Preliminary and Final Drainage Report
The Beach at Woodmoor Filing No. 1
El Paso County, Colorado

Prepared for:
Lake Woodmoor Development Inc.
1755 Telstar Drive, Suite 211
Colorado Springs, Colorado 80920

Prepared by:

Kiowa
Engineering Corporation
1604 South 21st Street
Colorado Springs, Colorado 80904
Ph: (719)630-7342

Kiowa Project No. 16059

January 16, 2018

PCD Project No. SF-17-015

January 16, 2018

Ms. Elizabeth Nijkamp, P.E.
El Paso County Development Services
2880 International Circle
Colorado Springs, Colorado 80910

RE: The Beach at Woodmoor Filing No. 1
(Kiowa Project No. 16059)

Dear Elizabeth:

This report is titled *Preliminary and Final Drainage Report The Beach at Woodmoor Filing No. 1* and addresses the drainage issues for the property. The report was prepared according to current County drainage criteria and is being submitted for approval.

If there are any questions or if we may be of further assistance, please feel free to call at any time.

Sincerely,
Kiowa Engineering Corporation

Christopher J. Castelli, P.E.
Senior Civil Engineer

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STATEMENTS AND APPROVALS

ENGINEER'S STATEMENT:

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

Kiowa Engineering Corporation, 1604 South 21st Street, Colorado Springs, Colorado 80904

________________________________ ________________

Christopher J. Castelli, P.E. (PE #38842) Date

For and on Behalf of Kiowa Engineering Corporation

DEVELOPER'S STATEMENT:

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

By: ________________________________ ________________

Thomas Taylor, Director of Development Services Date

Lake Woodmoor Development Inc.

Print Name: ________________________________

Address: Lake Woodmoor Development Inc.
1755 Telstar Drive, Suite 211
Colorado Springs, Colorado 80920

EL PASO COUNTY:

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

________________________________ ________________

Jennifer Irvine, P.E. Date

El Paso County Engineer/ECM Administrator

I. GENERAL LOCATION AND DESCRIPTION

The Beach at Woodmoor Filing No. 1 will be developed as a single-family residential subdivision located in the Woodmoor area of El Paso County near Monument, Colorado. The subject property is split north and south of Lake Woodmoor Drive and just west of Lower Lake Road. The site is located in the northeast quarter of Section 14, Township 11 South, Range 67 West of the 6th Principal Meridian, in El Paso County, Colorado. The site is bounded to the north by Lake Woodmoor reservoir and the Lake Woodmoor single-family residential subdivision, to the west by the Lake Woodmoor spillway, to the east by Lower Lake Road and to the south by the Brookmoor Filing No. 3 development. The property covers approximately 12.30 acres and is currently undeveloped. A vicinity map of the site is shown on Figure 1 included in the Appendix.

The existing vegetative cover within the property consists primarily of native grasses in fair to good condition throughout the site. There are a few trees along the north property boundary and adjacent to the existing drainageway at the south property boundary. There are scattered riparian shrubs along the Lake Woodmoor shoreline. The existing ground slopes within the property range from approximately 2.5 to 7 percent for the majority of the site, and approximately 4:1 (horizontal:vertical) closer to Lake Woodmoor. The majority of the soils within the subject site are classified to be within Hydrologic Soil Group B (Tomah-Crowfoot loamy sands #92). As shown in the El Paso County Custom Soil Resource Report, there is also a small area in the northeast corner of the site that is identified to be within Hydrologic Soil Group D (Alamosa loam #1). Excerpts from the report are included in the Appendix. Hydrologic Soil Group B was used for the purposes of computing the existing and proposed hydrology for the site, since it is the dominant soil group that encompasses the majority of the site area.

The site drains northwest to Lake Woodmoor, west to the Lake Woodmoor spillway, and south to an existing storm sewer system within the Brookmoor Filing No. 3 development, where runoff is conveyed west to the Lake Fork Dirty Woman Creek. Lake Woodmoor is located along Lake Fork Dirty Woman Creek, which continues in a southerly direction to the Dirty Woman Creek main branch. Dirty Woman Creek is a tributary to Monument Creek.

There are no active irrigation ditches or facilities within or adjacent to the site.

Existing utilities within the site include a sanitary sewer main that runs southwest parallel to Lake Woodmoor and southeast across Lake Woodmoor Drive to the Brookmoor Filing No. 3 development. There is also a waterline that runs from Lake Woodmoor generally along the west property line to Lake Woodmoor Drive. There is an underground electric line with associated meter and two vaults near the west property boundary, that connects between two electric transformers and runs generally south between the west property line and the Lake Woodmoor spillway. There is a 30-inch RCP located at the south property boundary that conveys offsite flows south through the Brookmoor Filing No. 3 development. Existing utilities within public rights-of-way include a sanitary sewer line, water line and two telephone lines along Lake Woodmoor Drive, and a water line, gas line and telephone line along Lower Lake Road. There are two 24-inch culverts and two 30-inch culverts that cross Lake Woodmoor Drive and Lower Lake Road near the intersection of these two streets. The culverts convey offsite flows from the east to the 30-inch RCP at the south property boundary. The existing water and sanitary sewer lines are owned by the Woodmoor Water and Sanitation District.

MAJOR DRAINAGE BASINS AND SUBBASINS

The site lies within the Dirty Woman Creek drainage basin. The northern approximately 15% of the site drains by sheet flow northwest directly to Lake Woodmoor (Sub-basin EX-2). Sub-basin EX-1 is located south of Sub-basin EX-2 and drains by sheet flow southwest to Lake Woodmoor Drive. Lake

Woodmoor drive has a rural street section with roadside ditches that convey runoff west from its high point (located approximately 200 feet west of Lower Lake Road) to its low point at the Lake Woodmoor spillway (DP EX1). Since the spillway/Lake Woodmoor Drive crossing is an at-grade crossing (is not grade separated), runoff that reaches the spillway on the north side of Lake Woodmoor Drive sheet flows south across Lake Woodmoor Drive, then continues south along the concrete-lined spillway channel. Sub-Basin EX-3 is located south of Lake Woodmoor Drive and just north of the existing Brookmoor Filing No. 3 development. Sub-basin EX-3 drains west by sheet flow to the Lake Woodmoor spillway (DP EX3). Runoff that reaches the spillway on the south side of Lake Woodmoor Drive combines with runoff from Sub-basin EX-1 and continues south along the concrete-lined spillway channel.

