MASTER DEVELOPMENT DRAINAGE PLAN &
FINAL DRAINAGE REPORT for

Broadview Business Park Filing No. 6 Colorado Springs, CO

Prepared for:

Scannell Properties #298, LLC.
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Prepared by:

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Project #: 09441008

August 12, 2019
Revised December 16, 2019

CERTIFICATION

ENGINEERS STATEMENT

This report and plan for the drainage design of Broadview Business Park Filing No. 6 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 12/16/19
Colorado P.E. No. 49487 Date

COLORADO REGISTERED ERIC JON GUNDERSON 49487 PROFESSIONAL ENGINEER

DEVELOPER'S STATEMENT

Scannell Properties, LLC hereby certifies that the drainage facilities for Broadview Business Park Filing No. 6 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 Broadview Business Park Filing No. 6, guarantee that final drainage design review will absolve Scannell Properties, LLC 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, LLC
Name of Developer

12/17/19
Authorized Signature Date

Marc Pfleging
Printed Name

Manager
Title

800 E. 96th Street, Suite 175, Indianapolis, Indiana 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.

12/23/2019

For City Engineer Date

Conditions:

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INTRODUCTION

PURPOSE AND SCOPE OF STUDY

The purpose of this master development drainage plan ("MDDP") / final drainage report ("FDR") is to outline the drainage arrangement for the Broadview Business Park Filing No. 6 located northwest of the intersection of Aviation Way and Zeppelin Road (the "Property"), City of Colorado Springs, Colorado (the "City"). This MDDP/FDR 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 14.66 acres in size. The Property is currently unplatted and is being platted and subdivided into Lots 1 and 2 Block 1 of the Broadview Business Park Filing No. 6. Lot 1 is the northern lot and is 6.13 acres in size. Lot 2 will consist of the southern lot that is 8.53 acres in size. Lot 1 and Lot 2 make up the entirety of the "Site".

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

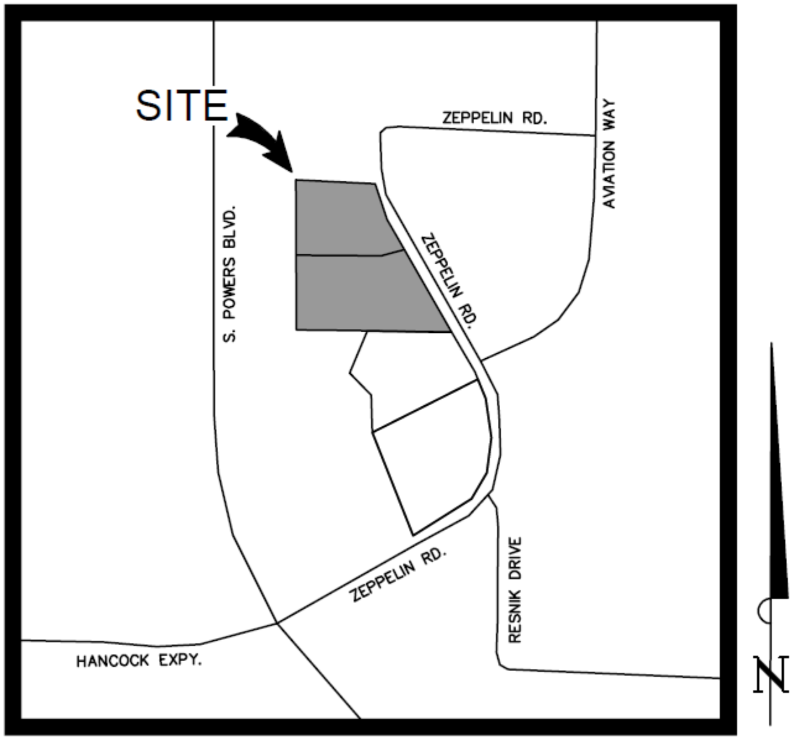
The proposed improvements consist of the construction of an approximately 91,520-gross square-foot, industrial warehouse/distribution building and parking lot within Lot 1 of the Property and construction of an approximately 131,040-gross square-foot, industrial warehouse/distribution building and parking lot within Lot 2 of the Property (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 and industrial distribution site to the south (Lot 1 BLK 1 Broadview Business Park Filing No. 3 & Lot 1 Broadview Business Park Filing No. 5), the James Irwin Charter Elementary School to the north (Lot 1 Sci Technology Sub Filing No. 1), Powers Boulevard to the west and Zeppelin Road to the east. The Property is currently undeveloped and does not include any existing site improvements except for a concrete drainage channel on the west side of the Property. The Property generally slopes northeast to southwest with the anticipated stormwater outfall for both Lot 1 and Lot 2 being the existing regional concrete trapezoidal channel to the west, which ultimately drains to the Powers Boulevard Detention Facility (herein the "regional detention pond") to the south of the Property.

An ALTA and topographic field survey was completed for the Project by Forth Land Surveying Inc. dated April 9th, 2019 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 Drainage Criteria Manual, Volumes 1 and 2 (2014) (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.



VICINITY MAP
NTS

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. There are two undeveloped offsite basins (Sub-basin D2 and Sub-basin D3) sheet flowing onto the site from the north. During a 100-year storm event, approximately 0.29 and 0.88 cfs (q_{peak})sheet flow onto the site from these offsite basins, respectively. Additionally, there is an existing regional concrete trapezoidal channel (43’ top width, 16’ bottom width and 6’ depth) along the western boundary of the Property that conveys on-site flows from Sub-basin D1 south to the regional detention pond. The Project is in compliance with the approved DBPS and there are no other previously approved reports or studies which impact this site.

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 1.2%, the western project boundary slopes from north to south at approximately 1.0%, 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 F.

NRCS soil data is available for this Site and it has been noted that soils onsite have been identified as 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 buildings, parking lot, paved drives, and other impervious surfaces comprise 82.2 percent (524,955 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 17.8 percent (113,669 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 historic drainage patterns of the Property. The proposed improvements align with the intent of the original drainage design of the Peterson Field Drainage Basin.

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 Appendix G for reference. Per the DETENTION REPORT, the Site lies within Sub-basin 3 which is 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. The existing regional detention pond was designed with a WQCV drain time of 24-hours, which differs from the current criteria of 40-hours.

Additionally, the design plans for the regional detention pond have been included in Appendix G 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 private water quality-only extended detention basins each with water quality outlet structures and two private water quality-only rain gardens that discharge to the existing regional detention concrete drainage channel. The private water quality-only extended detention basins will be constructed along the south western and central western boundaries of the Site and will only detain the proposed water quality capture volume. The water quality-only extended detention basins will discharge to the existing channel. The private water quality-only rain gardens will also only be sized to only detain the proposed water quality capture volume and they will discharge to the existing channel. Detention for the minor (5-year) and major (100-year) storm events will be provided in the existing regional detention pond.

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 regional extended detention basin to the south of the Site. There are no known major irrigation facilities within 100 feet of the property.

EXISTING CONDITIONS SUB-BASIN DESCRIPTION

The existing runoff within the Property generally drains from northeast to southwest to the regional detention pond. 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 7.14 acres of the property and is currently undeveloped vacant land. Drainage flows overland from northeast to southwest at approximately 1.5% to the existing concrete channel which outfalls to the existing regional detention pond. Runoff during the 5-year and 100-year events are 2.85 cfs and 15.45 cfs respectively.

Sub-Basin E2.

Sub-basin E2 consists of the southern 7.51 acres of the property and is currently undeveloped vacant land. Drainage flows overland from northeast to southwest at approximately 1.8% to the existing regional detention pond. Runoff during the 5-year and 100-year events are 3.79 cfs and 17.44 cfs respectively.

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 buildings will be conveyed to either the private water quality-only extended detention basins at the southwestern or central western edges of the Site or one of the two private water quality-only rain gardens at the north end of the Site. The controlled stormwater release from the central extended detention basin outlet structure will be conveyed through a private 24” HDPE storm sewer pipe. The controlled stormwater release from the southern extended detention basin outlet structure will be conveyed through a private 24” HDPE storm sewer pipe. The controlled stormwater release from both private water quality-only rain gardens will be conveyed through an 18” HDPE storm sewer pipe. All water quality features will outfall into the existing City owned regional concrete trapezoidal swale (43’ top width, 16’ bottom width and 6’ depth) which discharges to the pond to the south 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 an ultimate outfall into Sand Creek.

The Property has been divided into eleven sub-basins, A1-A2, B1-B4, C1-C2, and D1-D3. The runoff generated on the building roof area is collected and conveyed via private roof drain systems which outfall to the proposed water quality-only extended detention basins. Each of the sub-basins drain either directly to a private water quality-only rain garden or to an inlet upstream of the private water quality-only extended detention basins. A proposed conditions map has been provided in Appendix F.

Sub-Basin A1

Sub-basin A1 is located at the northeast corner of the Site and consists of 0.62 acres of parking and landscape area with a basin impervious value of 72.6%. Developed direct runoff for the 5-year and 100-year storm events are 2.00 and 3.94 cfs, respectively. The sub-basin flows overland to the proposed private water quality-only rain garden. Stormwater runoff will flow into the proposed private water quality-only rain garden through 2, 2-ft wide curb cuts. Energy dissipation for the curb cuts will be provided by a 6'x2' Type L Riprap area placed at each curb cut. A Private Type C Inlet (DP A1), which has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin A1 (refer to Appendix D), will capture overflow from the rain garden (stormwater runoff above the WQCV). In addition, a 4" PVC underdrain system that discharges to the inlet will capture excess stormwater that does not infiltrate within the rain garden. An 18" HDPE Storm Drain, Private Storm Line A, will convey flows from the Private Type C Inlet within the rain garden to the proposed outfall within the existing concrete swale to the west.

Sub-Basin A2

Sub-basin A2 is located along the northern property boundary and consists of 0.60 acres of parking and landscape area with a basin impervious value of 75.9%. Developed direct runoff for the 5-year and 100-year storm events are 2.15 and 4.19 cfs, respectively. The sub-basin flows overland to the proposed private water quality-only rain garden. Stormwater runoff will flow into the proposed private water quality-only rain garden through 2, 2-ft wide curb cuts. Energy dissipation for the curb cuts will be provided by a 6'x2' Type L Riprap area placed at each curb cut. A Private Type C Inlet (DP A2), which has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin A2 (refer to Appendix D), will capture overflow from the rain garden (stormwater runoff above the WQCV). In addition, a 4" PVC underdrain system that discharges to the inlet will capture excess stormwater that does not infiltrate within the rain garden. An 18" HDPE Storm Drain, Private Storm Line A, will convey flows from the Private Type C Inlet within the rain garden to the proposed outfall within the existing concrete swale to the west.

Sub-Basin B1

Sub-basin B1 is located along the eastern property boundary, between both lots, and consists of the eastern 1/3 of the north building, a portion of the south building, and the shared truck court with minimal landscape area. The sub-basin has an area of 3.04 acres with a basin impervious value of 82.5%. Developed direct runoff for the 5-year and 100-year storm events are 10.18 and 19.40 cfs, respectively. This sub-basin will flow overland to a proposed Private Double Type 13 Inlet in sump (DP B1) at the center of the sub-basin before discharging to the central private water quality-only extended detention basin via a 30" HDPE Storm Drain, private Storm Line B. The proposed private inlet has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin B1 (refer to Appendix D). If this inlet becomes clogged, the emergency overflow path for the stormwater will be west into the private inlet within Sub-Basin B2.

Sub-Basin B2

Sub-basin B2 is located near the center of the property and consists of the center 1/3 of the north building, a portion of the south building, and the shared truck court with minimal landscape area. The sub-basin has an area of 2.46 acres with a basin impervious value of 93.6%. Developed direct runoff for the 5-year and 100-year storm events are 9.71 and 17.79 cfs, respectively. This sub-basin will flow overland to a proposed Private Double Type 13 Inlet in sump (DP B2) at the center of the sub-basin before discharging to the central private water quality-only extended detention basin via a 30" HDPE Storm Drain, private Storm Line B. The proposed private inlet has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin B2 (refer to Appendix D). If this inlet becomes clogged, the emergency overflow path for the stormwater will be split as stormwater will flow either east into the private inlet within Sub-Basin B1 or west into the private inlet within Sub-Basin B3.

Sub-Basin B3

Sub-basin B3 is located near the western edge of the property, between both lots, and consists of the western 1/3 of the north building, a portion of the south building, private drive aisles, and the shared truck court with minimal landscape area. The sub-basin has an area of 2.72 acres with a basin impervious value of 90.1%. Developed runoff for the 5-year and 100-year storm events are 9.67 and 17.93 cfs, respectively. This sub-basin will flow overland to a proposed Private Double Type 13 Inlet in sump (DP B3) at the center of the sub-basin before discharging to the central private water quality-only extended detention basin via a 30" HDPE Storm Drain, private Storm Line B. The proposed private inlet has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin B3 (refer to Appendix D). If this inlet becomes clogged, the emergency overflow path for the stormwater will be east into the private inlet within Sub-Basin B2. The overall emergency path for Sub-Basins B1 – B3 will be west directly into the private water quality-only extended detention basin located within Sub-Basin B4.

Sub-Basin B4

Sub-basin B4 is located along the western property boundary and consists primarily of the proposed central private water quality-only extended detention basin. The sub-basin has an area of 0.42 acres with a basin impervious value of 26.5%. Developed runoff for the 5-year and 100-year storm events are 0.63 and 1.84 cfs, respectively. The sub-basin flows are captured within the central private water quality-only extended detention basin which outfalls to the adjacent regional concrete channel to the west. An emergency spillway will be provided for the detention basin contained within Sub-Basin B4. The emergency spillway will be 9-ft wide, include 4:1 max side slopes and will be stabilized with Type L Riprap. The emergency spillway will direct flows from the detention basin to the existing regional concrete trapezoidal channel.

Sub-Basin C1

Sub-basin C1 is located at the southeast corner of the Site and consists of 2.37 acres of a portion of the south building, parking lot, and landscape area with a basin impervious value of 69.5%. Developed runoff for the 5-year and 100-year storm events are 6.11 and 12.33 cfs, respectively. This sub-basin will flow overland to a proposed Private Type R Inlet in sump (DP C1) at the center of the sub-basin before discharging to the southern private water quality-only extended detention basin via a 24" HDPE Storm Drain, private Storm Line C. The proposed private inlet has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin C1 (refer to Appendix D). If this inlet becomes clogged, the emergency overflow path for the stormwater will be south, off-site into a large landscape area that drains directly to the regional extended detention basin.

Sub-Basin C2

Sub-basin C2 is located along the southern property boundary and consists of 1.66 acres of a portion of the south building, parking lot, the southern water quality-only extended detention basin and a landscape area with a basin impervious value of 69.6%. Developed runoff for the 5-year and 100-year storm events are 4.58 and 9.26 cfs, respectively. This sub-basin will flow overland to a proposed Private Type C inlet in sump (DP C2) at the center of the sub-basin before discharging to the southern private water quality-only extended detention basin via a 24" HDPE Storm Drain, private Storm Line C. The proposed private inlet has been sized to intercept 100% of the 100-yr storm event stormwater runoff within Sub-Basin C2 (refer to Appendix D). If this inlet becomes clogged, the emergency overflow path for the stormwater will be south, off-site directly to the regional extended detention basin. An emergency spillway will be provided for the detention basin contained within Sub-Basin C2. The emergency spillway will be 33-ft wide, include 4:1 max side slopes and will be stabilized with Type L Riprap. The emergency spillway will direct flows from the detention basin to the existing regional concrete trapezoidal channel.

Sub-Basins D1

Sub-basin D1 consists of the portion of the existing regional concrete trapezoidal channel (43' top width, 16' bottom width and 6' depth) within the Property. There are no changes proposed to the channel except for the three proposed outfall connections from Storm Lines A, B, and C. Sub-basin D1 is 0.51 acres with a basin imperviousness of 100.0%. Developed runoff for the 5-year and 100-year storm events are 2.02 and 3.61 cfs, respectively, and flows from north to south to the existing regional detention pond.

Sub-Basins D2

Sub-basin D2 is an offsite sub-basin that consists of undeveloped land north of the Property. There are no changes proposed to the sub-basin. Sub-basin D2 is 0.11 acres with a basin imperviousness of 2.0%. Developed runoff for the 5-year and 100-year storm events are 0.04 and 0.29 cfs, respectively. The sub-basin flows from north to south and sheet flows into sub-basin A1.

Sub-Basins D3

Sub-basin D3 is an offsite sub-basin that consists of undeveloped land north of the Property. There are no changes proposed to the sub-basin. Sub-basin D2 is 0.29 acres with a basin imperviousness of 2.0%. Developed runoff for the 5-year and 100-year storm events are 0.13 and 0.88 cfs, respectively. The sub-basin flows from north to south and sheet flows into sub-basin A2.

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 methods as specified in the CRITERIA and MANUAL. The water quality-only detention basin outlet structures were 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 Manual, dated May 2014, for the proposed development.

HYDRAULIC ANALYSIS

MAJOR DRAINAGEWAYS

There is an existing regional concrete trapezoidal channel (43' top width, 16' bottom width and 6' depth) that runs along the western boundary of the property. This channel conveys flows from areas north of the Site southward to the regional detention pond. No changes or impacts to this channel are proposed with the Project except for the proposed pond outfall pipe connections to the channel.

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 Appendix C. There are no additional provisions selected or deviations from the City of Colorado Springs Drainage Criteria Manual, dated May 2014, for the proposed development.

Inlet capacity calculations have been provided in Appendix D for each inlet on Site. The capacity of each type of inlet is adequate for the 5 and 100-year storm event developed flows for each sub-basin.

The Project will consist of the removal of the onsite vegetation of native weeds, brush, grasses, and trees. The proposed improvements consist of the construction of an approximately 91,520-gross square-foot, industrial warehouse/distribution building and parking lot within Lot 1 of the Property and construction of an approximately 131,040-gross square-foot, industrial warehouse/distribution building and parking lot within Lot 2 of the Property.

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 H. On-site water quality treatment will be provided by means of two (2) private water quality-only extended detention basins with water quality outlet structures and two (2) private water quality-only rain gardens. The water quality-only extended detention basins will be constructed along the western boundary of the Site. Each water quality-only extended detention basin 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 concrete swale. The outlet pipes are sized to be equal in diameter or greater to the inflow pipes that enter the extended detention basin, thereby passing the developed 100-year flows through the extended detention basin, directly to the regional detention concrete swale to the west of the Site. The proposed onsite water quality-only extended detention basins are designed to detain for the required WQCV only. The proposed private water quality-only rain gardens have also been sized to accommodate the WQCV. Stormwater flows above the WQCV water surface elevation within the rain gardens will be captured within a Private Type C Inlet and discharged directly to the regional detention concrete swale. The regional detention pond, south of the regional detention concrete swale, provides additional detention for the minor and major events.

Four-Step Process

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. The four-step process per the CRITERIA provides guidance and requirements for the selection of siting of structural Best Management Practices (BMPs) for new development and significant redevelopment.

Step 1: Employ Runoff Reduction Practices

Currently the site is vacant land. Development of the site will increase current runoff conditions due to the site being vacant. However, implementation of landscaping throughout the site, the proposed storm sewer infrastructure, the two proposed private water quality-only extended detention basins and the two proposed private water quality rain gardens will help slow runoff and encourage infiltration. 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. Reference Appendix for the UDFCD Imperviousness Reduction Factor (IRF) spreadsheet.

Step 2: Provide Water Quality Capture Volume (WQCV)
The water quality capture volume will be detained using two private water quality extended detention basins and two proposed private water quality rain gardens with water quality outlet structures located in the northwest and southwest corners of the property. The outfall pipes from the water quality outlet structures will convey the 100-year storm event to the existing 40' wide concrete drainage channel that runs along the western boundary of the property.

Step 3: Stabilize Drainageways
There is an existing regional concrete trapezoidal stabilized drainage channel (43' top width, 16' bottom width and 6' depth) that runs along the western boundary of the property. The existing channel is stabilized and is the drainageway that conveys flows from areas east of Powers Boulevard, southward to the existing regional detention pond. The historical drainage patterns and the proposed drainage patterns for the Site are tributary to this stabilized channel. No changes or impacts to this channel are proposed with the Project outside of the three proposed outfall connections from the Site.

Step 4: Implement Site Specific and Other Source Control BMPs
Day to day operations of the Project will include the arrival and departure of numerous semi-trucks that will be delivering and receiving packages from the proposed building. These trucks will be loaded via fork lifts and equipment that is internal to the building. All operations and material storage will be internal to the building, therefore site specific and other source control BMPs will not be required for outdoor material storage. Additionally, specific permanent BMPs for spill prevention exterior to the building is not anticipated to be required as all operations will be internal to the building. Internal to the building, sand/oil interceptors will be installed that will be connected to the sanitary system. These interceptors will treat chemical or oil spills internal to the building. A spill prevention, containment and control plan will be developed and implemented by the future building tenants.

STRUCTURE CHARACTERISTICS

Water Quality Storage Required

Calculations included in Appendix C provide calculations for the private water quality-only extended detention basins and the private water quality-only rain gardens. The calculations include determination of the storage volumes required for the WQCV only, and allowable release rates. Overall, 0.012 acre-feet of water quality capture volume is required for the northeast water quality-only rain garden (Sub-Basin A1) and the proposed rain garden provides 0.012 acre-feet of storage. Overall, 0.012 acre-feet of water quality capture volume is required for the northwest water quality-only rain garden (Sub-Basin A2) and the proposed rain garden provides 0.015 acre-feet of storage. Overall, 0.262 acre-feet of water quality capture volume is required for the center private water quality-only extended detention basin and the proposed basin provides 0.479 acre-feet of storage. Sub-basins B1-B4 have a total area of 8.64 acres (85.3% imperviousness) contributing flow to the central extended detention basin. Overall, 0.091 acre-feet of water quality capture volume is required for the south private water quality-only extended detention basin and the proposed basin provides 0.240 acre-feet of storage. Sub-basins C1-C2 have a total area of 4.02 acres (69.5% imperviousness) contributing flow to the southern extended detention basin.

The required 5-year and 100-year detention volumes are 0.049 acre-feet and 0.084 acre-feet respectively for the northeast water quality-only rain garden, 0.050 acre-feet and 0.085 acre-feet respectively for the northwest water quality-only rain garden, 0.843 acre-feet and 1.390 acre-feet respectively for the center private water quality-only extended detention basin, and 0.301 acre-feet and 0.519 acre-feet respectively for the south private water quality-only extended detention basin 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 water quality-only extended detention basins and water quality-only rain gardens. 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 structures are presented in Appendix C.

Storm Sewer Requirements

Calculations which determine the storm sewer capacity, type of flow, pipe losses, and hydraulic grade line calculations are included in Appendix D 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

Each private water quality-only extended detention basin is designed to include a forebay structure, concrete trickle channel, micropool and outlet structure per the CRITERIA.

FLOODPLAINS

The Flood Insurance Rate Maps (FIRM) 08041C0761G effective date December 7, 2018, by FEMA, indicates that the Site is located in Zone X (outside of the 500-year flood plain). This panel is included in Appendix A.

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, in connection with the subject property.”

EROSION CONTROL PLAN

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

For the initial erosion control plan, temporary sediment basins will be provided in the same proposed locations as the private water quality-only extended detention basins and private water quality-only rain gardens. Because the site drains from northeast to southwest, a diversion swale will be proposed along the south property line to direct the flows to either of the detention basins. The temporary sediment basins will be designed with an emergency spillway that would direct flow to the concrete channel to the west. The design for each pond will include an outfall pipe that directs flow from the ponds to the concrete channel to the west. Vehicle tracking control, soil stockpile, concrete washout, and stabilized staging area will be proposed near the site entrances. Silt fence will be utilized where necessary to protect adjacent land.

The final erosion control plan will use 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 will be 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. All landscape islands will be permanently stabilized with Kentucky bluegrass. The slopes and bottoms of the sediment basins will be stabilized with a detention basin mix by Applewood seed. Reference landscape plans for complete permanent stabilization details.

FEES DEVELOPMENT

DRAINAGE, BRIDGE, POND AND SURCHARGE FEES

The required fees for the Peterson Field Drainage Basin based upon the 2019 fee schedule, are listed below. Fees will be paid prior to plat recordation.

-	Drainage Fee/Acre =	$12,925	x	14.66 acres =	$189,480.50
-	Bridge Fee/Acre	$595	x	14.66 acres =	$8,722.70
				Total =	**$198,203.20**

CONSTRUCTION COST OPINION

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

MAINTENANCE AND OPERATIONS

It is our recommendation that the private water quality-only extended detention basin and private water quality-only rain garden 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. In addition, media replacement and mowing may need to occur after each inspection within the rain gardens. 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 water quality-only BMPs 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. 6 includes one variance request, which has been included as Appendix I. The variance has been requested to allow inlets to be used as junctions on a trunk line, which is not allowed by the Drainage Criteria Manual (Chapter 9, Section 6.2). With the exception of the variance request, the drainage design for the development conforms to the City of Colorado Springs Drainage Criteria Manual 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 Volumes 1 and 2, 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 08041C0761G effective date December 7, 2018, prepared by the Federal Emergency Management Agency (FEMA).

4. Peterson Field Drainage Basin Master Plan Update, Colorado Springs, Colorado, September 28, 1984, prepared by URS.

5. Powers Boulevard Detention Facility Final Drainage Report, Colorado Springs, Colorado, January 1990, prepared by Kiowa Engineering Corporation

APPENDIX

APPENDIX A – FEMA FIRM MAP

National Flood Hazard Layer FIRMette

FEMA

38°47'56.53"N

104°43'57.55"W

104°43'20.09"W

38°47'28.49"N

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

1:6,000

Legend

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

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

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

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

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

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

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

N

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

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 6/20/2019 at 5:16:11 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

APPENDIX B – SITE SOIL DATA

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

El Paso County Area, Colorado

Zeppelin III and IV

June 20, 2019

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

The U.S. Department of Agriculture (USDA) prohibits discrimination in all its programs and activities on the basis of race, color, national origin, age, disability, and where applicable, sex, marital status, familial status, parental status, religion, sexual orientation, genetic information, political beliefs, reprisal, or because all or a part of an individual's income is derived from any public assistance program. (Not all prohibited bases apply to all programs.) Persons with disabilities who require alternative means for communication of program information (Braille, large print, audiotape, etc.) should contact USDA's TARGET Center at (202) 720-2600 (voice and TDD). To file a complaint of discrimination, write to USDA, Director, Office of Civil Rights, 1400 Independence Avenue, S.W., Washington, D.C. 20250-9410 or call (800) 795-3272 (voice) or (202) 720-6382 (TDD). USDA is an equal opportunity provider and employer.

Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map


104° 43' 52" W
104° 43' 30" W
523350
523400
523450
523500
523550
523600
523650
523700
523750
523800
523850
38° 47' 45" N
38° 47' 35" N
4294170
4294120
4294070
4294020
4293970
4293920
4293870
Powers
S Powers Blvd
Driveway
Zeppelin Rd
95
8
Soil Map may not be valid at this scale.
Map Scale: 1:2,360 if printed on A landscape (11" x 8.5") sheet.
Meters
0
30
60
120
180
Feet
0
100
200
400
600
N
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

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

Special Point Features

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

Water Features

- Streams and Canals

Transportation

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

Background

- Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 16, Sep 10, 2018

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

Date(s) aerial images were photographed: Jun 3, 2014—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

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
8	Blakeland loamy sand, 1 to 9 percent slopes	0.4	2.6%
95	Truckton loamy sand, 1 to 9 percent slopes	16.9	97.4%
Totals for Area of Interest		**17.3**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

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: Hills, flats
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

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: Hills, flats
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

Soil Information for All Uses

Soil Properties and Qualities

The Soil Properties and Qualities section includes various soil properties and qualities displayed as thematic maps with a summary table for the soil map units in the selected area of interest. A single value or rating for each map unit is generated by aggregating the interpretive ratings of individual map unit components. This aggregation process is defined for each property or quality.

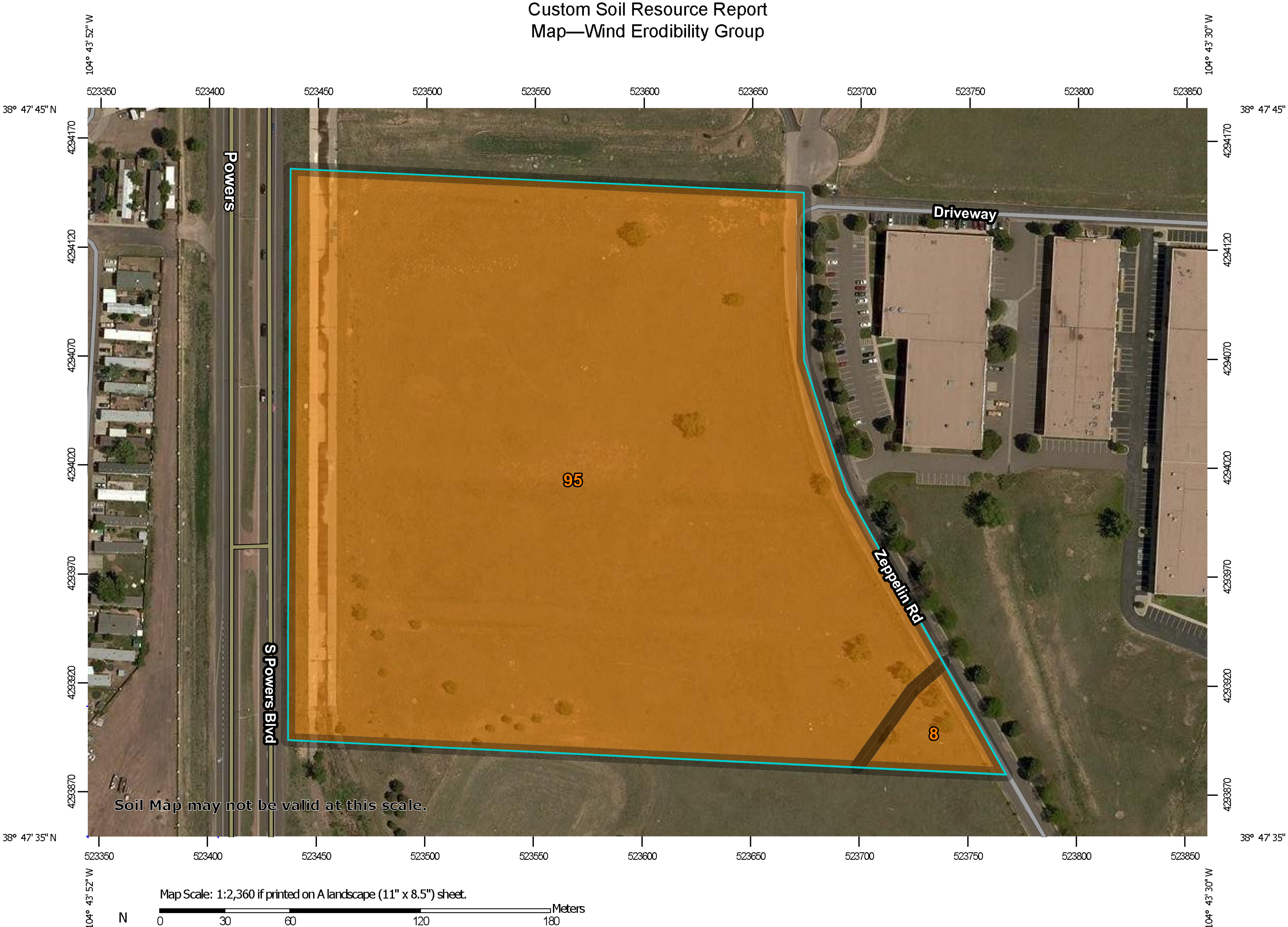
Soil Erosion Factors

Soil Erosion Factors are soil properties and interpretations used in evaluating the soil for potential erosion. Example soil erosion factors can include K factor for the whole soil or on a rock free basis, T factor, wind erodibility group and wind erodibility index.

Wind Erodibility Group

A wind erodibility group (WEG) consists of soils that have similar properties affecting their susceptibility to wind erosion in cultivated areas. The soils assigned to group 1 are the most susceptible to wind erosion, and those assigned to group 8 are the least susceptible.

Map—Wind Erodibility Group



Map Scale: 1:2,360 if printed on A landscape (11" x 8.5") sheet.

N

0 30 60 120 180 Meters

0 100 200 400 600 Feet

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

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- 1
- 2
- 3
- 4
- 4L
- 5
- 6
- 7
- 8
- Not rated or not available

Soil Rating Lines

- 1
- 2
- 3
- 4
- 4L
- 5
- 6
- 7
- 8
- Not rated or not available

Soil Rating Points

- 1
- 2
- 3
- 4
- 4L
- 5
- 6
- 7
- 8
- Not rated or not available

Water Features

Streams and Canals

Transportation

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

Background

Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 16, Sep 10, 2018

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

Date(s) aerial images were photographed: Jun 3, 2014—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.

Table—Wind Erodibility Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
8	Blakeland loamy sand, 1 to 9 percent slopes	2	0.4	2.6%
95	Truckton loamy sand, 1 to 9 percent slopes	2	16.9	97.4%
Totals for Area of Interest			**17.3**	**100.0%**

Rating Options—Wind Erodibility Group

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Lower

K Factor, Whole Soil

Erosion factor K indicates the susceptibility of a soil to sheet and rill erosion by water. Factor K is one of six factors used in the Universal Soil Loss Equation (USLE) and the Revised Universal Soil Loss Equation (RUSLE) to predict the average annual rate of soil loss by sheet and rill erosion in tons per acre per year. The estimates are based primarily on percentage of silt, sand, and organic matter and on soil structure and saturated hydraulic conductivity (Ksat). Values of K range from 0.02 to 0.69. Other factors being equal, the higher the value, the more susceptible the soil is to sheet and rill erosion by water.

"Erosion factor Kw (whole soil)" indicates the erodibility of the whole soil. The estimates are modified by the presence of rock fragments.

Map—K Factor, Whole Soil

Map Scale: 1:2,360 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

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

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- .02
- .05
- .10
- .15
- .17
- .20
- .24
- .28
- .32
- .37
- .43
- .49
- .55
- .64
- Not rated or not available

Soil Rating Lines

- .02
- .05
- .10
- .15
- .17
- .20
- .24
- .28
- .32
- .37
- .43
- .49
- .55
- .64
- Not rated or not available

Soil Rating Points

- .02
- .05
- .10
- .15
- .17
- .20
- .24
- .28
- .32
- .37
- .43
- .49
- .55
- .64
- Not rated or not available

Water Features

Streams and Canals

Transportation

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

Background

Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 16, Sep 10, 2018

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

Date(s) aerial images were photographed: Jun 3, 2014—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.

Table—K Factor, Whole Soil

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
8	Blakeland loamy sand, 1 to 9 percent slopes	.10	0.4	2.6%
95	Truckton loamy sand, 1 to 9 percent slopes	.17	16.9	97.4%
Totals for Area of Interest			**17.3**	**100.0%**

Rating Options—K Factor, Whole Soil

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Layer Options (Horizon Aggregation Method): Surface Layer (Not applicable)

Soil Qualities and Features

Soil qualities are behavior and performance attributes that are not directly measured, but are inferred from observations of dynamic conditions and from soil properties. Example soil qualities include natural drainage, and frost action. Soil features are attributes that are not directly part of the soil. Example soil features include slope and depth to restrictive layer. These features can greatly impact the use and management of the soil.

