for any given rainfall event.

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Because development results in greater surface runoff rates when compared with undeveloped land, it is common for local governments to attempt mitigating these runoff increases by requiring developers to construct on-site stormwater detention facilities. The concept is to hold back runoff for a short period from each development in small ponds, on parking lots, or wherever space can be found at the site to temporarily store the water. However, on-site detention criteria varies considerably from community to community, the impact of muliples of on-site facilities is uncertain, and long term maintenance is is not a sure thing when it comes to these randomly placed on-site detention facilities.

The alternative to developer constructed on-site detention facilities is regional detention sites. Most people agree that regoinal facilities are more cost efficient and are much more likely to be properly maintained because they would be owned and operated by a public entity. While preferred, it is difficult to fund regional detention. As a result, individual on-site detention requirements are still commonly enforced and the use of on-site detention is the most common approach.

Urban stormwater management, however, is changing quite rapidly from a focus on quantity to a focus on quantity and quality. Two basic issues have and are exerting considerable influence for this change. The first is a fundamental heightening of environmental awareness and concern by the public. There seems to be public support for environmental programs. Stormwater quality in general is probably not a serious problem in relation to concerns such as global warming, Love Canal, sludge disposal, or the Alaska oil spill, and except in some specific situations the impact of urban stormwater on receiving water bodies is not documented or understood. Nevertheless, urban stormwater along with non-point runoff from non-urban sources contribute pollutants to the receiving waters and efforts to do something about it are slowly picking up support and momentum.

The second factor causing a shift toward urban stormwater quality is the Water Quality Act of 1987 (WQA), which amended the Federal Water Pollution Control Act. The WQA of 1987 is a reflection of the public's support for pollution control, and such legislation gives focus and direction to general issues. The WQA requires the Environmental Protection Agency (EPA) to develop a National Pollutant Discharge Elimination System (NPDES) permit program for separate urban stormwater discharges. How the 1987 WQA may impact the citizens, communities, local governments, industry, consultants and the water quality across the United States is yet to be seen. Nevertheless, local governments and industries throughout the United States have a mandate from Congress to control pollutants in urban runoff to the "maximum extent practicable" (MEP). This hopefully means that Congress expects solutions to be practical, pragmatic, and economical.

In order to be practical and effective it is important that technologies for dealing with urban stormwater runoff be available that get the job done. Several simple technologies are emerging that willl be able to be used to remove pollutants from urban stormwater (Urbonas and Roesner, 1986), (Roesner, Urbonas and Sonnen, 1989). These include detention and retention basins, infiltration and percolation at the source of runoff, wetlands, sand filters, and combinations of these techniques. It is important to realize that the same design criteria used to design detention ponds to reduce peak flows cannot be used to design detention and retention basins for stormwater quality purposes.

It is clear from reading the 1986 and 1989 references cited above that the size of runoff event to be captured and treated is a critical factor in the design of stormwater quality detention and retention basins. For example, if the design runoff event is too small, the effectiveness will be reduced because too many storms will exceed the capacity of the facility. Or if the design event is too large, the smaller runoff events will tend to empty faster than desired for adequate settling of pollutants. Thus the larger basins may not provide the needed retention time for the predominant number of smaller events.

A balance between the storage size and water quality treatment effectiveness is needed. Grizzard et. al. (1986) reported results from a field study of basins with extended detention times in the Washington, D. C. area. Based on their observations they suggested that these basins provide good levels of treatment when they are sized to have an average drain time of 24 hours, which equates to a 40 hour drain time for a brim-full basin.

EPA (1986) suggested an analytical methodology for estimating the removal efficiencies of sediments in ponds that have surcharge storage above a permanent pool. Subsequently, Schueler (1987) suggested that the surcharge volume be equivalent to the average runoff event volume. Analysis by the authors in Denver using the EPA analysis technique indicates that wet ponds can be very effective in removing settleable pollutants (i.e., annual TSS removal rates in excess of 80 percent). However, this analysis was limited to ponds that have brim-full surcharge volume equal to one-half inch of runoff from the tributary impervious surfaces, with this volume being drained in 12 hours. Never-the-less, there remains little rationale for the sizing of the capture volume that results in reasonable pollutant load removal while providing reasonably sized cost effective facilities.

Until recently, the primary interest was in drainage and flood control. As a result, the focus was on the larger storm events such as the 2- to 100-year floods. Although drainage and flood control engineers traditionally consider the 2-year event as small, at least in the Denver area it is larger than 95 percent of all the runoff events that typically occur in an urban watershed. Also, through experience we have learned that a detention facility designed to control a 100-year, or even a 2-year flood has little, if any, effect on water quality. Thus, focusing on the traditional drainage design storms is not practical or desirable when considering stormwater quality.

This paper will discuss a method that can be used to find a point of diminishing returns for the sizing of water quality detention facilities. It utilizes rainstorm records as its base instead of synthesized design storms. An example based on the National Weather Service long term precipitation record in Denver is used to illustrate the suggested methodology.

MAXIMIZATION OF STORMWATER RUNOFF CAPTURE VOLUME

Rain Point Diagram.

In 1976 von den Herik (1976) suggested in Holland a rainfall data-based method for estimating runoff volumes. This method is based on long term record of total rainfall and duration of storms. Subsequently Pecher (1978 & 1979) suggested modifications to von den Herik's work to use in the sizing of detention facilities through the use of a Rain Point Diagram (RPD). The authors modified the original method to transform the RPD to a Runoff Volume Point Diagram (RVPD) by multiplying the individual rainstorm depths on the RPD by the runoff coefficient of the tributary watershed.

The PVPD method approximates continuous modelling without setting up a continuous model. The method requires combining individual recorded hourly or 15 minute rainfall increments in a given period of record into separate storm depth totals. Separate storms are identified by a period of time when no rainfall occurs. Very small storms that are not likely to produce runoff can be then be purged from the record. Rainfall storm totals were then converted to runoff depths (i.e., volumes) by multiplying the rainfall depth by the watersheds runoff coefficient (C).

Because the RVPD procedure has not taken into account the effects of several successive rainstorms, it would have a tendency to underestimate the capture effectivenes of detention facilites that have very low release rates. This is because the volume captured during one storm may not be fully drained before the next storm occurs. The RVPD assumes an empty basin for each event.

The procedures used to develop the RVPD method and a case study using the Denver rain gage data will be discussed subsequently. However, to illustrate the use of the RVPD a plot of 63 storms is shown in Figure 1, where the individual storm runoff depth in inches is plotted agaist storm duration. A runoff capture envelope is also plotted on this same figure. This captured storage envelope is bases on the "brim-full" volume of the detention facility and its emptying time. In Figure 1 the runoff capture envelope is based on a detention basin that has a brim-full capacity of 0.3 watershed inches which can be emtied throughthe outlet in 12 hours (sometimes called drawdown time).

All the points above the capture volume envelope line represent individual storms that have sufficient runoff to exceed the available storage volume (i.e., brim-full volume) of the detention facility. Obviously, plotting and counting all points for a long record of rainstorms is a very tedious job. As a result, the authors developed a software package to perform this task.

While this procedure is a simplification of a continuous modelling process, the results should be sufficiently accurate for general planning purposes. This conclusion is supported by the fact that the true accuracy of hydrologic calculations is significantly less than the precision implied by stormwater hydrology models (ASCE, 1984) that are commonly used.

To compensate for storms that may be closely spaced, the authors used a storm separation interval equal to one-half of the emptying time of the brim-full volume. In other words, a storm was defined as separate from a previous storm when this separation condition was satisfied between the end of the last recorded rainfall increment and the beginning of the next one.