There is currently offsite runoff that enters the site from the east. The offsite basins consist of existing residential developments, a portion of the Lewis Palmer Elementary School and the Lake Woodmoor Drive right-of-way.

<u>Sub-basin OS-1</u> is located south of Lake Woodmoor Drive and East of Lower Lake Road. Runoff from Sub-basin OS-1 is conveyed east in a 30-inch CMP and 24-inch CMP across an existing gravel access road (across Lake Woodmoor Drive from Lower Lake Road), where it is combined with runoff from Sub-basin OS-2 and conveyed south through an eroded channel to an existing 30-inch RCP at the south property boundary (DP OS13). Flow is conveyed south in the 30-inch RCP, then west in a 36-inch RCP through the Brookmoor Filing No. 3 development to a 66-inch CMP trunkline. The trunkline discharges to a riprap-lined channel (Lake Fork Dirty Woman Creek), where flow is conveyed southerly to Dirty Woman Creek.

<u>Sub-basin OS-2</u> is located north of Lake Woodmoor Drive and east of Lower Lake Road. Runoff from Sub-basin OS-2 collects in the ditch along the north side of Lake Woodmoor Drive, is conveyed west in a 24-inch CMP across Lower Lake Road, then continues in the ditch for approximately 100 feet to a 30-inch RCP that conveys flow south across Lake Woodmoor Drive to Sub-basin OS-1.

<u>Sub-basin OS-3</u> is located south of Sub-basin OS-1 and drains by sheet flow to the southeast corner of the site. Runoff from Sub-basin OS-3 combines with runoff from Sub-basin OS-1 along the south property boundary to DP OS13.

The existing drainage patterns for the site are shown on the Existing Condition Drainage Plan (Sheet DP1) provided in a map pocket at the end of this report.

The reports and plans that were reviewed in the process of preparing this drainage report are included in the References section. The Beach at Woodmoor Filing No. 1 area was studied as a part of the *Dirty Woman and Crystal Creeks Drainage Basin Planning Study (DBPS)* (Reference 1). As previously stated, Lake Woodmoor is located along the Lake Fork tributary to Dirty Woman Creek. The portion of the Lake Fork tributary that is adjacent to The Beach at Woodmoor Filing No. 1 property is identified as "Reach LFDW-A-24" in the DBPS. The DBPS recommends improvements on the adjacent property owned by the Woodmoor Water and Sanitation District and within the Lake Woodmoor Drive right-of-way, including lowering the Lake Woodmoor spillway so it can be conveyed through a 16' wide by 4' high box culvert (instead of the current at-grade crossing), and installation of a check structure and series of drop structures to stabilize the downstream channel section (Lake Fork Dirty Woman Creek) that flows in a southerly direction to Dirty Woman Creek. In lieu of the DBPS recommendations, the Brookmoor Preliminary/Final Drainage Report (Reference 2) states that a 66-inch CMP trunkline would be constructed to collect runoff from the development and the Lake Woodmoor spillway and convey it south to a riprap-lined channel (Lake Fork Dirty Woman Creek) to outlet at Dirty Woman Creek. These drainage improvements appear to have been constructed as described in the Brookmoor report.

Runoff leaving the subject site will be at or less than the existing (undeveloped) condition, so the development of the property will not adversely affect or impact any downstream properties, improvements or drainageways. Therefore, there are no outstanding improvements identified in the DBPS which need to be constructed as part of the development of this site.

The subject property limits are shown on Flood Insurance Rate Map (FIRM) 08041C0276 F (with an effective date of March 17, 1997). The FIRM was subsequently revised to reflect a Letter of Map Revision (LOMR) with Case Number 99-08-012P and an effective date of November 9, 1998. The FIRM showing the project site is included in the Appendix. The Developed Condition Drainage Plan shows that the portion of the property to be developed with buildable lots is located outside of the FEMA regulated floodplain in an unshaded Zone X area, which is described as "Areas determined to be outside the 500-year floodplain".

DRAINAGE DESIGN CRITERIA

Hydrologic and hydraulic calculations for the site were performed using the methods outlined in the *El Paso County Drainage Criteria Manual* (DCM). Topography for the site was compiled using a one-foot contour interval and is presented on the drainage plans. The hydrologic calculations were made for the existing and proposed site conditions. The drainage plans present the drainage patterns for the site, including the sub-basins. The peak flow rates for the sub-basins were estimated using the Rational Method. The 5-year (Minor Storm) and 100-year (Major Storm) recurrence intervals were determined. The one-hour rainfall depth was determined from Table 6-2 in Volume 1 of the City of Colorado Springs Drainage Criteria Manual. These depths are shown in the runoff calculations spreadsheet. The peak flow data generated using the rational method was used to verify street capacities and to size inlets, storm sewers, culverts and swales within the development. The drainage basin area, time of concentration, and rainfall intensity were determined for each of the sub-basins within the property. As discussed in the General Location and Description section, Hydrologic Soil Group B was used for the purposes of computing the existing and proposed hydrology for the site. For existing conditions, runoff coefficients for the on-site basins were determined using historic, packed gravel and pavement land uses. The land uses for the proposed development will be paved streets, lawns and residential with a density of approximately 4.1 lots per acre for the proposed lots north of Lake Woodmoor Drive and 3.6 lots per acre for the proposed lots south of Lake Woodmoor Drive. Runoff coefficients for the offsite basins were obtained from the Brookmoor Preliminary/Final Drainage Report (Reference 2).

The sizing of the onsite hydraulic structures was made using the methods outlined in both the El Paso County and City of Colorado Springs Drainage Criteria Manuals. Colorado Department of Transportation (CDOT) Type R curb inlets and a Type C grated inlet will be used within the site. The hydraulic capacities of the Type R curb inlets were determined using the UDINLET spreadsheet developed by the Urban Drainage and Flood Control District (UDFCD), and Figure 8-10 (refer to Appendix C) was utilized for the Type C grated inlet capacities.

El Paso County Type C curbs will be used throughout the development, except between curb returns and at curb inlets, where a 6-inch vertical curb will be used. The UD-Inlet spreadsheet was used to determine the capacity of each street within the site, considering the County criteria for the Minor (5-year) and Major (100-year) Storms.