Hydrologic Soil Group

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

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

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

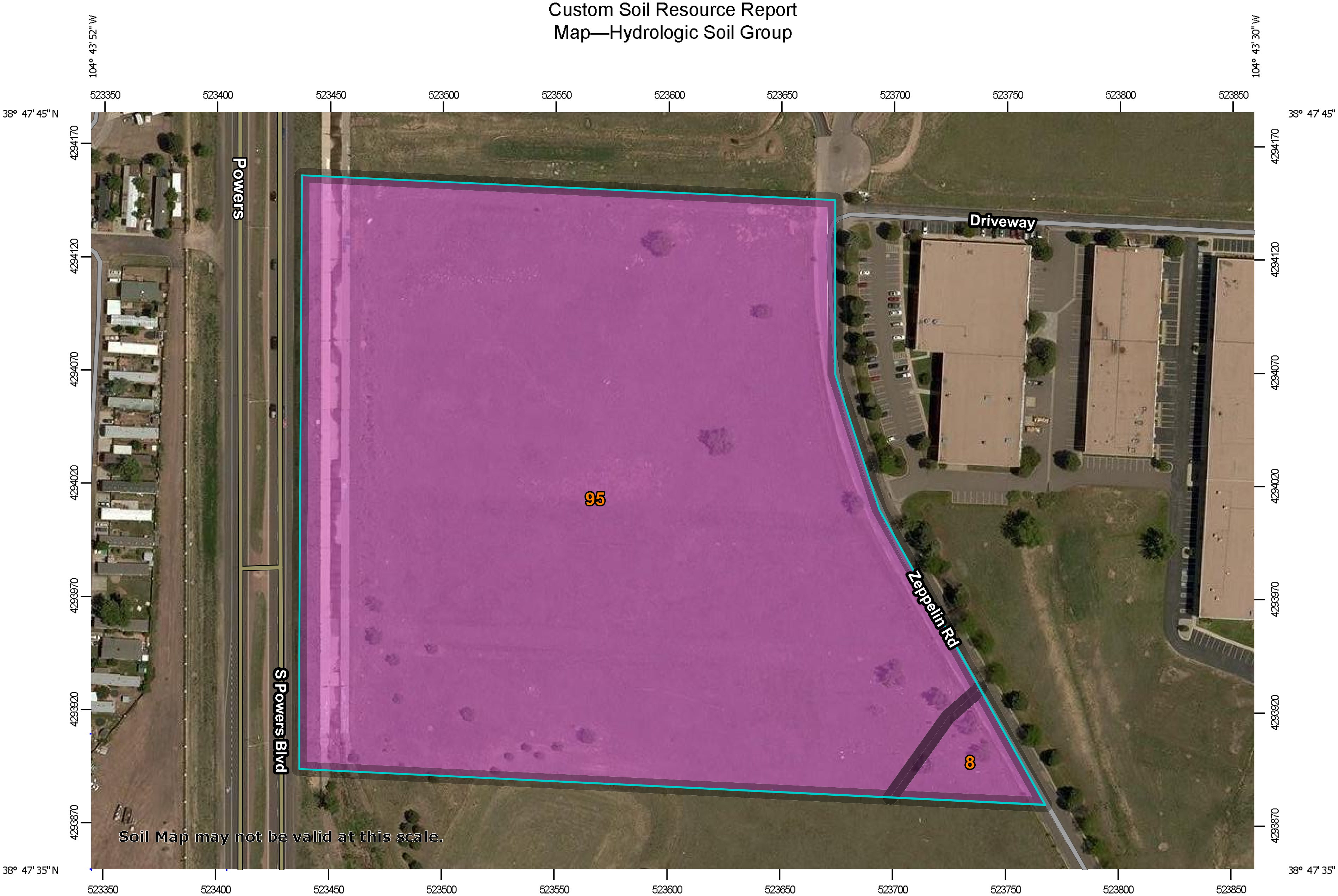
Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

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

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

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

Map—Hydrologic Soil Group



Map Scale: 1:2,360 if printed on A landscape (11" x 8.5") sheet.

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

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

Soil Rating Polygons

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

Soil Rating Lines

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

Soil Rating Points

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

Water Features

- Streams and Canals

Transportation

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

Background

- Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 16, Sep 10, 2018

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

Date(s) aerial images were photographed: Jun 3, 2014—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.

Table—Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
8	Blakeland loamy sand, 1 to 9 percent slopes	A	0.4	2.6%
95	Truckton loamy sand, 1 to 9 percent slopes	A	16.9	97.4%
Totals for Area of Interest			**17.3**	**100.0%**

Rating Options—Hydrologic Soil Group

Aggregation Method: Dominant Condition

Aggregation is the process by which a set of component attribute values is reduced to a single value that represents the map unit as a whole.

A map unit is typically composed of one or more "components". A component is either some type of soil or some nonsoil entity, e.g., rock outcrop. For the attribute being aggregated, the first step of the aggregation process is to derive one attribute value for each of a map unit's components. From this set of component attributes, the next step of the aggregation process derives a single value that represents the map unit as a whole. Once a single value for each map unit is derived, a thematic map for soil map units can be rendered. Aggregation must be done because, on any soil map, map units are delineated but components are not.

For each of a map unit's components, a corresponding percent composition is recorded. A percent composition of 60 indicates that the corresponding component typically makes up approximately 60% of the map unit. Percent composition is a critical factor in some, but not all, aggregation methods.

The aggregation method "Dominant Condition" first groups like attribute values for the components in a map unit. For each group, percent composition is set to the sum of the percent composition of all components participating in that group. These groups now represent "conditions" rather than components. The attribute value associated with the group with the highest cumulative percent composition is returned. If more than one group shares the highest cumulative percent composition, the corresponding "tie-break" rule determines which value should be returned. The "tie-break" rule indicates whether the lower or higher group value should be returned in the case of a percent composition tie. The result returned by this aggregation method represents the dominant condition throughout the map unit only when no tie has occurred.

Component Percent Cutoff: None Specified

Components whose percent composition is below the cutoff value will not be considered. If no cutoff value is specified, all components in the database will be considered. The data for some contrasting soils of minor extent may not be in the database, and therefore are not considered.

Tie-break Rule: Higher

The tie-break rule indicates which value should be selected from a set of multiple candidate values, or which value should be selected in the event of a percent composition tie.

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APPENDIX C – CIA CALCULATIONS AND WATER QUALITY BMP 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-BASIN	AREA (SF)	AREA (Acres)	ROOF AREA	ROOF IMPERVIOUSNESS	ROOF				LANDSCAPE AREA	LANDSCAPE IMPERVIOUSNESS	LANDSCAPE				PAVEMENT AREA	PAVEMENT IMPERVIOUSNESS	PAVEMENT				WEIGHTED IMPERVIOUSNESS	WEIGHTED COEFFICIENTS			
					C2	C5	C10	C100			C2	C5	C10	C100			C2	C5	C10	C100		C2	C5	C10	C100
E1	310,929	7.14	0	90%	0.71	0.73	0.75	0.81	300,159	2%	0.03	0.09	0.17	0.36	10,770	100%	0.89	0.90	0.92	0.96	5.4%	0.06	0.12	0.20	0.38
E2	327,347	7.51	0	90%	0.71	0.73	0.75	0.81	304,483	2%	0.03	0.09	0.17	0.36	22,864	100%	0.89	0.90	0.92	0.96	8.8%	0.09	0.15	0.22	0.40
OS-1	17,306	0.40	0	90%	0.71	0.73	0.75	0.81	17,306	2%	0.03	0.09	0.17	0.36	0	100%	0.89	0.90	0.92	0.96	2.0%	0.03	0.09	0.17	0.36
TOTAL	655,582	15.05	0	90%	0.71	0.73	0.75	0.81	621,948	2%	0.03	0.09	0.17	0.36	33,634	100%	0.89	0.90	0.92	0.96	7.0%	0.07	0.13	0.21	0.39

2520 and 2540 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.
1	E1	310,929	7.14	0.12	0	1.5%	0.0	935	1.5%	7.00	0.9	18.2	18.2	935	15.2	15.2
2	E2	327,347	7.51	0.15	0	1.8%	0.0	840	1.8%	7.00	0.9	14.9	14.9	840	14.7	14.7
3	OS-1	17,306	0.40	0.09	0	2.8%	0.0	45	2.8%	7.00	1.2	0.6	5.0	45	10.3	5.0

2520 and 2540 Zeppelin Road - Drainage Report
Existing Runoff Calculations *Design Storm 5 Year*
(Rational Method Procedure)

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
1	E1	7.14	0.12	15.2	0.84	3.38	2.85				2.85	
2	E2	7.51	0.15	14.7	1.10	3.44	3.79				3.79	
3	OS-1	0.40	0.09	5.0	0.04	5.09	0.18				0.18	

2520 and 2540 Zeppelin Road - Drainage Report
Existing Runoff Calculations
Design Storm 100 Year
(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				CUMULATIVE 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	
1	E1	7.14	0.38	15.2	2.72	5.69	15.45				15.45	
2	E2	7.51	0.40	14.7	3.02	5.78	17.44				17.44	
3	OS-1	0.40	0.36	5.0	0.14	8.55	1.22				1.22	

2520 and 2540 Zeppelin Road - Drainage Report
Existing 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	
1	E1	7.138	0.20	15.2	1.40	3.95	5.52				5.52	
2	E2	7.515	0.22	14.7	1.67	4.01	6.70	14.7	3.07	3.44	10.55	
3	OS-1	0.397	0.17	5.0	0.07	5.94	0.40				0.40	

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)
1	E1	7.14	2.85	15.45	2.85	15.45
2	E2	7.51	3.79	17.44	3.79	17.44

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

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	IMPERVIOUSNESS	C2	C5	C10	C100	AREA	IMPERVIOUSNESS	C2	C5	C10	C100	AREA	IMPERVIOUSNESS	C2	C5	C10	C100	IMPERVIOUSNESS	C2	C5	C10	C100
A1	27,102	0.62	0	90%	0.71	0.73	0.75	0.81	7,584	2%	0.03	0.09	0.17	0.36	19,518	100%	0.89	0.90	0.92	0.96	72.6%	0.65	0.67	0.71	0.79
A2	26,266	0.60	0	90%	0.71	0.73	0.75	0.81	6,459	2%	0.03	0.09	0.17	0.36	19,807	100%	0.89	0.90	0.92	0.96	75.9%	0.68	0.70	0.74	0.81
B1	132,338	3.04	52,549	90%	0.71	0.73	0.75	0.81	18,259	2%	0.03	0.09	0.17	0.36	61,530	100%	0.89	0.90	0.92	0.96	82.5%	0.70	0.72	0.75	0.82
B2	107,155	2.46	52,224	90%	0.71	0.73	0.75	0.81	1,704	2%	0.03	0.09	0.17	0.36	53,227	100%	0.89	0.90	0.92	0.96	93.6%	0.79	0.80	0.83	0.88
B3	118,492	2.72	52,238	90%	0.71	0.73	0.75	0.81	6,697	2%	0.03	0.09	0.17	0.36	59,557	100%	0.89	0.90	0.92	0.96	90.1%	0.76	0.78	0.80	0.86
B4	18,413	0.42	0	90%	0.71	0.73	0.75	0.81	17,622	2%	0.03	0.09	0.17	0.36	791	100%	0.89	0.90	0.92	0.96	6.2%	0.07	0.12	0.20	0.39
C1	103,103	2.37	33,368	90%	0.71	0.73	0.75	0.81	28,648	2%	0.03	0.09	0.17	0.36	41,087	100%	0.89	0.90	0.92	0.96	69.5%	0.59	0.62	0.66	0.74
C2	72,128	1.66	31,973	90%	0.71	0.73	0.75	0.81	19,138	2%	0.03	0.09	0.17	0.36	21,017	100%	0.89	0.90	0.92	0.96	69.6%	0.58	0.61	0.65	0.73
D1	22,254	0.51	0	90%	0.71	0.73	0.75	0.81	0	2%	0.03	0.09	0.17	0.36	22,254	100%	0.89	0.90	0.92	0.96	100.0%	0.89	0.90	0.92	0.96
D2	4,601	0.11	0	90%	0.71	0.73	0.75	0.81	4,601	2%	0.03	0.09	0.17	0.36	0	100%	0.89	0.90	0.92	0.96	2.0%	0.03	0.09	0.17	0.36
D3	12,714	0.29	0	90%	0.71	0.73	0.75	0.81	12,714	2%	0.03	0.09	0.17	0.36	0	100%	0.89	0.90	0.92	0.96	2.0%	0.03	0.09	0.17	0.36
TOTAL	644,566	14.80	222,352	90%	0.71	0.73	0.75	0.81	123,426	2%	0.03	0.09	0.17	0.36	298,788	100%	0.89	0.90	0.92	0.96	77.8%	0.66	0.69	0.72	0.79
CENTER POND (B1-B4)	376,398	8.64	157,011	90%	0.71	0.73	0.75	0.81	44,282	2%	0.03	0.09	0.17	0.36	175,105	100%	0.89	0.90	0.92	0.96	84.3%	0.71	0.73	0.76	0.83
SOUTH POND (C1-C2)	175,231	4.02	65,341	90%	0.71	0.73	0.75	0.81	47,786	2%	0.03	0.09	0.17	0.36	62,104	100%	0.89	0.90	0.92	0.96	69.5%	0.59	0.62	0.65	0.74

2520 and 2540 Zeppelin Road - Drainage Report
Proposed Runoff Calculations
Time of Concentration

Watercourse Coefficient

Forest & Meadow	2.50	Short Grass Pasture & Lawns	7.00
Fallow or Cultivation	5.00	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	27,102	0.62	0.67	68	1.3%	5.8	83	1.9%	20.00	2.7	0.5	6.3	151	10.8	6.3
A2	A2	26,266	0.60	0.70	51	3.0%	3.6	163	1.2%	20.00	2.2	1.2	5.0	214	11.2	5.0
B1	B1	132,338	3.04	0.72	81	1.2%	5.9	128	1.3%	20.00	2.3	0.9	6.8	209	11.2	6.8
B2	B2	107,155	2.46	0.80	82	1.1%	4.7	136	1.3%	20.00	2.3	1.0	5.7	219	11.2	5.7
B3	B3	118,492	2.72	0.78	69	0.7%	5.4	297	1.8%	20.00	2.7	1.8	7.2	366	12.0	7.2
B4	B4	18,413	0.42	0.12	30	9.7%	4.5	120	8.3%	15.00	4.3	0.5	5.0	149	10.8	5.0
C1	C1	103,103	2.37	0.62	68	1.3%	6.6	296	0.8%	20.00	1.8	2.8	9.4	364	12.0	9.4
C2	C2	72,128	1.66	0.61	38	1.1%	5.4	223	0.9%	20.00	1.9	2.0	7.4	261	11.4	7.4
D1	D1	22,254	0.51	0.90	9	48.4%	0.3	832	0.8%	20.00	1.8	7.8	8.1	841	14.7	8.1
D2	D2	4,601	0.11	0.09	45	4.4%	7.5	0	1.0%	15.00	1.5	0.0	7.5	45	10.3	7.5
D3	D3	12,714	0.29	0.09	30	6.7%	5.4	0	1.0%	15.00	1.5	0.0	5.4	30	10.2	5.4

2520 and 2540 Zeppelin Road - Drainage Report
Proposed Runoff Calculations

Design Storm *5 Year*

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	0.62	0.67	6.3	0.42	4.76	2.00	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.60	0.70	5.0	0.42	5.09	2.15	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
B1	B1	3.04	0.72	6.8	2.19	4.65	10.18	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
B2	B2	2.46	0.80	5.7	1.98	4.91	9.71	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
B3	B3	2.72	0.78	7.2	2.12	4.56	9.67	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
B4	B4	0.42	0.12	5.0	0.05	5.09	0.27	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
C1	C1	2.37	0.62	9.4	1.47	4.16	6.11	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
C2	C2	1.66	0.61	7.4	1.01	4.54	4.58	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.51	0.90	8.1	0.46	4.39	2.02	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.11	0.09	7.5	0.01	4.51	0.04	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.29	0.09	5.4	0.03	4.98	0.13	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

2520 and 2540 Zeppelin Road - Drainage Report

Proposed Runoff Calculations

Design Storm **100 Year**

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	0.62	0.79	6.3	0.49	8.00	3.94	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.60	0.81	5.0	0.49	8.55	4.19	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
B1	B1	3.04	0.82	6.8	2.48	7.81	19.40	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
B2	B2	2.46	0.88	5.7	2.16	8.25	17.79	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
B3	B3	2.72	0.86	7.2	2.34	7.67	17.93	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
B4	B4	0.42	0.39	5.0	0.16	8.55	1.39	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
C1	C1	2.37	0.74	9.4	1.76	6.99	12.33	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
C2	C2	1.66	0.73	7.4	1.22	7.62	9.26	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.51	0.96	8.1	0.49	7.37	3.61	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.11	0.36	7.5	0.04	7.57	0.29	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.29	0.36	5.4	0.11	8.37	0.88	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

2520 and 2540 Zeppelin Road - Drainage Report
Proposed Runoff Calculations **Design Storm** **10 Year**
(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	0.622	0.71	6.3	0.44	5.56	2.46	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.603	0.74	5.0	0.44	5.94	2.63	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
B1	B1	3.038	0.75	6.8	2.28	5.42	12.34	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
B2	B2	2.46	0.83	5.7	2.03	5.73	11.62	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
B3	B3	2.72	0.80	7.2	2.18	5.32	11.62	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
B4	B4	0.423	0.20	5.0	0.09	5.94	0.51	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
C1	C1	2.367	0.66	9.4	1.55	4.86	7.55	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
C2	C2	1.656	0.65	7.4	1.07	5.29	5.66	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.511	0.92	8.1	0.47	5.12	2.41	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.106	0.17	7.5	0.02	5.26	0.09	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.292	0.17	5.4	0.05	5.81	0.29	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

SUMMARY - PROPOSED RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)
A1	A1	0.62	2.00	3.94
A2	A2	0.60	2.15	4.19
B1	B1	3.04	10.18	19.40
B2	B2	2.46	9.71	17.79
B3	B3	2.72	9.67	17.93
B4	B4	0.42	0.27	1.39
C1	C1	2.37	6.11	12.33
C2	C2	1.66	4.58	9.26
D1	D1	0.51	2.02	3.61
D2	D2	0.11	0.04	0.29
D3	D3	0.29	0.13	0.88

Weighted Imperviousness Calculations (No Roof-Drains)

SUB-BASIN	AREA (SF)	AREA (Acres)	ROOF AREA	ROOF IMPERVIOUSNESS	ROOF				LANDSCAPE AREA	LANDSCAPE IMPERVIOUSNESS	LANDSCAPE				PAVEMENT AREA	PAVEMENT IMPERVIOUSNESS	PAVEMENT				WEIGHTED IMPERVIOUSNESS	WEIGHTED COEFFICIENTS			
					C2	C5	C10	C100			C2	C5	C10	C100			C2	C5	C10	C100		C2	C5	C10	C100
A1	27,102	0.62	0	90%	0.71	0.73	0.75	0.81	7,584	2%	0.03	0.09	0.17	0.36	19,518	100%	0.89	0.90	0.92	0.96	72.6%	0.65	0.67	0.71	0.79
A2	26,266	0.60	0	90%	0.71	0.73	0.75	0.81	6,459	2%	0.03	0.09	0.17	0.36	19,807	100%	0.89	0.90	0.92	0.96	75.9%	0.68	0.70	0.74	0.81
*B1	79,789	1.83	0	90%	0.71	0.73	0.75	0.81	18,259	2%	0.03	0.09	0.17	0.36	61,530	100%	0.89	0.90	0.92	0.96	77.6%	0.69	0.71	0.75	0.82
*B2	54,931	1.26	0	90%	0.71	0.73	0.75	0.81	1,704	2%	0.03	0.09	0.17	0.36	53,227	100%	0.89	0.90	0.92	0.96	97.0%	0.86	0.87	0.90	0.94
*B3	66,254	1.52	0	90%	0.71	0.73	0.75	0.81	6,697	2%	0.03	0.09	0.17	0.36	59,557	100%	0.89	0.90	0.92	0.96	90.1%	0.80	0.82	0.84	0.90
*B4	18,413	0.42	0	90%	0.71	0.73	0.75	0.81	17,622	2%	0.03	0.09	0.17	0.36	791	100%	0.89	0.90	0.92	0.96	6.2%	0.07	0.12	0.20	0.39
*C1	69,735	1.60	0	90%	0.71	0.73	0.75	0.81	28,648	2%	0.03	0.09	0.17	0.36	41,087	100%	0.89	0.90	0.92	0.96	59.7%	0.54	0.57	0.61	0.71
*C2	40,155	0.92	0	90%	0.71	0.73	0.75	0.81	19,138	2%	0.03	0.09	0.17	0.36	21,017	100%	0.89	0.90	0.92	0.96	53.3%	0.48	0.51	0.56	0.67
D1	22,254	0.51	0	90%	0.71	0.73	0.75	0.81	0	2%	0.03	0.09	0.17	0.36	22,254	100%	0.89	0.90	0.92	0.96	100.0%	0.89	0.90	0.92	0.96
D2	4,601	0.11	0	90%	0.71	0.73	0.75	0.81	4,601	2%	0.03	0.09	0.17	0.36	0	100%	0.89	0.90	0.92	0.96	2.0%	0.03	0.09	0.17	0.36
D3	12,714	0.29	0	90%	0.71	0.73	0.75	0.81	12,714	2%	0.03	0.09	0.17	0.36	0	100%	0.89	0.90	0.92	0.96	2.0%	0.03	0.09	0.17	0.36
TOTAL	422,214	9.69	0	90%	0.71	0.73	0.75	0.81	123,426	2%	0.03	0.09	0.17	0.36	298,788	100%	0.89	0.90	0.92	0.96	71.4%	0.64	0.66	0.70	0.78
CENTER POND (B1-B4)	219,387	5.04	0	90%	0.71	0.73	0.75	0.81	44,282	2%	0.03	0.09	0.17	0.36	175,105	100%	0.89	0.90	0.92	0.96	80.2%	0.72	0.74	0.77	0.84
SOUTH POND (C1-C2)	109,890	2.52	0	90%	0.71	0.73	0.75	0.81	47,786	2%	0.03	0.09	0.17	0.36	62,104	100%	0.89	0.90	0.92	0.96	57.4%	0.52	0.55	0.59	0.70

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes.
The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the sizing the inlets.

2520 and 2540 Zeppelin Road - Drainage Report
Proposed Runoff Calculations (No Roof-Drains)
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.
A1	A1	27,102	0.62	0.67	68	1.3%	5.8	83	1.9%	20.00	2.7	0.5	6.3	151	10.8	6.3
A2	A2	26,266	0.60	0.70	51	3.0%	3.6	163	1.2%	20.00	2.2	1.2	5.0	214	11.2	5.0
*B1	*B1	79,789	1.83	0.71	81	1.2%	6.0	128	1.3%	20.00	2.3	0.9	6.9	209	11.2	6.9
*B2	*B2	54,931	1.26	0.87	82	1.1%	3.6	136	1.3%	20.00	2.3	1.0	5.0	219	11.2	5.0
*B3	*B3	66,254	1.52	0.82	69	0.7%	4.8	297	1.8%	20.00	2.7	1.8	6.6	366	12.0	6.6
*B4	*B4	18,413	0.42	0.12	30	9.7%	4.5	120	8.3%	15.00	4.3	0.5	5.0	149	10.8	5.0
*C1	*C1	69,735	1.60	0.57	68	1.3%	7.3	296	0.8%	20.00	1.8	2.8	10.1	364	12.0	10.1
*C2	*C2	40,155	0.92	0.51	38	1.1%	6.5	223	0.9%	20.00	1.9	2.0	8.5	261	11.4	8.5
D1	D1	22,254	0.51	0.90	9	48.4%	0.3	832	0.8%	20.00	1.8	7.8	8.1	841	14.7	8.1
D2	D2	4,601	0.11	0.09	45	4.4%	7.5	0	1.0%	15.00	1.5	0.0	7.5	45	10.3	7.5
D3	D3	12,714	0.29	0.09	30	6.7%	5.4	0	1.0%	15.00	1.5	0.0	5.4	30	10.2	5.4

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes. The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the size of the inlets.

2520 and 2540 Zeppelin Road - Drainage Report
Proposed Runoff Calculations **(No Roof-Drains)** ***Design Storm 5 Year***
(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	NOTES
A1	A1	0.62	0.67	6.3	0.42	4.76	2.00	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.60	0.70	5.0	0.42	5.09	2.15	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
*B1	*B1	1.83	0.71	6.9	1.31	4.63	6.06	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
*B2	*B2	1.26	0.87	5.0	1.10	5.09	5.61	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
*B3	*B3	1.52	0.82	6.6	1.24	4.69	5.84	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
*B4	*B4	0.42	0.12	5.0	0.05	5.09	0.27	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
*C1	*C1	1.60	0.57	10.1	0.91	4.05	3.68	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
*C2	*C2	0.92	0.51	8.5	0.47	4.32	2.05	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.51	0.90	8.1	0.46	4.39	2.02	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.11	0.09	7.5	0.01	4.51	0.04	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.29	0.09	5.4	0.03	4.98	0.13	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes. The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the size of the inlets.

2520 and 2540 Zeppelin Road - Drainage Report

Proposed Runoff Calculations **(No Roof-Drains)** ***Design Storm 100 Year***

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				NOTES
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	
A1	A1	0.62	0.79	6.3	0.49	8.00	3.94	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.60	0.81	5.0	0.49	8.55	4.19	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
*B1	*B1	1.83	0.82	6.9	1.51	7.77	11.71	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
*B2	*B2	1.26	0.94	5.0	1.19	8.55	10.15	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
*B3	*B3	1.52	0.90	6.6	1.37	7.88	10.78	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
*B4	*B4	0.42	0.39	5.0	0.16	8.55	1.39	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
*C1	*C1	1.60	0.71	10.1	1.14	6.80	7.77	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
*C2	*C2	0.92	0.67	8.5	0.62	7.26	4.51	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.51	0.96	8.1	0.49	7.37	3.61	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.11	0.36	7.5	0.04	7.57	0.29	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.29	0.36	5.4	0.11	8.37	0.88	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes. The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the size of the inlets.

2520 and 2540 Zeppelin Road - Drainage Report

Proposed Runoff Calculations **(No Roof-Drains)** **Design Storm** **10 Year**

(Rational Method Procedure)

BASIN INFORMATION				DIRECT RUNOFF				
DESIGN POINT	DRAIN BASIN	AREA ac.	RUNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs	NOTES
A1	A1	0.622	0.71	6.3	0.44	5.56	2.46	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A1.
A2	A2	0.603	0.74	5.0	0.44	5.94	2.63	Flows convey to a rain garden. Overflow stormwater outfalls via a Private Type C Inlet and 18" HDPE Pipe at Design Point A2.
*B1	*B1	1.832	0.75	6.9	1.37	5.40	7.40	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B1.
*B2	*B2	1.261	0.90	5.0	1.13	5.94	6.71	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B2.
*B3	*B3	1.521	0.84	6.6	1.28	5.47	7.03	Flows convey to a Private Double Type 13 Inlet and 30" HDPE Pipe at Design Point B3.
*B4	*B4	0.423	0.20	5.0	0.09	5.94	0.51	Flows convey to the bottom of the center extended detention basin and then to the basin outlet structure at Design Point B4.
*C1	*C1	1.601	0.61	10.1	0.98	4.72	4.63	Flows convey to a Private Type R Inlet and 24" HDPE Pipe at Design Point C1.
*C2	*C2	0.922	0.56	8.5	0.52	5.04	2.61	Flows convey to a Private Type C Inlet and 24" HDPE Pipe at Design Point C2.
D1	D1	0.511	0.92	8.1	0.47	5.12	2.41	Offsite flows, directly into concrete swale west of site, outfall at Design Point D1.
D2	D2	0.106	0.17	7.5	0.02	5.26	0.09	Offsite additional flows at Design Point D2, which enters the rain garden at Design Point A1.
D3	D3	0.292	0.17	5.4	0.05	5.81	0.29	Offsite additional flows at Design Point D3, which enters the rain garden at Design Point A2.

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes. The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the size of the inlets.

SUMMARY - PROPOSED RUNOFF TABLE (No Roof-Drains)

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)
A1	A1	0.62	2.00	3.94
A2	A2	0.60	2.15	4.19
*B1	*B1	1.83	6.06	11.71
*B2	*B2	1.26	5.61	10.15
*B3	*B3	1.52	5.84	10.78
*B4	*B4	0.42	0.27	1.39
*C1	*C1	1.60	3.68	7.77
*C2	*C2	0.92	2.05	4.51
D1	D1	0.51	2.02	3.61
D2	D2	0.11	0.04	0.29
D3	D3	0.29	0.13	0.88

*Sub-Basins marked have been revised to only include the areas and resultant flows that will enter the storm drain inlets through the grates of each inlet. These values are used for inlet sizing purposes. The building roof drains have been removed from these spreadsheets all roof drains will be piped underground directly to the storm drain pipes flows from these areas should not impact the size of the inlets.

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
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Northeast Private Water Quality-Only Rain Garden (Sub-Basin A1)

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	2-Year Event	1.19	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	NRCS Method		
(NRCS Type II Method) 24-Hour Storm Event and Rainfall Depth for User Defined Storm	100-Year Event		
Max Intensity for Optional User Defined Storm	0		

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	**A1**													
Receiving Pervious Area Soil Type	Loamy Sand													
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	0.620													
Directly Connected Impervious Area (DCIA, acres)	0.446													
Unconnected Impervious Area (UIA, acres)	0.000													
Receiving Pervious Area (RPA, acres)	0.000													
Separate Pervious Area (SPA, acres)	0.174													
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	C													

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	0.620													
Directly Connected Impervious Area (DCIA, %)	71.9%													
Unconnected Impervious Area (UIA, %)	0.0%													
Receiving Pervious Area (RPA, %)	0.0%													
Separate Pervious Area (SPA, %)	28.1%													
A_R (RPA / UIA)	0.000													
I_a Check	1.000													
f / I for WQCV Event:	3.2													
f / I for 2-Year Event:	0.6													
f / I for 100-Year Event:	0.4													
f / I for Optional User Defined Storm NRCS Method:														
IRF for WQCV Event:	1.00													
IRF for 2-Year Event:	1.00													
IRF for 100-Year Event:	1.00													
IRF for Optional User Defined Storm NRCS Method:														
Total Site Imperviousness: I_{total}	71.9%													
Effective Imperviousness for WQCV Event:	71.9%													
Effective Imperviousness for 2-Year Event:	71.9%													
Effective Imperviousness for 100-Year Event:	71.9%													
Effective Imperviousness for Optional User Defined Storm NRCS Method:														

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV 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
This line only for 10-Year Event	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
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 NRCS Method CREDIT: Reduce Detention By:														

Total Site Imperviousness:	**71.9%**
Total Site Effective Imperviousness for WQCV Event:	**71.9%**
Total Site Effective Imperviousness for 2-Year Event:	**71.9%**
Total Site Effective Imperviousness for 100-Year Event:	**71.9%**
Total Site Effective Imperviousness for Optional User Defined Storm NRCS Method:	

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

Design Procedure Form: Rain Garden (RG)

UD-BMP (Version 3.06, November 2016)

Designer: Mitchell Hess
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Northeast Private Water Quality-Only Rain Garden (Sub-Basin A1)

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a (100% if all paved and roofed areas upstream of rain garden)	I_a = 71.9 %
B) Tributary Area's Imperviousness Ratio ($i = I_a/100$)	i = 0.719
C) Water Quality Capture Volume (WQCV) for a 12-hour Drain Time (WQCV= 0.8 * (0.91* i^3 - 1.19 * i^2 + 0.78 * i)	WQCV = 0.23 watershed inches
D) Contributing Watershed Area (including rain garden area)	Area = 27,007 sq ft
E) Water Quality Capture Volume (WQCV) Design Volume Vol = (WQCV / 12) * Area	V_{WQCV} = ____ cu ft
F) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = 0.43 in
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume	$V_{WQCV\ OTHER}$ = 511.1 cu ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{WQCV\ USER}$ = ____ cu ft
2. Basin Geometry	
A) WQCV Depth (12-inch maximum)	D_{WQCV} = 12 in
B) Rain Garden Side Slopes (Z = 4 min., horiz. dist per unit vertical) (Use "0" if rain garden has vertical walls)	Z = 0.00 ft / ft
C) Mimimum Flat Surface Area	A_{Min} = 388 sq ft
D) Actual Flat Surface Area	A_{Actual} = 489 sq ft
E) Area at Design Depth (Top Surface Area)	A_{Top} = 489 sq ft
F) Rain Garden Total Volume (V_T= ((A_{Top} + A_{Actual}) / 2) * Depth)	V_T= 489 cu ft
3. Growing Media	Choose One (●) 18" Rain Garden Growing Media ( ) Other (Explain):
4. Underdrain System	
A) Are underdrains provided?	Choose One (●) YES ( ) NO
B) Underdrain system orifice diameter for 12 hour drain time	
i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y = 1.8 ft
ii) Volume to Drain in 12 Hours	Vol_{12} = 511 cu ft
iii) Orifice Diameter, 3/8" Minimum	D_O = 1/2 in

Design Procedure Form: Rain Garden (RG)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**Northeast Private Water Quality-Only Rain Garden (Sub-Basin A1)**

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

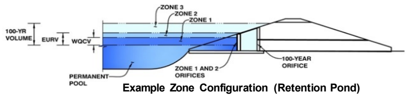
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: Northeast Private Water Quality-Only Rain Garden (Sub-Basin A1)



Example Zone Configuration (Retention Pond)