The sensitivity of the storm separation period was tested using a storm separation period equal to the brim-full volume emptying time. Virtually no difference was found in the capture volume effectiveness between the separation set at brim-full and one-half of the brim-full emptying time. Such sensitivity tests are suggested whenever other precipitation data are used for this procedure.

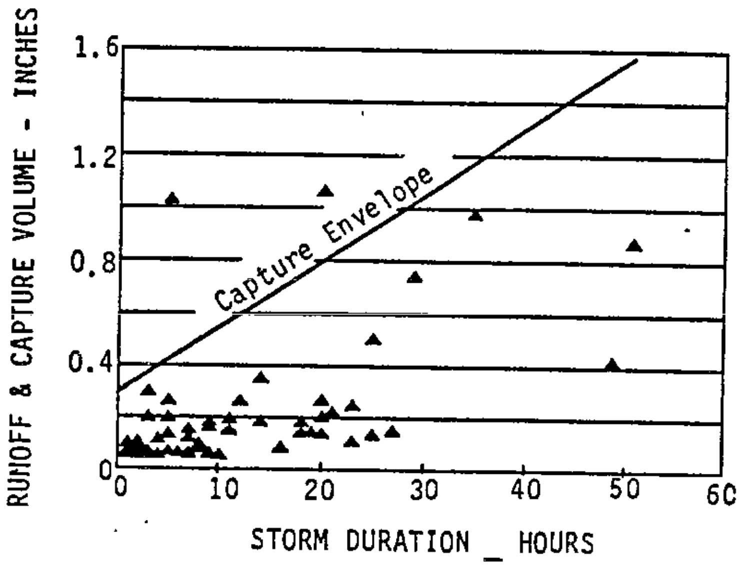

Figure 1. Runoff Volume Point Diagram and Capture Volume Envelope. (1-inch = 24.5 millimeters)

Storage Volume Optimization Procedure

After the total rainfall record is separated into individual storm events, the runoff volume for each storm can be estimated using:

$$V_r = C\ P_t \tag{1}$$

in which, V_r = total runoff volume for a storm, in watershed inches or meters
C = runoff coefficient
P_t = total precipitation over the watershed for the storm in inches of meters.

For a given detention pond or basin that has a brim-full volume V_r with an emptying time T_e, its average release rate, q, is

$$q = V_r\ /\ T_e \tag{2}$$

The runoff volume capture capacity, V_m, of the detention basin for any storm may be estimated using:

$$V_m = V_r + q\ T_d \tag{3}$$

in which, T_d = storm duration. The function (q T_d) represents the storage beyond the brim-full volume that becomes available during the strom as the result of releases from the basin during the storm's duration.

The actual runoff volume captured and processed for quality improvement through the basin for a given storm is equal to V_r, namely storm runoff volume, when V_r is less than V_m; otherwise it is equal to V_m with the excess runoff volume assumed to overflow without any treatment. Adding the volumes captured for all the storms occurring during the record period gives the total volume captured and treated, V_t, within the period. Thus, the volume capture ratio for the period of rainfall record is defined as,

$$R_v = V_t \;/\; V_{tr} \tag{4}$$

in which, R_v = volume capture ratio for the record period
V_t = total volume captured during the period
V_{tr}= total runoff volume during the same period.

Similarly, the runoff event capture ratio is defined:

$$R_e = N_f \;/\; N \tag{5}$$

in which, R_e = runoff event capture ratio for the period
N_f = number of runoff events that are less than or equal to V_m in runoff volume
N = total number of runoff events.

For the total set of runoff events in the record there is a detention volume that will capture all of the runoff events of record. For practical reasons this maximum pond volume, P_m, was defined to be equal to the 99.9 percent probability runoff event volume for the record period. For the Denver raingage period of record studied (1944-1984) this is equal to to the runoff from 3.04 inches (77.2 mm) of precipitation, or 6.9 times the precipitation of an average runoff producing storm for this period of record. This 99.9 percentile value, namely P_m, was then used to normalize all pond sizes being tested using the following equation:

$$P_r = P \;/\; P_m \tag{6}$$

in which, P_r = relative pond size normalized to P_m
P = pond size being tested
P_m = maximum runoff volume (i.e., 99.9% probability).

The maximization procedure incrementally increases the relative (i.e., normalized) pond size and calculates the runoff volume and event capture ratios (i.e., R_v and R_e) using the RVPD method. Figure 2 illustrates an example of the results of such an analysis using the precipitation record at the Denver gauge between 1944 and 1984. In this example the capture volume was maximized using storms defined by a 6-hour period of separation, 12-hour emptying time for the brim-full basin, and a runoff coefficient C = 0.5 for the watershed.

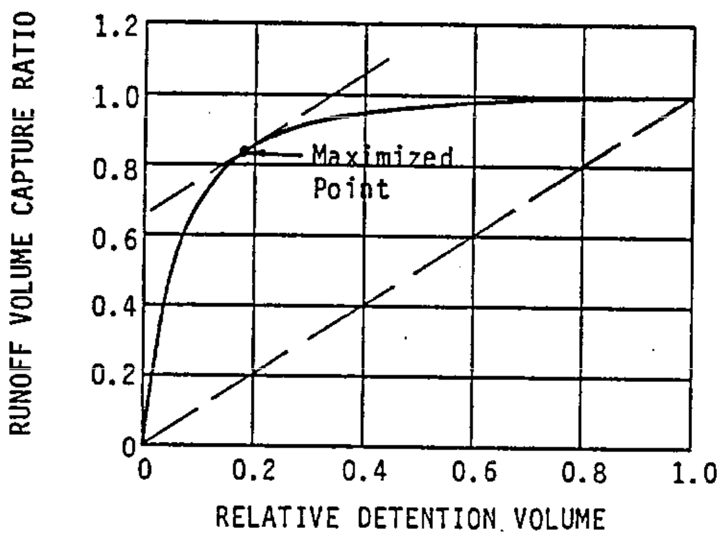

Figure 2. Maximizing Capture Volume.

The maximized pond size occurs where the 1:1 slope is tangent to the runoff capture rate function. Before this point is reached the capture rate increases faster than the relative capure volume size. After this point is reached the increases in the capture rate become less than than corresponding increases in relative capture volume size. In other words, when the point of maximization is passed, diminishing returns are experienced if the capture volume is increased any further. In Figure 2 example, the maximized point occurs when the relative capture volume is equal to 0.18. At this point we capture in total and release slowly approximately 82 percent of the entire runoff depth that has occured during the 40 year study period. This relative capture volume is then converted to actual volume using Equation 6, namely,

$$\begin{aligned} P &= P_r\ P_m \\ &= (0.18)\ (0.5\ 3.04) \\ &= 0.27 \text{ watershed inches (6.86 millimeters)} \end{aligned}$$

in which, 0.5 is the watershed's runoff coefficient and P_m = 3.04 inches (77.2 mm), namely the depth of rain during the 99.9 percent probability storm.

CASE STUDY USING DENVER RAIN GAUGE DATA

Developing Regional Detention Sizing Guidelines.

The authors investigated the Denver Gauge precipitation data using several storm separation periods, which has been defined as the time between the end of one storm and the beginning of the next. A statistical summary of rainfall characteristics for all storms that exceeded a total of 0.1 inch (2.54 mm) is given in Table 1. A 0.1 inch (2.54 mm) "filter" was used to eliminate from the record the very small storms, of which most are likely not to produce runoff. The urban rainfall and runoff data in the Denver area indicate that approximately 0.08 to 0.15 inches (2.03 to 3.81 mm) of rainfall depth is the point of incipient runoff.