Storm sewer pipes and culverts were sized based on their full-flow capacity using the Manning's equation. Hydraulic calculations are provided in Appendix C for the proposed street, inlet, pipe and culvert capacities.

The UD-Culvert spreadsheet was used to determine the extent and size of riprap erosion protection for storm sewer system and culvert outlets. These calculations are also included in Appendix C.

The on-site detention basin is planned to be Extended Detention Basins that use Full Spectrum Detention. The UD-Detention spreadsheet created by the UDFCD was used to size and design the detention basin with water quality enhancement, per the County's recommendation. The supporting calculations associated with the sizing of the hydraulic facilities for this development are included in Appendix B of this report.

II. DRAINAGE FACILITY DESIGN

The drainage of the site will be accomplished through a combination of sheet flow, open channel flow, gutter flow and storm sewer flow. Curb inlets will be placed at low points (sump areas) and on a continuous grade within the site to accept the developed runoff and convey it to the proposed detention basin prior to being discharged off site. Riprap outlet protection will be placed at the end of each pipe outfall and culvert outlet to reduce erosion.

Detention Basin A will include a concrete-lined presedimentation forebay at each proposed storm sewer outlet and a concrete trickle channel to convey flow to the outlet structure. The detention basin will have a micropool and water quality orifice plate onto an outlet structure, an emergency spillway and a maintenance access trail. The detention basin will be a private facility owned and maintained by the homeowner's association for The Beach at Woodmoor Filing No. 1 development.

The evaluation related to the sizing of the onsite drainage improvements was carried out in accordance with the *El Paso County Drainage Criteria Manual*. The capacities of the proposed onsite facilities were calculated in accordance with the Criteria Manual.

The proposed drainage patterns for the site are shown on the Developed Condition Drainage Plan (Sheet DP2) provided in the map pocket at the end of this report. The hydrologic and hydraulic calculations are provided in Appendices A, B and C, refer to the Drainage Design Criteria section for additional information on the hydrologic and hydraulic calculations.

Following is a description of the on-site drainage sub-basins:

<u>Sub-basin A-1</u> is approximately 0.70 acres in area, is located north of Lake Woodmoor Drive and just east of the Lake Woodmoor spillway. Runoff from this basin will sheet flow south, combine with carry-over flow from Sub-basin A-3, and gutter flow south in Coronado Beach Drive to a swale along the north side of Lake Woodmoor Drive. The combined runoff will be conveyed west to the Lake Woodmoor spillway (DP A1), then sheet flow south across Lake Woodmoor Drive (as it does currently) to the concrete-lined spillway.

<u>Sub-basin A-2</u> is approximately 2.09 acres in area and is located north of Sub-basins A-1 and A-3 along the north property boundary. This basin includes backyards and undeveloped land, and will drain by sheet flow northwest directly to Lake Woodmoor.

<u>Sub-basin A-3</u> is approximately 2.08 acres in area and is located north of Coronado Beach Drive between Sub-basins A-1 and A-6. Runoff from this basin will sheet flow southwest then gutter flow westerly along the north flowline of Coronado Beach Drive to a 15' Type R Inlet on a continuous grade connected to an 18-inch RCP (DP A3).

<u>Sub-basin A-4</u> is approximately 1.31 acres in area and is located south of Coronado Beach Drive and north of Sub-basin A-5. Runoff from this basin will sheet flow west-northwest then gutter flow westerly along the south flowline of Coronado Beach Drive to a 10' Type R Inlet on a continuous grade (DP A4). Captured runoff at DP A4 will combine with captured runoff from DP A3 and be conveyed in a 24" RCP to a Type C Inlet in a sump condition within Sub-basin A-5 (DP A5).

<u>Sub-basin A-5</u> is approximately 2.22 acres in area and is located north of Lake Woodmoor Drive between Sub-basins A-1 and A-7. This basin includes backyards, open space and a portion of the

north half of Lake Woodmoor Drive. Grass-lined Swale 3 captures and conveys runoff to a Type C Inlet in a sump condition (DP A5). Carry-over flow from Sub-basin A-4 is also conveyed to the Type C Inlet at DP A5. The combined runoff is then conveyed in an 24-inch RCP south across Lake Woodmoor Drive to Detention Basin A.

Sub-basin A-6 is approximately 0.10 acres in area and is located just west of Lower Lake Road and north of Coronado Beach Drive. Runoff from this basin will sheet flow to grass-lined Swale 1 and be conveyed south to an 18-inch RCP culvert across Coronado Beach Drive (DP A6).

Sub-basin A-7 is approximately 0.38 acres in area and is located just west of Lower Lake Road and north of Lake Woodmoor Drive. Runoff from this basin will sheet flow south and west to a proposed Type C Inlet in a sump condition. An existing 24-inch CMP that crosses Lower Lake Road just north of Lake Woodmoor Drive will be extended west with a 24-inch RCP to the Type C Inlet, where offsite runoff from Sub-basin OS-5 will combine with runoff from Sub-basins A-6 and A-7 (DP 8). The combined runoff will continue south in an existing 30-inch RCP that crosses Lake Woodmoor Drive.

Sub-basin B-1 is approximately 0.25 acres in area and is located south of Lake Woodmoor Drive and west of the public access to Captiva Beach Lane. This basin will accept runoff from Sub-basin B-2, include a portion of the south half of Lake Woodmoor Drive, and convey runoff through grass-lined Swale 6 to an 18-inch RCP (DP 11). The 18-inch RCP will connect to the 24-inch RCP that will cross Lake Woodmoor Drive from the north at a 4' diameter manhole, where runoff from Sub-basins A-3, A-4, A-5, B-1 and B-2 will combine and be conveyed south in an 24-inch RCP to Detention Basin A.

Sub-basin B-2 is approximately 0.47 acres in area and is located south of Lake Woodmoor Drive and west of Lower Lake Road. This basin includes a portion of the south half of Lake Woodmoor Drive and grass-lined Swale 5, and will convey runoff to an 18-inch RCP culvert across the public access to Captiva Beach Lane (DP B2). Offsite runoff from Sub-basin OS-4 will be conveyed west in a 30-inch RCP storm sewer system that will combine with runoff from Sub-basins OS-5, A-6 and A-7 and continue south in a 30-inch RCP through Sub-basin B-4.

Sub-basin B-3 is approximately 0.77 acres in area and is located south of Lake Woodmoor Drive and just east of the Lake Woodmoor spillway. This basin represents the area directly tributary to and including proposed Detention Basin A.