Required Volume Calculation

			Optional User Override 1-hr Precipitation	
Selected BMP Type =	**RG**			
Watershed Area =	0.62	acres		
Watershed Length =	340	ft		
Watershed Slope =	0.020	ft/ft		
Watershed Imperviousness =	71.90%	percent		
Percentage Hydrologic Soil Group A =	100.0%	percent		
Percentage Hydrologic Soil Group B =	0.0%	percent		
Percentage Hydrologic Soil Groups C/D =	0.0%	percent		
Desired WQCV Drain Time =	12.0	hours		
Location for 1-hr Rainfall Depths =	User Input			
Water Quality Capture Volume (WQCV) =	0.012	acre-feet		
Excess Urban Runoff Volume (EURV) =	0.057	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.039	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.051	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.062	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.074	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.087	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.102	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	0.037	acre-feet		
Approximate 5-yr Detention Volume =	0.048	acre-feet		
Approximate 10-yr Detention Volume =	0.058	acre-feet		
Approximate 25-yr Detention Volume =	0.070	acre-feet		
Approximate 50-yr Detention Volume =	0.076	acre-feet		
Approximate 100-yr Detention Volume =	0.083	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.012	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 =	0.012	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 = [ ] 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)
Media Surface	--	0.00	--	--	--	504	0.012		
	--	0.10	--	--	--	504	0.012	50	0.001
	--	0.20	--	--	--	504	0.012	96	0.002
	--	0.30	--	--	--	504	0.012	146	0.003
	--	0.40	--	--	--	504	0.012	197	0.005
	--	0.50	--	--	--	504	0.012	247	0.006
	--	0.60	--	--	--	504	0.012	297	0.007
	--	0.70	--	--	--	504	0.012	348	0.008
	--	0.80	--	--	--	504	0.012	398	0.009
	--	0.90	--	--	--	504	0.012	449	0.010
	--	1.00	--	--	--	504	0.012	499	0.011
	--	1.03	--	--	--	504	0.012	514	0.012
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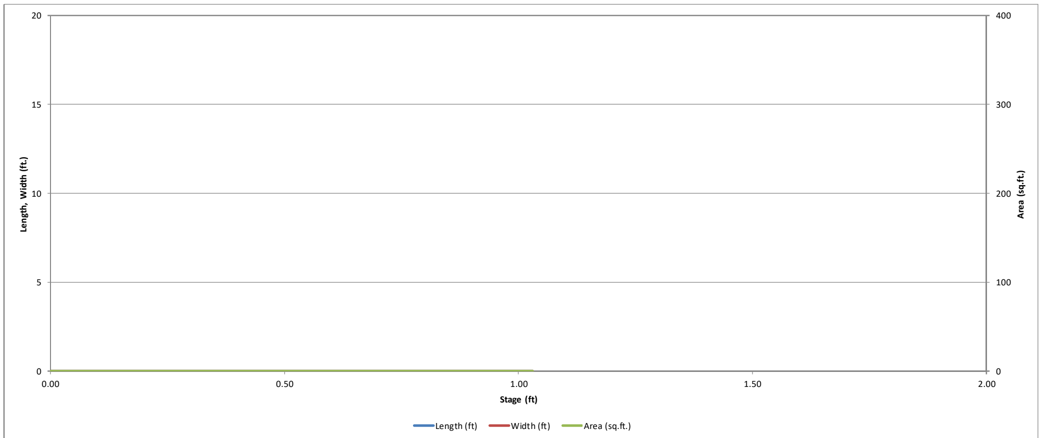
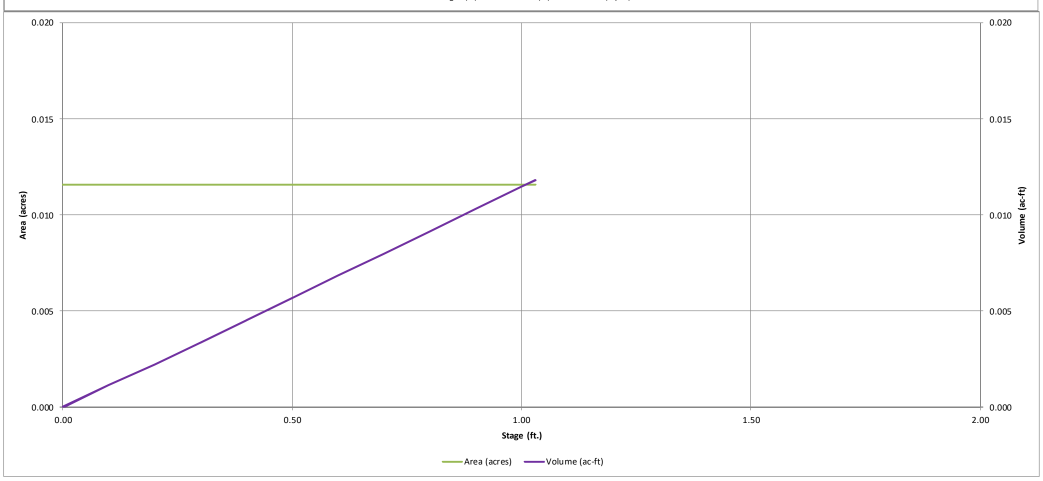
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)





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
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Northwest Private Water Quality-Only Rain Garden (Sub-Basin A2)

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	2-Year Event	1.19	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	NRCS Method		
(NRCS Type II Method) 24-Hour Storm Event and Rainfall Depth for User Defined Storm	100-Year Event		
Max Intensity for Optional User Defined Storm	0		

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	**A2**													
Receiving Pervious Area Soil Type	Loamy Sand													
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	0.600													
Directly Connected Impervious Area (DCIA, acres)	0.452													
Unconnected Impervious Area (UIA, acres)	0.000													
Receiving Pervious Area (RPA, acres)	0.000													
Separate Pervious Area (SPA, acres)	0.148													
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	C													

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	0.600													
Directly Connected Impervious Area (DCIA, %)	75.3%													
Unconnected Impervious Area (UIA, %)	0.0%													
Receiving Pervious Area (RPA, %)	0.0%													
Separate Pervious Area (SPA, %)	24.7%													
A_R (RPA / UIA)	0.000													
I_a Check	1.000													
f / I for WQCV Event:	3.2													
f / I for 2-Year Event:	0.6													
f / I for 100-Year Event:	0.4													
f / I for Optional User Defined Storm NRCS Method:														
IRF for WQCV Event:	1.00													
IRF for 2-Year Event:	1.00													
IRF for 100-Year Event:	1.00													
IRF for Optional User Defined Storm NRCS Method:														
Total Site Imperviousness: I_{total}	75.3%													
Effective Imperviousness for WQCV Event:	75.3%													
Effective Imperviousness for 2-Year Event:	75.3%													
Effective Imperviousness for 100-Year Event:	75.3%													
Effective Imperviousness for Optional User Defined Storm NRCS Method:														

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV 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
This line only for 10-Year Event	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
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 NRCS Method CREDIT: Reduce Detention By:														

Total Site Imperviousness:	**75.3%**
Total Site Effective Imperviousness for WQCV Event:	**75.3%**
Total Site Effective Imperviousness for 2-Year Event:	**75.3%**
Total Site Effective Imperviousness for 100-Year Event:	**75.3%**
Total Site Effective Imperviousness for Optional User Defined Storm NRCS Method:	

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

Design Procedure Form: Rain Garden (RG)

UD-BMP (Version 3.06, November 2016) **Sheet 1 of 2**

Designer: Mitchell Hess
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Northwest Private Water Quality-Only Rain Garden (Sub-Basin A2)

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a (100% if all paved and roofed areas upstream of rain garden)	I_a = 75.3 %
B) Tributary Area's Imperviousness Ratio ($i = I_a/100$)	i = 0.753
C) Water Quality Capture Volume (WQCV) for a 12-hour Drain Time (WQCV= 0.8 * (0.91* i^3 - 1.19 * i^2 + 0.78 * i)	WQCV = 0.24 watershed inches
D) Contributing Watershed Area (including rain garden area)	Area = 26,226 sq ft
E) Water Quality Capture Volume (WQCV) Design Volume Vol = (WQCV / 12) * Area	V_{WQCV} = cu ft
F) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = 0.43 in
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume	$V_{WQCV\ OTHER}$ = 526.5 cu ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{WQCV\ USER}$ = cu ft
2. Basin Geometry	
A) WQCV Depth (12-inch maximum)	D_{WQCV} = 12 in
B) Rain Garden Side Slopes (Z = 4 min., horiz. dist per unit vertical) (Use "0" if rain garden has vertical walls)	Z = 0.00 ft / ft
C) Mimimum Flat Surface Area	A_{Min} = 395 sq ft
D) Actual Flat Surface Area	A_{Actual} = 664 sq ft
E) Area at Design Depth (Top Surface Area)	A_{Top} = 664 sq ft
F) Rain Garden Total Volume (V_T= ((A_{Top} + A_{Actual}) / 2) * Depth)	V_T= 664 cu ft
3. Growing Media	Choose One ◉ 18" Rain Garden Growing Media ○ Other (Explain):
4. Underdrain System	
A) Are underdrains provided?	Choose One ◉ YES ○ NO
B) Underdrain system orifice diameter for 12 hour drain time	
i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y = 1.8 ft
ii) Volume to Drain in 12 Hours	Vol_{12} = 526 cu ft
iii) Orifice Diameter, 3/8" Minimum	D_O = 1/2 in

Design Procedure Form: Rain Garden (RG)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**Northwest Private Water Quality-Only Rain Garden (Sub-Basin A2)**

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

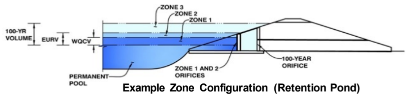
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: Northwest Private Water Quality-Only Rain Garden (Sub-Basin A2)



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	RG			
Watershed Area =	0.60	acres		
Watershed Length =	305	ft		
Watershed Slope =	0.008	ft/ft		
Watershed Imperviousness =	75.30%	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 =	12.0	hours		
Location for 1-hr Rainfall Depths =	User Input			
Water Quality Capture Volume (WQCV) =	0.012	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.058	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.040	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.053	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.064	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.076	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.088	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.103	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	0.038	acre-feet		
Approximate 5-yr Detention Volume =	0.050	acre-feet		
Approximate 10-yr Detention Volume =	0.060	acre-feet		
Approximate 25-yr Detention Volume =	0.071	acre-feet		
Approximate 50-yr Detention Volume =	0.078	acre-feet		
Approximate 100-yr Detention Volume =	0.084	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.012	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 =	0.012	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 = 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)
Media Surface	--	0.00	--	--	--	665	0.015		
	--	0.10	--	--	--	665	0.015	67	0.002
	--	0.20	--	--	--	665	0.015	126	0.003
	--	0.30	--	--	--	665	0.015	193	0.004
	--	0.40	--	--	--	665	0.015	259	0.006
	--	0.50	--	--	--	665	0.015	326	0.007
	--	0.60	--	--	--	665	0.015	392	0.009
	--	0.70	--	--	--	665	0.015	459	0.011
	--	0.80	--	--	--	665	0.015	525	0.012
	--	0.90	--	--	--	665	0.015	592	0.014
	--	1.00	--	--	--	665	0.015	658	0.015
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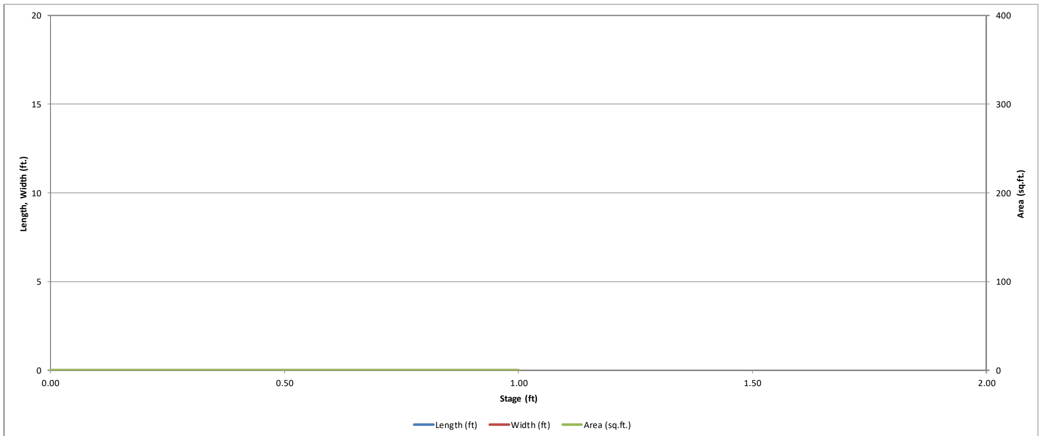
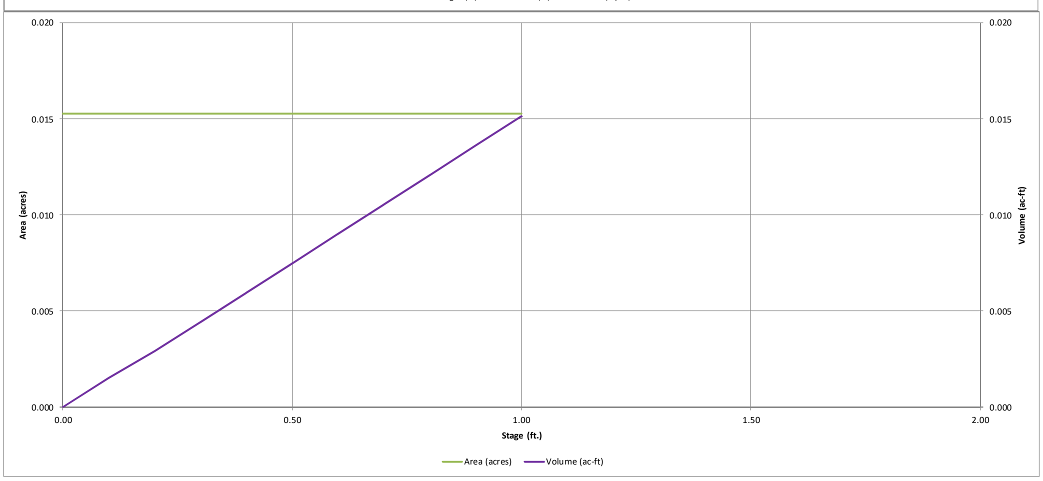
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)





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

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	2-Year Event	1.19	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	NRCS Method		
(NRCS Type II Method) 24-Hour Storm Event and Rainfall Depth for User Defined Storm	100-Year Event		
Max Intensity for Optional User Defined Storm	0		

Designer: Eric Gunderson
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	B1	B2	B3	B4										
Receiving Pervious Area Soil Type	Loamy Sand	Loamy Sand	Loamy Sand	Loamy Sand										
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	3.040	2.460	2.720	0.423										
Directly Connected Impervious Area (DCIA, acres)	2.589	2.420	2.570	0.018										
Unconnected Impervious Area (UIA, acres)	0.031	0.000	0.000	0.000										
Receiving Pervious Area (RPA, acres)	0.000	0.000	0.000	0.405										
Separate Pervious Area (SPA, acres)	0.420	0.040	0.150	0.000										
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	C	C	C	V										

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	3.040	2.460	2.720	0.423										
Directly Connected Impervious Area (DCIA, %)	85.2%	98.4%	94.5%	4.3%										
Unconnected Impervious Area (UIA, %)	1.0%	0.0%	0.0%	0.0%										
Receiving Pervious Area (RPA, %)	0.0%	0.0%	0.0%	95.7%										
Separate Pervious Area (SPA, %)	13.8%	1.6%	5.5%	0.0%										
A_R (RPA / UIA)	0.000	0.000	0.000	0.000										
I_a Check	1.000	1.000	1.000	1.000										
f / I for WQCV Event:	3.2	3.2	3.2	3.2										
f / I for 2-Year Event:	0.6	0.6	0.6	0.6										
f / I for 100-Year Event:	0.4	0.4	0.4	0.4										
f / I for Optional User Defined Storm NRCS Method:														
IRF for WQCV Event:	1.00	1.00	1.00	0.00										
IRF for 2-Year Event:	1.00	1.00	1.00	1.00										
IRF for 100-Year Event:	1.00	1.00	1.00	1.00										
IRF for Optional User Defined Storm NRCS Method:														
Total Site Imperviousness: I_{total}	86.2%	98.4%	94.5%	4.3%										
Effective Imperviousness for WQCV Event:	86.2%	98.4%	94.5%	4.3%										
Effective Imperviousness for 2-Year Event:	86.2%	98.4%	94.5%	4.3%										
Effective Imperviousness for 100-Year Event:	86.2%	98.4%	94.5%	4.3%										
Effective Imperviousness for Optional User Defined Storm NRCS Method:														

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV Event CREDIT: Reduce Detention By:	0.0%	0.0%	0.0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
This line only for 10-Year Event	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
100-Year Event CREDIT**: Reduce Detention By:	0.0%	0.0%	0.0%	2.4%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
User Defined NRCS Method CREDIT: Reduce Detention By:														

Total Site Imperviousness:	**88.3%**
Total Site Effective Imperviousness for WQCV Event:	**88.3%**
Total Site Effective Imperviousness for 2-Year Event:	**88.3%**
Total Site Effective Imperviousness for 100-Year Event:	**88.3%**
Total Site Effective Imperviousness for Optional User Defined Storm NRCS Method:	

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

Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.06, November 2016)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)**

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a	I_a = 88.3 %
B) Tributary Area's Imperviousness Ratio (i = I_a / 100)	i = 0.883
C) Contributing Watershed Area	Area = 8.640 ac
D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = 0.43 in
E) Design Concept (Select EURV when also designing for flood control)	Choose One: (●) Water Quality Capture Volume (WQCV); ( ) Excess Urban Runoff Volume (EURV)
F) Design Volume (WQCV) Based on 40-hour Drain Time (V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area)	V_{DESIGN}= 0.279 ac-ft
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume ($V_{WQCV\ OTHER}$ = (d_6*(V_{DESIGN}/0.43))	$V_{DESIGN\ OTHER}$= 0.279 ac-ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{DESIGN\ USER}$= ac-ft
I) Predominant Watershed NRCS Soil Group	Choose One: ( ) A; ( ) B; ( ) C / D — **WQCV selected. Soil group not required.**
J) Excess Urban Runoff Volume (EURV) Design Volume For HSG A: $EURV_A = 1.68 * i^{1.28}$ For HSG B: $EURV_B = 1.36 * i^{1.08}$ For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$	EURV = ac-f t
2. Basin Shape: Length to Width Ratio (A basin length to width ratio of at least 2:1 will improve TSS reduction.)	L : W = 4.5 : 1
3. Basin Side Slopes	
A) Basin Maximum Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred)	Z = 4.00 ft / ft
4. Inlet	
A) Describe means of providing energy dissipation at concentrated inflow locations:	A concrete forebay with concrete baffle blocks will be used for energy dissipation.

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Mitchell Hess
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)

5. Forebay

A) Minimum Forebay Volume (V_{FMIN} = 3% of the WQCV) — V_{FMIN} = 0.008 ac-ft

B) Actual Forebay Volume — V_F = 0.009 ac-ft

C) Forebay Depth (D_F = 18 inch maximum) — D_F = 18.0 in

D) Forebay Discharge

i) Undetained 100-year Peak Discharge — Q_{100} = 55.12 cfs

ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$) — Q_F = 1.10 cfs

E) Forebay Discharge Design

Choose One
- ○ Berm With Pipe
- ◉ Wall with Rect. Notch
- ○ Wall with V-Notch Weir

(flow too small for berm w/ pipe)

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

G) Rectangular Notch Width — Calculated W_N = 5.8 in

6. Trickle Channel

A) Type of Trickle Channel

Choose One
- ◉ Concrete
- ○ Soft Bottom

F) Slope of Trickle Channel — S = 0.0050 ft / ft

7. Micropool and Outlet Structure

A) Depth of Micropool (2.5-feet minimum) — D_M = 2.5 ft

B) Surface Area of Micropool (10 ft^2 minimum) — A_M = 16 sq ft

C) Outlet Type

Choose One
- ◉ Orifice Plate
- ○ Other (Describe):

D) Smallest Dimension of Orifice Opening Based on Hydrograph Routing (Use UD-Detention) — $D_{orifice}$ = 1.06 inches

E) Total Outlet Area — A_{ot} = 3.44 square inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)**

8. Initial Surcharge Volume			
A) Depth of Initial Surcharge Volume (Minimum recommended depth is 4 inches)	D_{IS} =	4	in
B) Minimum Initial Surcharge Volume (Minimum volume of 0.3% of the WQCV)	V_{IS} =	36.5	cu ft
C) Initial Surcharge Provided Above Micropool	V_s=	5.3	cu ft
9. Trash Rack			
A) Water Quality Screen Open Area: $A_t = A_{ot} * 38.5*(e^{-0.095D})$	A_t =	120	square inches
B) Type of Screen (If specifying an alternative to the materials recommended in the USDCM, indicate "other" and enter the ratio of the total open are to the total screen are for the material specified.) Other (Y/N): N		*S.S. Well Screen with 60% Open Area*	
C) Ratio of Total Open Area to Total Area (only for type 'Other')	User Ratio =		
D) Total Water Quality Screen Area (based on screen type)	A_{total} =	200	sq. in.
E) Depth of Design Volume (EURV or WQCV) (Based on design concept chosen under 1E)	H=	2.8	feet
F) Height of Water Quality Screen (H_{TR})	H_{TR}=	61.6	inches
G) Width of Water Quality Screen Opening ($W_{opening}$) (Minimum of 12 inches is recommended)	$W_{opening}$ =	12.0	inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)**

10. Overflow Embankment A) Describe embankment protection for 100-year and greater overtopping: B) Slope of Overflow Embankment (Horizontal distance per unit vertical, 4:1 or flatter preferred)	Overflow is for anything above the Water Quality WSEL as this is a Water Quality-Only EDB Embankment protection will consist of 2-ft deep Type L Riprap 4.00
11. Vegetation	Choose One ◉ Irrigated ○ Not Irrigated AVOID PLACING IRRIGATION HEADS IN THE BOTTOM OF THE BASIN
12. Access A) Describe Sediment Removal Procedures	An access has been provided for the detention pond so vehicles can access the bottom of the pond to maintain the forebay, trickle channel and outlet structure.

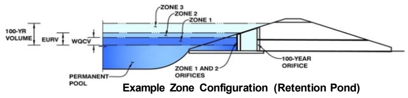
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: Center Private Water Quality-Only Extended Detention Basin (Sub-Basins B1-B4)



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**				
Watershed Area =	8.64	acres	**Note: L / W Ratio < 1**		
Watershed Length =	510	ft	**L / W Ratio = 0.7**		
Watershed Slope =	0.014	ft/ft			
Watershed Imperviousness =	88.30%	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 =	User Input				
Water Quality Capture Volume (WQCV) =	0.279	acre-feet	Optional User Override 1-hr Precipitation		
Excess Urban Runoff Volume (EURV) =	1.032	acre-feet			
2-yr Runoff Volume (P1 = 1.19 in.) =	0.716	acre-feet		1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.928	acre-feet		1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	1.114	acre-feet		1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	1.306	acre-feet		2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	1.481	acre-feet		2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	1.698	acre-feet		2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet			inches
Approximate 2-yr Detention Volume =	0.680	acre-feet			
Approximate 5-yr Detention Volume =	0.882	acre-feet			
Approximate 10-yr Detention Volume =	1.048	acre-feet			
Approximate 25-yr Detention Volume =	1.237	acre-feet			
Approximate 50-yr Detention Volume =	1.347	acre-feet			
Approximate 100-yr Detention Volume =	1.443	acre-feet			

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.279	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 =	0.279	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 Increement = [ ] ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	0	0.000		
	--	0.10	--	--	--	197	0.005	10	0.000
	--	0.20	--	--	--	359	0.008	34	0.001
	--	0.30	--	--	--	485	0.011	75	0.002
	--	0.40	--	--	--	607	0.014	128	0.003
	--	0.50	--	--	--	744	0.017	195	0.004
	--	0.60	--	--	--	1,063	0.024	282	0.006
	--	0.70	--	--	--	1,263	0.029	396	0.009
	--	0.80	--	--	--	1,403	0.032	528	0.012
	--	0.90	--	--	--	1,558	0.036	675	0.015
	--	1.00	--	--	--	1,734	0.040	837	0.019
	--	1.10	--	--	--	1,975	0.045	1,020	0.023
	--	1.20	--	--	--	2,198	0.050	1,227	0.028
	--	1.30	--	--	--	2,405	0.055	1,455	0.033
	--	1.40	--	--	--	2,607	0.060	1,704	0.039
	--	1.50	--	--	--	2,805	0.064	1,972	0.045
	--	1.60	--	--	--	3,005	0.069	2,261	0.052
	--	1.70	--	--	--	3,209	0.074	2,569	0.059
	--	1.80	--	--	--	3,423	0.079	2,899	0.067
	--	1.90	--	--	--	3,651	0.084	3,250	0.075
	--	2.00	--	--	--	3,916	0.090	3,626	0.083
	--	2.10	--	--	--	4,200	0.096	4,071	0.093
	--	2.20	--	--	--	4,385	0.101	4,500	0.103
	--	2.30	--	--	--	4,583	0.105	4,948	0.114
	--	2.40	--	--	--	4,792	0.110	5,417	0.124
	--	2.50	--	--	--	5,011	0.115	5,907	0.136
	--	2.60	--	--	--	5,240	0.120	6,420	0.147
	--	2.70	--	--	--	5,478	0.126	6,956	0.160
	--	2.80	--	--	--	5,730	0.132	7,516	0.173
	--	2.90	--	--	--	6,022	0.138	8,104	0.186
	--	3.00	--	--	--	6,394	0.147	8,725	0.200
	--	3.10	--	--	--	6,713	0.154	9,380	0.215
	--	3.20	--	--	--	6,952	0.160	10,063	0.231
	--	3.30	--	--	--	7,153	0.164	10,768	0.247
	--	3.40	--	--	--	7,356	0.169	11,494	0.264
WQCV WSEL	--	3.50	--	--	--	7,561	0.174	12,240	0.281
	--	3.60	--	--	--	7,769	0.178	13,006	0.299
	--	3.70	--	--	--	7,978	0.183	13,794	0.317
	--	3.80	--	--	--	8,191	0.188	14,602	0.335
	--	3.90	--	--	--	8,405	0.193	15,432	0.354
	--	4.00	--	--	--	8,621	0.198	16,283	0.374
	--	4.10	--	--	--	8,841	0.203	17,156	0.394
	--	4.20	--	--	--	9,062	0.208	18,051	0.414
	--	4.30	--	--	--	9,285	0.213	18,969	0.435
	--	4.40	--	--	--	9,508	0.218	19,908	0.457
Freeboard	--	4.50	--	--	--	9,727	0.223	20,870	0.479
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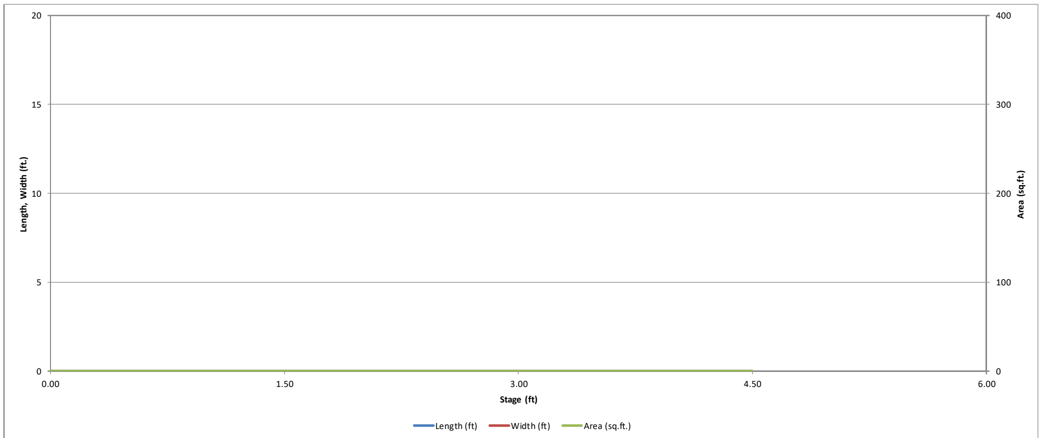
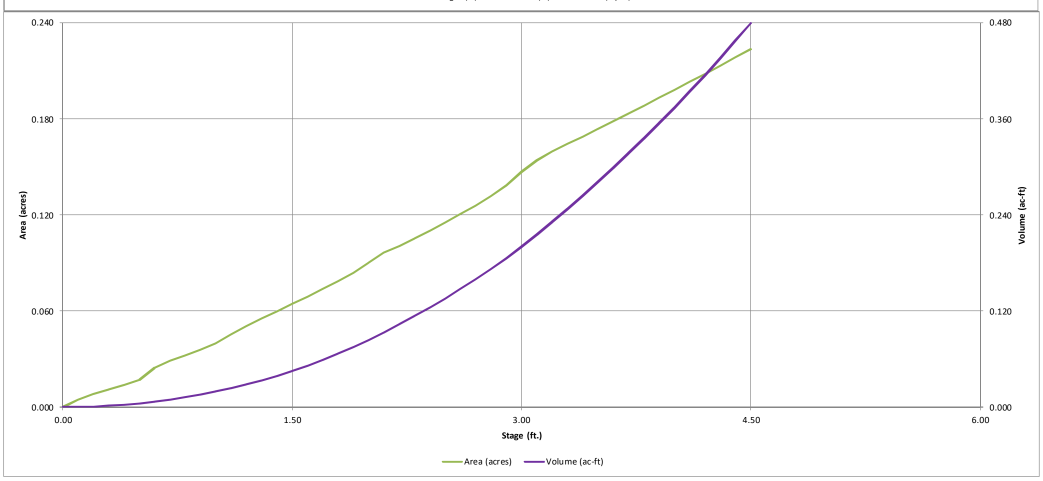
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)



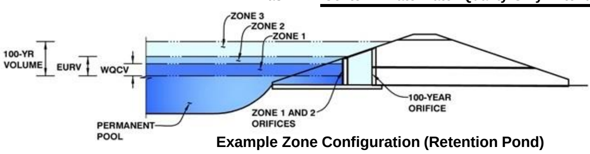


Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: Center Private Water Quality-Only Extended Detention Basin (Sub-Bains B1-B4)



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	3.49	0.279	Orifice Plate
Zone 2			Weir&Pipe (Circular)
Zone 3			Not Utilized
		0.279	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 =	3.50	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	11.00	inches
Orifice Plate: Orifice Area per Row =	0.86	sq. inches (diameter = 1-1/16 inches)

Calculated Parameters for Plate

WQ Orifice Area per Row =	5.972E-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	0.90	1.80	2.70				
Orifice Area (sq. inches)	0.86	0.86	0.86	0.86				

	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 =	3.50	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	5.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	5.00	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 =	3.50	N/A	feet
Over Flow Weir Slope Length =	5.00	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 =	17.50	N/A	ft^2
Overflow Grate Open Area w/ Debris =	8.75	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 =	0.25	N/A	ft (distance below basin bottom at Stage = 0 ft)
*Circular Orifice Diameter =	30.00	N/A	inches

*Please note that the outlet pipe sizing calculations (StormCAD profiles and tables) are included in Appendix D of this Report.

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=	3.50	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	9.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	0.12	feet

Calculated Parameters for Spillway

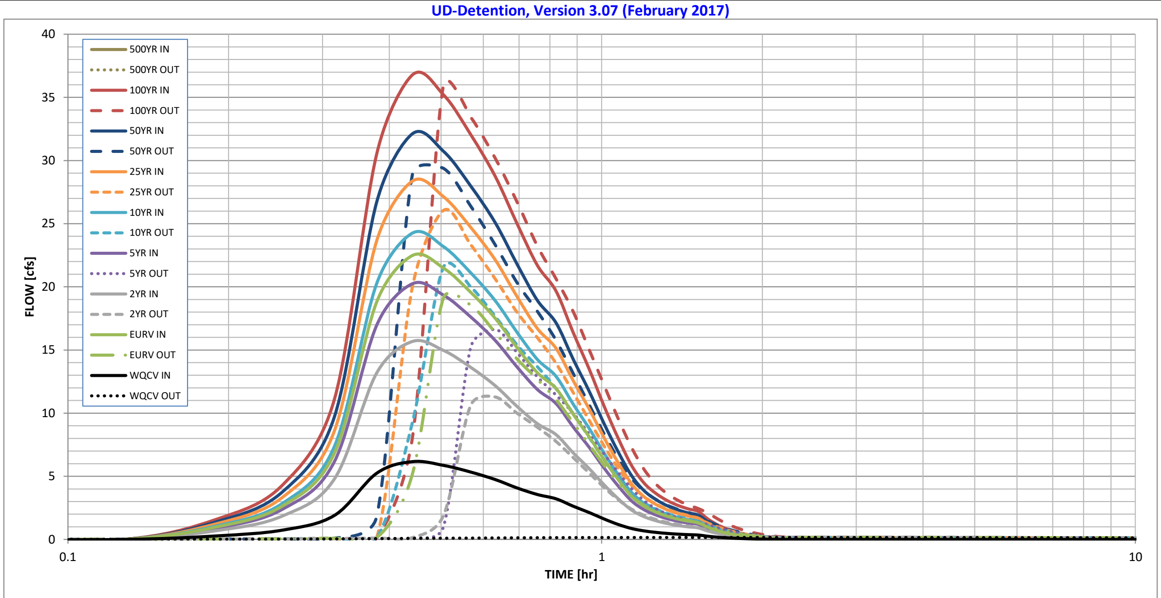
Spillway Design Flow Depth=	0.98	feet
Stage at Top of Freeboard =	4.60	feet
Basin Area at Top of Freeboard =	0.22	acres

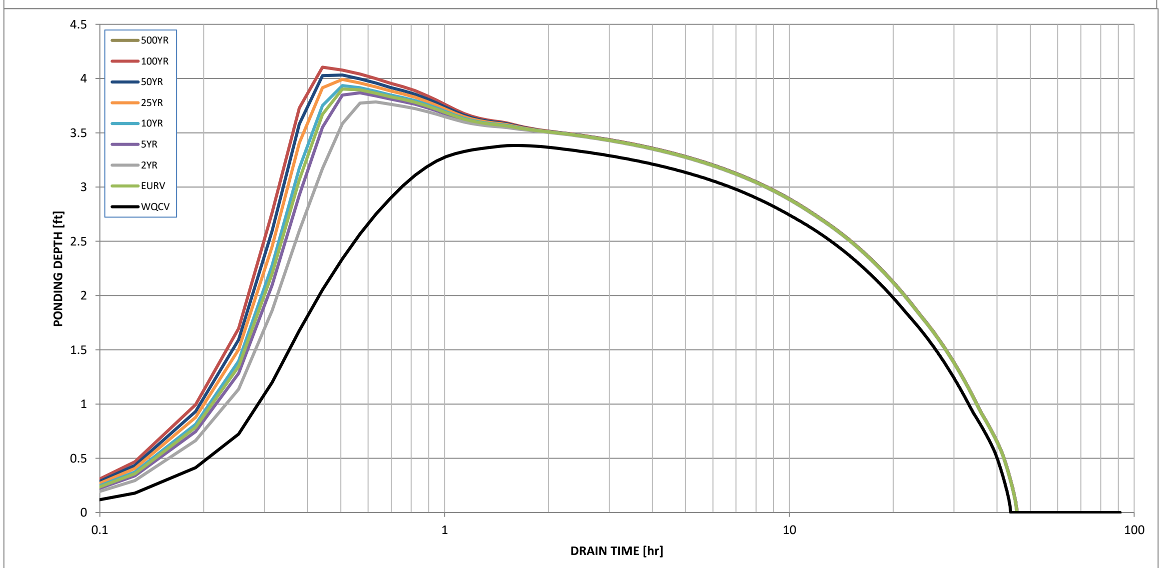
Routed Hydrograph Results

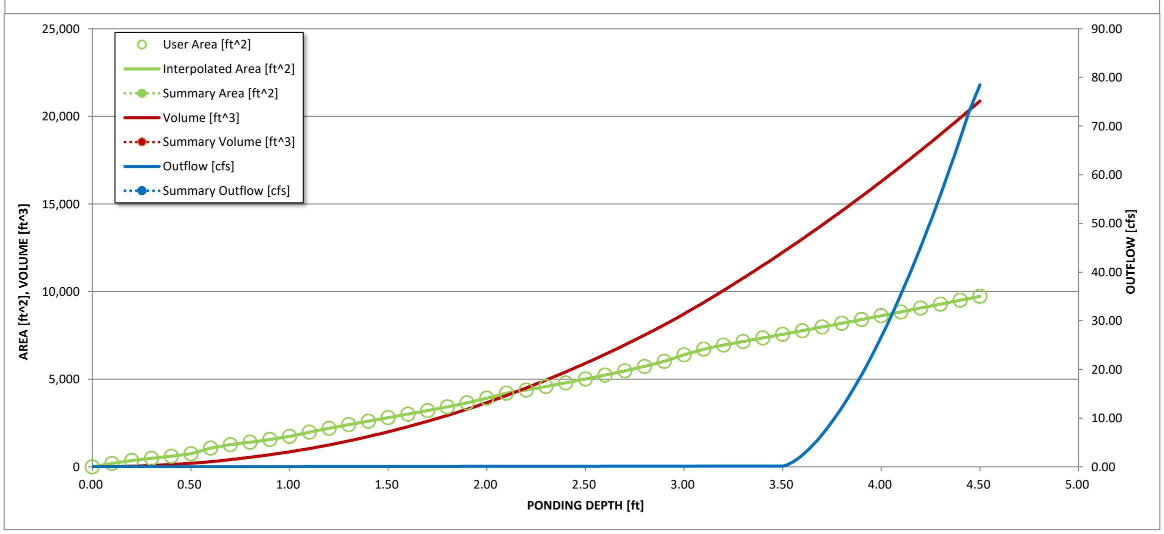
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.279	1.032	0.716	0.928	1.114	1.306	1.481	1.698	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.278	1.032	0.717	0.928	1.114	1.306	1.481	1.699	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.00	0.01	0.02	0.03	0.25	0.60	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.1	0.1	0.3	2.1	5.2	0.0
Peak Inflow Q (cfs) =	6.2	22.5	15.7	20.3	24.3	28.4	32.1	36.8	#N/A
Peak Outflow Q (cfs) =	0.2	19.1	11.3	16.6	21.6	26.1	29.4	36.0	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	298.0	165.8	91.3	13.7	7.0	#N/A
Structure Controlling Flow =	Plate	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	#N/A
Max Velocity through Grate 1 (fps) =	N/A	0.62	0.38	0.6	0.7	0.8	1.0	1.2	#N/A
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	#N/A
Time to Drain 97% of Inflow Volume (hours) =	38	32	34	33	31	30	29	28	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**41**	39	40	39	39	38	37	36	#N/A
Maximum Ponding Depth (ft) =	3.38	3.90	3.79	3.87	3.94	3.99	4.03	4.11	#N/A
Area at Maximum Ponding Depth (acres) =	0.17	0.19	0.19	0.19	0.19	0.20	0.20	0.20	#N/A
Maximum Volume Stored (acre-ft) =	0.260	0.354	0.331	0.347	0.360	0.372	0.380	0.394	#N/A