TABLE 1. DENVER RAIN GAUGE HOURLY DATA SUMMARY 1944-1984
STORMS LARGER THAN 0.1 INCHES (2.54 mm) IN DEPTH

SEPARATION BASIS FOR NEW STORM (HOURS)	NUMBER OF STORMS	AVERAGE DEPTH (INCHES)	AVERAGE STORM DURATION (HOURS)	AVERAGE TIME BETWEEN STORMS (HOURS)	NUMBER OF STORMS SMALLER THAN AV.	PERCENT OF STORMS SMALLER THAN AV.
1	1131	0.39*	7	267	802	70.9
3	1091	0.42*	9	275	782	71.7
6	1084	0.44*	11	275	766	70.7
12	1056	0.46*	14	280	748	70.8
24	983	0.51*	23	293	686	69.8
48	876	0.58*	43	310	613	70.0

* Multiply values by 25.4 to convert to millimeters.

A skewed statistical distribution exists with more than two-thirds of the storms having less precipitation than the 40 year average storm depth. Appearently in the Denver area the average runoff producing rain storm depth is a relatively large event.

The distribution of all (i.e., unfiltered) storms vs. total storm precipitation depth when individual storms are defined by a six hours separation period is shown in Figure 3. Note that sixty percent of the precipitation events produced 0.1-inches (2.54 mm) or less of rainfall depth. Over ninety percent of all recorded storms had 0.5-inches or less of rainfall depth. This indicates that the focus, at least in the Denver area should be on the smaller, more frequently occurring storms whenever water quality is being considered.

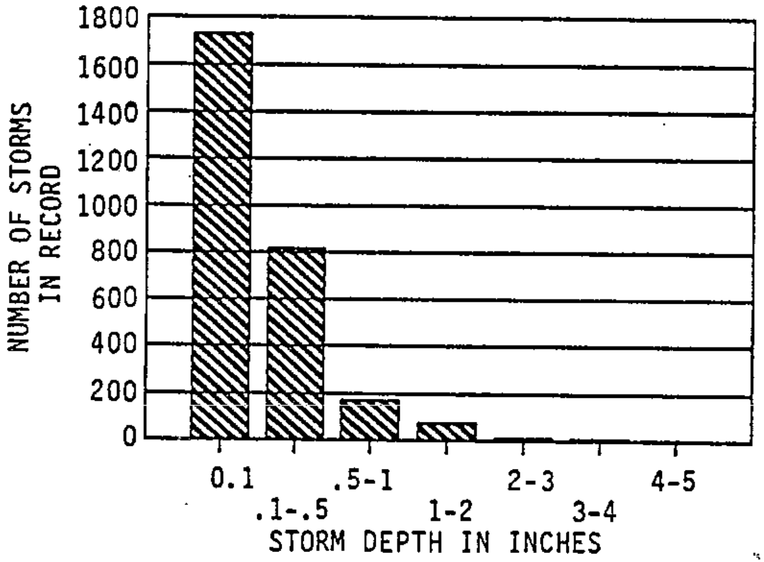

Figure 3. Number of Storms in Denver vs. Storm depth. (One inch = 25.4 millimeters)

Once the precipitation and runoff probabilities were understood, an attempt was made to find a simple yet reasonably accurate relationships for approximating the maximized capture volume of water quality detention basins. As described earlier, the maximized point was defined when additional storage resulted in rapidly diminishing numbers of storms or in the storm runoff volume being totally captured. The final result of this analysis is illustrated in Figure 4, which relates the maximized capture volume to the watershed's runoff coefficient. Separate relationships are shown for the brim-full storage volume emptying time of 12-, 24- and 40-hours.

The captured volume ratio for this relationship exceeds 80 percent and the storm event capture ratio exceeds 86 percent. The storm event capture ratio is of greater importance to the receiving waters because it is the frequency of the shock loads that has the greatest negative effect on the aquatic life in the receiving streams. On the other hand, examination of the precipitation records (i.e., Figure 3) indicates that the volume capture ratio is influenced significantly by the very few very large storms. During these very large runoff events catastrophic flooding is likely to be of primary concern and stromwater quality. It should also be noted that even in these larger events some degree of capture and treatment occurs, although at somewhat reduced efficiency since the detention capacity is exceeded.

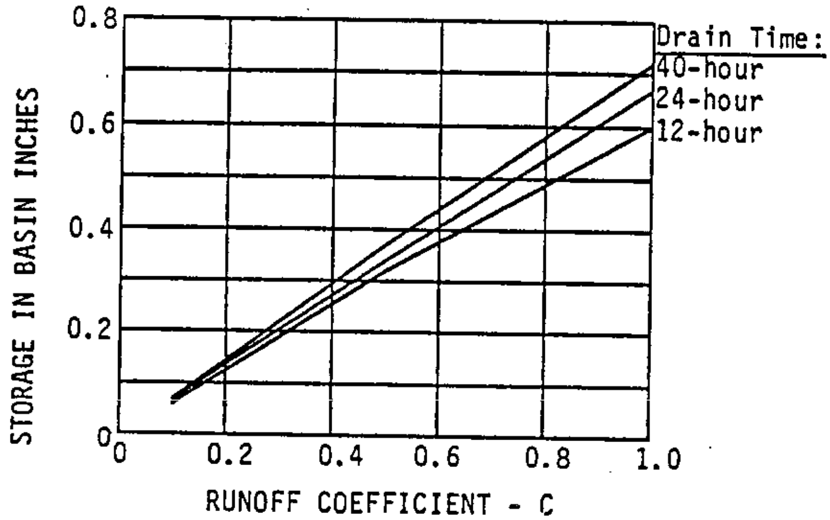

Figure 4. Maximized Capture Volume for Water Quality, Denver Rain Gauge 1944-84 Period. (One inch = 25.4 millimeters)

SENSITIVITY OF PROCEDURE

Capture Volume

Understanding the sensitivity of the event capture ratios to a change in the design capture volume (i.e.. brim-full volume) helps to rationally size water quality facilities. To help define this sensitivity a watershed having a runoff coefficient of C = 1.0 and a storage basin having the maximized volume draining in 12 hours was analyzed. The design capture volume of the basin was increased and decreased in increments and the results were normalized around the maximized volume point. Figure 5 illustrates the findings for this particular case. Although the results varied somewhat between similar tests, the trend was virtually the same for each test that were made using the Denver rain rauge data.

At the ratio of 1.0 on the abcissa, the capture volume has to be almost doubled to capture an additional 10 persent of the runoff events in the fecord. On the other hand, reducing the capture volume by 25 percent results in the reduction of only eight percent in the runoff events that are not captured in total. It needs to be understood that failure to capture a runoff event in total does not mean that the facility will not remove suspended solids. Suspended solids will be removed, but at a somewhat diminished efficiency. The sensitivity of the facility's solids capture efficiency will be discussed next.