Sub-basin B-4 is approximately 1.26 acres in area and is located south of Lake Woodmoor Drive and north of Captiva Beach Lane. This basin will drain by sheet flow and gutter flow south to Captiva Beach Lane, then continue west along the north Captiva Beach Lane flowline to a 5' curb inlet in a sump condition (DP B4). Runoff captured at DP B4 will be conveyed south in an 18-inch RCP to DP 13. Offsite runoff from Sub-basin OS-4 will be conveyed west in a 24-inch RCP storm sewer system that will combine with runoff from Sub-basins OS-5, A-6 and A-7 and continue southerly in a 30-inch RCP through Sub-basin B-5.

Sub-basin B-5 is approximately 2.14 acres in area and is located south of Captiva Beach Lane along the south property boundary. This basin will accept runoff from Sub-basin OS-6, continue north as sheet flow to Captiva Beach Lane, then west along the south Captiva Beach Lane flowline to a 5' curb inlet in a sump condition (DP 13). At DP 13, runoff from Sub-basins OS-6 and B-5 will combine with runoff from Sub-basin B-4, and be conveyed through an 18-inch RCP storm sewer system to proposed Detention Basin A (DP 14). The 30-inch RCP storm sewer system described for Sub-basin B-4 crosses Sub-basin B-5 and connects to an existing 30-inch RCP (DP 9), where it continues south then west through the Brookmoor Filing No. 3 development and eventually south to Dirty Woman Creek (refer to the Major Drainage Basins and Subbasins section for additional discussion of this storm sewer system).

Sub-basin C-1 is approximately 0.24 acres in area and is located south of Lake Woodmoor Drive, and just east of the Lake Woodmoor spillway. Runoff from a portion of this basin will sheet flow north to a swale along the north side of Lake Woodmoor Drive (Swale 7), while the remaining area will sheet flow west to the property adjacent to the Lake Woodmoor spillway. The 24-inch RCP outlet pipe from Detention Basin A will cross Sub-basin C-1 and daylight to Swale 7 along the south side of Lake Woodmoor Drive. A swale capacity analysis has been performed to handle the major and minor storms from Detention Basin A and can be seen in Appendix C.

The offsite drainage sub-basins are described in detail in the Major Drainage Basins and Subbasins section.

A. STORMWATER DETENTION AND WATER QUALITY DESIGN

Stormwater Detention

One full spectrum detention basin is planned for the proposed development:

Detention Basin A is a private detention basin with the home owners association being responsible for maintenance of the subject drainage facilities. The required WQCV for a 40-hour drain time is 0.15 acre-feet. The required excess urban runoff volume (EURV) for a 72-hour drain time is 0.40 acre-feet. The storage volume required for detention is 0.87 acre-feet, which includes 0.79 acre-feet for the 100-year storm event plus one-half of the WQCV in accordance with County criteria. The proposed outlet structure will include an external micropool and one chamber that controls the release of the WQCV and the EURV. An orifice plate will drain the WQCV and EURV into the chamber of the outlet structure. Approximately Q_{100}=31.6 cfs (DP 15) will drain to the proposed detention basin. 100-year storm event or greater flows will spill over the top of the chamber through a steel grate. Runoff released from the detention basin will be restricted to 15.4 cfs for the 100-year storm event. A proposed 24-inch RCP equipped with a restrictor plate will convey runoff released from the detention basin to the south roadside swale along Lake Woodmoor Drive just east of the Lake Woodmoor spillway. If the outlet structure becomes plugged, a 35-foot wide emergency spillway will convey the runoff to the roadside swale.

Stormwater Quality

Stormwater quality measures are required as stated in the County's Drainage Criteria Manual. The selection of appropriate BMPs is based on the site's characteristics and potential pollutants. The County requires that a Four-Step Process be followed in the BMP selection process:

Step 1: Employ Runoff Reduction Practices

The natural drainage patterns were generally maintained for the site. The proposed site includes the construction of streets, driveways and sidewalks to the minimum widths necessary in order to minimize imperviousness while still maintaining the functionality of the site as intended, providing for adequate parking, snow management, public safety and fire access. Low Impact Development (LID) techniques were implemented as much as possible (and practical) through the use of long grass-lined swales adjacent to Lower Lake Road and Lake Woodmoor Drive. Runoff was also routed by sheet flow through grass areas in select locations to encourage infiltration.

Step 2: Stabilize Drainageways

There are no drainageways within the site that require stabilization. There are grass-lined swales proposed along the existing streets as stated under Step 1. The swales will be designed such that runoff will be conveyed along them at non-erosive velocities, and riprap erosion protection will be provided at culvert outlets.

Step 3: Provide Water Quality Capture Volume (WQCV)
Since water quality capture volume (WQCV) will be required for the proposed development, full spectrum Detention Basin A will also be used for stormwater quality treatment. A presedimentation forebay will be installed at each storm sewer outlet into the detention basin. The outlet structure will include a water quality orifice plate and a micropool.

Step 4: Consider Need for Industrial and Commercial BMPs
The proposed development is not an industrial or commercial site, so no specialized BMPs were considered.

B. COST OF PROPOSED PRIVATE DRAINAGE FACILITIES

Table 2 presents a cost estimate for the construction of private drainage improvements for The Beach at Woodmoor Filing No. 1 development.

C. DRAINAGE AND BRIDGE FEES

The site lies within the Dirty Woman Creek Drainage Basin. The current drainage basin fee associated with the Dirty Woman Creek Drainage Basin is $17,197 per impervious acre. The current bridge fee associated with the Dirty Woman Creek Drainage Basin is $941 per impervious acre. The Beach at Woodmoor Filing No. 1 development encompasses 12.30 acres. Table 1 details the fees due as part of this development.

III. CONCLUSIONS

The Beach at Woodmoor Filing No. 1 will be a single-family residential development covering approximately 12.30 acres. Onsite drainage will include the use of curb inlets, storm sewers, culverts and grass-lined swales to route runoff from the site to an extended detention basin. Detained runoff from the site will be conveyed to the Lake Woodmoor spillway, which is located along the Lake Fork Tributary of Dirty Woman Creek. With the site discharging its runoff at or below existing rates to an armored spillway located along a major drainageway, the development of The Beach at Woodmoor Filing No. 1 property will not adversely impact or deteriorate improvements or natural drainageways downstream of the property.