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	3.79 min	
Hydrograph Constant	1.320	

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	#N/A
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	#N/A
0:03:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:07:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:11:22	0.28	0.99	0.69	0.89	1.06	1.24	1.39	1.59	#N/A
0:15:10	0.74	2.67	1.87	2.40	2.87	3.35	3.79	4.33	#N/A
0:18:57	1.91	6.85	4.80	6.18	7.38	8.62	9.74	11.12	#N/A
0:22:44	5.25	18.82	13.19	16.97	20.28	23.67	26.73	30.54	#N/A
0:26:32	6.16	22.49	15.68	20.25	24.26	28.38	32.13	36.79	#N/A
0:30:19	5.86	21.49	14.97	19.35	23.19	27.14	30.74	35.21	#N/A
0:34:07	5.34	19.56	13.63	17.61	21.11	24.71	27.98	32.05	#N/A
0:37:54	4.74	17.51	12.18	15.76	18.91	22.14	25.09	28.76	#N/A
0:41:41	4.07	15.16	10.52	13.64	16.38	19.20	21.77	24.97	#N/A
0:45:29	3.56	13.19	9.16	11.87	14.25	16.69	18.92	21.69	#N/A
0:49:16	3.22	11.96	8.30	10.76	12.92	15.14	17.16	19.67	#N/A
0:53:04	2.63	9.91	6.86	8.91	10.71	12.57	14.27	16.38	#N/A
0:56:51	2.13	8.13	5.61	7.30	8.79	10.33	11.74	13.49	#N/A
1:00:38	1.62	6.31	4.33	5.65	6.83	8.05	9.16	10.56	#N/A
1:04:26	1.19	4.74	3.23	4.24	5.14	6.08	6.94	8.02	#N/A
1:08:13	0.87	3.43	2.33	3.06	3.73	4.43	5.07	5.88	#N/A
1:12:01	0.68	2.64	1.81	2.36	2.87	3.39	3.87	4.47	#N/A
1:15:48	0.56	2.17	1.49	1.94	2.35	2.77	3.16	3.65	#N/A
1:19:35	0.48	1.84	1.26	1.64	1.99	2.35	2.67	3.08	#N/A
1:23:23	0.42	1.61	1.11	1.44	1.74	2.05	2.34	2.70	#N/A
1:27:10	0.38	1.45	1.00	1.30	1.57	1.85	2.10	2.42	#N/A
1:30:58	0.35	1.33	0.92	1.20	1.44	1.70	1.93	2.22	#N/A
1:34:45	0.26	0.98	0.67	0.88	1.06	1.25	1.42	1.64	#N/A
1:38:32	0.19	0.72	0.49	0.64	0.78	0.91	1.04	1.19	#N/A
1:42:20	0.14	0.53	0.36	0.47	0.57	0.67	0.76	0.88	#N/A
1:46:07	0.10	0.39	0.27	0.35	0.42	0.50	0.56	0.65	#N/A
1:49:55	0.07	0.28	0.19	0.25	0.30	0.36	0.41	0.47	#N/A
1:53:42	0.05	0.20	0.14	0.18	0.22	0.25	0.29	0.33	#N/A
1:57:29	0.03	0.14	0.10	0.13	0.15	0.18	0.21	0.24	#N/A
2:01:17	0.02	0.10	0.06	0.08	0.10	0.12	0.14	0.17	#N/A
2:05:04	0.01	0.06	0.04	0.05	0.06	0.08	0.09	0.10	#N/A
2:08:52	0.01	0.03	0.02	0.03	0.03	0.04	0.05	0.06	#N/A
2:12:39	0.00	0.01	0.01	0.01	0.01	0.01	0.02	0.02	#N/A
2:16:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:20:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:24:01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:27:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:31:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:35:23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:39:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:42:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:46:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:50:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:54:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:58:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:01:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:05:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:09:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:13:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:17:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:20:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:24:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:28:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:32:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:36:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:39:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:43:37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:47:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:51:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:54:59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:58:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:02:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:06:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:10:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:13:56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:17:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:21:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:25:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:29:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:32:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A

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
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: South Private Water Quality-Only Extended Detention Basin (Sub-Basins C1-C2)

***Design Storm: 1-Hour Rain Depth	WQCV Event	0.60	inches
***Minor Storm: 1-Hour Rain Depth	2-Year Event	1.19	inches
***Major Storm: 1-Hour Rain Depth	100-Year Event	2.52	inches
Optional User Defined Storm	NRCS Method		
(NRCS Type II Method) 24-Hour Storm Event and Rainfall Depth for User Defined Storm	100-Year Event		
Max Intensity for Optional User Defined Storm	0		

SITE INFORMATION (USER-INPUT)

Sub-basin Identifier	C1	C2												
Receiving Pervious Area Soil Type	Loamy Sand	Loamy Sand												
Total Area (ac., Sum of DCIA, UIA, RPA, & SPA)	2.370	1.660												
Directly Connected Impervious Area (DCIA, acres)	1.698	1.221												
Unconnected Impervious Area (UIA, acres)	0.014	0.000												
Receiving Pervious Area (RPA, acres)	0.000	0.000												
Separate Pervious Area (SPA, acres)	0.658	0.439												
RPA Treatment Type: Conveyance (C), Volume (V), or Permeable Pavement (PP)	C	V												

CALCULATED RESULTS (OUTPUT)

Total Calculated Area (ac, check against input)	2.370	1.660												
Directly Connected Impervious Area (DCIA, %)	71.6%	73.6%												
Unconnected Impervious Area (UIA, %)	0.6%	0.0%												
Receiving Pervious Area (RPA, %)	0.0%	0.0%												
Separate Pervious Area (SPA, %)	27.8%	26.4%												
A_R (RPA / UIA)	0.000	0.000												
I_a Check	1.000	1.000												
f / I for WQCV Event:	3.2	3.2												
f / I for 2-Year Event:	0.6	0.6												
f / I for 100-Year Event:	0.4	0.4												
f / I for Optional User Defined Storm NRCS Method:														
IRF for WQCV Event:	1.00	0.00												
IRF for 2-Year Event:	1.00	1.00												
IRF for 100-Year Event:	1.00	1.00												
IRF for Optional User Defined Storm NRCS Method:														
Total Site Imperviousness: I_{total}	72.2%	73.6%												
Effective Imperviousness for WQCV Event:	72.2%	73.6%												
Effective Imperviousness for 2-Year Event:	72.2%	73.6%												
Effective Imperviousness for 100-Year Event:	72.2%	73.6%												
Effective Imperviousness for Optional User Defined Storm NRCS Method:														

LID / EFFECTIVE IMPERVIOUSNESS CREDITS

WQCV 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
This line only for 10-Year Event	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
100-Year Event CREDIT**: Reduce Detention By:	0.0%	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
User Defined NRCS Method CREDIT: Reduce Detention By:														

Total Site Imperviousness:	**72.8%**
Total Site Effective Imperviousness for WQCV Event:	**72.8%**
Total Site Effective Imperviousness for 2-Year Event:	**72.8%**
Total Site Effective Imperviousness for 100-Year Event:	**72.8%**
Total Site Effective Imperviousness for Optional User Defined Storm NRCS Method:	

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

Design Procedure Form: Extended Detention Basin (EDB)

UD-BMP (Version 3.06, November 2016)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**South Private Water Quality-Only Extended Detention Basin (Sub-Basins C1-C2)**

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a	I_a = 72.8 %
B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	i = 0.728
C) Contributing Watershed Area	Area = 4.030 ac
D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = 0.43 in
E) Design Concept (Select EURV when also designing for flood control)	Choose One ◉ Water Quality Capture Volume (WQCV) ○ Excess Urban Runoff Volume (EURV)
F) Design Volume (WQCV) Based on 40-hour Drain Time ($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area$)	V_{DESIGN} = 0.097 ac-ft
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume ($V_{WQCV\ OTHER} = (d_6*(V_{DESIGN}/0.43))$	$V_{DESIGN\ OTHER}$ = 0.097 ac-ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{DESIGN\ USER}$ = ac-ft
I) Predominant Watershed NRCS Soil Group	Choose One ○ A ○ B ○ C / D **WQCV selected. Soil group not required.**
J) Excess Urban Runoff Volume (EURV) Design Volume For HSG A: $EURV_A = 1.68 * i^{1.28}$ For HSG B: $EURV_B = 1.36 * i^{1.08}$ For HSG C/D: $EURV_{C/D} = 1.20 * i^{1.08}$	EURV = ac-f t
2. Basin Shape: Length to Width Ratio (A basin length to width ratio of at least 2:1 will improve TSS reduction.)	L : W = 10.4 : 1
3. Basin Side Slopes	
A) Basin Maximum Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred)	Z = 4.00 ft / ft
4. Inlet	
A) Describe means of providing energy dissipation at concentrated inflow locations:	A concrete forebay with concrete baffle blocks will be used for energy dissipation.

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Mitchell Hess
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: South Private Water Quality-Only Extended Detention Basin (Sub-Basins C1-C2)

5. Forebay

A) Minimum Forebay Volume (V_{FMIN} = 2% of the WQCV) — V_{FMIN} = 0.002 ac-ft

B) Actual Forebay Volume — V_F = 95.520 ac-ft

C) Forebay Depth (D_F = 18 inch maximum) — D_F = 12.0 in

D) Forebay Discharge

i) Undetained 100-year Peak Discharge — Q_{100} = 21.59 cfs

ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$) — Q_F = 0.43 cfs

E) Forebay Discharge Design

Choose One
- ○ Berm With Pipe
- ◉ Wall with Rect. Notch
- ○ Wall with V-Notch Weir

(flow too small for berm w/ pipe)

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

G) Rectangular Notch Width — Calculated W_N = 4.0 in

6. Trickle Channel

A) Type of Trickle Channel

Choose One
- ◉ Concrete
- ○ Soft Bottom

F) Slope of Trickle Channel — S = 0.0050 ft / ft

7. Micropool and Outlet Structure

A) Depth of Micropool (2.5-feet minimum) — D_M = 2.5 ft

B) Surface Area of Micropool (10 ft^2 minimum) — A_M = 16 sq ft

C) Outlet Type

Choose One
- ◉ Orifice Plate
- ○ Other (Describe):

D) Smallest Dimension of Orifice Opening Based on Hydrograph Routing (Use UD-Detention) — $D_{orifice}$ = 0.34 inches

E) Total Outlet Area — A_{ot} = 1.02 square inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	**Mitchell Hess**
Company:	**Kimley-Horn and Associates, Inc.**
Date:	**November 27, 2019**
Project:	**Zeppelin 3 and 4**
Location:	**South Private Water Quality-Only Extended Detention Basin (Sub-Basins C1-C2)**

8. Initial Surcharge Volume

A) Depth of Initial Surcharge Volume (Minimum recommended depth is 4 inches)	D_{IS} = 4 in
B) Minimum Initial Surcharge Volume (Minimum volume of 0.3% of the WQCV)	V_{IS} = cu ft
C) Initial Surcharge Provided Above Micropool	V_s = 5.3 cu ft

9. Trash Rack

A) Water Quality Screen Open Area: $A_t = A_{ot} * 38.5*(e^{-0.095D})$	A_t = 38 square inches
B) Type of Screen (If specifying an alternative to the materials recommended in the USDCM, indicate "other" and enter the ratio of the total open are to the total screen are for the material specified.) Other (Y/N): N	*S.S. Well Screen with 60% Open Area*
C) Ratio of Total Open Area to Total Area (only for type 'Other')	User Ratio =
D) Total Water Quality Screen Area (based on screen type)	A_{total} = 63 sq. in.
E) Depth of Design Volume (EURV or WQCV) (Based on design concept chosen under 1E)	H= 2.5 feet
F) Height of Water Quality Screen (H_{TR})	H_{TR}= 58 inches
G) Width of Water Quality Screen Opening ($W_{opening}$) (Minimum of 12 inches is recommended)	$W_{opening}$ = 12.0 inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Mitchell Hess
Company: Kimley-Horn and Associates, Inc.
Date: November 27, 2019
Project: Zeppelin 3 and 4
Location: South Private Water Quality-Only Extended Detention Basin (Sub-Basins C1-C2)

10. Overflow Embankment A) Describe embankment protection for 100-year and greater overtopping:	Overflow is for anything above the Water Quality WSEL as this is a Water Quality-Only EDB Embankment protection will consist of 2-ft deep Type L Riprap
B) Slope of Overflow Embankment (Horizontal distance per unit vertical, 4:1 or flatter preferred)	
11. Vegetation	Choose One ◉ Irrigated ○ Not Irrigated AVOID PLACING IRRIGATION HEADS IN THE BOTTOM OF THE BASIN
12. Access A) Describe Sediment Removal Procedures	An access has been provided for the detention pond so vehicles can access the bottom of the pond to maintain the forebay, trickle channel and outlet structure.

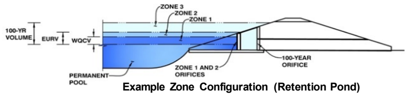
Notes:

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: South Private Water Quality-Only Extended Detention Basin (C1-C2)



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	4.03	acres		
Watershed Length =	536	ft		
Watershed Slope =	0.014	ft/ft		
Watershed Imperviousness =	72.80%	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 =	User Input			
Water Quality Capture Volume (WQCV) =	0.097	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.376	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.259	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.338	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.409	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.489	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.573	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.671	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 0 in.) =	0.000	acre-feet		inches
Approximate 2-yr Detention Volume =	0.246	acre-feet		
Approximate 5-yr Detention Volume =	0.320	acre-feet		
Approximate 10-yr Detention Volume =	0.384	acre-feet		
Approximate 25-yr Detention Volume =	0.459	acre-feet		
Approximate 50-yr Detention Volume =	0.503	acre-feet		
Approximate 100-yr Detention Volume =	0.546	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.097	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 =	0.097	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 = ft

Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Top of Micropool	--	0.00	--	--	--	20	0.000		
	--	0.10	--	--	--	51	0.001	4	0.000
	--	0.20	--	--	--	91	0.002	10	0.000
	--	0.30	--	--	--	133	0.003	21	0.000
	--	0.40	--	--	--	179	0.004	36	0.001
	--	0.50	--	--	--	233	0.005	56	0.001
	--	0.60	--	--	--	300	0.007	82	0.002
	--	0.70	--	--	--	387	0.009	115	0.003
	--	0.80	--	--	--	498	0.011	158	0.004
	--	0.90	--	--	--	643	0.015	214	0.005
	--	1.00	--	--	--	861	0.020	287	0.007
	--	1.10	--	--	--	1,165	0.027	385	0.009
	--	1.20	--	--	--	1,331	0.031	508	0.012
	--	1.30	--	--	--	1,492	0.034	648	0.015
	--	1.40	--	--	--	1,656	0.038	804	0.018
	--	1.50	--	--	--	1,827	0.042	976	0.022
	--	1.60	--	--	--	2,035	0.047	1,167	0.027
	--	1.70	--	--	--	2,241	0.051	1,379	0.032
	--	1.80	--	--	--	2,466	0.057	1,612	0.037
	--	1.90	--	--	--	2,731	0.063	1,869	0.043
	--	2.00	--	--	--	3,081	0.071	2,156	0.050
	--	2.10	--	--	--	3,581	0.082	2,520	0.058
	--	2.20	--	--	--	3,967	0.091	2,898	0.067
	--	2.30	--	--	--	4,328	0.099	3,312	0.076
	--	2.40	--	--	--	4,677	0.107	3,763	0.086
WQCV WSEL	--	2.50	--	--	--	5,027	0.115	4,248	0.098
	--	2.60	--	--	--	5,313	0.122	4,765	0.109
	--	2.70	--	--	--	5,564	0.128	5,309	0.122
	--	2.80	--	--	--	5,777	0.133	5,876	0.135
	--	2.90	--	--	--	5,992	0.138	6,464	0.148
	--	3.00	--	--	--	6,208	0.143	7,074	0.162
	--	3.10	--	--	--	6,426	0.148	7,706	0.177
	--	3.20	--	--	--	6,648	0.153	8,360	0.192
	--	3.30	--	--	--	6,875	0.158	9,036	0.207
	--	3.40	--	--	--	7,108	0.163	9,735	0.223
Freeboard	--	3.50	--	--	--	7,351	0.169	10,458	0.240
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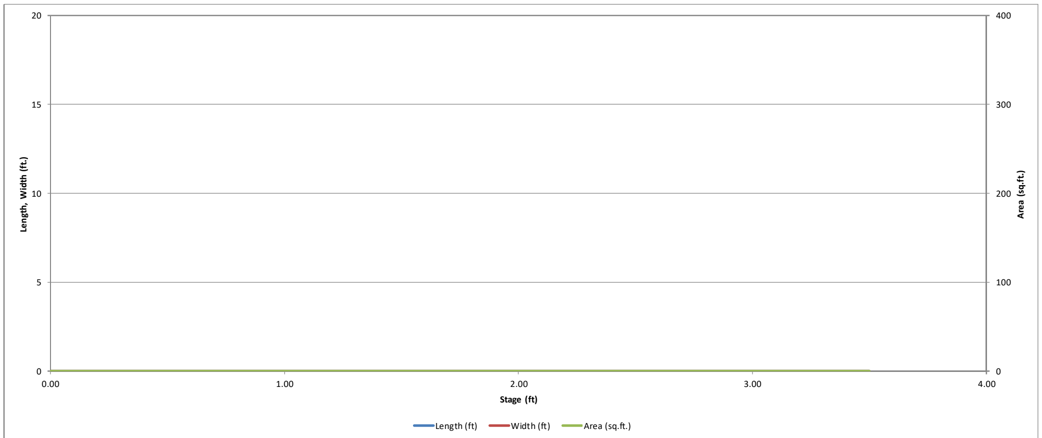
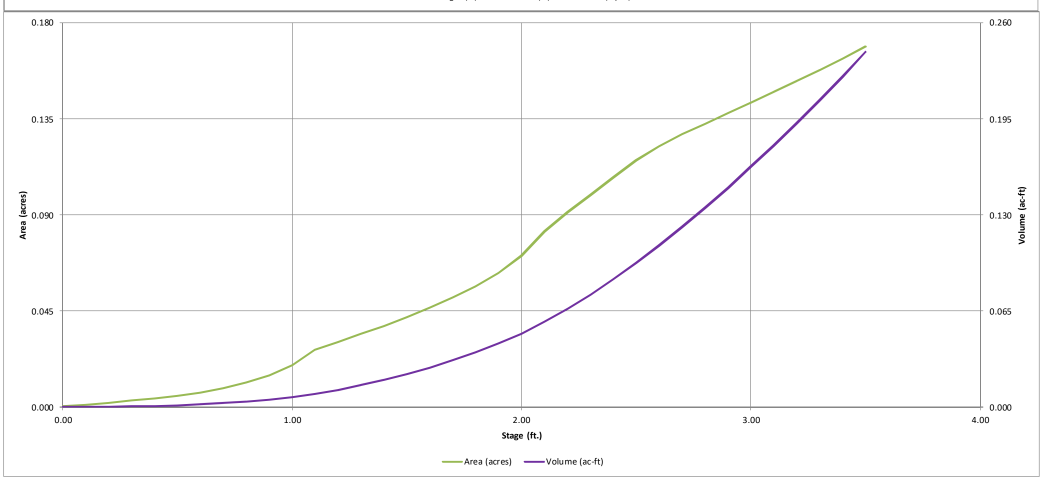
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)



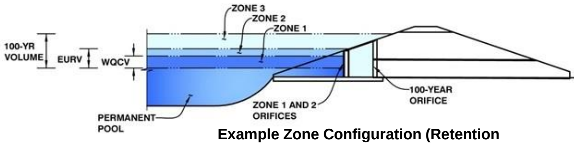


Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: Zeppelin 3 and 4

Basin ID: South Private Water Quality-Only Extended Detention Basin (C1-C2)



Example Zone Configuration (Retention

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
1 (WQCV)	2.50	0.097	Orifice Plate
Zone 2			Weir&Pipe (Circular)
Zone 3			Not Utilized
		0.097	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 =	2.50	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	N/A	inches
Orifice Plate: Orifice Area per Row =	0.34	sq. inches (diameter = 5/8 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	2.361E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	0.80	1.60					
Orifice Area (sq. inches)	0.34	0.34	0.34					

	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 =	2.50	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	5.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	5.00	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 =	2.50	N/A	feet
Over Flow Weir Slope Length =	5.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	5.57	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	17.50	N/A	ft^2
Overflow Grate Open Area w/ Debris =	8.75	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 =	1.00	N/A	ft (distance below basin bottom at Stage = 0 ft)
*Circular Orifice Diameter =	24.00	N/A	inches

*Please note that the outlet pipe sizing calculations (StormCAD profiles and tables) are included in Appendix D of this Report.

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=	2.50	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	13.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	0.01	feet

Calculated Parameters for Spillway

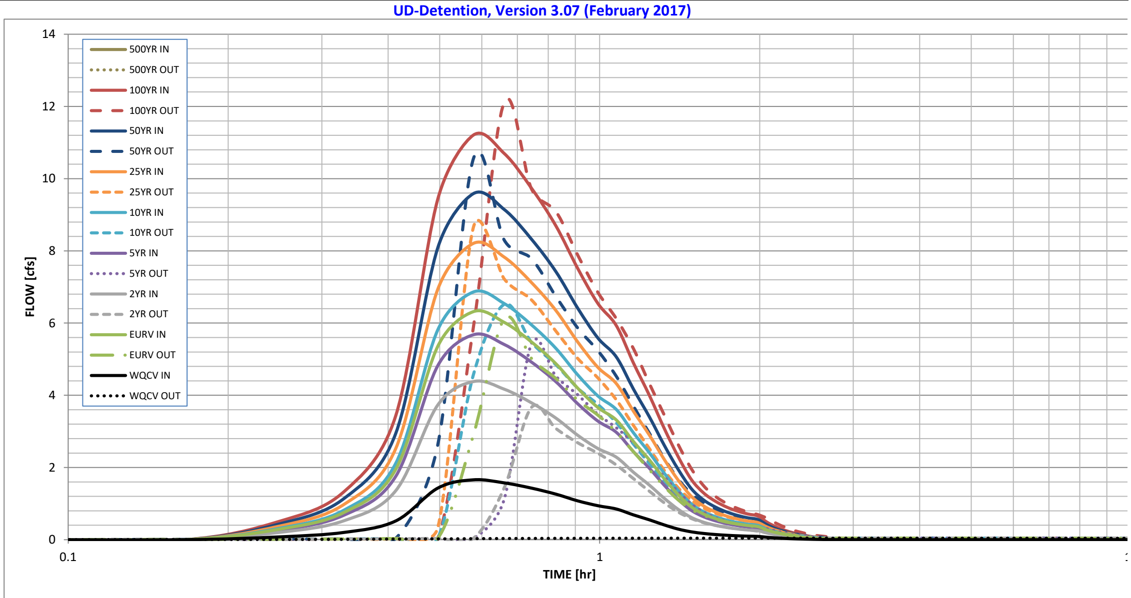
Spillway Design Flow Depth=	0.39	feet
Stage at Top of Freeboard =	2.90	feet
Basin Area at Top of Freeboard =	0.14	acres

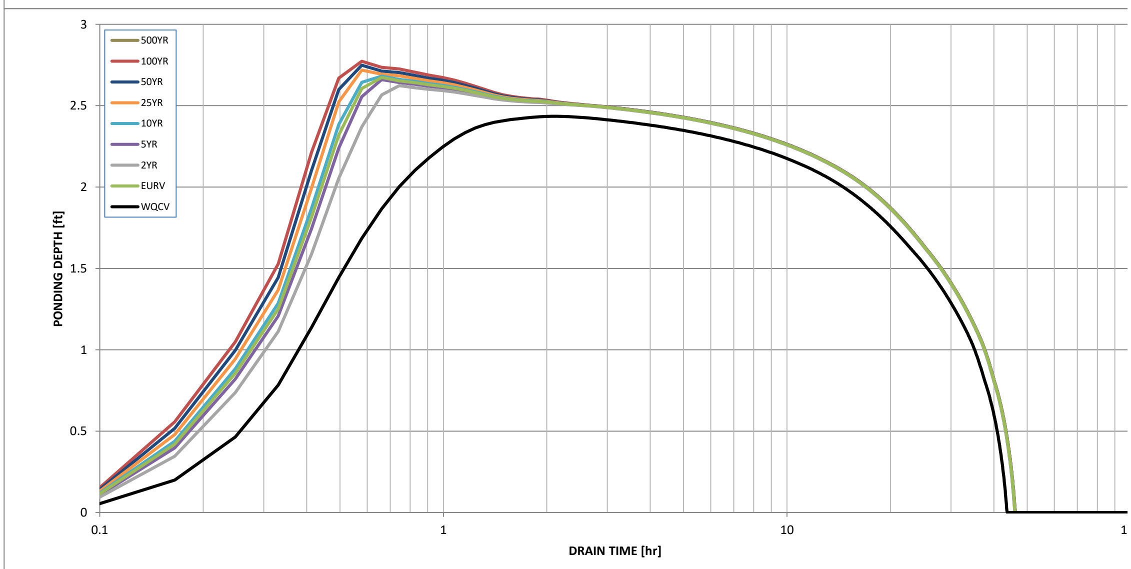
Routed Hydrograph Results

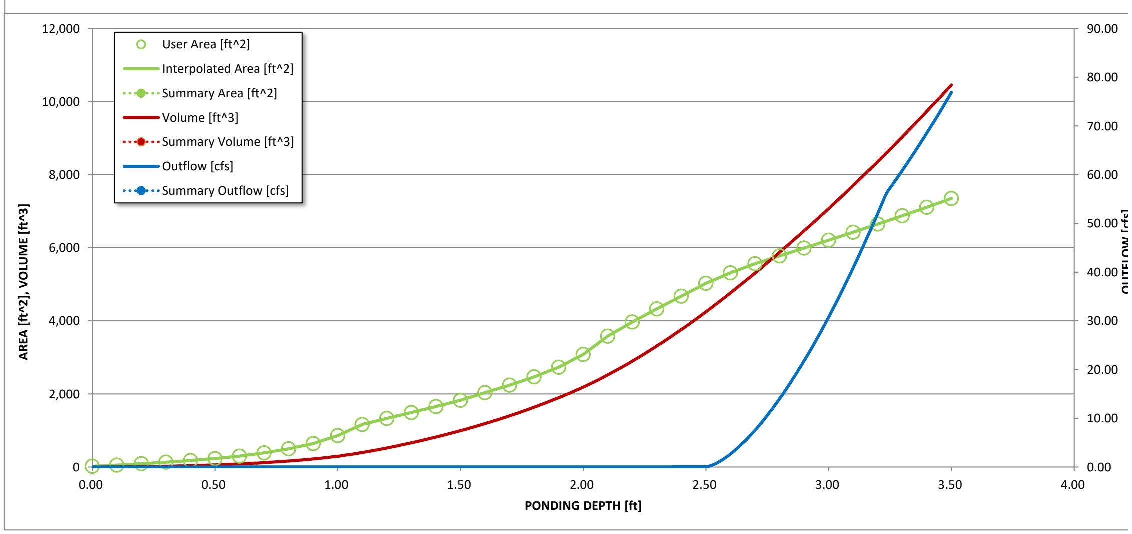
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	0.00
Calculated Runoff Volume (acre-ft) =	0.097	0.376	0.259	0.338	0.409	0.489	0.573	0.671	0.000
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.097	0.375	0.259	0.337	0.408	0.489	0.572	0.670	#N/A
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.00	0.01	0.01	0.03	0.20	0.48	0.00
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.0	0.0	0.1	0.8	2.0	0.0
Peak Inflow Q (cfs) =	1.7	6.3	4.4	5.7	6.9	8.2	9.6	11.2	#N/A
Peak Outflow Q (cfs) =	0.0	6.1	3.7	5.4	6.5	8.7	10.6	12.1	#N/A
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	259.5	133.9	80.8	13.1	6.2	#N/A
Structure Controlling Flow =	Plate	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	#N/A
Max Velocity through Grate 1 (fps) =	N/A	0.17	0.10	0.2	0.2	0.3	0.3	0.3	#N/A
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	#N/A
Time to Drain 97% of Inflow Volume (hours) =	39	35	37	35	34	33	31	30	#N/A
Time to Drain 99% of Inflow Volume (hours) =	**42**	40	42	41	40	39	39	38	#N/A
Maximum Ponding Depth (ft) =	2.43	2.67	2.62	2.66	2.68	2.72	2.75	2.77	#N/A
Area at Maximum Ponding Depth (acres) =	0.11	0.13	0.12	0.13	0.13	0.13	0.13	0.13	#N/A
Maximum Volume Stored (acre-ft) =	0.090	0.118	0.112	0.117	0.119	0.123	0.127	0.131	#N/A

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:

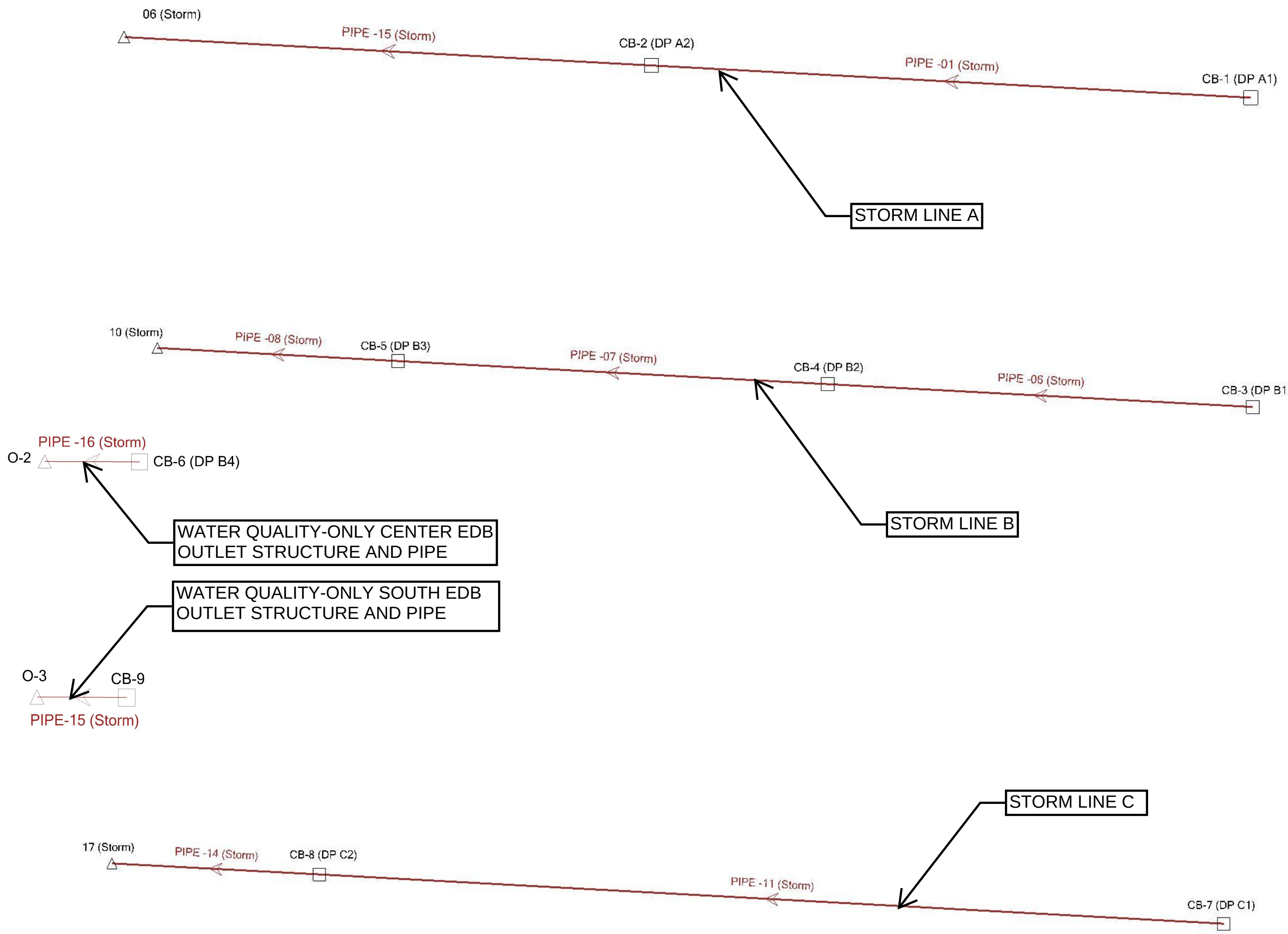
Storm Inflow Hydrographs **UD-Detention, Version 3.07 (February 2017)**