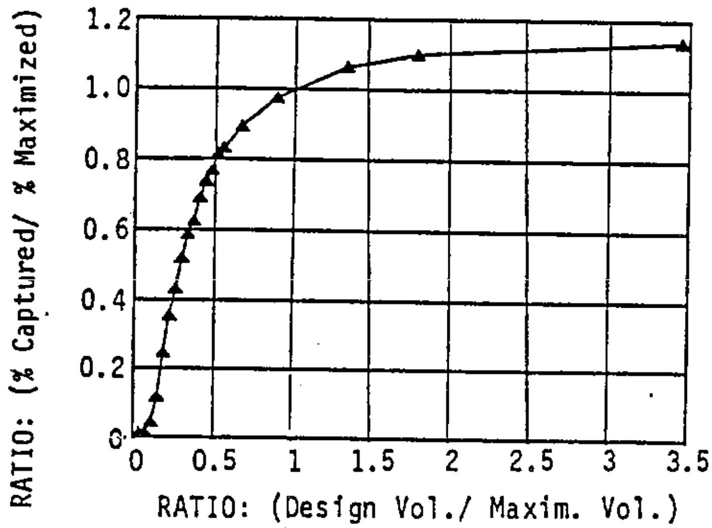

Figure 5. Sensitivity of Capture Volume Size.

Removal of Suspended Sediments

An attempt was made to test the sensitivity of the surcharge detention volume above the permanent pool level on the annual removal rates of total suspended solids in stormwater. For lack of local data on sediment settling velocities, the data given by EPA (1986) was use for several capture volume sizes. Estimates were made of the dynamic removals during the runoff events and the quiescent removals in the pond between storms. When using a surcharge capture volume equal to 70 percent of the maximized volume, the annual removal of TSS by the pond is estimated at 86 percent. This compares to an estimated rate of 88 percent annual removal of TSS when using the maximized capture volume, and only a 90 percent removal rate when using twice the maximized volume.

It appears from the preliminary estimates made using the Denver rain gauge records that it is posible to reduce the capture volume for a wet detention pond and see virtually no effect on the annual removal efficiency of the facility. Figure 5 suggests that the the design volume could be set 25 to 35 percent less than the maximized capture volume. Obviously this suggestion needs more testing. If verified, savings in the construction of water quality enhancement facilities should be possible. Continuous modelling and field testing are suggested as posible methods to test this premise.

Extending the Design Procedure

It is clear from the sensitivity analysis that the capture volume may be reduced somewhat from the maximized point without a significant loss in effectiveness. The designer or the water quality administrator may want to target the capture volume size to serve a runoff event of a desired recurrence probability such as the 85%, 80% or lesser runoff event. Figure 6 illustrates the type of relationships that can be developed if such a goal is desired. Obviously economics and practicality of the capture volume size should be considered when selecting the stormwater quality sizing criteria.

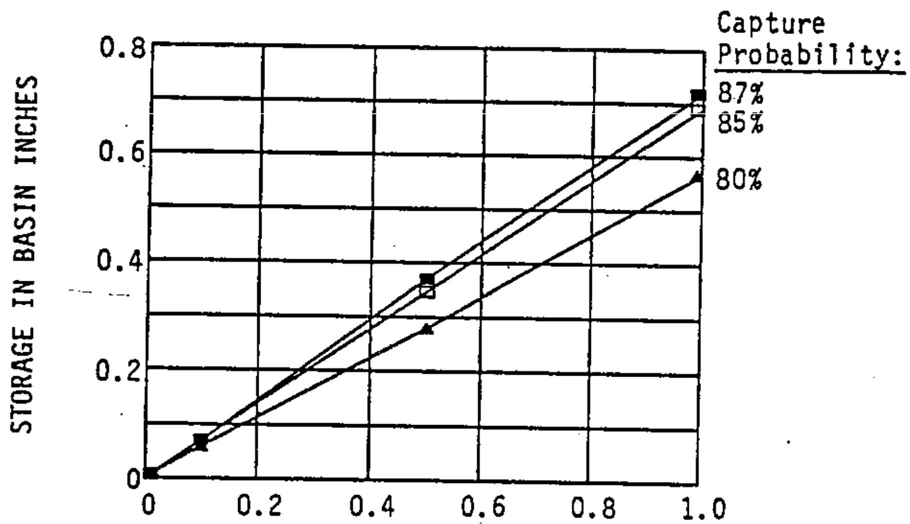

Figure 6. Capture Volumes for a 40-hour Drain Time and Several Runoff Event Capture Probabilities.

From our analysis of the Denver rain gauge data, it looks reasonable, logical and prudent to target the capture of approximately 80th percentile runoff event. This means that the detention facility can be reduced by about 25 to 30 percent in size make it more affordable, while still capturing in total 92 percent of the storm events. When the reduced detention facility is analyzed for impact on the average annual removal in total suspended solids, the difference from the maximized size in water quality being released to the receiving waters is practically not measurable. In other words, the 80 percentile capture volume should provide very good long term TSS removal rates. Also, basins of this size should fit easily within either on-site detention facilities designed for control of runoff peaks or within most landscaping areas of new developments.

At the same time, the removal of dissolved nutrients, such as phosphorous or nitrates, is primarily the function of residence time within the permanent water pool of the "wet pond" between storms. Increasing the capture volume above this pool should have little effect on the removal efficiencies of these compounds. Similarly, "dry ponds" have limited removal efficiencies of dissolved nutrients since their primary removal mechanism is sedimentation (Grizzard, et. al., 1986; Schueler, 1987; Roesner, et. al., 1988; Stahre and Urbonas, 1988).

DETERMINATION OF RUNOFF COEFFICIENT

Using Figure 4 or Figure 6 it is possible to quickly estimate an effective size of a stormwater quality detention basin. Since the engineer has to address smaller runoff events when dealing with stormwater quality, an appropriate runoff coefficient needs to be used. In 1982 EPA published data as part of the NURP study on rainfall depth vs. runoff volume. Although EPA did acknowledge some regional differences, much of the United States was found to be well represented by the data plotted in Figure 7. The curve in this figure is a third order regressed polynomial with the regression coefficient $R^2 = 0.79$. This value of R^2 implies a reasonably strong correlation between the watershed imperviousness, I, in percent and the runoff coefficient, C, for the range of data collected by EPA. Since the NURP study covered two year period, in our opinion this relationship is justified for 2-year recurrence probability and smaller storms.

EXAMPLE OF BASIN SIZING

An example is used next to demonstrate how to determine a "maximized" capture volume for an extended detention basin. A 100 acre (40.5 hectares) multi-family residential tributary watershed that has 60 percent of its area covered by impervious surfaces is used as the example conditions.

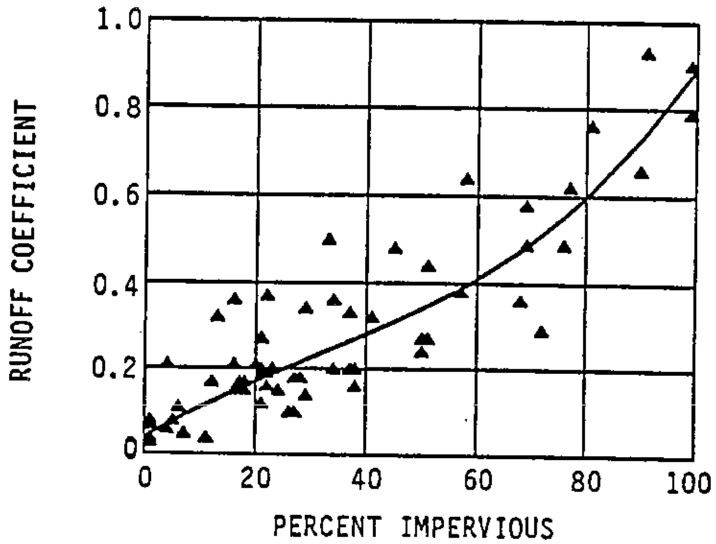

Figure 7. Runoff Coefficient Based on NURP Data for 2-year and Smaller Storms.