IV. REFERENCES

1) Drainage Basin Planning Study, Dirty Woman Creek and Crystal Creek, El Paso County, Colorado, prepared by Kiowa Engineering Corporation, dated September 1993.

2) Brookmoor Preliminary/Final Drainage Report, prepared by Nolte and Associates, dated June 1995.

3) Flood Insurance Study, El Paso County, Colorado and Incorporated Areas, prepared by the Federal Emergency Management Agency, dated August 1999.

4) El Paso County Drainage Criteria Manual (Volumes 1 and 2) and Engineering Criteria Manual, current editions.

5) City of Colorado Springs Drainage Criteria Manual, Volumes 1 and 2, May 2014.

6) Flood Insurance Rate Map, Map Number 08041C0276F, by Federal Emergency Management Agency, dated March 17, 1997.

7) Letter of Map Change, Letter of Map Revision Case Number 99-08-012P, Community Number 080059, by Federal Emergency Management Agency, dated November 9, 1998.

8) Excerpts from the Custom Soil Resource Report for El Paso County Area, Colorado, prepared by United States Department of Agriculture Natural Resources Conservation Service, dated April 12, 2017.

APPENDIX TABLE OF CONTENTS

APPENDIX

Figures and Exhibits

Figure 1: Vicinity Map

Excerpts from USDA NRCS Custom Soil Resource Report

FEMA Flood Insurance Rate Map (Panel 276)

Table 1: Impervious Area and Drainage Basin & Bridge Fee Calculations

Table 2: Opinion of Cost – Private Drainage Facilities

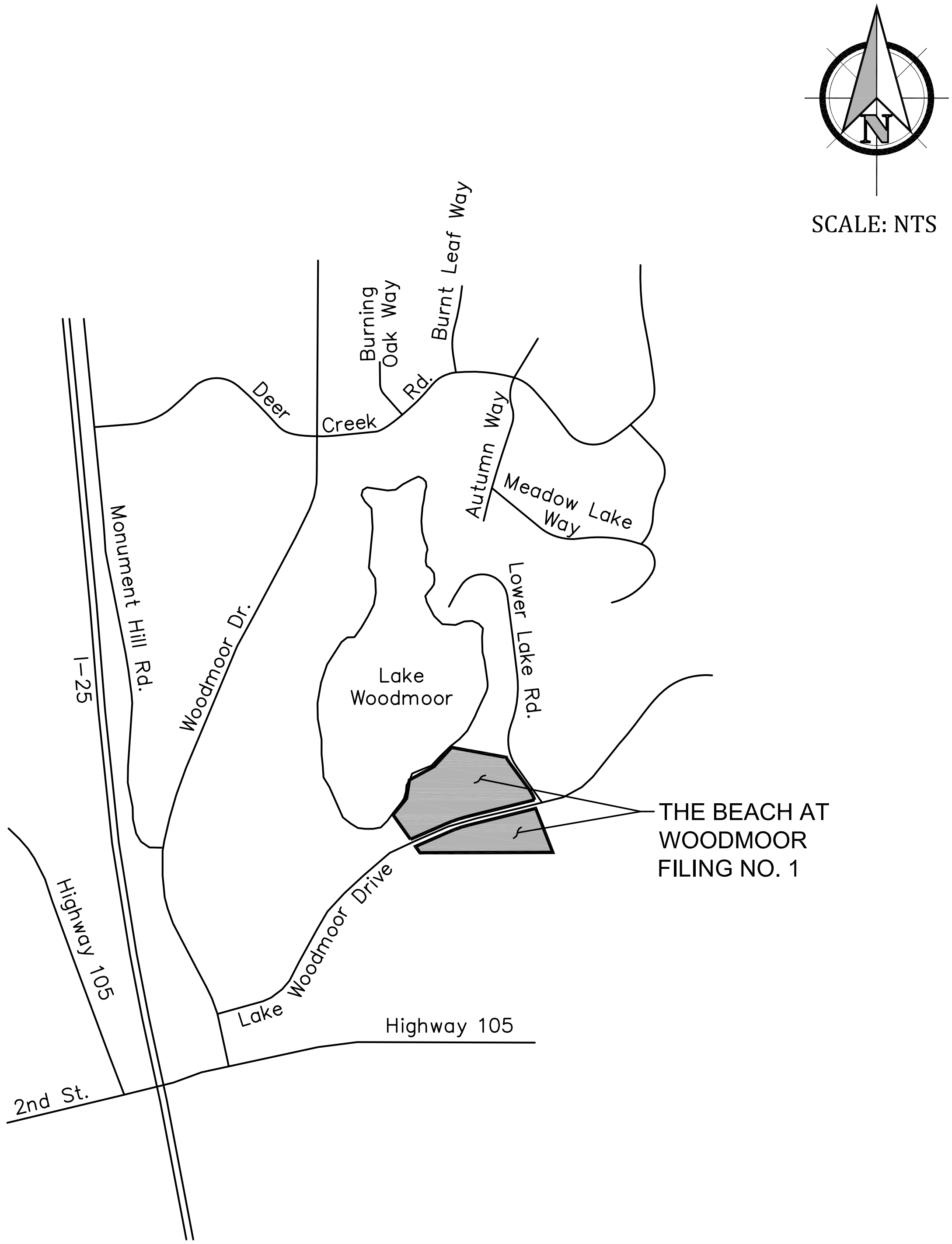


FIGURE 1

The Beach at Woodmoor Filing No. 1

Vicinity Map

El Paso County, Colorado

PROJECT NO. 16059

Celebrating 30 years
Kiowa Engineering Corporation
1604 South 21st Street
Colorado Springs, Colorado 80904
(719) 630-7342

16059 Drainage Plan.dwg/May 12, 2017

USDA
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

The Beach at Woodmoor Filing No. 1

April 12, 2017

Soil Map

Map Scale: 1:2,550 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 35, 70, 140, 210

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 14, Sep 23, 2016

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

Date(s) aerial images were photographed: Apr 15, 2011—Sep 22, 2011

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

Map Unit Legend

El Paso County Area, Colorado (CO625)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
1	Alamosa loam, 1 to 3 percent slopes	1.8	7.4%
92	Tomah-Crowfoot loamy sands, 3 to 8 percent slopes	18.0	73.0%
111	Water	4.8	19.6%
Totals for Area of Interest		**24.6**	**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