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

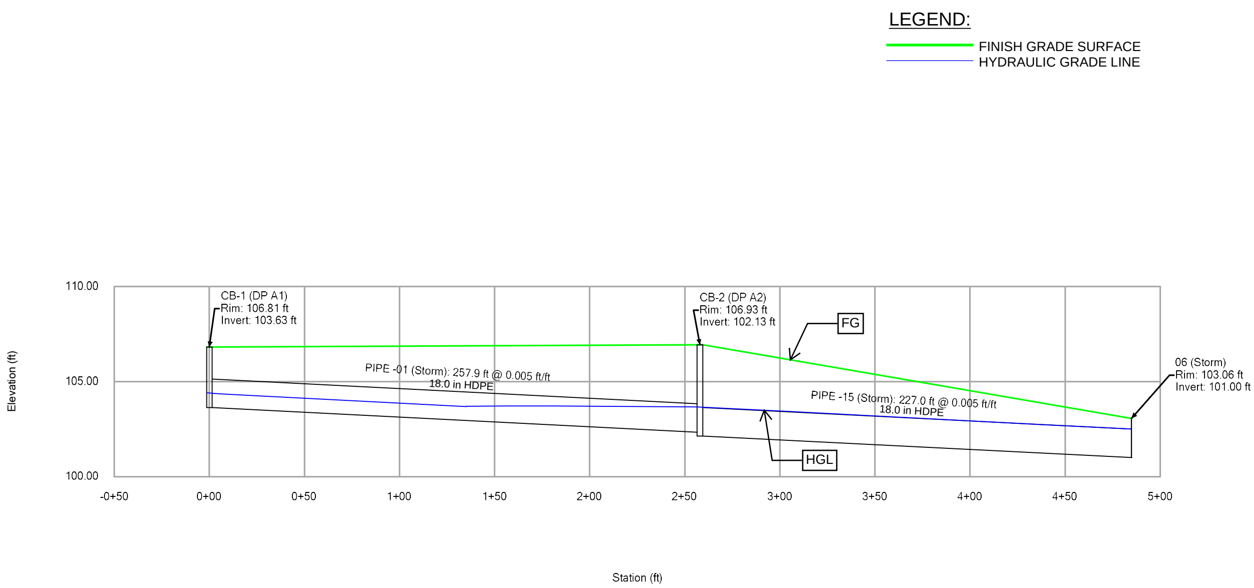
SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	#N/A
TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:04:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:09:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
0:14:53	0.08	0.28	0.20	0.26	0.31	0.37	0.43	0.50	#N/A
0:19:50	0.20	0.76	0.53	0.68	0.82	0.98	1.15	1.34	#N/A
0:24:48	0.52	1.95	1.36	1.75	2.12	2.53	2.94	3.43	#N/A
0:29:46	1.43	5.36	3.73	4.82	5.82	6.94	8.09	9.44	#N/A
0:34:43	1.66	6.32	4.38	5.68	6.86	8.21	9.59	11.21	#N/A
0:39:41	1.57	6.02	4.17	5.41	6.54	7.83	9.15	10.70	#N/A
0:44:38	1.43	5.48	3.79	4.92	5.95	7.12	8.33	9.74	#N/A
0:49:36	1.26	4.88	3.37	4.38	5.30	6.35	7.43	8.70	#N/A
0:54:34	1.08	4.20	2.89	3.76	4.56	5.48	6.41	7.51	#N/A
0:59:31	0.94	3.66	2.53	3.29	3.98	4.78	5.59	6.55	#N/A
1:04:29	0.85	3.32	2.29	2.97	3.61	4.32	5.06	5.93	#N/A
1:09:26	0.69	2.72	1.87	2.44	2.96	3.56	4.17	4.90	#N/A
1:14:24	0.55	2.21	1.51	1.98	2.41	2.90	3.40	4.00	#N/A
1:19:22	0.41	1.69	1.15	1.51	1.84	2.22	2.62	3.08	#N/A
1:24:19	0.29	1.25	0.84	1.11	1.36	1.65	1.94	2.30	#N/A
1:29:17	0.22	0.91	0.61	0.81	0.99	1.20	1.41	1.66	#N/A
1:34:14	0.17	0.71	0.48	0.63	0.77	0.93	1.09	1.29	#N/A
1:39:12	0.14	0.58	0.40	0.52	0.63	0.77	0.90	1.06	#N/A
1:44:10	0.12	0.50	0.34	0.44	0.54	0.65	0.76	0.90	#N/A
1:49:07	0.11	0.44	0.30	0.39	0.47	0.57	0.67	0.79	#N/A
1:54:05	0.10	0.39	0.27	0.35	0.43	0.52	0.60	0.71	#N/A
1:59:02	0.09	0.36	0.25	0.33	0.40	0.48	0.56	0.66	#N/A
2:04:00	0.07	0.27	0.18	0.24	0.29	0.35	0.41	0.48	#N/A
2:08:58	0.05	0.20	0.13	0.17	0.21	0.26	0.30	0.35	#N/A
2:13:55	0.04	0.14	0.10	0.13	0.16	0.19	0.22	0.26	#N/A
2:18:53	0.03	0.10	0.07	0.09	0.11	0.14	0.16	0.19	#N/A
2:23:50	0.02	0.07	0.05	0.07	0.08	0.10	0.12	0.14	#N/A
2:28:48	0.01	0.05	0.04	0.05	0.06	0.07	0.08	0.10	#N/A
2:33:46	0.01	0.04	0.02	0.03	0.04	0.05	0.06	0.07	#N/A
2:38:43	0.01	0.02	0.02	0.02	0.03	0.03	0.04	0.05	#N/A
2:43:41	0.00	0.01	0.01	0.01	0.02	0.02	0.02	0.03	#N/A
2:48:38	0.00	0.01	0.00	0.01	0.01	0.01	0.01	0.01	#N/A
2:53:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
2:58:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:03:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:08:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:13:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:18:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:23:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:28:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:33:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:38:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:43:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:48:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:53:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
3:58:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:03:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:08:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:12:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:17:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:22:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:27:50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:32:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:37:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:42:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:47:41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:52:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
4:57:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:02:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:07:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:12:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:17:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:22:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:27:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:32:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:37:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:42:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:47:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:52:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A
5:57:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	#N/A

APPENDIX D – STORMCADD PROFILES AND DATA AND INLET SIZING

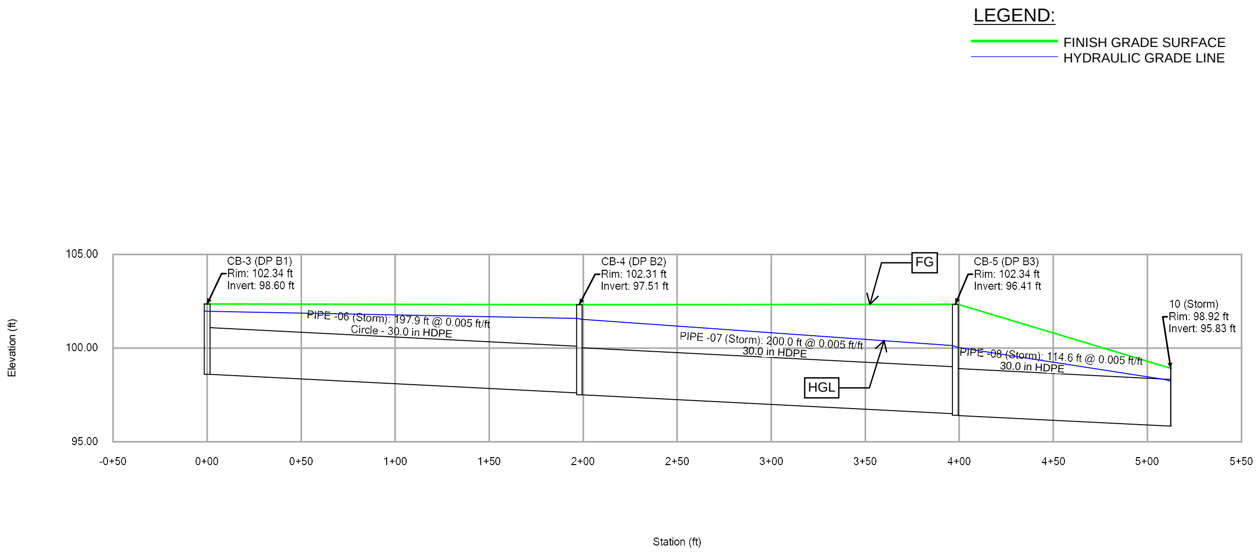
Zeppelin 3 StormCAD Model
Scenario: 100-Year
Active Scenario: 100-Year



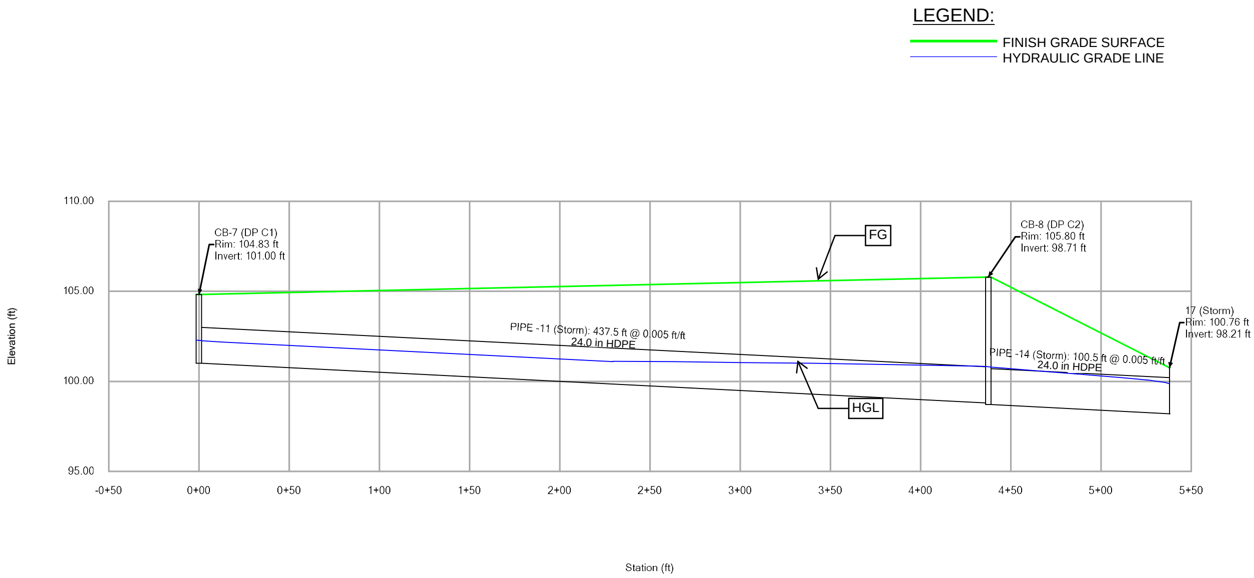
Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm A (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year



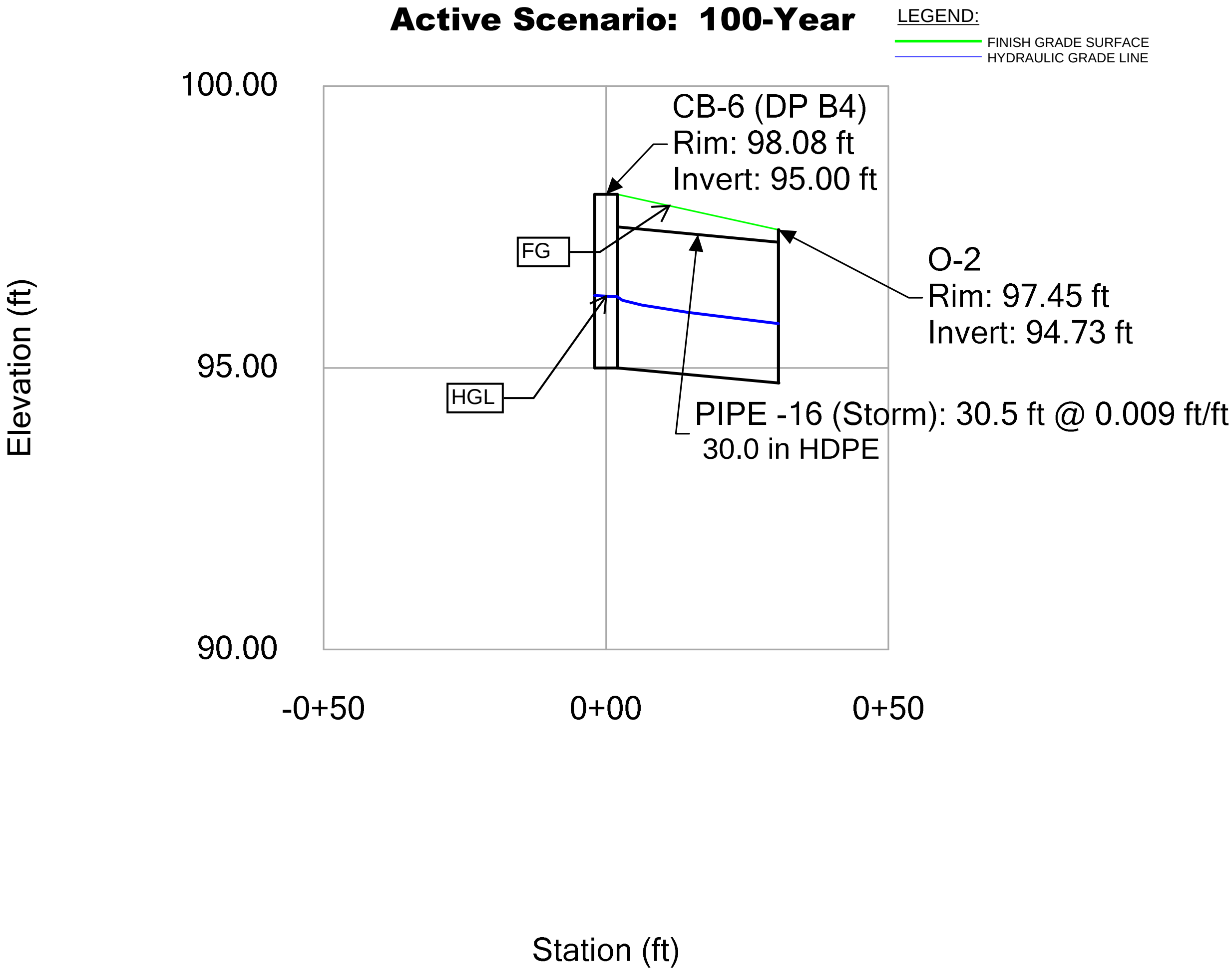
Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm B (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year



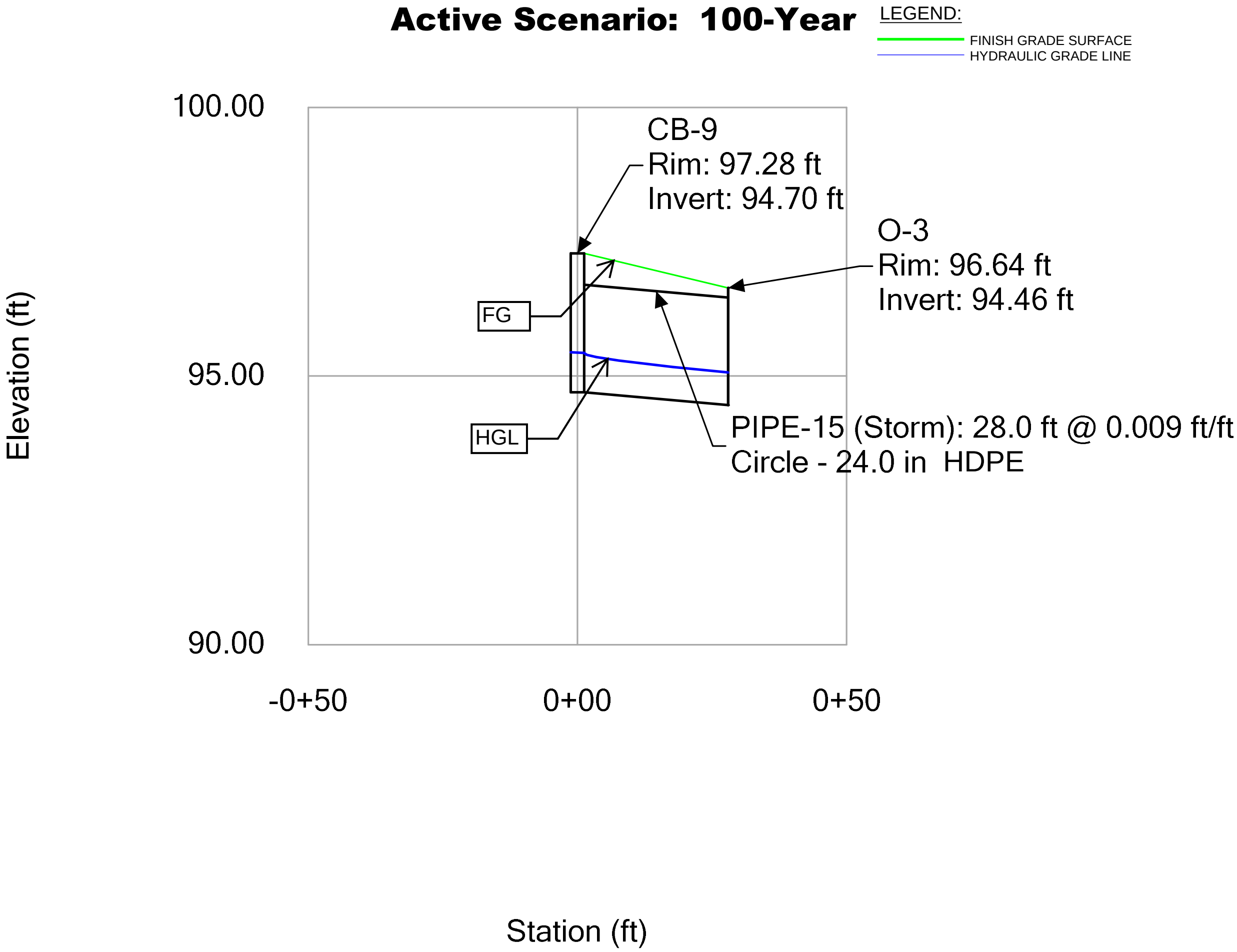
Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm C (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year



Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Center Pond (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year



Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - South Pond (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year



Zeppelin 3 StormCAD Model
FlexTable: Conduit Table
Active Scenario: 100-Year

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)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
PIPE -01 (Storm)	103.63	102.34	257.9	0.005	18.0	0.012	3.94	4.54	104.39	103.68
PIPE -06 (Storm)	98.60	97.61	197.9	0.005	30.0	0.012	19.40	3.95	101.96	101.58
PIPE -07 (Storm)	97.51	96.51	200.0	0.005	30.0	0.012	37.19	7.58	101.54	100.14
PIPE -08 (Storm)	96.41	95.83	114.6	0.005	30.0	0.012	55.12	11.23	100.04	98.19
PIPE -11 (Storm)	101.00	98.81	437.5	0.005	24.0	0.012	12.33	5.99	102.26	100.83
PIPE -14 (Storm)	98.71	98.21	100.5	0.005	24.0	0.012	21.59	6.87	100.79	99.87
PIPE -15 (Storm)	102.13	101.00	227.0	0.005	18.0	0.012	8.13	5.18	103.66	102.50
PIPE -16 (Storm)	95.00	94.73	30.5	0.009	30.0	0.012	14.00	7.67	96.26	95.79
PIPE-15 (Storm)	94.70	94.46	28.0	0.009	24.0	0.012	4.30	5.55	95.43	95.07

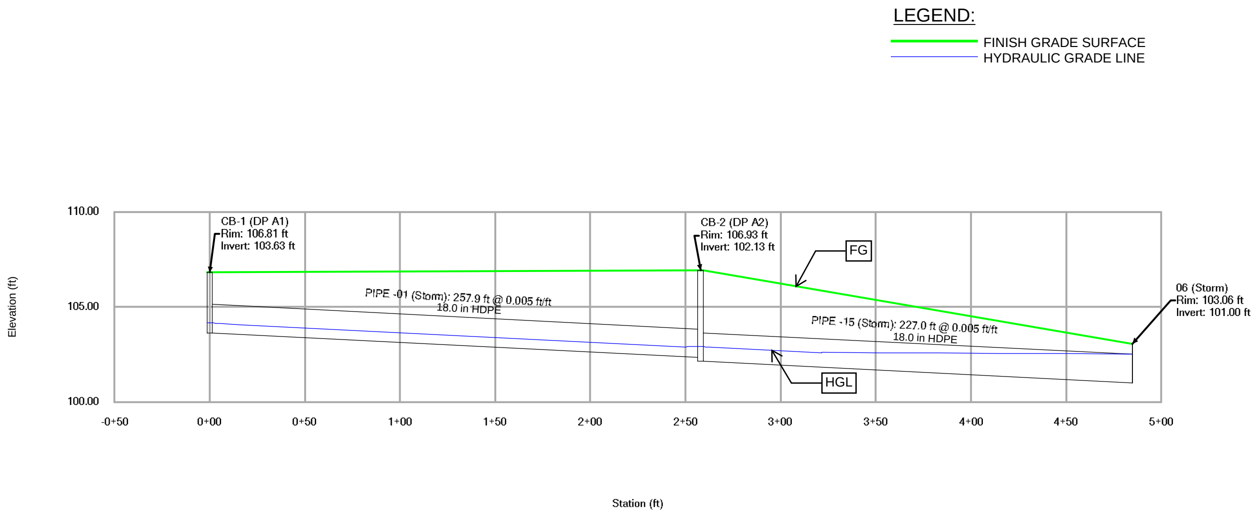
Zeppelin 3 StormCAD Model
FlexTable: Catch Basin Table
Active Scenario: 100-Year

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Headloss Coefficient (Standard)	Flow (Captured) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
CB-1 (DP A1)	106.81	103.63	0.050	0.00	104.40	104.39
CB-2 (DP A2)	106.93	102.13	0.050	0.00	103.68	103.66
CB-3 (DP B1)	102.34	98.60	0.050	0.00	101.97	101.96
CB-4 (DP B2)	102.31	97.51	0.050	0.00	101.58	101.54
CB-5 (DP B3)	102.34	96.41	0.050	0.00	100.14	100.04
CB-6 (DP B4)	98.08	95.00	0.050	0.00	96.28	96.26
CB-7 (DP C1)	104.83	101.00	0.050	0.00	102.29	102.26
CB-8 (DP C2)	105.80	98.71	0.050	0.00	100.83	100.79
CB-9	97.28	94.70	0.050	0.00	95.44	95.43

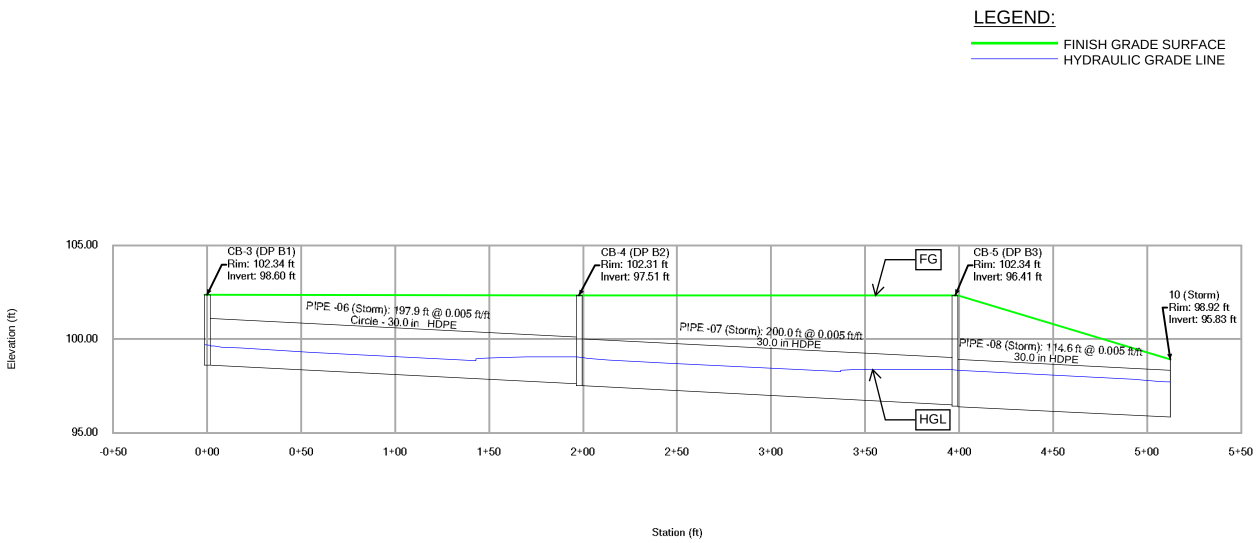
Zeppelin 3 StormCAD Model
FlexTable: Outfall Table
Active Scenario: 100-Year

Label	Elevation (Ground) (ft)	Elevation (Invert) (ft)	Boundary Condition Type	Hydraulic Grade (ft)	Flow (Total Out) (cfs)
06 (Storm)	103.06	101.00	Crown	102.50	8.13
10 (Storm)	98.92	95.83	User Defined Tailwater	98.19	55.12
17 (Storm)	100.76	98.21	User Defined Tailwater	99.87	21.59
O-2	97.45	94.73	Free Outfall	95.79	14.00
O-3	96.64	94.46	Free Outfall	95.07	4.30

Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm A (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 5-Year



Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm B (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 5-Year

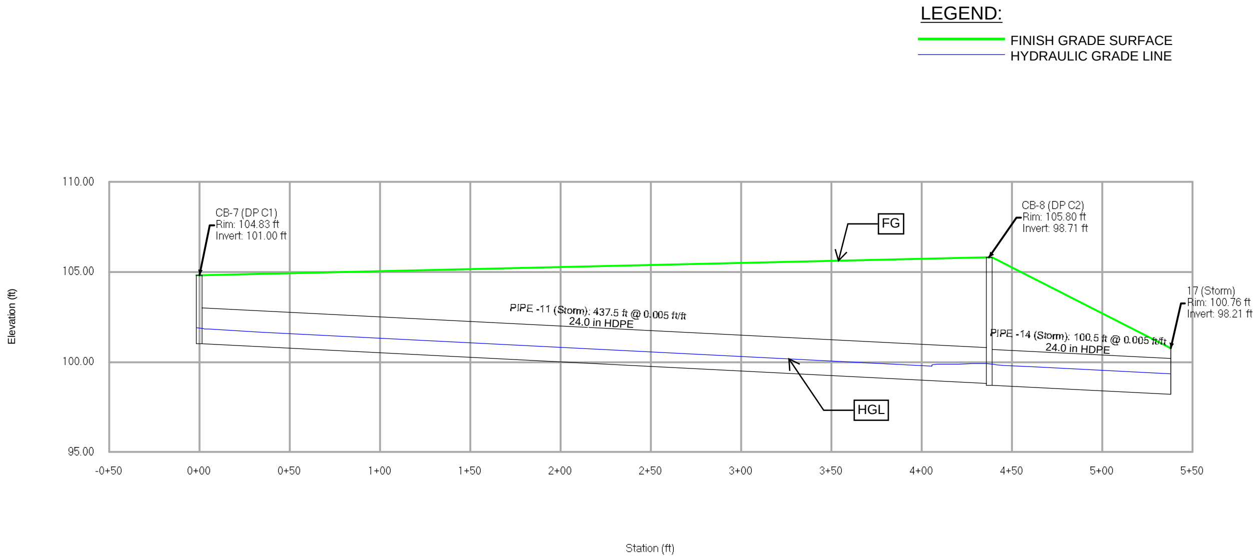


Zeppelin 3 StormCAD Model

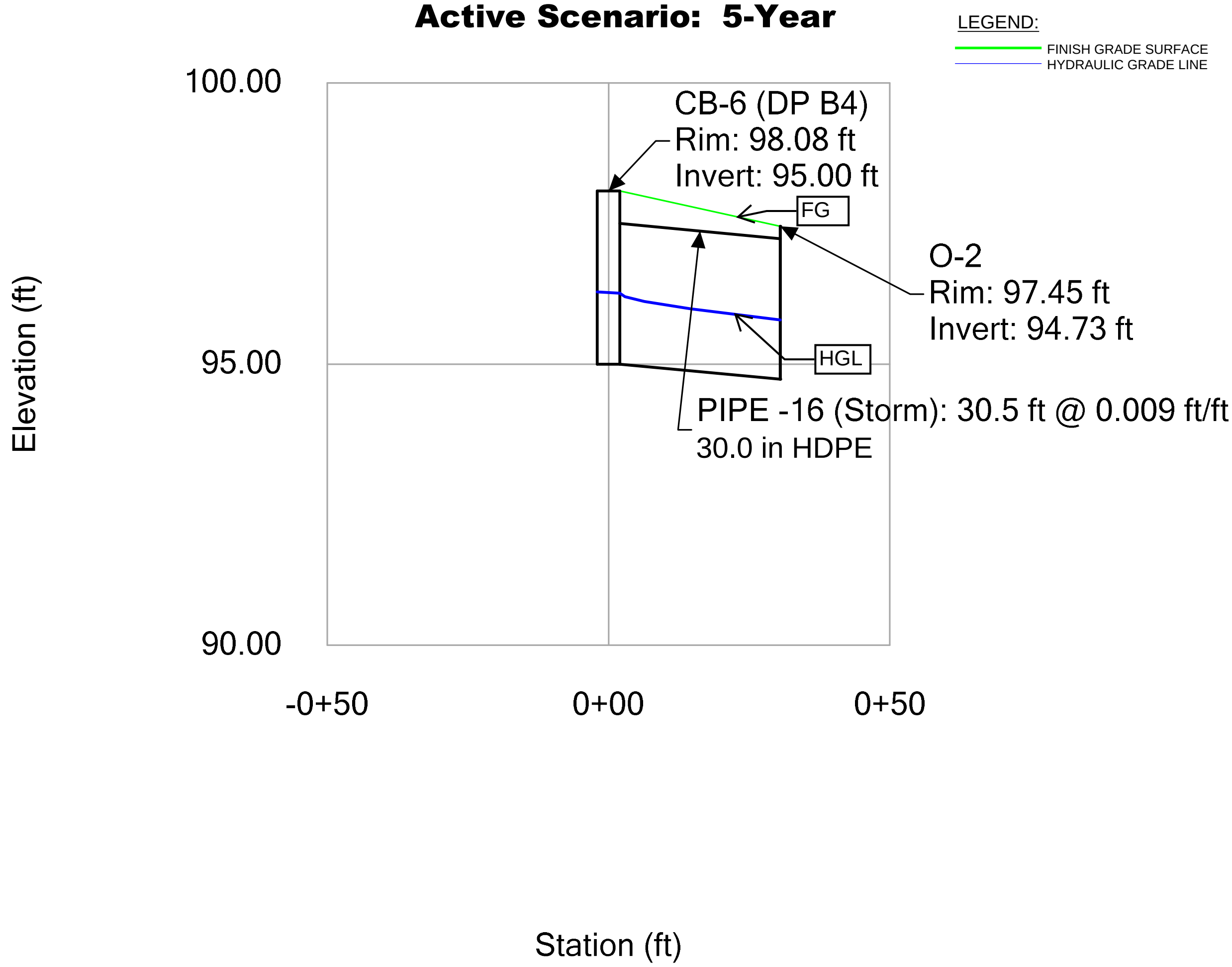
Profile Report

Engineering Profile - Storm C (Zeppelin 3&4 StormCAD.stsw)

Active Scenario: 5-Year



Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Center Pond (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 5-Year

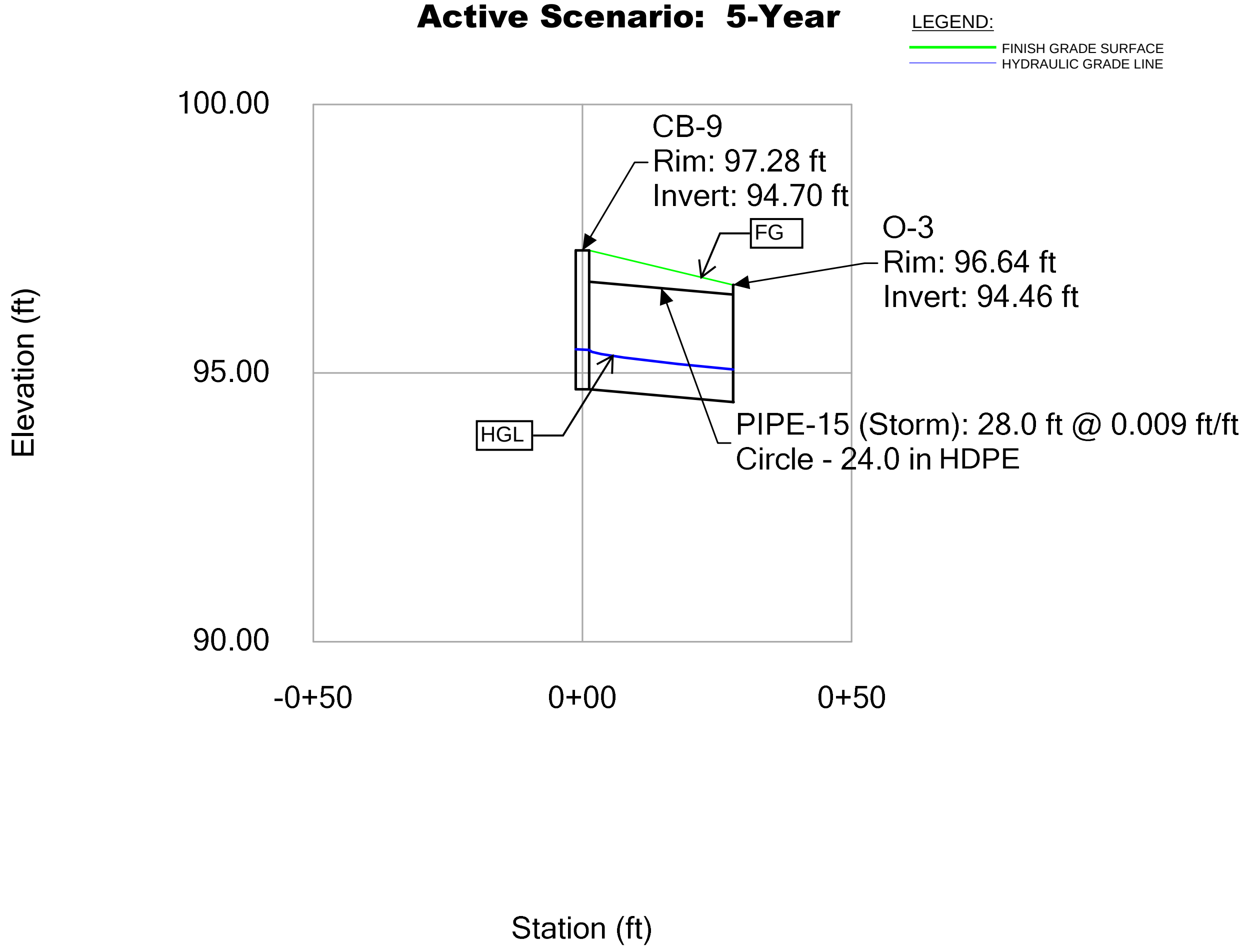


Zeppelin 3 StormCAD Model

Profile Report

Engineering Profile - South Pond (Zeppelin 3&4 StormCAD.stsw)

Active Scenario: 5-Year



Zeppelin 3 StormCAD Model

FlexTable: Conduit Table

Active Scenario: 5-Year

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)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
PIPE -01 (Storm)	103.63	102.34	257.9	0.005	18.0	0.012	2.00	3.78	104.16	102.93
PIPE -06 (Storm)	98.60	97.61	197.9	0.005	30.0	0.012	10.18	5.71	99.66	99.05
PIPE -07 (Storm)	97.51	96.51	200.0	0.005	30.0	0.012	19.89	6.77	99.02	98.37
PIPE -08 (Storm)	96.41	95.83	114.6	0.005	30.0	0.012	29.56	7.28	98.33	97.69
PIPE -11 (Storm)	101.00	98.81	437.5	0.005	24.0	0.012	6.11	5.04	101.87	99.91
PIPE -14 (Storm)	98.71	98.21	100.5	0.005	24.0	0.012	10.69	5.80	99.88	99.34
PIPE -15 (Storm)	102.13	101.00	227.0	0.005	18.0	0.012	4.15	4.58	102.91	102.50
PIPE -16 (Storm)	95.00	94.73	30.5	0.009	30.0	0.012	14.00	7.67	96.26	95.79
PIPE-15 (Storm)	94.70	94.46	28.0	0.009	24.0	0.012	4.30	5.55	95.43	95.07

Zeppelin 3 StormCAD Model

FlexTable: Catch Basin Table

Active Scenario: 5-Year

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Headloss Coefficient (Standard)	Flow (Captured) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
CB-1 (DP A1)	106.81	103.63	0.050	0.00	104.17	104.16
CB-2 (DP A2)	106.93	102.13	0.050	0.00	102.93	102.91
CB-3 (DP B1)	102.34	98.60	0.050	0.00	99.68	99.66
CB-4 (DP B2)	102.31	97.51	0.050	0.00	99.05	99.02
CB-5 (DP B3)	102.34	96.41	0.050	0.00	98.37	98.33
CB-6 (DP B4)	98.08	95.00	0.050	0.00	96.28	96.26
CB-7 (DP C1)	104.83	101.00	0.050	0.00	101.89	101.87
CB-8 (DP C2)	105.80	98.71	0.050	0.00	99.91	99.88
CB-9	97.28	94.70	0.050	0.00	95.44	95.43

Zeppelin 3 StormCAD Model
FlexTable: Outfall Table
Active Scenario: 5-Year

Label	Elevation (Ground) (ft)	Elevation (Invert) (ft)	Boundary Condition Type	Hydraulic Grade (ft)	Flow (Total Out) (cfs)
06 (Storm)	103.06	101.00	Crown	102.50	4.15
10 (Storm)	98.92	95.83	User Defined Tailwater	97.69	29.56
17 (Storm)	100.76	98.21	User Defined Tailwater	99.34	10.69
O-2	97.45	94.73	Free Outfall	95.79	14.00
O-3	96.64	94.46	Free Outfall	95.07	4.30

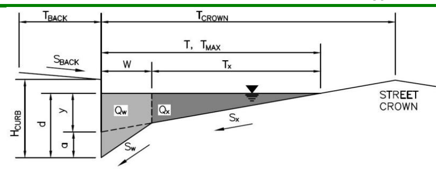
Version 4.05 Released March 2017

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

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

Project: Zeppelin 3 & 4

Inlet ID: Inlet A1



Gutter Geometry (Enter data in the blue cells)

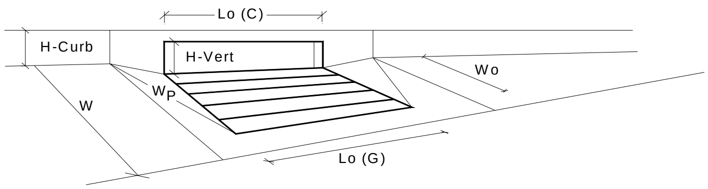
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	13.3	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.339	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	12.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	68.0	ft
Gutter Width	W =	3.00	ft
Street Transverse Slope	S_X =	0.039	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	68.0	68.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type C Grate)	Type =	CDOT Type C Grate		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	8.0	12.0	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} =	0.545	0.879	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} =	1.00	1.00	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**3.6**	**7.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.0	3.9	cfs

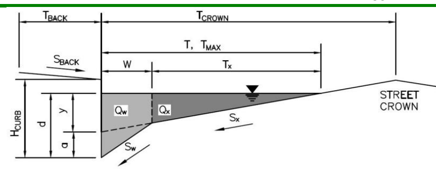
Version 4.05 Released March 2017

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

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

Project: Zeppelin 3 & 4

Inlet ID: Inlet A2



Gutter Geometry (Enter data in the blue cells)

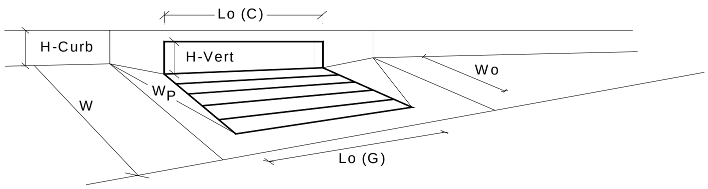
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	13.3	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.339	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	12.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	56.0	ft
Gutter Width	W =	3.00	ft
Street Transverse Slope	S_X =	0.030	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	56.0	56.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	12.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)	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} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	8.4	12.0	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} =	0.579	0.879	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} =	1.00	1.00	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**3.9**	**7.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	2.2	4.2	cfs

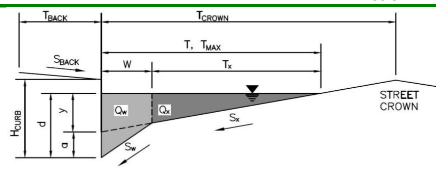
Version 4.05 Released March 2017

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

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

Project: **Zeppelin 3 & 4**

Inlet ID: **Inlet C1**



Gutter Geometry (Enter data in the blue cells)

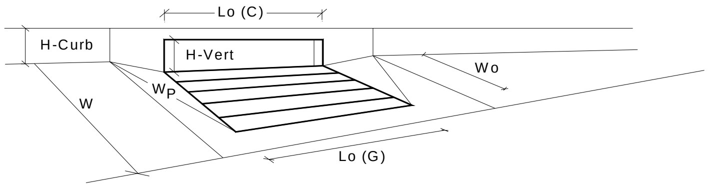
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.000	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	104.0	ft
Gutter Width	W =	4.00	ft
Street Transverse Slope	S_X =	0.008	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.008	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	60.0	104.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	6.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLEТ IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet — CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	5.4	6.0	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 =	4.00	4.00	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} =	0.42	0.47	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.69	0.77	
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 =	**8.4**	**8.8**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	*3.7	*7.8	cfs

*Please refer to the CIA Calculations Spreadsheets which do not include the roof drains as part of the Sub-Basin Qpeak values. The roof drain areas have been removed in those spreadsheets so a more accurate Q Peak value that will enter the storm drain inlets via the grates can be determined and used for the inlet sizing.