Using Figure 7 the runoff coefficient for the watershed, C = 0.4, is estimated. A well performing extended detention basin, according to Grizzard, et. al. (1986), needs to capture approximately the mean seasonal runoff and release it over a 24 hour period, which they suggested could be accomplished if the brim-full volume is drained in 40 to 48 hours. Thus, using the 80 percentile curve on Figure 6 and a brim-full drain time of 40 hours a design volume of 0.22 watershed inches (7.62 mm) is obtained. This is the runoff from a 0.55 inch (14 mm) storm and equates to 1.8 acre feet (2,300 cubic meters) of storage.

CONCLUSIONS

An investigation of sizing stormwater quality facilities for maximized capture of stormwater runoff events and their performance in removing settleable pollutants revealed that simplified design guidelines are possible. These guidelines can be developed using local or regional rain gauge records.

The procedure for the development of these simplified guidelines uses a Runoff Volume Point Diagram method to approximate a continuous simulation process in combination with an optimization routine. This procedure was converted by the authors into computer software.

Using the Denver rain gauge for the testing of this procedure, a figure was prepared that relates a watershed's runoff coefficient, required capture volume and the drain time for this volume. The procedure consists of the following steps:

1. Reduce the recorded rain gauge record (preferably hourly or 15-minute record) to a Rain Point Diagram using several storm separation periods.

2. Transform these Rain Point Diagrams into a Runoff Volume Point Diagrams by multiplying the individual rainfall depths by the watershed's Runoff Coefficient. This can be done for three or more values of C, such as C = 0.1, 0.5 and 1.0 to provide several points on the final design curves.

3. Process the Runoff Volume Point Diagrams through the optimization procedure described earlier using several capture volumes and brim-full storage volume drain times. Suggest using a Runoff Volume Point Diagram that was prepared using a time of storm separation equal to one-half of the desired brim-full drain time.

4. Plot all of the results on a figure similar to Figure 4 for the specific precipitation gauge being used.

5. Perform sensitivity analysis and if appropriate offer options for the sizing of capture volume for several levels of capture probability (eg. Figure 6) and/or TSS removal.

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HYDRAULIC DATA

	10-YR	100-YR
INFLOW (cfs)	1040	1890
OUTFLOW	180	540
WATER SURFACE ELEVATION	89.1	91.5
STORAGE (Ac. Ft.)	41.0	71.0

STATEMENT:

THE CITY OF COLORADO SPRINGS RECOGNIZES THE DESIGN ENGINEER AS HAVING RESPONSIBILITY FOR THE DESIGN; THE CITY HAS LIMITED ITS SCOPE OF REVIEW ACCORDINGLY.
RESUBMITTAL REQUIRED IF CONSTRUCTION HAS NOT COMMENCED WITHIN 180 DAYS AFTER REVIEW DATE.

REVIEW:

STREET DESIGN:
ROUGH CUT REVIEW ______ DATE ______
FINAL REVIEW ______ DATE ______
DRAINAGE DESIGN:
FILED IN ACCORDANCE WITH SECTION 15-3-906 OF THE CODE OF COLORADO SPRINGS 1980, AS AMENDED ______ DATE ______

DESIGN DATA:

SIDEWALKS: WIDTH ______
LOCATION: Attached ☐
Detached, 6" from P/L ☐
CURB TYPE 1 ☐ 2 ☐ 3 ☐
R/W WIDTH ______ F/C - F/C ______
STREET TYPE ______ HVEEM ______
ASPHALT THICKNESS:
AC Surface ______
AC Base ______
AGG. BASE THICKNESS:
Class 6 ______
Class 5 ______
Class 2 ______

SCALE: HORIZ. 1"=50' VERT. ______

BENCHMARK: BRASS CAP LOCATED AT THE S.W. cor OF THE BRIDGE 2000'± WEST OF POWERS BLVD. ON U.S. 24 BRIDGE OVER SAND CREEK EL. 6190.00 USGS.

REVISIONS:

NO.	DESCRIPTION	DATE
1	CONCRACT DOCUMENTS	7-16-90

ENGINEER:
Kiowa Engineering Corporation
419 W. BIJOU STREET
COLORADO SPRINGS
COLORADO 80905
DESIGNED BY: RNW/JYC DATE: 5/90
DRAWN BY: EAK DATE: 5/90
CHECKED BY: ______ DATE: ______

PROJECT POWERS BOULEVARD DETENTION POND
DETENTION POND PLAN
DRAINAGE BASIN PETERSON FIELD
JOB NO. 89-08-16 SHEET D4 OF 15

9005-07
Sheet 4 of 13
Facility: D
Detention Pond Plan
9005-07 Sheet 4

9005-07-4

STATEMENT:

THE CITY OF COLORADO SPRINGS RECOGNIZES THE DESIGN ENGINEER AS HAVING RESPONSIBILITY FOR THE DESIGN; THE CITY HAS LIMITED ITS SCOPE OF REVIEW ACCORDINGLY.
RESUBMITTAL REQUIRED IF CONSTRUCTION HAS NOT COMMENCED WITHIN 180 DAYS AFTER REVIEW DATE.

REVIEW:

STREET DESIGN:
ROUGH CUT REVIEW ______ DATE ______
FINAL REVIEW ______ DATE ______
DRAINAGE DESIGN:
FILED IN ACCORDANCE WITH SECTION 15-3-906 OF THE CODE OF COLORADO SPRINGS 1980, AS AMENDED ______ DATE ______

DESIGN DATA:

SIDEWALKS: WIDTH ______
LOCATION: Attached ☐
Detached, 6" from P/L ☐
CURB TYPE 1 ☐ 2 ☐ 3 ☐
R/W WIDTH ______ F/C - F/C ______
STREET TYPE ______ HVEEM ______

ASPHALT THICKNESS:
AC Surface ______
AC Base ______
AGG. BASE THICKNESS:
Class 6 ______
Class 5 ______
Class 2 ______

SCALE: HORIZ. ______ VERT. ______

BENCHMARK: BRASS CAP LOCATED AT THE S.W. cor OF THE BRIDGE, 2000'± WEST OF POWERS BLVD. ON US. 24 BRIDGE OVER SAND CREEK EL 6190.00 USGS

REVISIONS:

NO.	DESCRIPTION	DATE

ENGINEER:

Kiowa Engineering Corporation
419 W. BIJOU STREET
COLORADO SPRINGS
COLORADO 80905