El Paso County Area, Colorado

1—Alamosa loam, 1 to 3 percent slopes

Map Unit Setting

National map unit symbol: 3670
Elevation: 7,200 to 7,700 feet
Farmland classification: Prime farmland if irrigated and reclaimed of excess salts and sodium

Map Unit Composition

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

Description of Alamosa

Setting

Landform: Flood plains, fans
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium

Typical profile

A - 0 to 6 inches: loam
Bt - 6 to 14 inches: clay loam
Btk - 14 to 33 inches: clay loam
Cg1 - 33 to 53 inches: sandy clay loam
Cg2 - 53 to 60 inches: sandy loam

Properties and qualities

Slope: 1 to 3 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.60 in/hr)
Depth to water table: About 12 to 18 inches
Frequency of flooding: Frequent
Frequency of ponding: None
Calcium carbonate, maximum in profile: 5 percent
Salinity, maximum in profile: Very slightly saline to strongly saline (2.0 to 16.0 mmhos/cm)
Available water storage in profile: High (about 10.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 5w
Hydrologic Soil Group: D
Ecological site: Mountain Meadow (R048AY241CO)
Hydric soil rating: Yes

Minor Components

Other soils

Percent of map unit:
Hydric soil rating: No

92—Tomah-Crowfoot loamy sands, 3 to 8 percent slopes

Map Unit Setting

National map unit symbol: 36b9
Elevation: 7,300 to 7,600 feet
Farmland classification: Not prime farmland

Map Unit Composition

Tomah and similar soils: 50 percent
Crowfoot and similar soils: 30 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Tomah

Setting

Landform: Alluvial fans, hills
Landform position (three-dimensional): Side slope, crest
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium derived from arkose and/or residuum weathered from arkose

Typical profile

A - 0 to 10 inches: loamy sand
E - 10 to 22 inches: coarse sand
C - 48 to 60 inches: coarse sand

Properties and qualities

Slope: 3 to 8 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.60 to 2.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Very low (about 2.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: B
Ecological site: Sandy Divide (R049BY216CO)
Hydric soil rating: No

Description of Crowfoot

Setting

Landform: Alluvial fans, hills

Landform position (three-dimensional): Side slope, crest
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium

Typical profile

A - 0 to 12 inches: loamy sand
E - 12 to 23 inches: sand
Bt - 23 to 36 inches: sandy clay loam
C - 36 to 60 inches: coarse sand

Properties and qualities

Slope: 3 to 8 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.60 to 2.00 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 4.7 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: B
Ecological site: Sandy Divide (R049BY216CO)
Hydric soil rating: No

Minor Components

Other soils

Percent of map unit:
Hydric soil rating: No

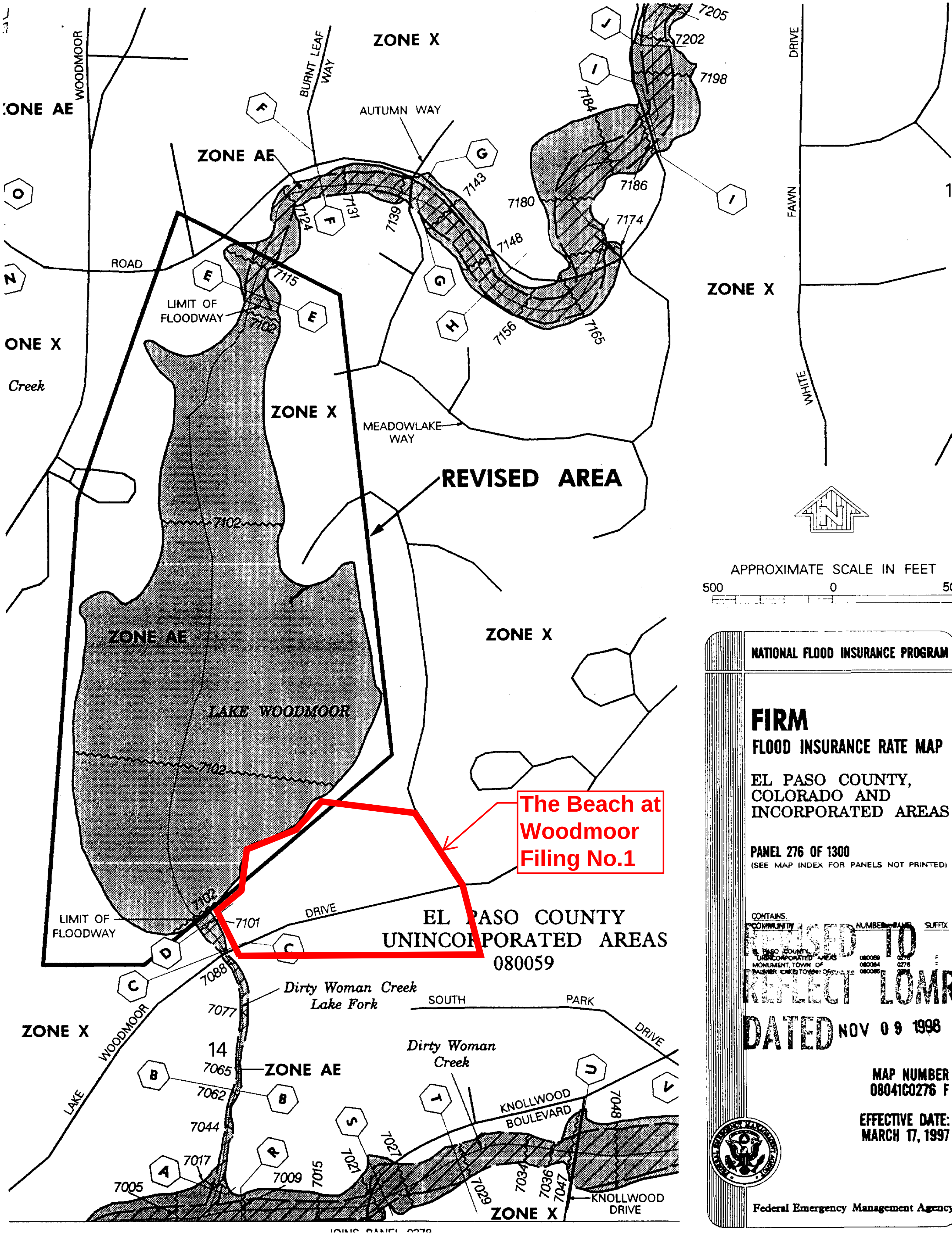
Pleasant

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

111—Water

Map Unit Composition

Water: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.