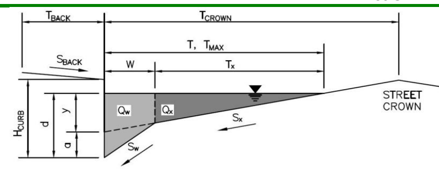
Version 4.05 Released March 2017

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

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

Project: **Zeppelin 3 & 4**

Inlet ID: **Inlet C2**



Gutter Geometry (Enter data in the blue cells)

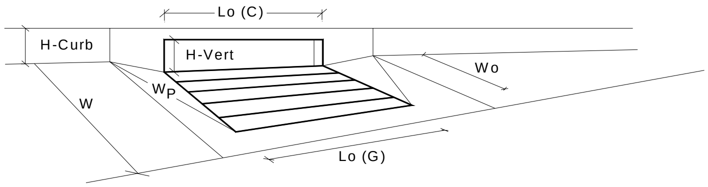
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.000	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	109.0	ft
Gutter Width	W =	4.00	ft
Street Transverse Slope	S_X =	0.016	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.016	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	80.0	109.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	6.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type C Grate)	Type =	CDOT Type C Grate		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	0.00	0.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	6	6	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	6.0	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} =	0.477	0.477	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} =	0.57	0.57	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**7.4**	**7.4**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	*2.1	*4.5	cfs

*Please refer to the CIA Calculations Spreadsheets which do not include the roof drains as part of the Sub-Basin Qpeak values. The roof drain areas have been removed in those spreadsheets so a more accurate Q Peak value that will enter the storm drain inlets via the grates can be determined and used for the inlet sizing.

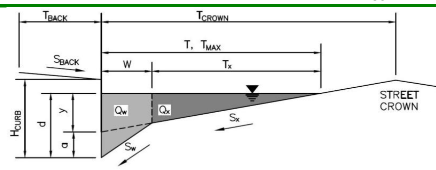
Version 4.05 Released March 2017

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

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

Project: Zeppelin 3 & 4

Inlet ID: Inlet B1



Gutter Geometry (Enter data in the blue cells)

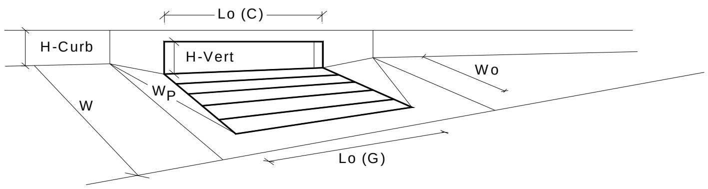
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	0.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.000	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	10.80	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	100.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.009	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.009	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	70.0	100.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	8.0	10.8	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet — CDOT/Denver 13 Valley Grate	Type =	CDOT/Denver 13 Valley Grate		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	2.00	2.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	2	2	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	7.6	10.8	inches
Grate Information		MINOR	MAJOR	☐ Override Depths
Length of a Unit Grate	L_o (G) =	3.00	3.00	feet
Width of a Unit Grate	W_o =	1.73	1.73	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	0.43	0.43	
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) =	3.30	3.30	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	0.60	0.60	
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} =	0.717	0.987	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} =	0.89	1.00	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**7.4**	**12.4**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	*6.1	*11.7	cfs

*Please refer to the CIA Calculations Spreadsheets which do not include the roof drains as part of the Sub-Basin Qpeak values. The roof drain areas have been removed in those spreadsheets so a more accurate Q Peak value that will enter the storm drain inlets via the grates can be determined and used for the inlet sizing.

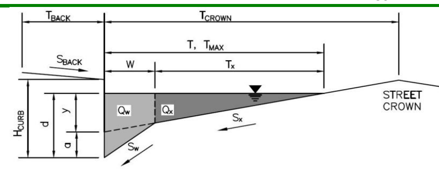
Version 4.05 Released March 2017

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

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

Project: **Zeppelin 3 & 4**

Inlet ID: **Inlet B2**



Gutter Geometry (Enter data in the blue cells)

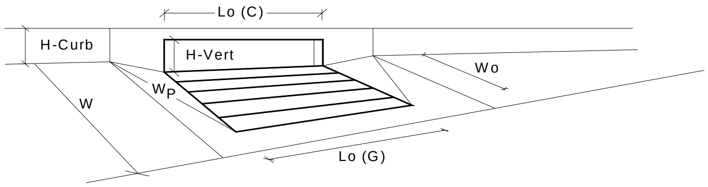
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	0.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.000	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.016	
Height of Curb at Gutter Flow Line	H_{CURB} =	10.80	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	100.0	ft
Gutter Width	W =	3.00	ft
Street Transverse Slope	S_X =	0.009	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.009	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	70.0	100.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	8.0	10.8	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT/Denver 13 Valley Grate)	Type =	CDOT/Denver 13 Valley Grate		
Local Depression (additional to continuous gutter depression 'a' from above)	a_{local} =	2.00	2.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	2	2	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	7.6	10.8	inches
Grate Information		MINOR	MAJOR	Override Depths
Length of a Unit Grate	L_o (G) =	3.00	3.00	feet
Width of a Unit Grate	W_o =	1.73	1.73	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	0.43	0.43	
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) =	3.30	3.30	
Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	0.60	0.60	
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} =	0.741	1.011	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} =	0.89	1.00	

		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**7.7**	**12.7**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	Q PEAK REQUIRED =	*5.6	*10.2	cfs

*Please refer to the CIA Calculations Spreadsheets which do not include the roof drains as part of the Sub-Basin Qpeak values. The roof drain areas have been removed in those spreadsheets so a more accurate Q Peak value that will enter the storm drain inlets via the grates can be determined and used for the inlet sizing.

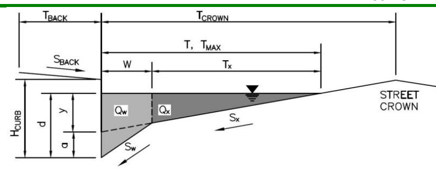
Version 4.05 Released March 2017

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

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

Project: Zeppelin 3 & 4

Inlet ID: Inlet B3



Gutter Geometry (Enter data in the blue cells)

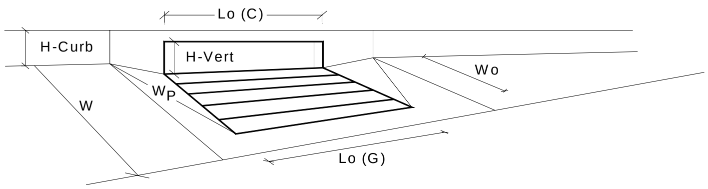
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	8.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.010	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	70.0	ft
Gutter Width	W =	4.00	ft
Street Transverse Slope	S_X =	0.025	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

			Minor Storm	Major Storm	
	Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	35.0	70.0	ft
Warning 02	Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	9.0	10.8	inches
	Check boxes are not applicable in SUMP conditions		☐	☐	

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

INLET IN A SUMP OR SAG LOCATION

Version 4.05 Released March 2017



Design Information (Input)	CDOT/Denver 13 Valley Grate		MINOR	MAJOR	
Type of Inlet		Type =	CDOT/Denver 13 Valley Grate		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	2.00	2.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	2	2	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	9.0	10.8	inches
Grate Information			MINOR	MAJOR	Override Depths
Length of a Unit Grate		L_o (G) =	3.00	3.00	feet
Width of a Unit Grate		W_o =	1.73	1.73	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)		A_{ratio} =	0.43	0.43	
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) =	3.30	3.30	
Grate Orifice Coefficient (typical value 0.60 - 0.80)		C_o (G) =	0.60	0.60	
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} =	0.809	0.959	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} =	1.00	1.00	
			MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)		Q_a =	**9.9**	**12.1**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)		Q PEAK REQUIRED =	*5.8	*10.8	cfs

*Please refer to the CIA Calculations Spreadsheets which do not include the roof drains as part of the Sub-Basin Qpeak values. The roof drain areas have been removed in those spreadsheets so a more accurate Q Peak value that will enter the storm drain inlets via the grates can be determined and used for the inlet sizing.

INLET PICTURES


CDOT Type R Curb Opening


Denver No. 14 Curb Opening

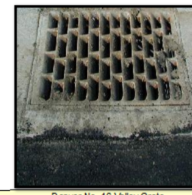

Colorado Springs D-10-R


CDOT/Denver 13 Valley Grate

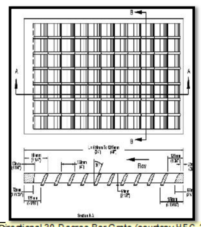

CDOT/Denver 13 Combination


Denver No. 16 Combination


Wheat Ridge Combination Inlet


Denver No. 16 Valley Grate

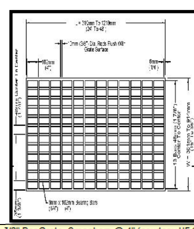

Directional Cast Vane Grate

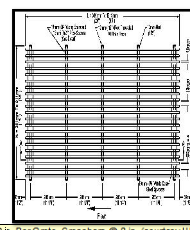

Directional 30-Degree Bar Grate (courtesy HEC-22)

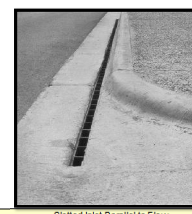

Directional 45-Degree Bar Grate


Reticuline Riveted Grate


1-7/8" Bar Grate, Crossbars @ 8"


1-7/8" Bar Grate, Crossbars @ 4" (courtesy HEC-22)


1-1/8 in. Bar Grate, Crossbars @ 8 in. (courtesy HEC-22)

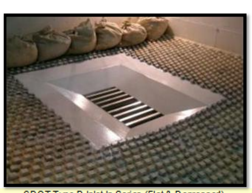

Slotted Inlet Parallel to Flow


CDOT Type C Grate (Close Mesh)


CDOT Type C Grate


CDOT Type C Inlet


CDOT Type C Inlet in Depression


CDOT Type D Inlet In Series (Flat & Depressed)


CDOT Type D Inlet In Series (10° Incline & Depressed)


CDOT Type D Inlet In Series (20° Incline & Depressed)


CDOT Type D Inlet In Series (30° Incline & Depressed)


CDOT Type D Inlet Parallel (Flat & Depressed)


CDOT Type D Inlet Parallel (10° Incline & Depressed)


CDOT Type D Inlet Parallel (20° Incline & Depressed)


CDOT Type D Inlet Parallel (30° Incline & Depressed)

APPENDIX E – STORMWATER EOPC

Kimley»Horn

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

Client:	**Scannell Properties**	**Date:**	**10/31/2019**
Project:	**2520 and 2540 Zeppelin Road**	**Prepared By:**	**MOH**
KHA No.:	**096441008**	**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 Northern Rain Gardens (Non-Reimbursible)				
1	Filter Media and Plants	1,170	SF	$25.00	$29,250
2	Underdrain	100	LF	$25.00	$2,500
3	CDOT Type C Inlet	2	EA	$5,000.00	$10,000
	Private Central Water Quality-Only Extended Detention Basin (Non-Reimbursible)				
1	Concrete Forebay	1	EA	$7,500.00	$7,500
2	Concrete Outlet Structure	1	EA	$10,000.00	$10,000
3	Micropool	1	EA	$6,000.00	$6,000
4	Concrete Trickle Channel	95	LF	$10.00	$950
5	Emergency Overflow	1	EA	$4,500.00	$4,500
	Private South Water Quality-Only Extended Detention Basin (Non-Reimbursible)				
1	Concrete Forebay	1	EA	$7,500.00	$7,500
2	Concrete Outlet Structure	1	EA	$10,000.00	$10,000
3	Micropool	1	EA	$6,000.00	$6,000
4	Concrete Trickle Channel	140	LF	$10.00	$1,400
5	Emergency Overflow	1	EA	$8,000.00	$8,000
		Subtotal Northern Rain Gardens:			$41,750
		Subtotal Central Detention Basin:			$28,950
		Subtotal South Detention Basin:			$32,900
		Subtotal (All BMPs):			**$103,600**
		Contingency (%,+/-)		10%	$10,360
		Project Total:			**$113,960**

Basis for Cost Projection:

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

Design Engineer:

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

Kimley»Horn

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

Client:	**SCANNELL PROPERTIES, LLC**	**Date:**	**9/11/2019**
Project:	**2520 and 2540 Zeppelin Road**	**Prepared By:**	**MOH**
KHA No.:	**096441008**	**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 (Non-Reimbursible)				
1	18" PVC Storm Pipe	1,010	LF	$125.00	$126,250
2	24" PVC Storm Pipe	225	LF	$150.00	$33,750
3	30" PVC Storm Pipe	340	LF	$165.00	$56,100
4	Double Type 13 Inlet	2	EA	$15,000.00	$30,000
5	CDOT Type C Inlet	3	EA	$17,000.00	$51,000
6	COS Type R 5' Inlet	1	EA	$4,500.00	$4,500
		Subtotal:			$175,350
		Contingency (%,+/-)		10%	$17,535
		Project Total:			**$192,885**

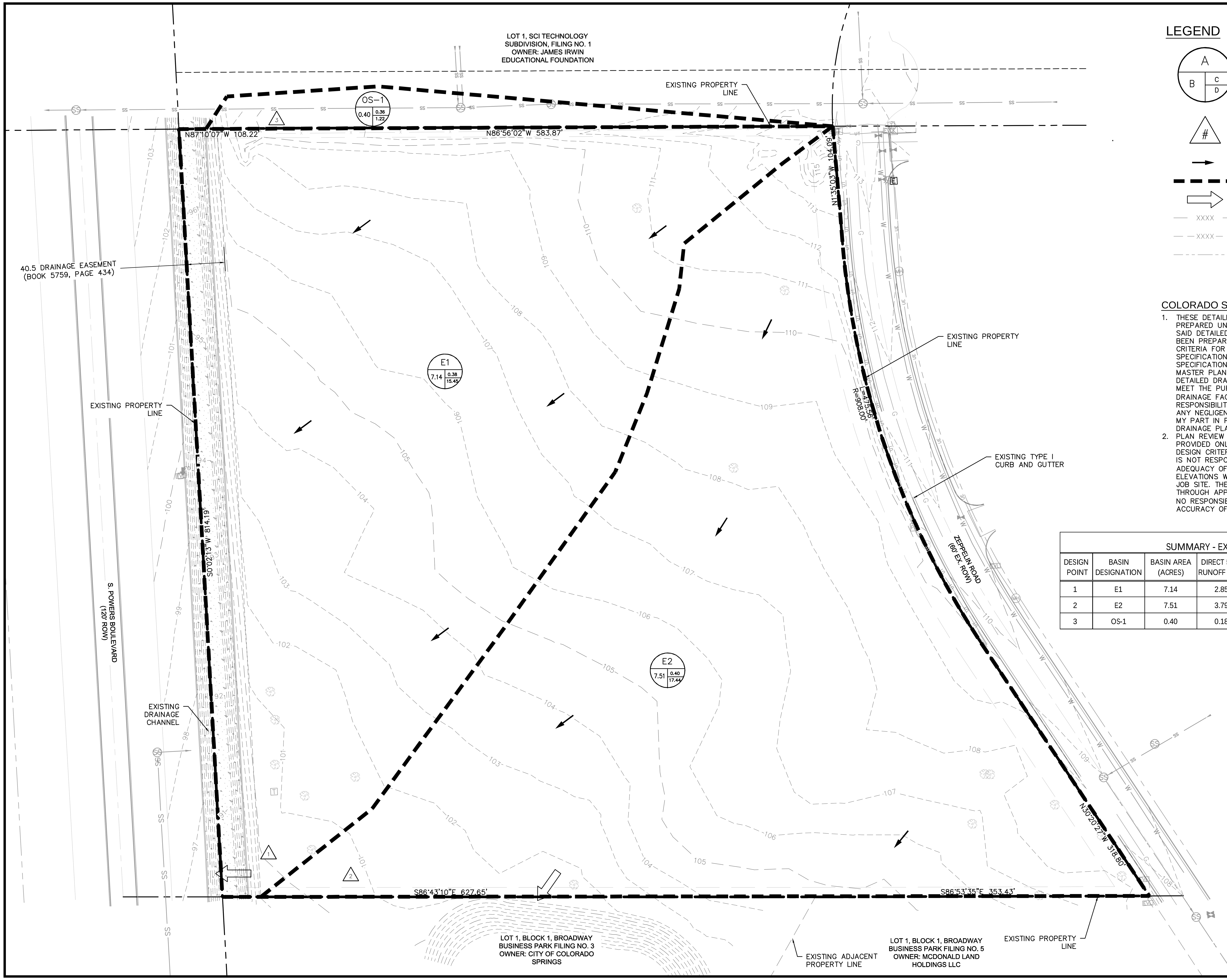
Basis for Cost Projection:

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

Design Engineer:

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

APPENDIX F – DRAINAGE MAPS



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
- EXISTING PROPERTY LINE

COLORADO SPRINGS GENERAL NOTE

1. THESE DETAILED PLANS AND SPECIFICATIONS WERE PREPARED UNDER MY DIRECTION AND SUPERVISION. SAID DETAILED PLANS AND SPECIFICATIONS HAVE BEEN PREPARED ACCORDING TO THE ESTABLISHED CRITERIA FOR DETAILED DRAINAGE PLANS AND SPECIFICATIONS, AND SAID DETAILED PLANS AND SPECIFICATIONS ARE IN CONFORMITY WITH THE MASTER PLAN OF THE DRAINAGE BASIN. SAID DETAILED DRAINAGE PLANS AND SPECIFICATIONS MEET THE PURPOSES FOR WHICH THE PARTICULAR DRAINAGE FACILITY(S) IS DESIGNED. I ACCEPT RESPONSIBILITY FOR ANY LIABILITY CAUSED BY ANY NEGLIGENT ACTS, ERRORS OR OMMISSIONS ON MY PART IN PREPARATION OF THE DETAILED DRAINAGE PLANS AND SPECIFICATIONS.
2. PLAN REVIEW BY CITY OF COLORADO SPRINGS IS PROVIDED ONLY FOR GENERAL CONFORMANCE WITH DESIGN CRITERIA. THE CITY OF COLORADO SPRINGS IS NOT RESPONSIBLE FOR THE ACCURACY AND ADEQUACY OF THE DESIGN, DIMENSIONS, AND/OR ELEVATIONS WHICH SHALL BE CONFIRMED AT THE JOB SITE. THE CITY OF COLORADO SPRINGS, THROUGH APPROVAL OF THIS DOCUMENT, ASSUMES NO RESPONSIBILITY FOR COMPLETENESS AND/OR ACCURACY OF THIS DOCUMENT.

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)
1	E1	7.14	2.85	15.45	2.85	15.45
2	E2	7.51	3.79	17.44	3.79	17.44
3	OS-1	0.40	0.18	1.22	0.18	1.22

NO.	REVISION	DATE	BY	APPR.

Kimley»Horn

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2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0180

DESIGNED BY: EJG
DRAWN BY: JAR
CHECKED BY: EJG
DATE: 10/31/2019

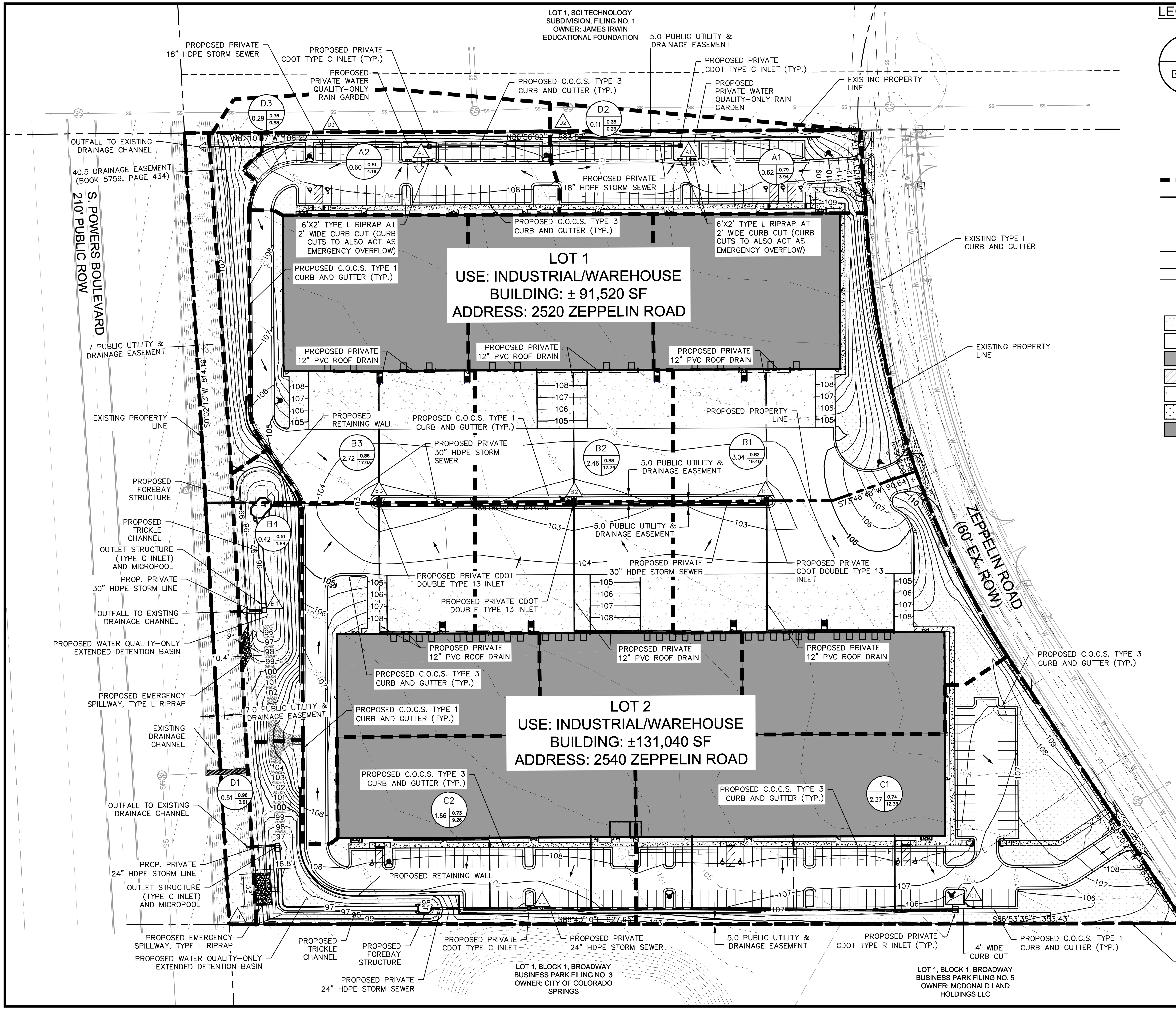
LOT 1&2, BROADVIEW BUSINESS PARK FILING NO. 6
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS
EXISTING DRAINAGE MAP

COLORADO REGISTERED PROFESSIONAL ENGINEER 48487 12/16/19

PROJECT NO. 096441008

SHEET 1

K:\COS_Civil\096441008_Zeppelin 3 and 4\CADD\Sheets\CDs\096441008DR_EX.dwg Hess, Mitchell 10/31/2019 1:41 PM



LEGEND

- A = BASIN DESIGNATION
- B = AREA (ACRES)
- C = 100-YR COMPOSITE RUNOFF COEFFICIENT
- D = 100-YR DIRECT STORM RUNOFF (CFS)
- DESIGN POINT
- FLOW DIRECTION
- EMERGENCY OVERFLOW PATH
- DRAINAGE BASIN BOUNDARY
- PROPERTY LINE
- EASEMENT
- EASEMENT
- SETBACK
- PROPOSED MAJOR CONTOUR
- PROPOSED MINOR CONTOUR
- EXISTING MAJOR CONTOUR
- EXISTING MAJOR CONTOUR
- CONCRETE SIDEWALK
- LIGHT DUTY ASPHALT
- STANDARD DUTY ASPHALT
- HEAVY DUTY ASPHALT
- HEAVY DUTY CONCRETE
- LANDSCAPE AREA (REF: LANDSCAPE PLAN)
- BUILDING HATCH

SUMMARY - PROPOSED RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)
A1	A1	0.62	2.00	3.94
A2	A2	0.60	2.15	4.19
B1	B1	3.04	10.18	19.40
B2	B2	2.46	9.71	17.79
B3	B3	2.72	9.67	17.93
B4	B4	0.42	0.27	1.39
C1	C1	2.37	6.11	12.33
C2	C2	1.66	4.58	9.26
D1	D1	0.51	2.02	3.61
D2	D2	0.11	0.04	0.29
D3	D3	0.29	0.13	0.88

NOTES

1. THESE DETAILED PLANS AND SPECIFICATIONS WERE PREPARED UNDER MY DIRECTION AND SUPERVISION. SAID DETAILED PLANS AND SPECIFICATIONS HAVE BEEN PREPARED ACCORDING TO THE ESTABLISHED CRITERIA FOR DETAILED DRAINAGE PLANS AND SPECIFICATIONS, AND SAID DETAILED PLANS AND SPECIFICATIONS ARE IN CONFORMITY WITH THE MASTER PLAN OF THE DRAINAGE BASIN. SAID DETAILED DRAINAGE PLANS AND SPECIFICATIONS MEET THE PURPOSES FOR WHICH THE PARTICULAR DRAINAGE FACILITY(S) IS DESIGNED. I ACCEPT RESPONSIBILITY FOR ANY LIABILITY CAUSED BY ANY NEGLIGENT ACTS, ERRORS OR COMMISSIONS ON MY PART IN PREPARATION OF THE DETAILED DRAINAGE PLANS AND SPECIFICATIONS.
2. PLAN REVIEW BY CITY OF COLORADO SPRINGS IS PROVIDED ONLY FOR GENERAL CONFORMANCE WITH DESIGN CRITERIA. THE CITY OF COLORADO SPRINGS IS NOT RESPONSIBLE FOR THE ACCURACY AND ADEQUACY OF THE DESIGN, DIMENSIONS, AND/OR ELEVATIONS WHICH SHALL BE CONFIRMED AT THE JOB SITE. THE CITY OF COLORADO SPRINGS, THROUGH APPROVAL OF THIS DOCUMENT, ASSUMES NO RESPONSIBILITY FOR COMPLETENESS AND/OR ACCURACY OF THIS DOCUMENT.

NORTH

GRAPHIC SCALE IN FEET
0 25 50 100

NO.	REVISION	DATE	BY	APPR.

Kimley»Horn
© 2019 KIMLEY-HORN AND ASSOCIATES, INC.
2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0180

DESIGNED BY: EJG
DRAWN BY: JAR
CHECKED BY: EJG
DATE: 11/26/2019

LOT 1&2, BROADVIEW BUSINESS PARK FILING NO. 6
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS
PROPOSED DRAINAGE MAP

COLORADO REGISTERED PROFESSIONAL ENGINEER 48487 12/16/19

PROJECT NO. 096441008

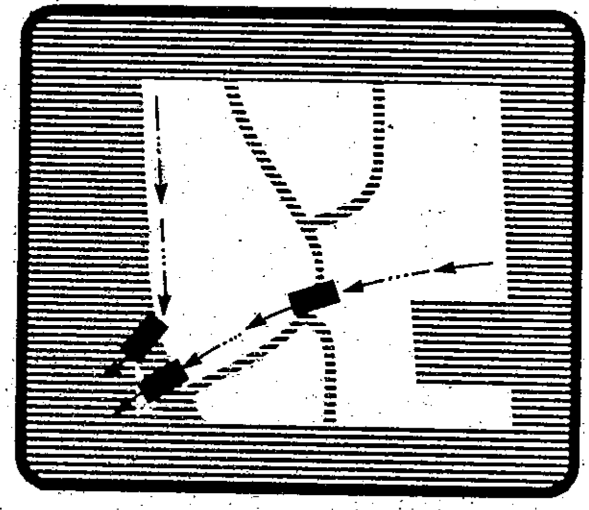
SHEET 2

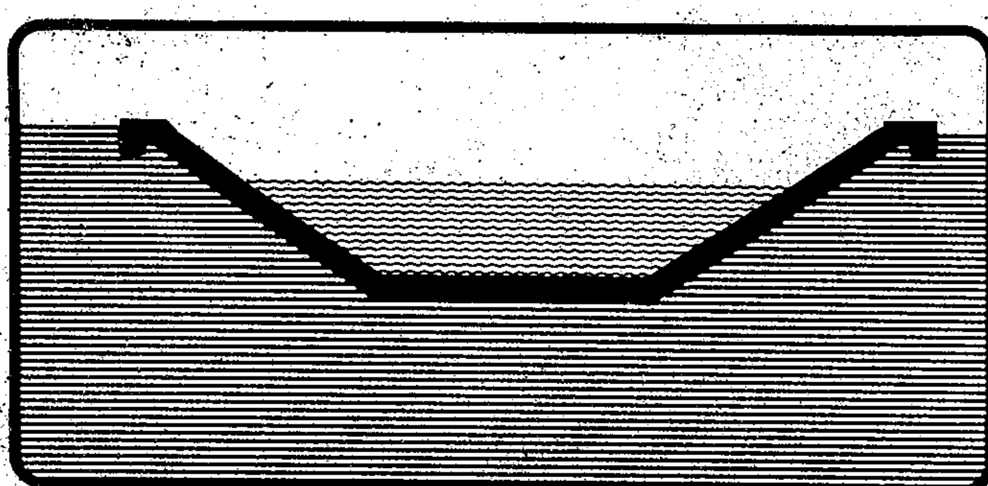
APPENDIX G – PETERSON AIRFIELD DRAINAGE STUDY

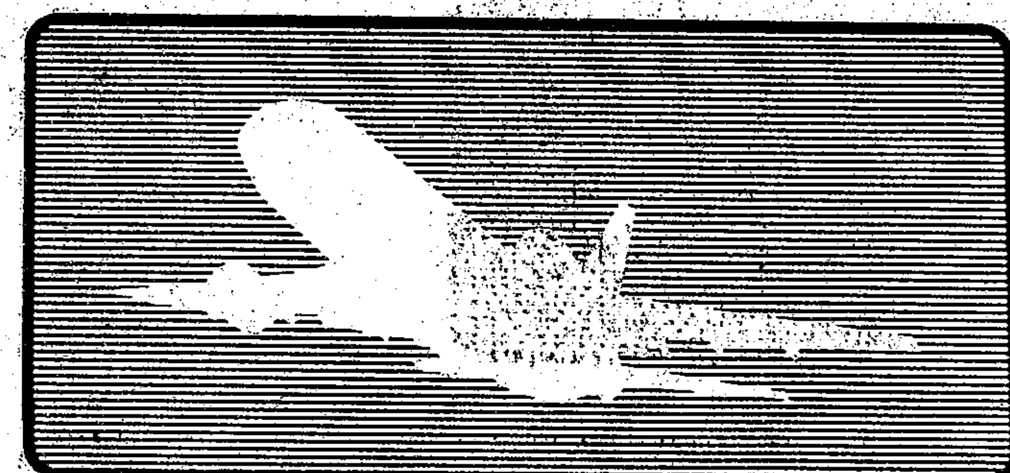
Peterson Field Drainage Basin Master Plan Update

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

CERTIFICATION

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 C. Behrens, P.E.

APPROVAL

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 7 8 9 10 11 12 1 2 3 4 5 6 PM

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

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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.