DESIGNED BY: JYC DATE: 4/90
DRAWN BY: EAK DATE: 5/90
CHECKED BY: RNW DATE: 5/90

PROJECT POWERS BOULEVARD DETENTION POND
GRADING PLAN

SUBDIVISION ______
DRAINAGE BASIN PETERSON FIELD
JOB NO. 89-08-16 SHEET 05 OF 15

9005-07-5

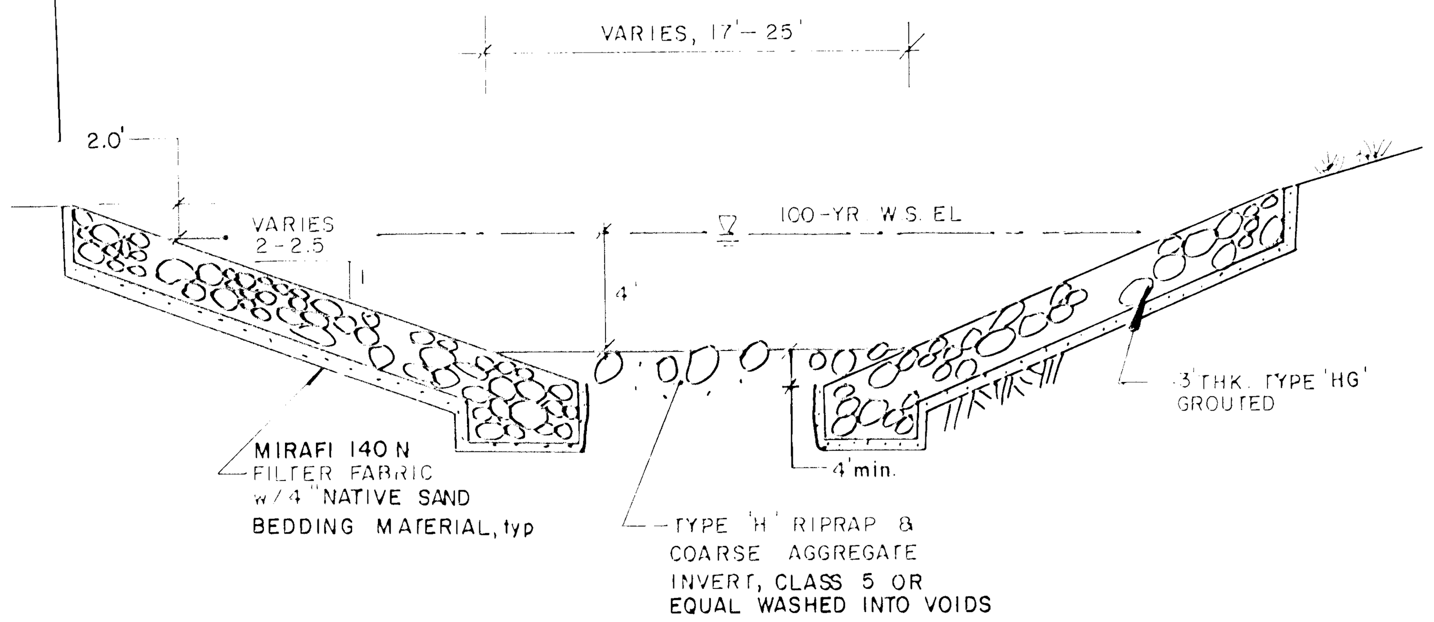
9005-07 Sheet 5 of 13 Facility: D

RIPRAP GRADATIONS

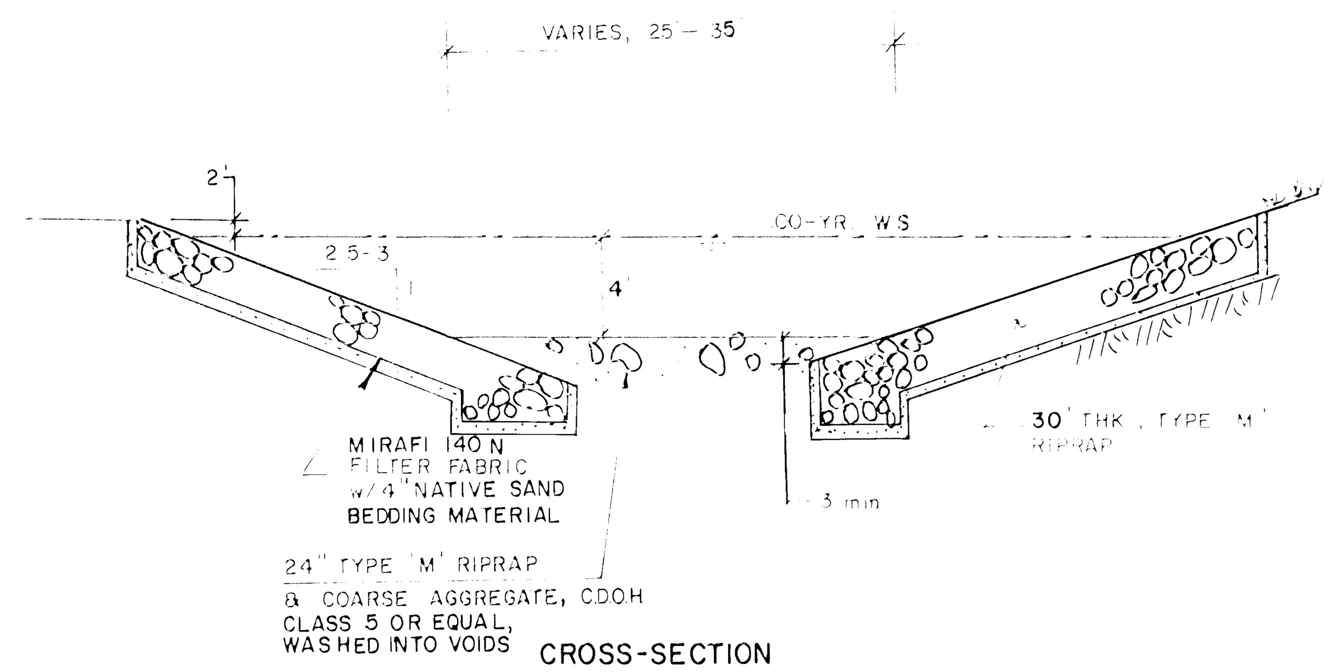
Rip Rap Designation	% Smaller Than Given Size by Weight	Intermediate Rock Dimension (inches)	d_{50}* (inches)
Type L	70 - 100	15	
	50 - 70	12	
	35 - 50	9	9**
	2 - 10	3	
Type M	70 - 100	21	
	50 - 70	18	
	35 - 50	12	12
	2 - 10	4	
Type H	100	30	
	50 - 70	24	
	35 - 50	18	18
	2 - 10	6	