ZONE X
ZONE AE
ZONE X
ZONE AE
ZONE X
ZONE AE
ZONE X
ZONE X
ZONE X
ZONE AE
ZONE X
WOODMOOR
BURNT LEAF WAY
AUTUMN WAY
ROAD
LIMIT OF FLOODWAY
MEADOWLAKE WAY
REVISED AREA
LAKE WOODMOOR
Creek
DRIVE
FAWN
WHITE
LIMIT OF FLOODWAY
EL PASO COUNTY
UNINCORPORATED AREAS
080059
Dirty Woman Creek
Lake Fork
SOUTH PARK DRIVE
LAKE WOODMOOR
Dirty Woman Creek
KNOLLWOOD BOULEVARD
KNOLLWOOD DRIVE
The Beach at Woodmoor Filing No.1
7205
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7048
14
APPROXIMATE SCALE IN FEET
500
0
NATIONAL FLOOD INSURANCE PROGRAM
FIRM
FLOOD INSURANCE RATE MAP
EL PASO COUNTY, COLORADO AND INCORPORATED AREAS
PANEL 276 OF 1300
(SEE MAP INDEX FOR PANELS NOT PRINTED)
CONTAINS:
REVISED TO REFLECT LOMR
DATED NOV 0 9 1998
MAP NUMBER
08041C0276 F
EFFECTIVE DATE:
MARCH 17, 1997
Federal Emergency Management Agency

Table 1: Impervious Area and Drainage Basin & Bridge Fee Calculation

Total Lots =	35 lots
Total Development Area =	12.304 ac
Total Area =	**12.304 ac**
Building/Patio/Drive Per Lot =	3,775 sf
Total Building/Patio/Drive Area =	3.033 ac
Total Street/Sidewalk Area =	1.385 ac
Total Impervious Area =	4.418 ac
% Impervious Area =	**35.90 %**

Dirty Woman Creek Drainage Basin

Drainage Basin Fee and Bridge Fee Calculations			
Drainage Basin Fee =	$17,197 / ac	**Drainage Basin Fee =**	**$ 75,968.87**
Bridge Fee =	$941 / ac	**Bridge Fee =**	**$ 4,156.93**

Impervious Area = Acreage x (% Impervious)
Drainage Basin Fee = Impervious Area x (Drainage Basin Fee per Acre)
Bridge Fee = Impervious Area x (Bridge Fee per Acre)

Table 2: Opinion of Cost - Private Drainage Facilities

Item	Quantity	Unit	Unit Cost	**Item Total**
Drainage Improvements				
18" Reinforced Concrete Pipe (RCP)	608	LF	$ 69.00	$ 41,952.00
24" Reinforced Concrete Pipe (RCP)	356	LF	$ 84.00	$ 29,904.00
30" Reinforced Concrete Pipe (RCP)	479	LF	$ 94.00	$ 45,026.00
Flared End Section (FES) RCP 18"	5	EA	$ 800.00	$ 4,000.00
Flared End Section (FES) RCP 24"	1	EA	$ 900.00	$ 900.00
Curb Inlet (Type R) L=5', Depth < 5 feet	2	EA	$ 3,791.00	$ 7,582.00
Curb Inlet (Type R) L=10', Depth < 5 feet	1	EA	$ 5,528.00	$ 5,528.00
Curb Inlet (Type R) L=15', Depth < 5 feet	1	EA	$ 7,923.00	$ 7,923.00
Grated Inlet (Type C), Depth < 5 feet	2	EA	$ 3,270.00	$ 6,540.00
5' Dia. Storm Sewer Manhole, Slab Base, Depth < 15 feet	12	EA	$ 4,575.00	$ 54,900.00
5' Dia. Storm Sewer Manhole, Slab Base, Depth < 15 feet	1	EA	$ 5,575.00	$ 5,575.00
Soil Riprap, d50 9" and 12"	57	CY	$ 98.00	$ 5,586.00
Channel Lining, Concrete (Trickle Channel)	12	CY	$ 450.00	$ 5,400.00
Channel Lining, Grass	0.83	AC	$ 1,287.00	$ 1,068.21
Concrete Cutoff Wall (18" RCP FES)	2	EA	$ 300.00	$ 600.00
Concrete Collar	2	EA	$ 500.00	$ 1,000.00
Detention Outlet Structure	1	EA	$ 8,000.00	$ 8,000.00
Detention Emergency Spillway (incl. riprap and cutoff wall)	1	EA	$ 14,000.00	$ 14,000.00
Presedimentation Forebay	2	EA	$ 3,000.00	$ 6,000.00
Gravel Maintenance Access Trail	440	SY	$ 20.00	$ 8,800.00
Type II Bedding	16	CY	$ 35.00	$ 560.00
Detention Basin Seeding and Mulch	0.7	AC	$ 520.00	$ 364.00
			Estimated Storm Drainage Facilities Cost	**$ 261,208.21**
			Engineering 10%	$ 26,120.82
			Contingency 5%	$ 13,060.41
			Total Estimated Cost	**$ 300,389.44**

APPENDIX A

Existing and Developed Condition Hydrologic Calculations

Runoff Coefficient Calculations
Time of Concentration Calculations
Runoff Calculations