URS

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

SECTION PAGE

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:

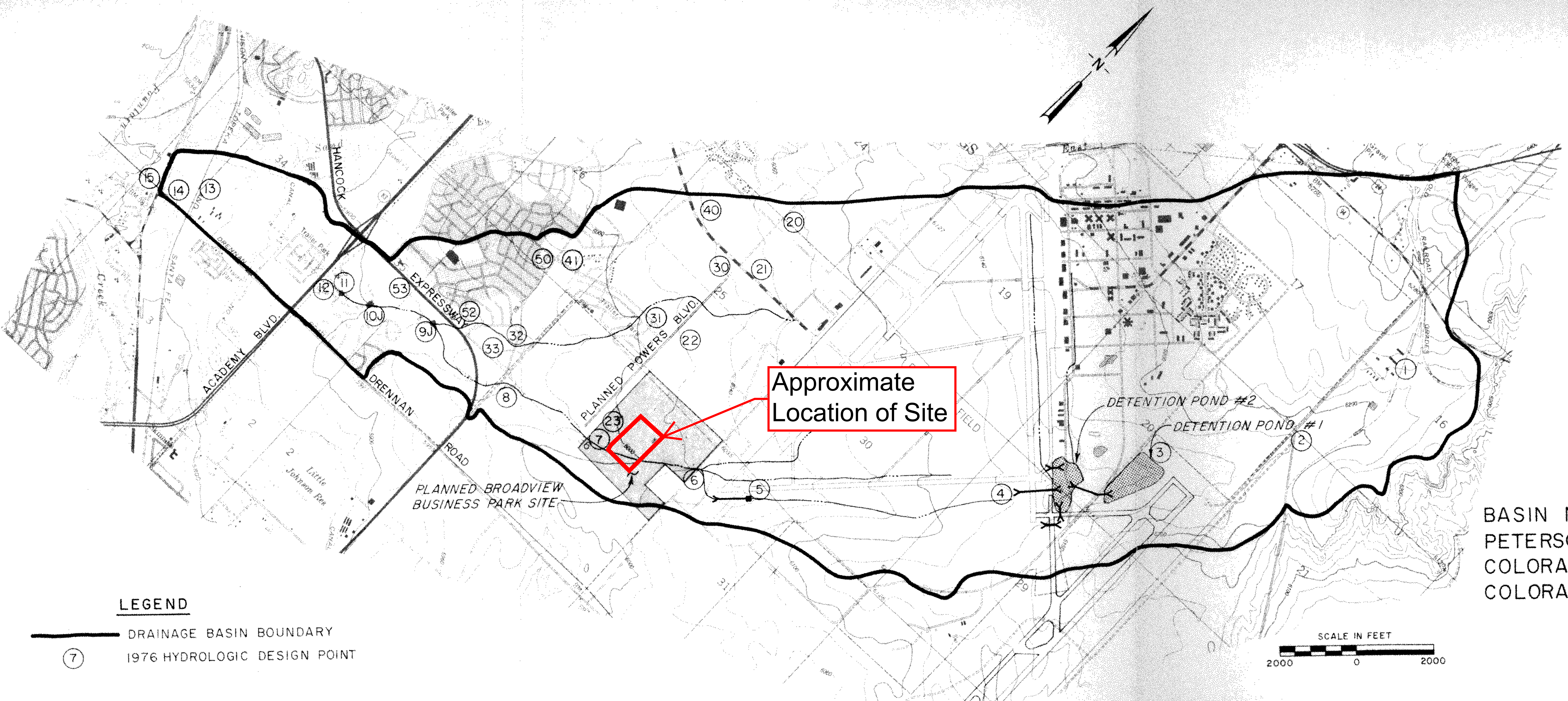
(1) the hydrologic impact of existing Peterson Field storm water detention ponds #1 and #2 on future fully developed 100-year flood flows and;

(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.

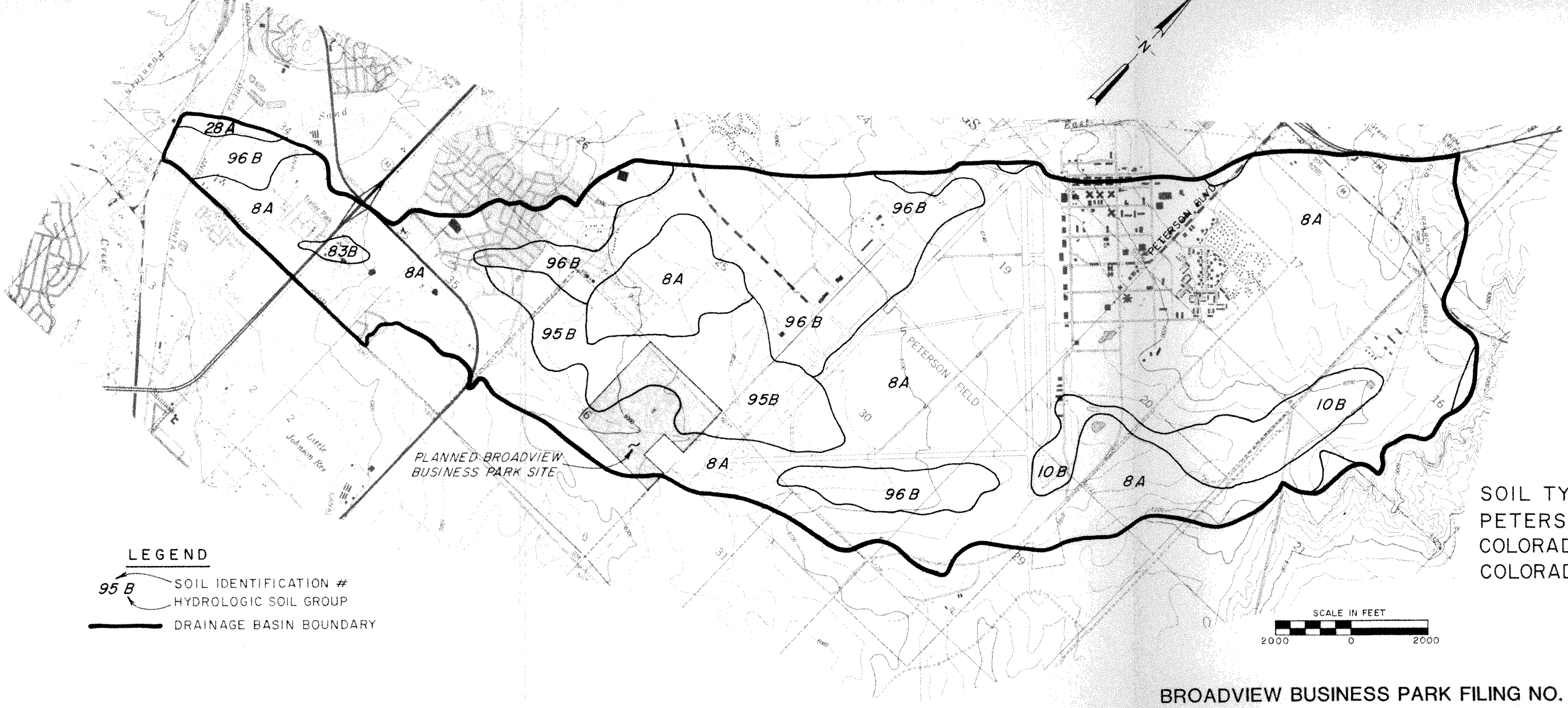
Peterson Field Basin outfalls to Sand Creek which in turn outfalls to Fountain Creek. Sand Creek Basin is a major drainage planning basin located north of the Peterson Field Basin. Chandelle and Windmill Gulch basins are major drainageway planning basins located south of the Peterson Field Basin. Peterson 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.

APPENDIX H – POWERS BOULEVARD DRAINAGE STUDY

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

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.

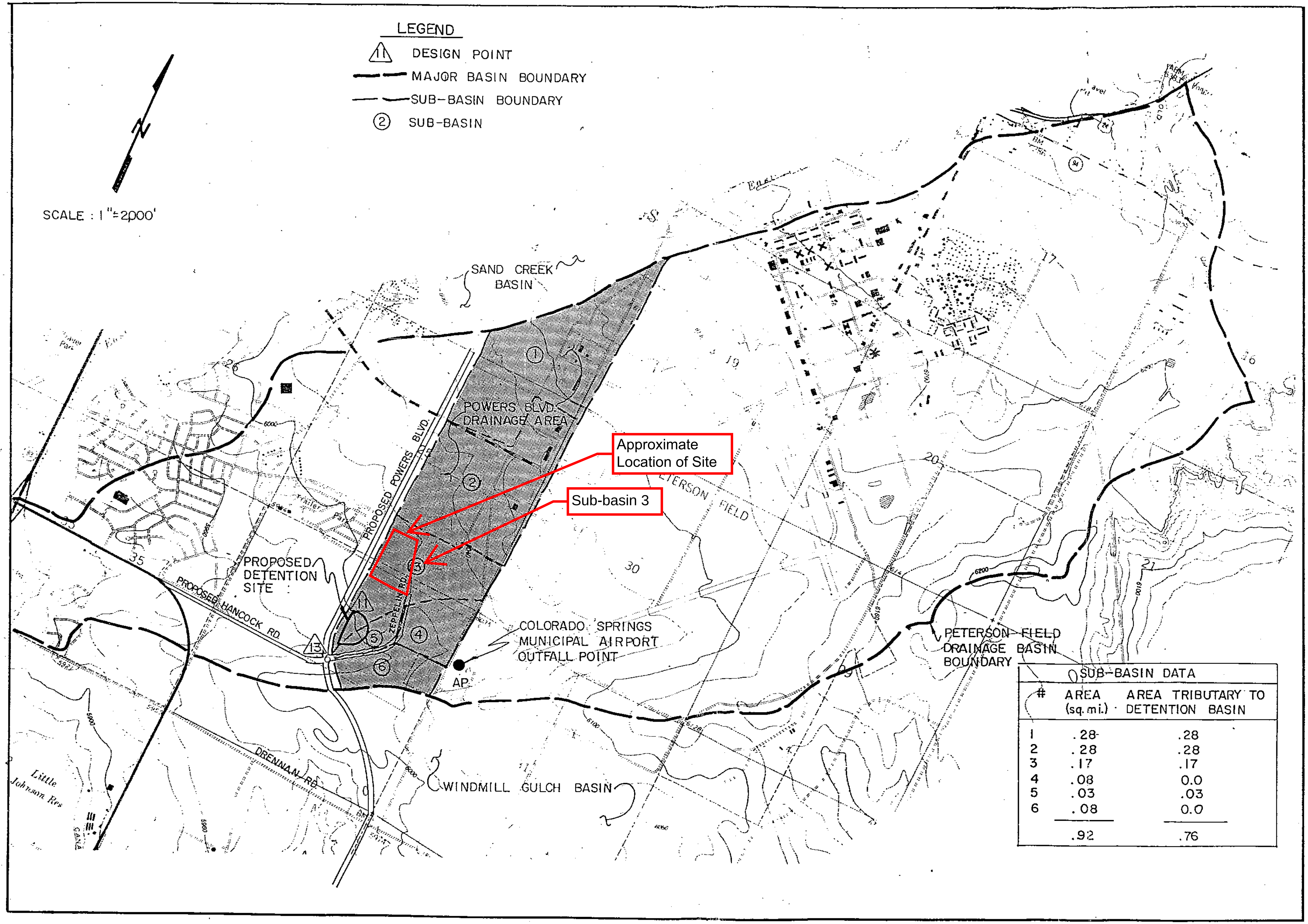
The emergency spillway has been sized to convey the developed 100-year peak flow out of the pond, assuming that the principal outlet is blocked. A riprap weir, of approximately 400 feet in length and a 100-year depth of 1.5 feet, has been sized for the detention basin. The crest elevation has been set 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

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

POWERS BLVD. DETENTION FACILITY
FINAL DRAINAGE REPORT
DRAINAGE BASIN BOUNDARIES

Project No. 89-08-16
Date: 9/89
Design: RNW
Drawn: EAK
Check:
Revisions:

FIGURE 1

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

XSECTION/ 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"
 ITLE     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
3                       82.5        0.          0.
8                       83.0        6.          0.4
8                       84.0        16.         2.3
)                       85.0        21.         7.
j                       86.0        50.         12.5
8                       87.0        100.        19.5
3                       88.0        160.        30.
)                       89.0        200.        40.5
8                       90.0        350.        52.5
R                       91.0        550.        65.
)                       91.5        610.        71.0
3                       92.5        670.        90.
 9 ENDTBL
 ! XSECTN     2         1.0
)                       6020.       0.          0.
8                       6021.       76.2        8.5
R                       6022.       260.9       20.
)                       6023.       556.7       34.5
3                       6024.       975.3       52.0
8                       6025.       1529.5      72.5
```

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

 9 ENDTBL
 ! XSECTN     3         1.0
)                       6020.       0.          0.
8                       6021.       101.8       11.5
3                       6022.       338.9       26.
)                       6023.       704.1       43.5
8                       6024.       1204.4      64.
                        6025.       [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
```

```
          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
```

********************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
3         168.9     168.7     168.5     168.4     168.2
8         168.1     168.1     168.0     167.9     167.9
3         167.9     167.8     167.8     167.8     167.8
3         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
}         168.2     168.2     168.3     168.3     168.4
d         168.4     168.5     168.6     168.6     168.7
8         168.8     168.9     168.9     169.0     169.1
}         169.2     169.3     169.3     169.4     169.5
}         169.6     169.7     168.9     167.0     165.1
8         163.3     161.9     160.7     159.9     159.3
9         159.0     158.9     158.4     158.1     158.0
}         158.0     158.1     158.1     158.0     157.9
d         157.9     158.0     158.2     158.2     158.2
8         158.1     158.1     158.2     158.4     158.4
          158.4     158.3     158.3     158.4     158.6
}         158.7     158.6     158.5     158.8     158.6
8         158.8     158.9     158.8     158.7     158.7
          158.8     158.9     159.0     159.0     158.9
:         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

3	167.90	168.00	168.00	168.10	168.10
3	168.20	168.20	168.30	168.30	168.40
8	168.40	168.50	168.60	168.60	168.70
7	168.80	168.90	168.90	169.00	169.10
}	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
}	159.00	158.90	158.40	158.10	158.00
J	158.00	158.10	158.10	158.00	157.90
8	157.90	158.00	158.20	158.20	158.20
}	158.10	158.10	158.20	158.40	158.40
}	158.40	158.30	158.30	158.40	158.60
8	158.70	158.60	158.50	158.80	158.60
9	158.80	158.90	158.80	158.70	158.70
}	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
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
3	6020.00	.00	.00
8	6021.00	50.40	7.50
8	6022.00	175.50	18.00
3	6023.00	379.50	31.50
3	6024.00	673.30	48.00
8	6025.00	1064.60	67.50
8	6026.00	1569.20	90.00
3	6027.00	2187.70	115.50
8	6028.00	2944.40	144.00

1

	ELEVATION	DISCHARGE	END AREA
3	6028.50	3359.80	159.40

ENDTBL

	XSECTN NO.	DRAINAGE AREA
2 XSECTN	5	1.0000

	ELEVATION	DISCHARGE	END AREA
	6020.00	.00	.00
8	6021.00	56.30	7.50
	6022.00	196.20	18.00
	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
	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
		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

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

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

[illegible]	.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

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.

TIME(HRS)											
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.

TIME(HRS)											
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

TIME(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

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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 = 1.62 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
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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
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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.

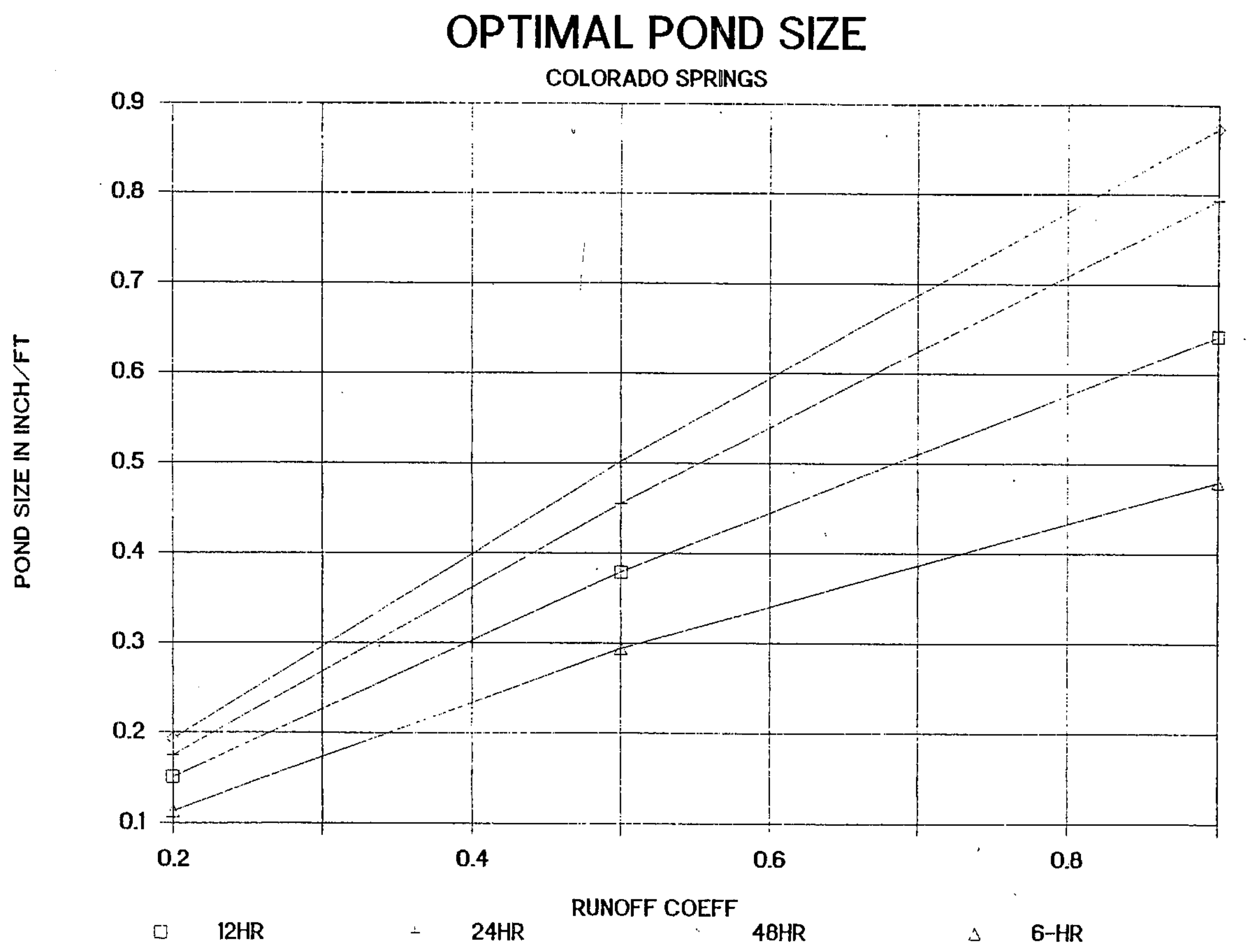
OPTIMAL POND SIZE AND RUNOFF CAPTURE RATE
FOR COLORADO SPRINGS

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.

[1] Chief, Master Planning Program, Urban Drainage and Flood Control District, Denver, Colorado.

[2] Associate Professor, University of Colorado at Denver

[3] Executive Director, Urban Drainage and Flood Control District, Denver, Colorado.

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.

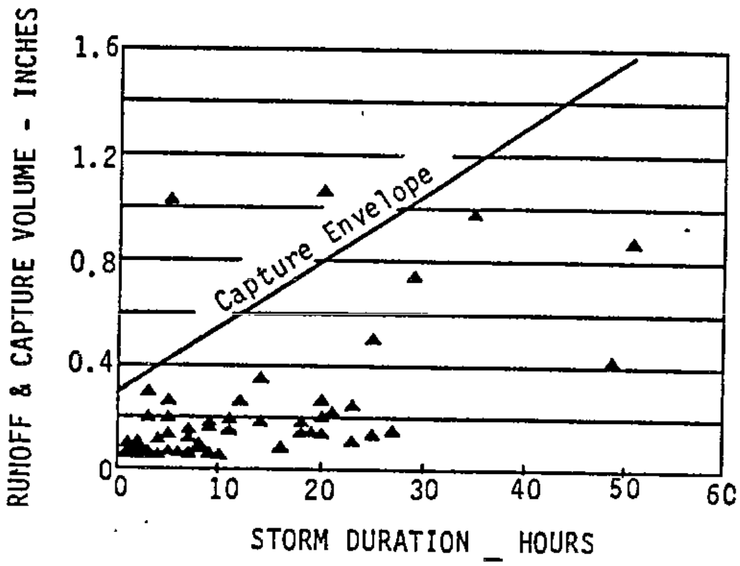


Figure 1. Runoff Volume Point Diagram and Capture Volume Envelope. (1-inch = 24.5 millimeters)

<u>Storage Volume Optimization Procedure</u>

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.

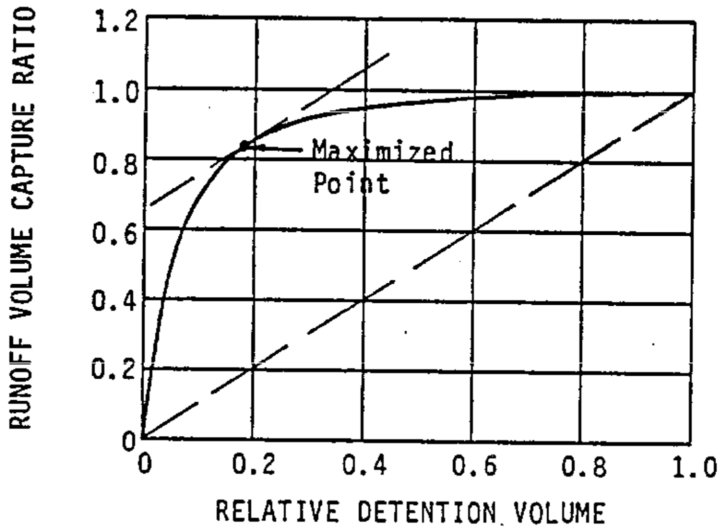


Figure 2. Maximizing Capture Volume.

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,

$$P = P_r \, P_m$$
$$= (0.18)\ (0.5\ 3.04)$$
$$= 0.27 \text{ watershed inches (6.86 millimeters)}$$

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.

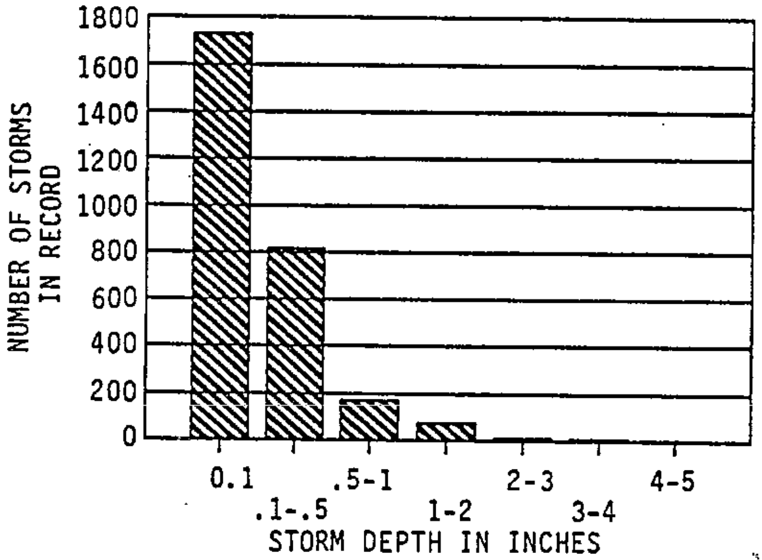


Figure 3. Number of Storms in Denver vs. Storm depth. (One inch = 25.4 millimeters)

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.

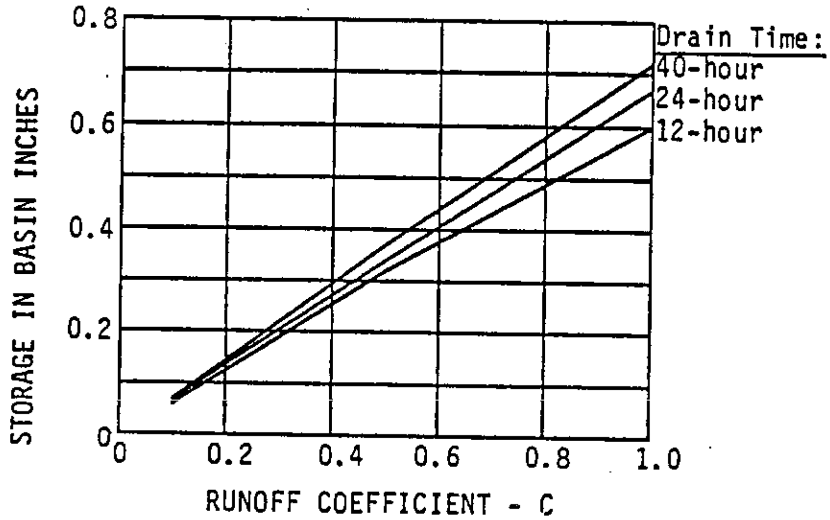


Figure 4. Maximized Capture Volume for Water Quality, Denver Rain Gauge 1944-84 Period. (One inch = 25.4 millimeters)

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.

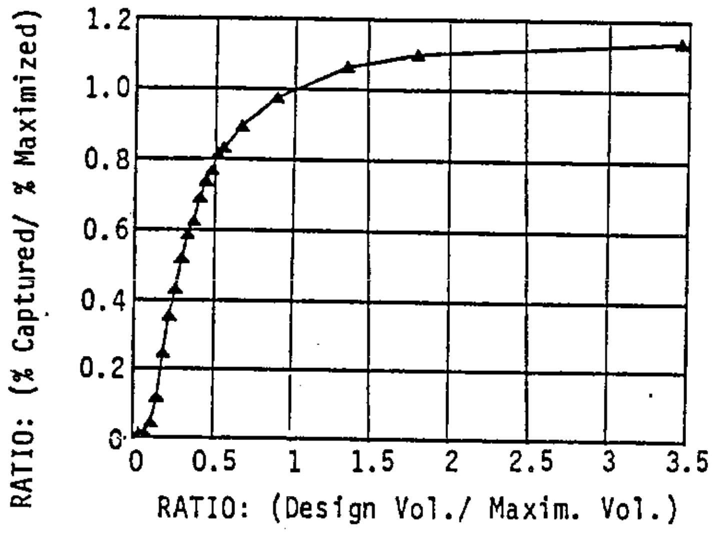


Figure 5. Sensitivity of Capture Volume Size.

<u>Removal of Suspended Sediments</u>

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.

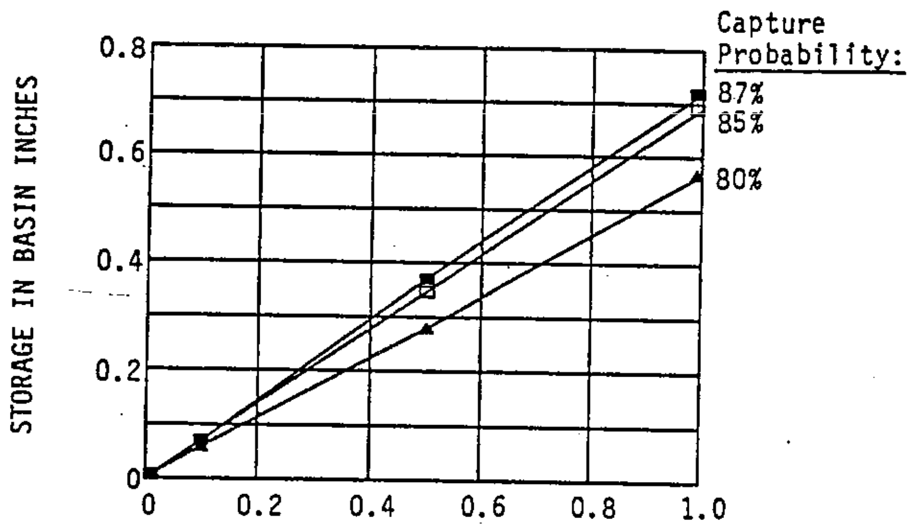


Figure 6. Capture Volumes for a 40-hour Drain Time and Several Runoff Event Capture Probabilities.

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.

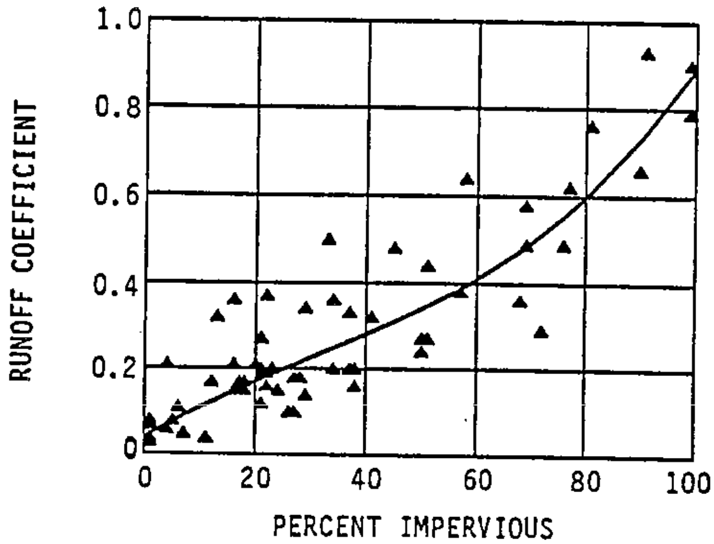


Figure 7. Runoff Coefficient Based on NURP Data for 2-year and Smaller Storms.

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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FOR COORDINATE DATA & GENERAL LAYOUT, REFER TO SHEET 3.

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.

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

Detention Pond Plan
9005-07 Sheet 4

PLAN	SURVEYED	PLOTTED	CHECKED
NOTE BOOK	REVIEWED	DESIGNED	

PROFILE

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	CONTRACT 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 Facility: D Sheet 4 of 13

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.
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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 ______
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ASPHALT THICKNESS:
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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 U.S.G.S.

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 Sheet 5 of 13 Facility: D

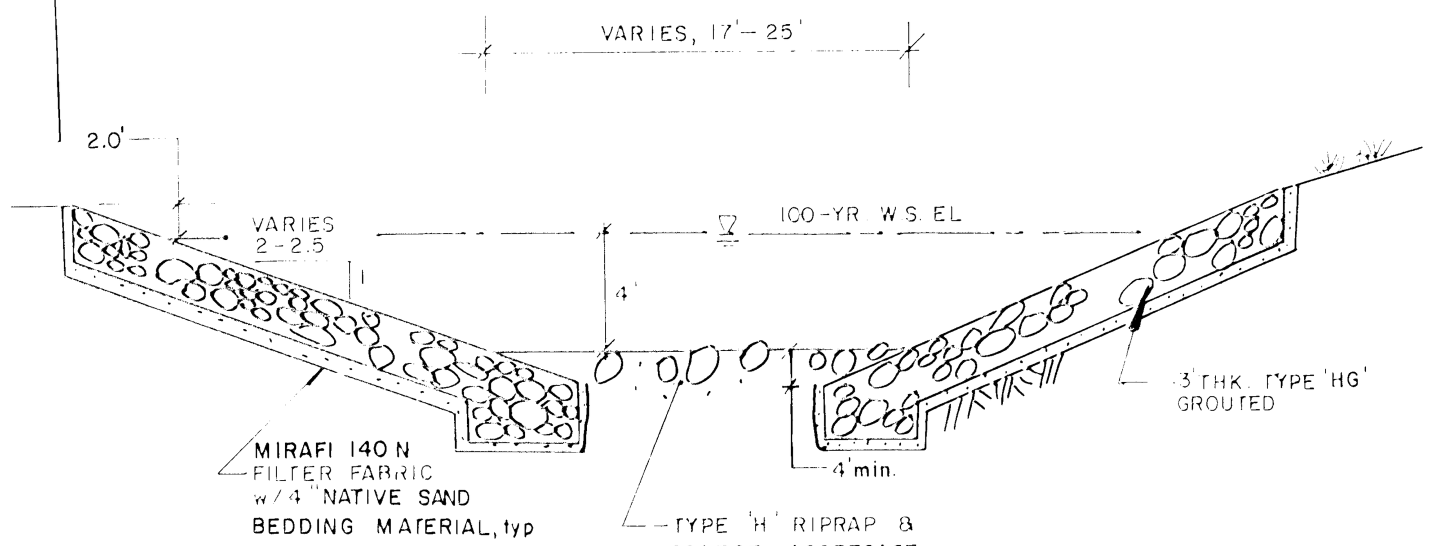
9005-07-5

RIPRAP GRADATIONS

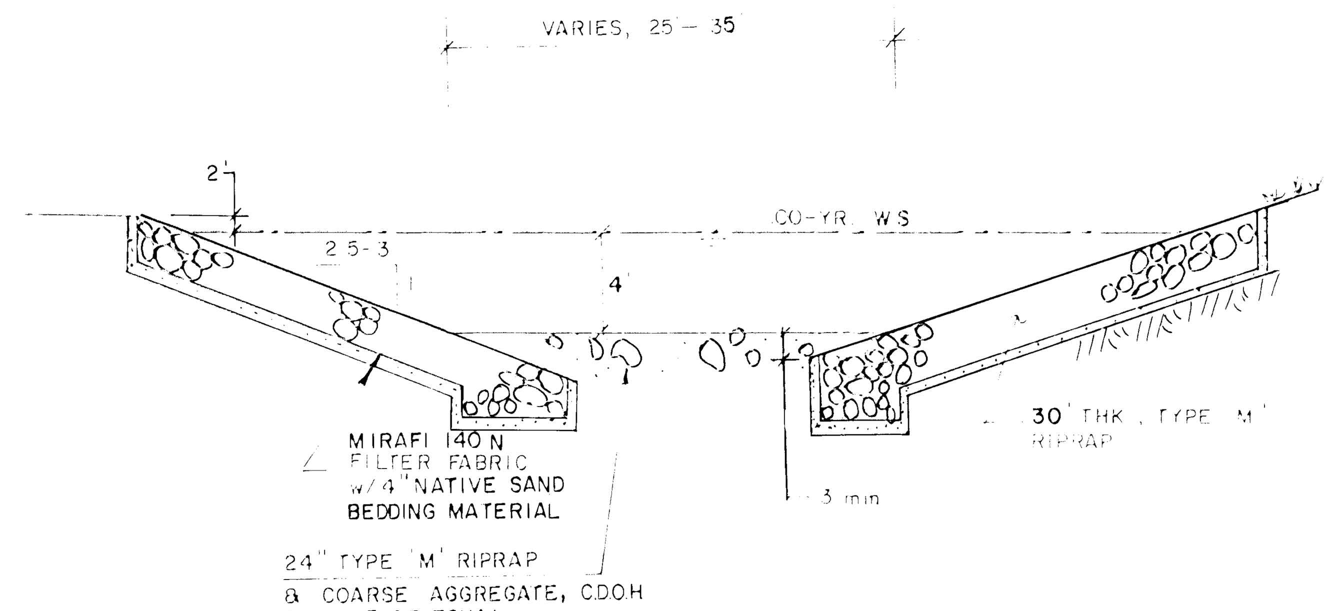
Rip Rap Designation	% Smaller Than Given Size by Weight	Intermediate Rock Dimension (inches)	d_{50}* (inches)
Type L	70 - 100	15	
	50 - 70	12	
	35 - 50	9	9**
	2 - 10	3	
Type M	70 - 100	21	
	50 - 70	18	
	35 - 50	12	12
	2 - 10	4	
Type H	100	30	
	50 - 70	24	
	35 - 50	18	18
	2 - 10	6	

GROUTED RIPRAP GRADATIONS

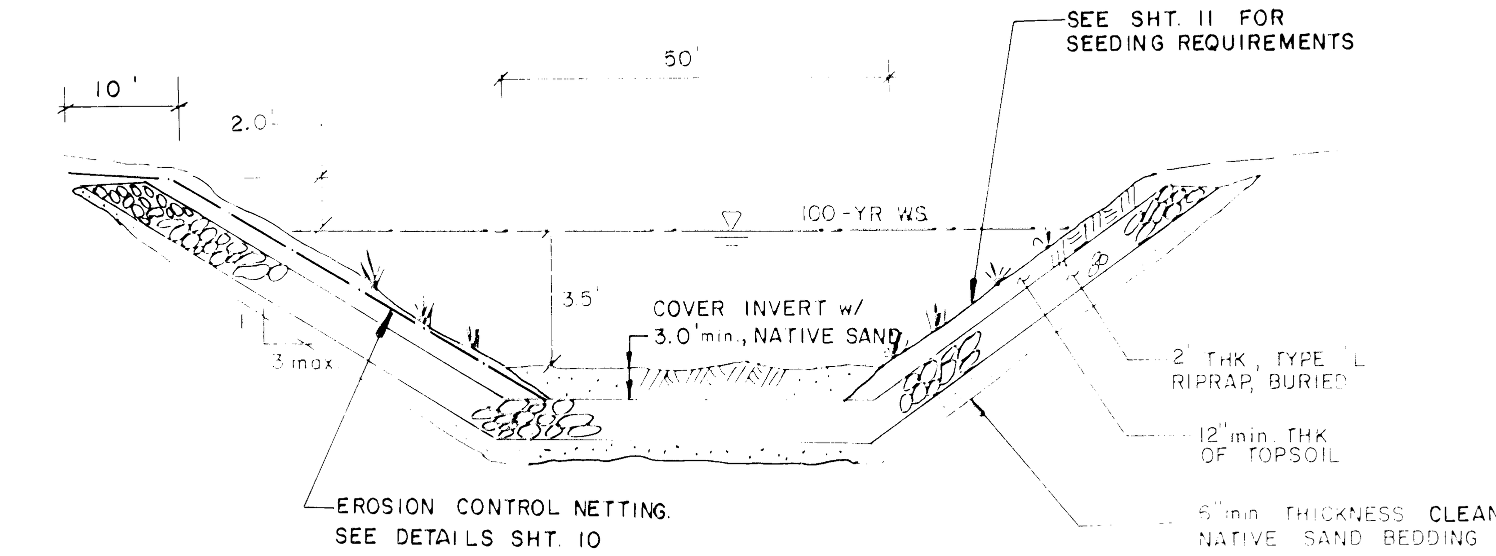
TYPE HG	100	30
	50-70	24
	0-5	18



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 STOCKPILED 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.