GROUTED RIPRAP GRADATIONS

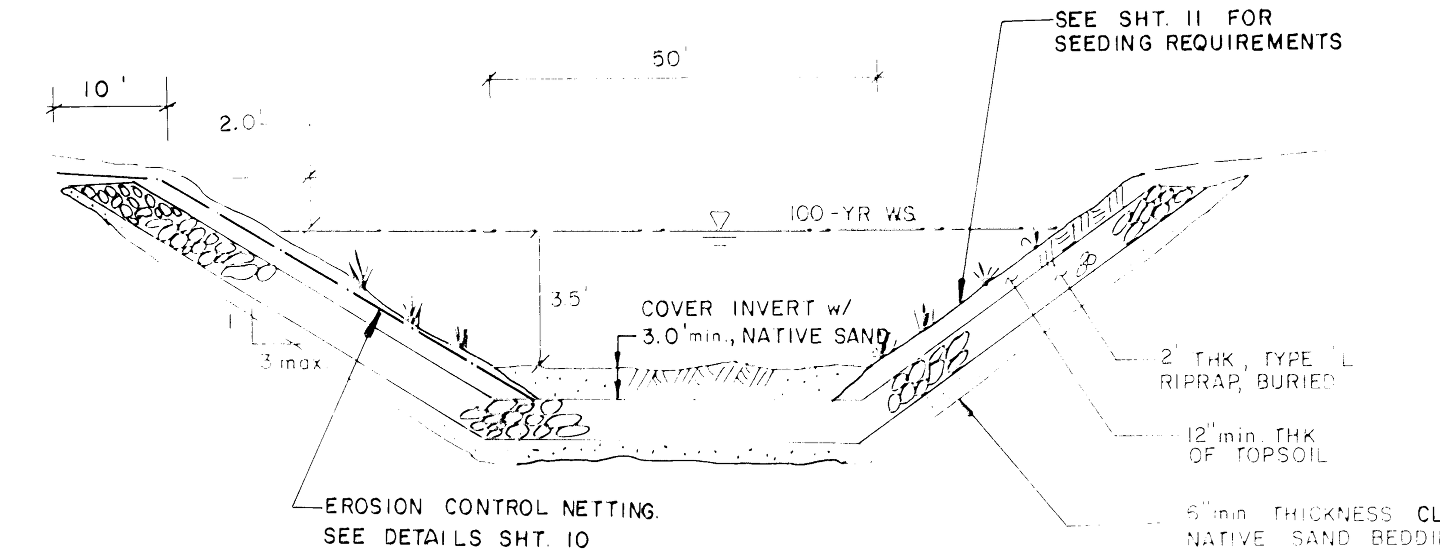
TYPE HG	100	30
	50-70	24
	0-5	18



CROSS-SECTION
PROPERTY LINE to STA.15+06 A
4|6



CROSS-SECTION
STA.15+06 to 14+24 B
4|6



CROSS-SECTION
STA. 13+94 to 12+70 C
4|6

NATIVE SAND MATERIAL TO BE USED AS BEDDING UNDER THE RIPRAP LININGS SHALL BE THE FINE TO COARSE SANDS IDENTIFIED IN THE SOIL REPORT FOR THIS PROJECT. NATIVE SAND BEDDING MATERIAL SHALL BE STOCKPILED AND CLEARED OF ORGANIC MATERIAL AND DEBRIS. ENGINEER SHALL OBSERVE STOCK-PILED MATERIAL PRIOR TO PLACEMENT OF THE NATIVE SAND OVER THE FILTER FABRIC MATERIAL.

RECORD DRAWINGS

Revisions Drawn by MOC Date 2-25-92

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS

STATEMENT

THE CITY OF COLORADO SPRINGS ... DESIGN ENGINEER AS HAVING RESPONSIB... THE DESIGN. THE CITY HAS LIMITED ... REVIEW ACCORDINGLY. RESUBMITTAL REQUIRED IF CONSTRUCTION HAS NOT COMMENCED WITHIN 730 DAYS AFTER ... DATE

1980 AS AMENDED

1" = 50' 1" = 5'

BRASS CAP LOCATED AT THE SW. cor. OF THE BRIDGE, 2000'± WEST OF POWERS BLVD. ON U.S. 24 BRIDGE OVER SAND CREEK. EL. 6190.00 U.S.G.S.

1 CONTRACT DOCUMENTS 7-16-90

Kiowa Engineering Corporation
419 W. BIJOU STREET
COLORADO SPRINGS
COLORADO 80905

DESIGNED BY	JYC	DATE	4/90
DRAWN BY	EAK	DATE	4/90
CHECKED BY		DATE	

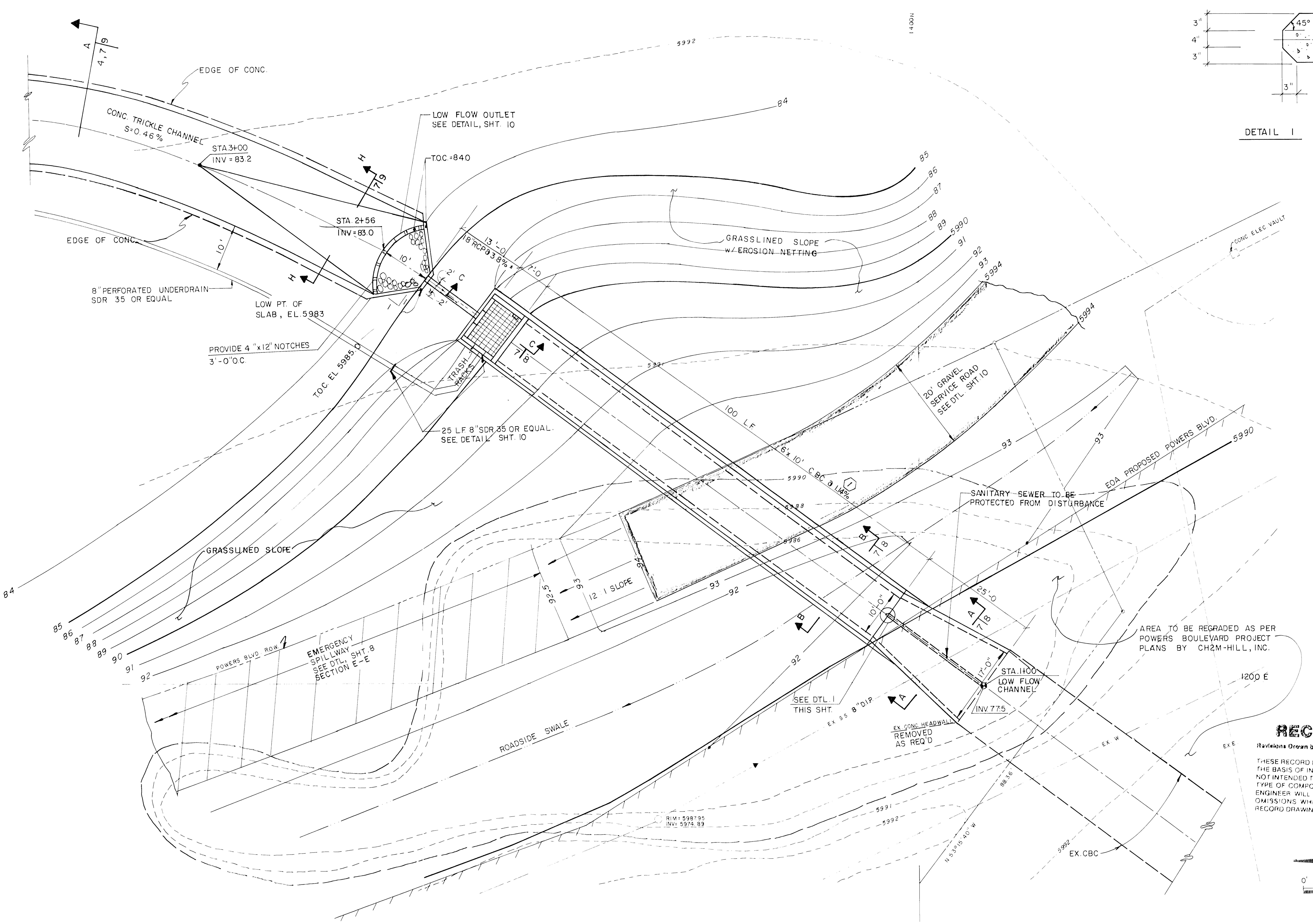
PROJECT POWERS BLVD. DETENTION POND
BASIN PROFILE & CHANNEL SECTIONS

DRAINAGE BASIN PETERSON FIELD

JOB NO. 89-08-16 SHEET 06 OF 15

9005-07-6

9005-07 Sheet 6 of 13 Facility: D



RECORD DRAWINGS

Revisions Drawn by MDC Date 2-25-92

THESE RECORD DRAWINGS HAVE BEEN PREPARED IN PART ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

Z

0' 5' 10' 15' 20' 25' 30

STATEMENT:

THE CITY OF COLORADO SPRINGS RECOGNIZES THE DESIGN ENGINEER AS HAVING RESPONSIBILITY FOR THE DESIGN. THE CITY HAS LIMITED ITS SCOPE OF REVIEW ACCORDINGLY.
RESUBMITTAL REQUIRED IF CONSTRUCTION HAS NOT COMMENCED WITHIN 180 DAYS AFTER REVIEW DATE

REVIEW:

STREET DESIGN
ROUGH CUT REVIEW DATE
FINAL REVIEW DATE
DRAINAGE DESIGN
FILED IN ACCORDANCE WITH SECTION 15-3-906 OF THE CODE OF COLORADO SPRINGS 1980, AS AMENDED DATE

DESIGN DATA:

SIDEWALKS WIDTH
LOCATION Attached ☐
Detached, 8" from P.L. ☐
CURB TYPE 1 ☐ 2 ☐ 3 ☐
R/W WIDTH F.C. F.C.
STREET TYPE HVEEM

ASPHALT THICKNESS
AC Surface
AC Base
AGG. BASE THICKNESS
Class 6
Class 5
Class 2

SCALE HORIZ 1"=10' VERT

BENCHMARK BRASS CAP LOCATED AT THE S.W. cor. OF THE BRIDGE, 2000'± WEST OF POWERS BLVD. ON U.S. 24 BRIDGE OVER SAND CREEK. EL. 6190.00 U.S.G.S.

REVISIONS

NO.	DESCRIPTION	DATE
1	CONTRACT DOCUMENTS	7-16-90

ENGINEER

Kiowa Engineering Corporation
419 W. Bijou Street
Colorado Springs, CO
80905

DESIGNED BY WHY DATE 4/90
DRAWN BY EAK DATE 4/90
CHECKED BY RNW DATE 5/90

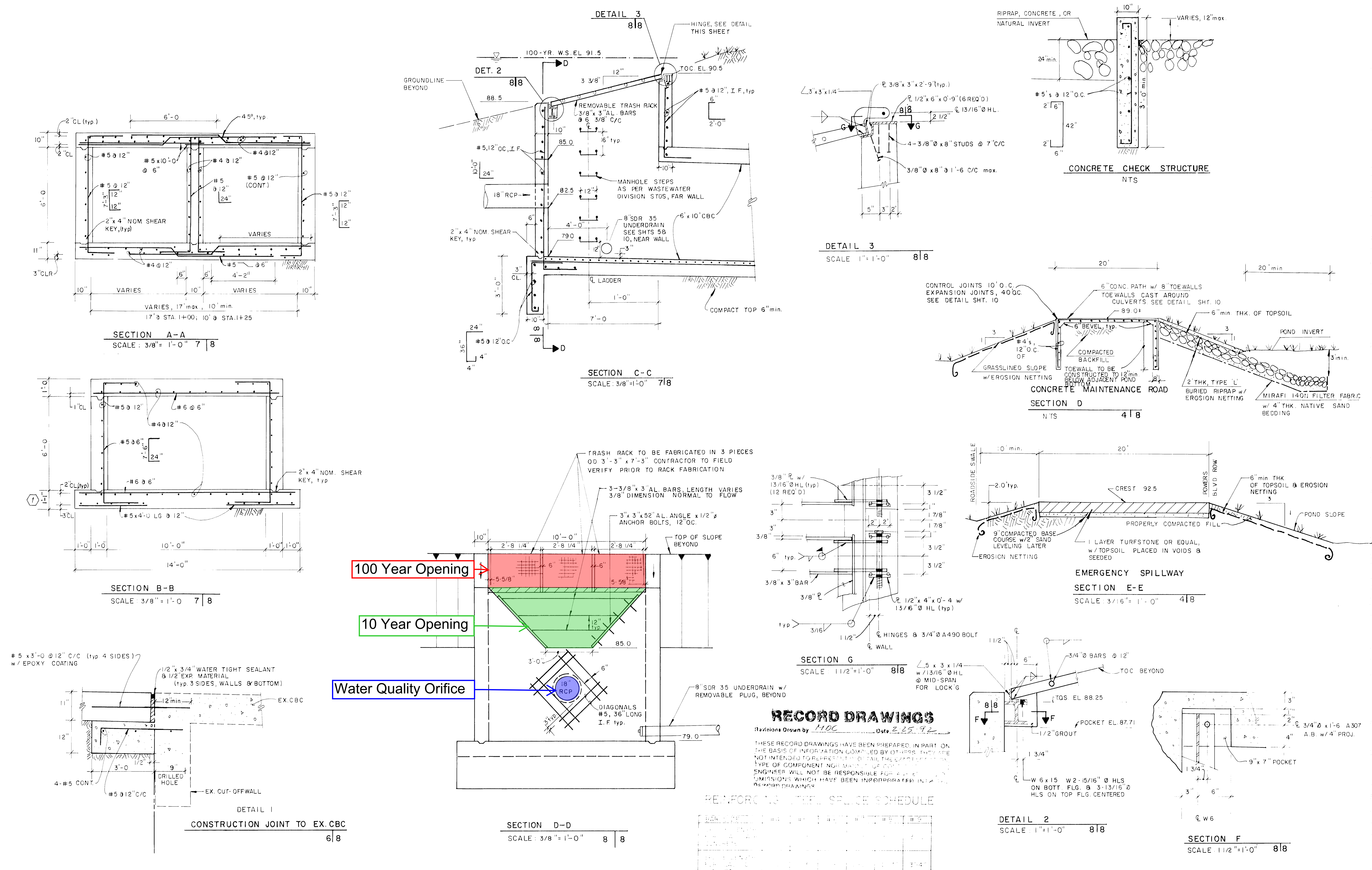
PROJECT POWERS BLVD. DETENTION POND
OUTLET STRUCTURE PLAN

DRAINAGE BASIN PETERSON FIELD
JOB NO 89-08-16 SHEET D7 OF 15

9005-07-7

9005-07 Sheet 7 of 13 Facility D

Outlet Structure Plan 9005-07 Sheet 7



STATEMENT

THE CITY OF COLORADO SPRINGS RECOGNIZES THE DESIGN ENGINEER AS HAVING RESPONSIBILITY FOR THE DESIGN. THE CITY HAS LIMITED ITS SCOPE OF REVIEW ACCORDINGLY.
RESUBMITTAL REQUIRED IF CONSTRUCTION HAS NOT COMMENCED WITHIN 180 DAYS AFTER REVIEW DATE

REVIEW

STREET DESIGN
ROUGH CUT REVIEW — DATE
FINAL REVIEW — DATE
DRAINAGE DESIGN
FILED IN ACCORDANCE WITH SECTION 15-3-908 OF THE CODE OF COLORADO SPRINGS 1980, AS AMENDED — DATE

DESIGN DATA

SIDEWALKS WIDTH
LOCATION
CURB TYPE
R.O.W. WIDTH
STREET TYPE

BENCHMARK: BRASS CAP LOCATED AT THE SW. cor. OF THE BRIDGE, 2000'± WEST OF POWERS BLVD. ON U.S. 24 BRIDGE OVER SAND CREEK. EL 6190.00 USGS.

REVISIONS

(1) CONTRACT DOCUMENTS 7-16-90

ENGINEER
Kiowa Engineering Corporation
419 W. BIJOU STREET
COLORADO SPRINGS
COLORADO 80905

	BY	DATE
DESIGNED		4/90
DRAWN	EAK	5/90
CHECKED	RNW	5/90

PROJECT POWERS BLVD. DETENTION POND
STRUCTURE DETAILS

DRAINAGE BASIN PETERSON FIELD
JOB NO 89-08-16
SHEET D8 OF 15

9005-07-8

9005-07
Sheet 8 of 13
Facility D