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 USGS.

(1) CONTRACT DOCUMENTS 7-16-90

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

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 Sheet 6 of 13 Facility: D

9005-07-6

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

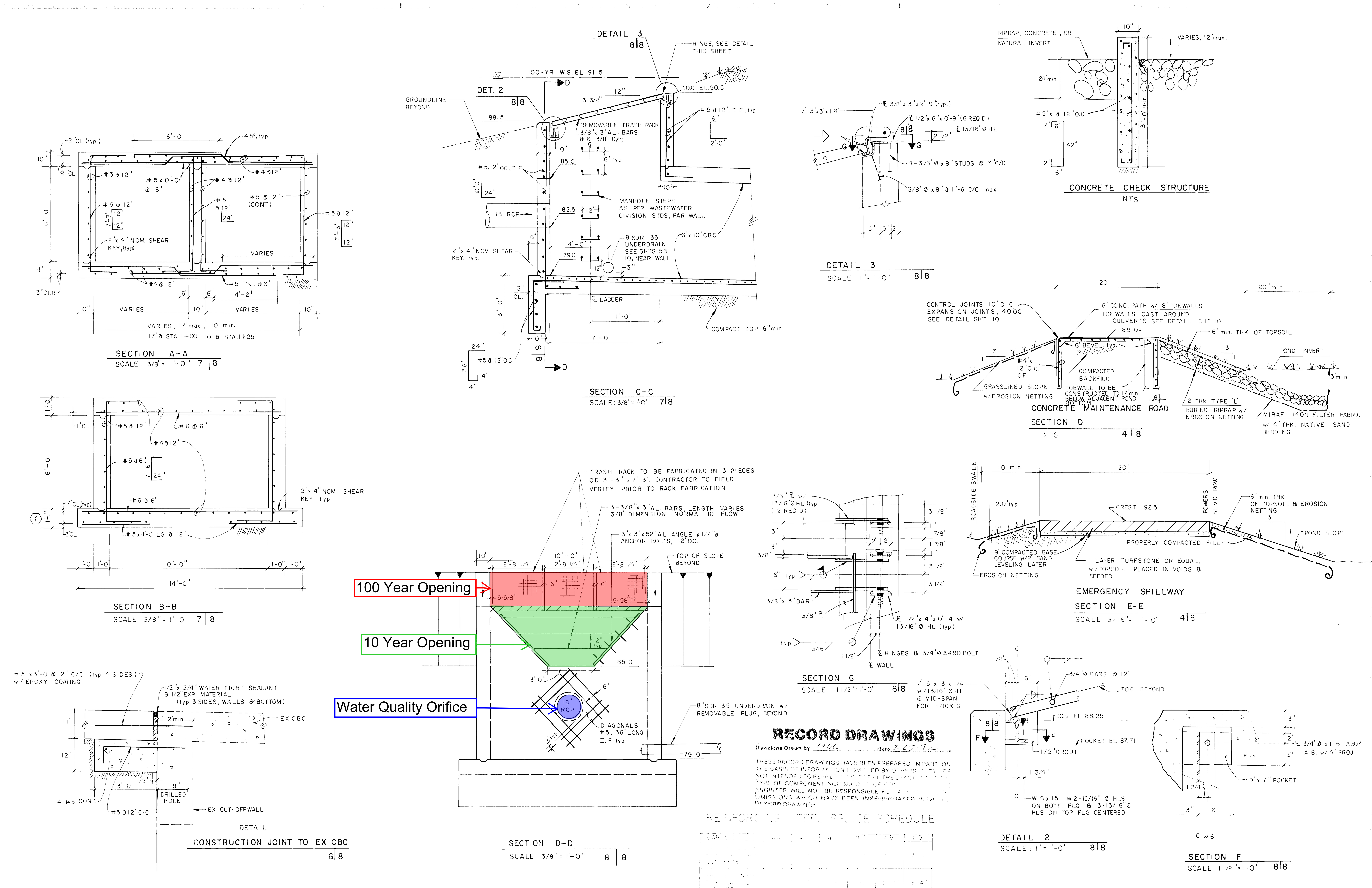
DESIGNED BY	WHY	DATE	4/90
DRAWN BY	EAK	DATE	4/90
CHECKED BY	RNW	DATE	5/90

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 Outlet Structure Plan Sheet 7
9005-07 Sheet 7 of 13 Facility: D



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 US 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

APPENDIX I – VARIANCE REQUEST

Kimley»Horn

November 16, 2019

City of Colorado Springs
Water Resources Engineering Review
Engineering Division
30 S. Nevada Ave., Suite 401
Colorado Springs, CO 80903

ATTN: Jonathan Scherer

RE: Broadview Business Park Filing 6 (Zeppelin III and IV) - Variance Letter

Dear Mr. Scherer:

We respectfully request the City's consideration of our request for a variance from the following criteria:

Inlet may not be used as junctions along trunk lines. DCM Volume 1 (Chapter 9, Section 6.2)

Background:
Broadview Business Park Filing No. 6 (Zeppelin III and IV) consists of a 14.66-acre development located on Parcel #64361000180 within the City of Colorado Springs, County of El Paso, State of Colorado. The development involves the construction of two industrial distribution warehouses, each located on a separate lot. The Property is bounded by a regional detention pond and industrial distribution site to the south (Lot 1 BLK 1 Broadview Business Park Filing No. 3 & Lot 1 Broadview Business Park Filing No. 5), the James Irwin Charter Elementary School to the north (Lot 1 Sci Technology Sub Filing No. 1), Powers Boulevard to the west and Zeppelin Road to the east.

The following variances are respectfully requested:

1. **Inlets may not be used as junctions along trunk lines (Chapter 9, Section 6.2)** – Per the DCM, "Inlets may be used as junction structures in place of manholes to connect adjacent inlets if the interconnecting pipe can be fit within the standard inlet dimensions without modification to the inlet and if the additional flow can be passed through the structure in accordance with standard hydraulic criteria. Inlets may not be used as junctions along trunk lines."

- One of the proposed private storm drain lines (Storm Drain B) connects three area inlets before outfalling into a proposed water quality-only extended detention basin. Storm Drain B is proposed to be located between the two truck courts that will service the two industrial distribution buildings. The storm drain will be located within a median landscape area between the two buildings and stormwater from both truck dock areas will flow to curb cuts in the landscape median before entering the storm drain inlets.

<u>Justification:</u>
In the above condition, the storm drain line will be more easily maintained because no laterals are proposed. If the proposed storm drain line cannot include inlets on the private mainline, an additional two to three storm drain structures would need to be added which would increase the number of structures that would need annual maintenance, needlessly complicating the storm drain system.

Furthermore, the additional structures (manholes) would need to be located within one of the distribution center's truck court areas. Maintenance of the system would require personnel to open and work in and around storm drain structures while in an often busy truck court used by semi-trucks. The current proposed storm drain system would be located entirely within a landscaped median, allowing maintenance personnel to safely access the storm drain system.

It is our professional engineering opinion that this variance is justified and that it will promote more efficient, effective and safe maintenance of the storm drain system proposed for this development. Based upon this request, the overall design approval will not negatively affect the downstream storm sewer and stormwater conditions. The design will not result in any increase in flows nor will it result in any decrease in water quality in Fountain Creek.

We respectfully request your favorable consideration of this request.

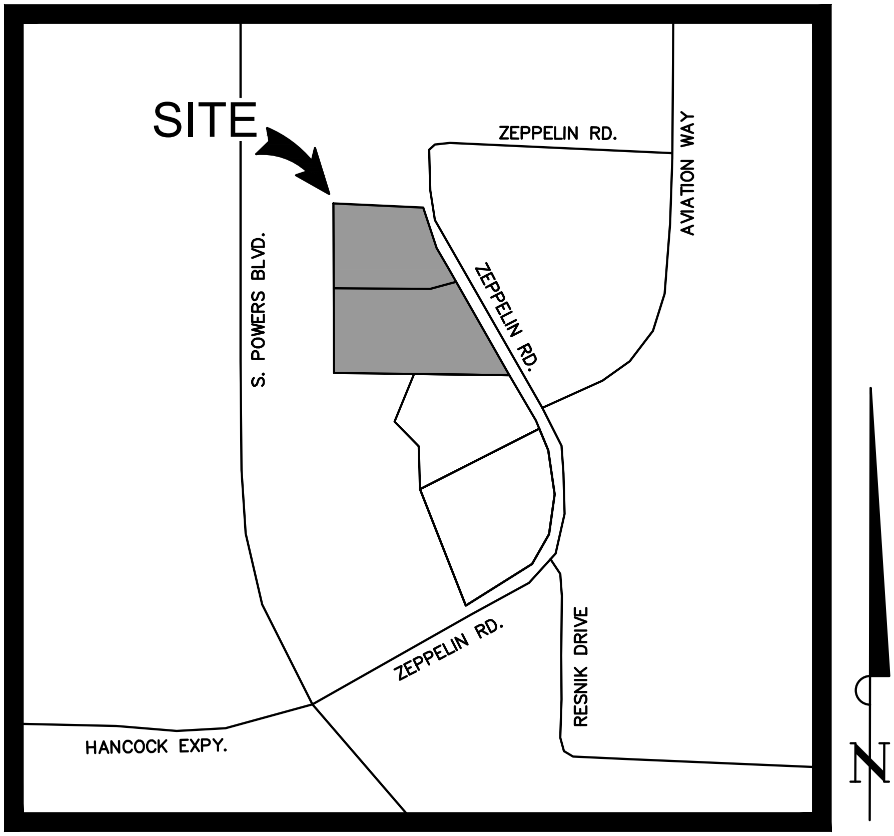
Please contact me at (719) 284-7281 or mitchell.hess@kimley-horn.com should you have any questions.

Sincerely,
KIMLEY-HORN AND ASSOCIATES, INC.

Mitchell Hess, P.E.
Project Manager

Attachments:
Vicinity Map
Storm Line B Profile
CDOT Type 13 Detail
Proposed Drainage Map

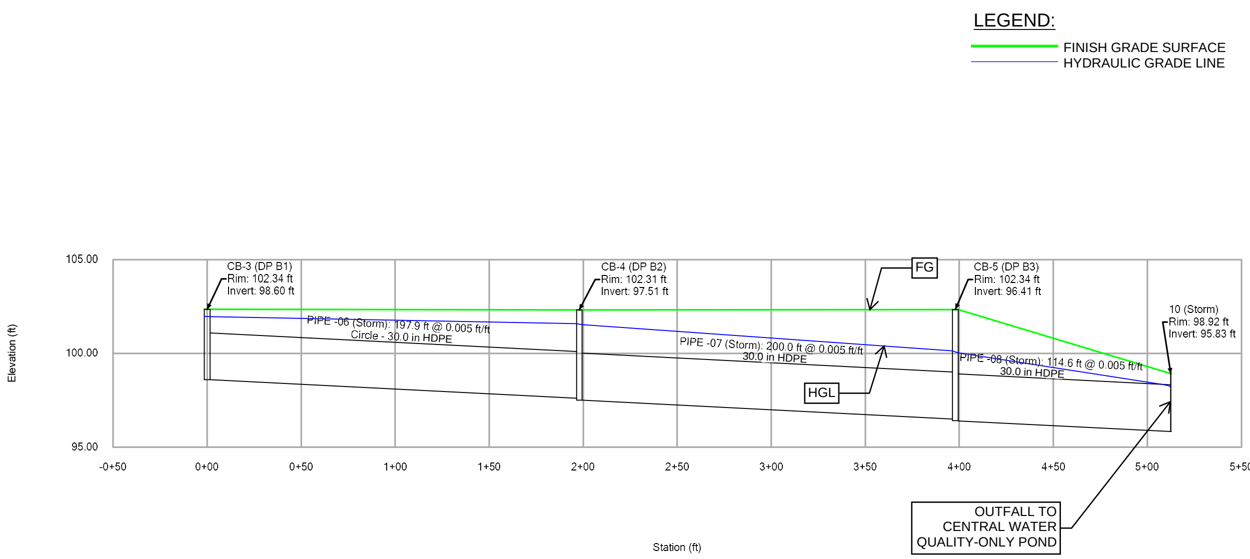
COLORADO LICENSED
MITCHELL O. HESS
53916
11/16/19
PROFESSIONAL ENGINEER

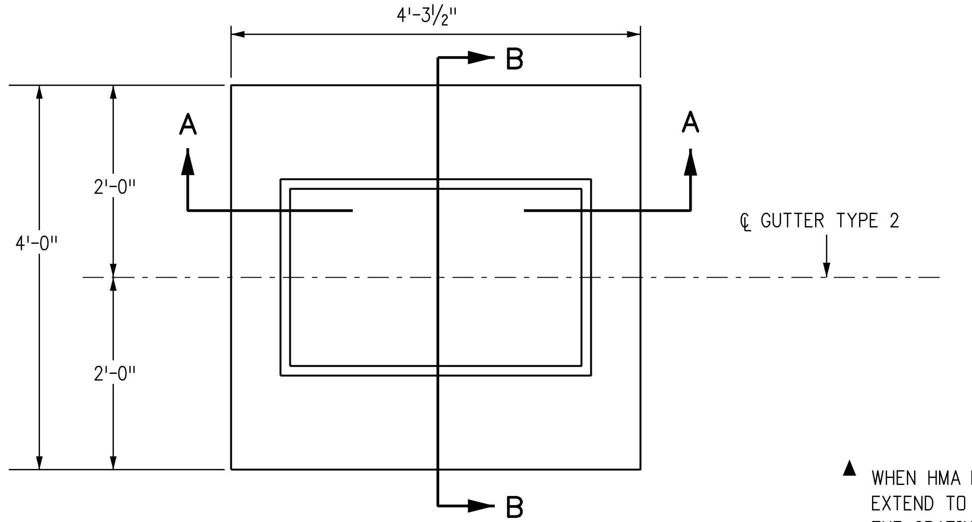


VICINITY MAP

NTS

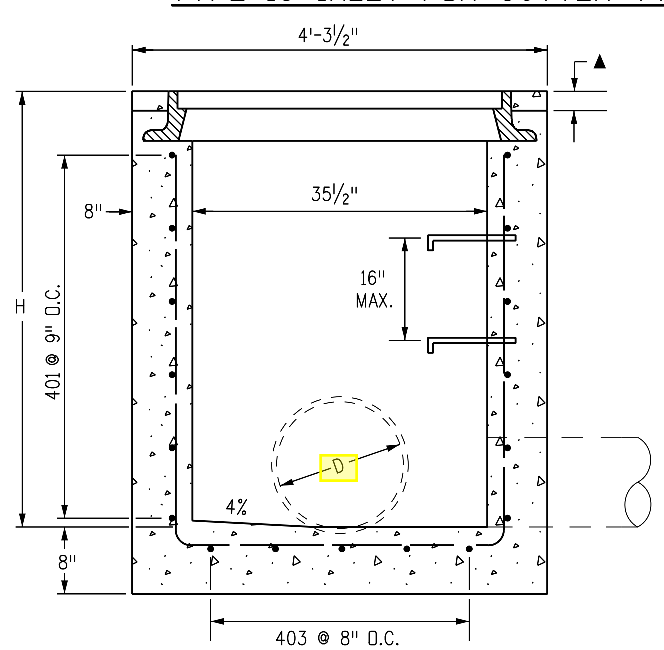
Zeppelin 3 StormCAD Model
Profile Report
Engineering Profile - Storm B (Zeppelin 3&4 StormCAD.stsw)
Active Scenario: 100-Year





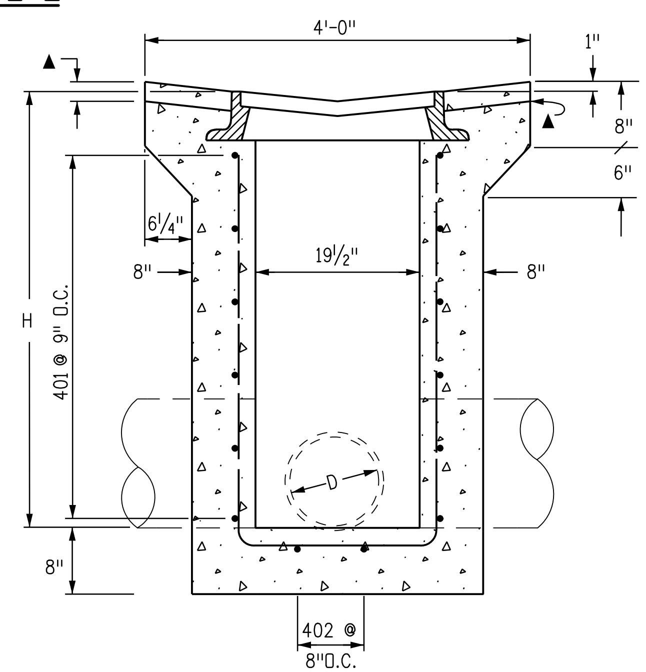
PLAN VIEW
TYPE 13 INLET FOR GUTTER TYPE 2

▲ WHEN HMA MATERIAL IS TO EXTEND TO THE EDGE OF THE GRATING FRAME, CONCRETE MAY BE DEPRESSED.



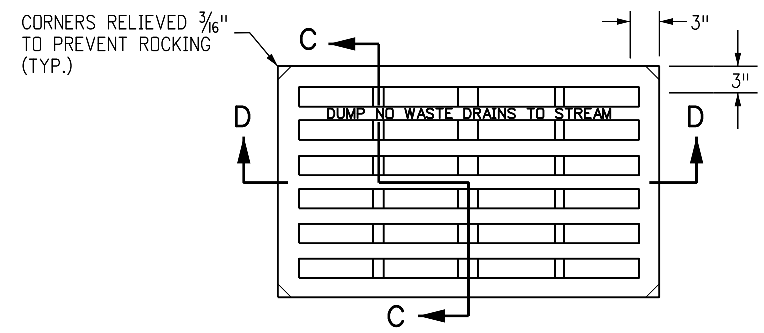
SECTION A-A

D MAX = 30 IN. FOR H > 4 FT.

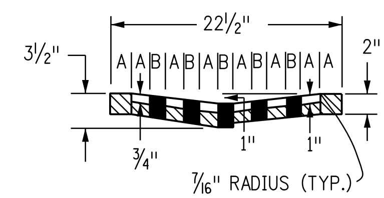


SECTION B-B

D MAX = 18 IN. FOR ALL H

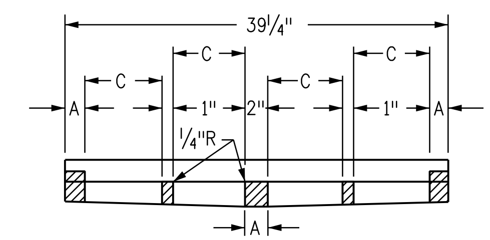


NO. 13 GRATE

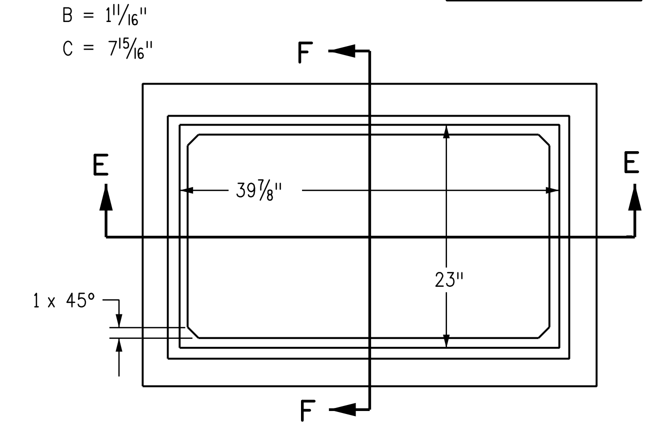


SECTION C-C

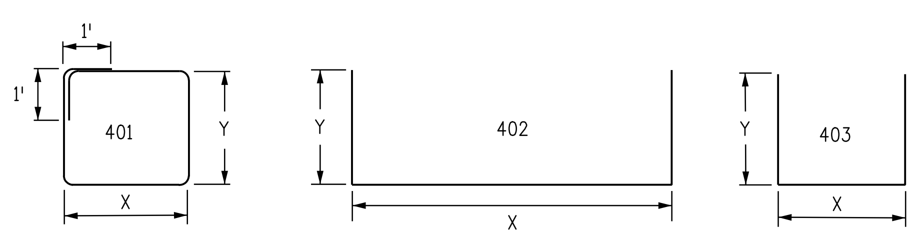
A = 1¾"
B = 1 11/16"
C = 7 15/16"



SECTION D-D

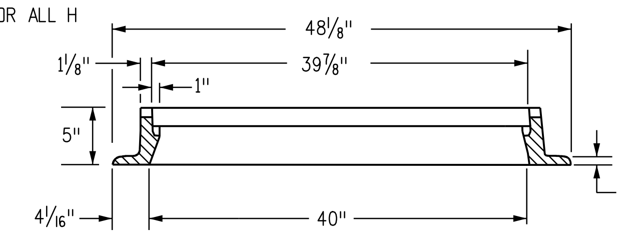


NO. 13 GRATING & FRAMES



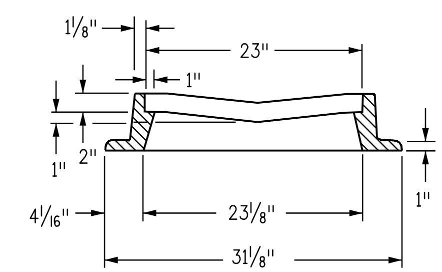
BENDING DIAGRAMS

ALL DIMENSIONS ARE OUT-TO-OUT OF BAR.



SECTION E-E

APPROXIMATE WEIGHT = 590 LBS.



SECTION F-F

GENERAL NOTES

1. CONCRETE SHALL BE CLASS B. INLET MAY BE CAST-IN-PLACE OR PRECAST.
2. CAST-IN-PLACE CONCRETE WALLS SHALL BE FORMED ON BOTH SIDES.
3. EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED ¾ IN.
4. REINFORCING BARS SHALL BE DEFORMED #4 AND SHALL HAVE A 2 IN. MINIMUM CLEARANCE. ALL REINFORCING BARS SHALL BE EPOXY COATED.
5. STEPS SHALL BE PROVIDED WHEN INLET DIMENSION "H" IS EQUAL TO OR GREATER THAN 3 FT.-6 IN. AND SHALL CONFORM TO AASHTO M 199.
6. ALL GRATES AND FRAMES SHALL BE GRAY OR DUCTILE CAST IRON IN ACCORDANCE WITH SUBSECTION 712.06. GRATES AND FRAMES SHALL BE DESIGNED TO WITHSTAND HS 20 LOADING.
7. STATION POINT IS AT THE CENTER OF THE INLET.
8. GRATE SHALL HAVE "DUMP NO WASTE DRAINS TO STREAM" MESSAGE CAST ON SURFACE.

Please note that maximum diameters shown below are only for Single Type 13's and not Doubles. 30" Storm Drain Pipes will be used, which fit within both the Single and Double Type 13 Inlets.

H	CONCRETE CU. YDS.	REINFORCING STEEL θ LB.	NO. OF 401 BARS REQ'D.	MAXIMUM PIPE I.D. SEC. A-A IN.	SEC. B-B IN.
3'-0"	1.3	72	4	18	18
3'-6"	1.5	76	4	24	18
4'-0"	1.6	90	5	30	18
4'-6"	1.8	104	6	30	18
5'-0"	1.9	109	6	30	18
5'-6"	2.1	122	7	30	18
6'-0"	2.2	136	8	30	18
6'-6"	2.4	141	8	30	18
7'-0"	2.5	154	9	30	18
7'-6"	2.7	168	10	30	18
8'-0"	2.8	173	10	30	18
8'-6"	3.0	187	11	30	18
9'-0"	3.1	200	12	30	18
9'-6"	3.3	205	12	30	18
10'-0"	3.4	219	13	30	18

θ INCLUDES 1% FOR OVERRUN.
NOTE: CONCRETE QUANTITIES INCLUDE VOLUME OCCUPIED BY PIPE.

QUANTITIES FOR ONE INLET

MARK	NO. REQ'D.	DIMENSIONS X	DIMENSIONS Y	LENGTH
401	4	3'-6"	2'-2"	13'-4"
402	2	3'-4½"	* 2'-6½"	8'-5½"
403	5	2'-½"	* 2'-7"	7'-2½"

* ADD 6 IN. TO THIS DIMENSION FOR EACH 6 IN. INCREASE OF "H" OVER 3 FT.-0 IN.

BAR LIST FOR H = 3 FT.-0 IN.

Computer File Information	
Creation Date: 07/04/12	Initials: DD
Last Modification Date: 07/04/12	Initials: LTA
Full Path: www.coloradodot.info/business/designsupport	
Drawing File Name: 6040130101.dgn	
CAD Ver.: MicroStation V8 Scale: Not to Scale	Units: English

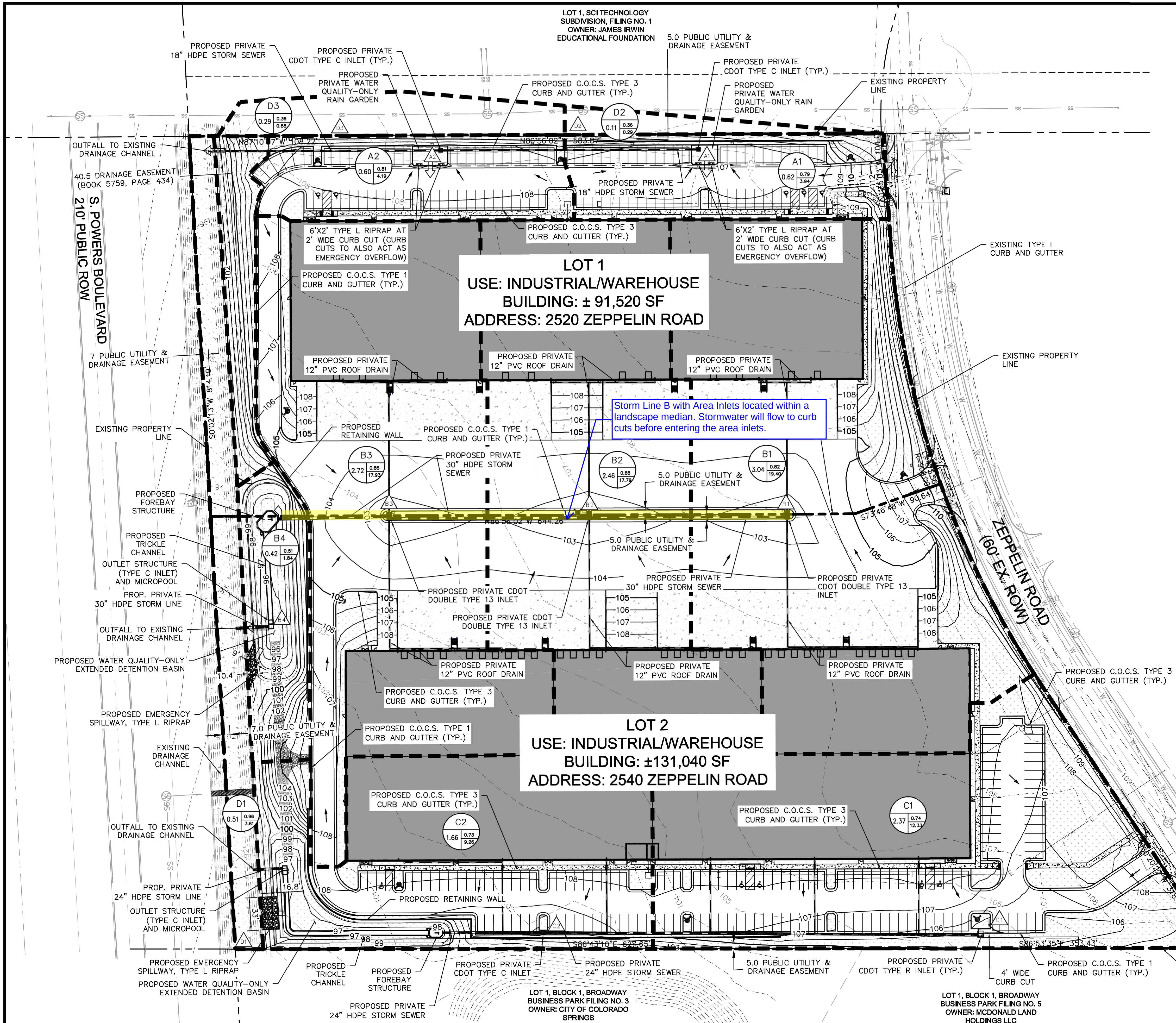
Sheet Revisions	Date:	Comments
(R-X)		
(R-X)		
(R-X)		
(R-X)		

Colorado Department of Transportation
4201 East Arkansas Avenue
Denver, Colorado 80222
Phone: (303) 757-9083
Fax: (303) 757-9820
Project Development Branch DD/LTA

CONCRETE INLET TYPE 13

Issued By: Project Development Branch July 4, 2012

STANDARD PLAN NO.
M-604-13
Sheet No. 1 of 1



LEGEND

A = BASIN DESIGNATION
B = AREA (ACRES)
C = 100-YR COMPOSITE RUNOFF COEFFICIENT
D = 100-YR DIRECT STORM RUNOFF (CFS)

DESIGN POINT
FLOW DIRECTION
EMERGENCY OVERFLOW PATH
DRAINAGE BASIN BOUNDARY
PROPERTY LINE
EASEMENT
EASEMENT
SETBACK
PROPOSED MAJOR CONTOUR
PROPOSED MINOR CONTOUR
EXISTING MAJOR CONTOUR
EXISTING MAJOR CONTOUR
CONCRETE SIDEWALK
LIGHT DUTY ASPHALT
STANDARD DUTY ASPHALT
HEAVY DUTY ASPHALT
HEAVY DUTY CONCRETE
LANDSCAPE AREA (REF: LANDSCAPE PLAN)
BUILDING HATCH

SUMMARY - PROPOSED RUNOFF TABLE

DESIGN POINT	BASIN DESIGNATION	BASIN AREA (ACRES)	DIRECT 5-YR RUNOFF (CFS)	DIRECT 100-YR RUNOFF (CFS)
A1	A1	0.62	2.00	3.94
A2	A2	0.60	2.15	4.19
B1	B1	3.04	10.18	19.40
B2	B2	2.46	9.71	17.79
B3	B3	2.72	9.67	17.93
B4	B4	0.42	0.63	1.84
C1	C1	2.37	6.11	12.33
C2	C2	1.66	4.58	9.26
D1	D1	0.51	2.02	3.61
D2	D2	0.11	0.04	0.29
D3	D3	0.29	0.13	0.88

NOTES

1. THESE DETAILED PLANS AND SPECIFICATIONS WERE PREPARED UNDER MY DIRECTION AND SUPERVISION. SAID DETAILED PLANS AND SPECIFICATIONS HAVE BEEN PREPARED ACCORDING TO THE ESTABLISHED CRITERIA FOR DETAILED DRAINAGE PLANS AND SPECIFICATIONS, AND SAID DETAILED PLANS AND SPECIFICATIONS ARE IN CONFORMITY WITH THE MASTER PLAN OF THE DRAINAGE BASIN. SAID DETAILED DRAINAGE PLANS AND SPECIFICATIONS MEET THE PURPOSES FOR WHICH THE PARTICULAR DRAINAGE FACILITY(S) IS DESIGNED. I ACCEPT RESPONSIBILITY FOR ANY LIABILITY CAUSED BY ANY NEGLIGENT ACTS, ERRORS OR COMMISSIONS ON MY PART IN PREPARATION OF THE DETAILED DRAINAGE PLANS AND SPECIFICATIONS.
2. PLAN REVIEW BY CITY OF COLORADO SPRINGS IS PROVIDED ONLY FOR GENERAL CONFORMANCE WITH DESIGN CRITERIA. THE CITY OF COLORADO SPRINGS IS NOT RESPONSIBLE FOR THE ACCURACY AND ADEQUACY OF THE DESIGN, DIMENSIONS, AND/OR ELEVATIONS WHICH SHALL BE CONFIRMED AT THE JOB SITE. THE CITY OF COLORADO SPRINGS, THROUGH APPROVAL OF THIS DOCUMENT, ASSUMES NO RESPONSIBILITY FOR COMPLETENESS AND/OR ACCURACY OF THIS DOCUMENT.

NORTH
GRAPHIC SCALE IN FEET
0 25 50 100

Kimley»Horn
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2 North Nevada Avenue Suite 300
Colorado Springs, Colorado 80903 (719) 453-0180

NO.	REVISION	DATE	BY	APPR.

DESIGNED BY: EJG
DRAWN BY: JAR
CHECKED BY: EJG
DATE: 11/16/2019

LOT 1&2, BROADVIEW BUSINESS PARK FILING NO. 6
COLORADO SPRINGS, COLORADO
CONSTRUCTION DOCUMENTS

PROPOSED DRAINAGE MAP

PROJECT NO. 096441008
SHEET
2

K:\COS_Civil\096441008_Zeppelin 3 and 4\CADD\Sheets\CDs\096441008DR_PR.dwg Hess, Mitchell 11/16/2019 11:32 AM