The Beach at Woodmoor Filing No. 1

Existing Condition Runoff Coeficient and Percent Impervious Calculation

Basin / DP	Basin or DP Area (DP contributing basins)		Soil Type	PV % Imperv	Area 1 Land Use: Land Use Area	% Area	Comp Land Use % Imp	HI % Imperv	Area 2 Land Use: Land Use Area	% Area	Comp Land Use % Imp	GR % Imperv	Area 3 Land Use: Land Use Area	% Area	Comp Land Use % Imp	RO % Imperv	Area 4 Land Use: Land Use Area	% Area	Comp Land Use % Imp	CO % Imperv	Area 5 Land Use: Land Use Area	% Area	Comp Land Use % Imp	Basin % Imperv	Basin Runoff Coeficient C_5	C_{100}
EX-1/EX1	247,950 sf	5.69ac	B	100%	0.19ac	3%	3%	2%	5.45ac	96%	2%	40%	0.05ac	1%	0%	90%		0%	0%	95%		0%	0%	**5.7%**	**0.11**	**0.38**
EX-2	84,646 sf	1.94ac	B	100%	0.00ac	0%	0%	2%	1.84ac	95%	2%	40%	0.10ac	5%	2%	90%		0%	0%	95%		0%	0%	**4.0%**	**0.09**	**0.37**
EX-3/EX3	133,500 sf	3.06ac	B	100%	0.19ac	6%	6%	2%	2.87ac	94%	2%	40%		0%	0%	90%		0%	0%	95%		0%	0%	**8.2%**	**0.12**	**0.39**
OS-1	548,856 sf	12.60ac	B																						**0.35**	**0.45**
OS-2	509,652 sf	11.70ac	B																						**0.35**	**0.45**
OS-3	22,185 sf	0.51ac	B	100%	0.01ac	2%	2%	2%	0.47ac	92%	2%	40%	0.03ac	5%	2%	90%	0.00ac	1%	1%	95%		0%	0%	**6.4%**	**0.11**	**0.39**
DP OS13	OS1,OS2,OS3	24.81ac	B																						**0.35**	**0.45**

Basin Runoff Coefficient is based on UDFCD % Imperviousness Calculation								
Runoff Coefficients and Percents Impervious								
Hydrologic Soil Type:	**B**	**Runoff Coef Calc Method**						%Imp
Land Use	Abb	%	C_2	C_5	C_{10}	C_{25}	C_{50}	C_{100}
Commercial Area	CO	95%	0.79	0.81	0.83	0.85	0.87	0.88
Drives and Walks	DR	90%	0.71	0.73	0.75	0.78	0.80	0.81
Streets - Gravel (Packed)	GR	40%	0.23	0.30	0.36	0.42	0.46	0.50
Historic Flow Analysis	HI	2%	0.03	0.08	0.17	0.26	0.31	0.36
Lawns	LA	0%	0.02	0.08	0.15	0.25	0.30	0.35
Off-site flow-Undeveloped	OF	45%	0.26	0.32	0.38	0.44	0.48	0.51
Park	PA	7%	0.05	0.12	0.20	0.29	0.34	0.39
Playground	PL	13%	0.07	0.16	0.24	0.32	0.37	0.42
Streets - Paved	PV	100%	0.89	0.90	0.92	0.94	0.95	0.96
Roofs	RO	90%	0.71	0.73	0.75	0.78	0.80	0.81
User Input 1	US1	50%	0.29	0.35	0.40	0.46	0.50	0.52
User Input 2	US2	42%	0.24	0.31	0.37	0.43	0.47	0.50

Weighted
%Imp
A
B
C
D

Equations (% Impervious Calculation):

$C_A = K_A + (1.31 i^3 - 1.44 i^2 + 1.135 i - 0.12)$ [Eqn RO-6]

$C_{CD} = K_{CD} + (0.858 i^3 - 0.786 i^2 + 0.774 i + 0.04)$ [Eqn RO-7]

$C_B = (C_A + C_{CD}) / 2$

I = % imperviousness/100 as a decimal (See Table RO-3)

C_A = Runoff coefficient for NRCS Type A Soils

C_B = Runoff coefficient for NRCS Type B Soils

C_{CD} = Runoff coefficient for NRCS Type C and D Soils

K_A = Correction Factor for NRCS Type A Soils - Table RO-4

K_A (2-yr)= 0

K_A (5-yr)= -0.08i + 0.09

K_A (10-yr)= -0.14i + 0.17

K_A (25-yr)= -0.19i + 0.24

K_A (50-yr)= -0.22i + 0.28

K_A (100-yr)= -0.25i + 0.32

K_{CD}=Correct Factor for NRCS Type C & D Soils-Table RO-4

K_{CD} (2-yr)= 0

K_{CD} (5-yr)= -0.10i + 0.11

K_{CD} (10-yr)= -0.18i + 0.21

K_{CD} (25-yr)= -0.28i + 0.33

K_{CD} (50-yr)= -0.33i + 0.40

K_{CD} (100-yr)= -0.39i + 0.46

The Beach at Woodmoor Filing No. 1
Existing Condition Time of Concentration Calculation

Sub-Basin Data				Time of Concentration Estimate										Final t_c
				Initial/Overland Time (t_i)			Travel Time (t_t)						Comp.	
Basin / Design Point	Contributing Basins	Area	C_5	Length	Slope	t_i	Length	Slope	Land Type	Cv	Velocity	t_t	t_c	
EX-1/EX1		5.69ac	0.11	300lf	5.0%	18.4 min.	500lf	4.0%	GW	15	3.0 ft/sec	2.8 min.	21.2 min.	**21.2 min.**
EX-2		1.94ac	0.09	50lf	4.0%	8.2 min.	300lf	12.0%	GW	15	5.2 ft/sec	1.0 min.	9.2 min.	**9.2 min.**
EX-3/EX3		3.06ac	0.12	120lf	8.8%	9.5 min.	630lf	2.5%	GW	15	2.4 ft/sec	4.4 min.	13.9 min.	**13.9 min.**
OS-1		12.60ac	0.35	230lf	8.0%	10.4 min.	1970lf	6.5%	GW	15	3.8 ft/sec	8.6 min.	19.0 min.	**19.0 min.**
OS-2		11.70ac	0.35	200lf	9.0%	9.3 min.	1850lf	11.5%	GW	15	5.1 ft/sec	6.1 min.	15.4 min.	**15.4 min.**
OS-3		0.51ac	0.11	50lf	6.0%	7.0 min.	300lf	6.0%	GW	15	3.7 ft/sec	1.4 min.	8.4 min.	**8.4 min.**
DP OS13	OS1,OS2,OS3	24.81ac	0.35	230lf	8.0%	10.5 min.	1970lf	6.5%	GW	15	3.8 ft/sec	8.6 min.	19.1 min.	**19.1 min.**

Equations:

t_i (Overland) = $0.395(1.1-C_5)L^{0.5} S^{-0.333}$

C_5 = Runoff coefficient for 5-year

L = Length of overland flow (ft)

S = Slope of flow path (ft/ft)

Velocity (Travel Time) = $CvS^{0.5}$

Cv = Conveyance Coef (see Table RO-2)

S = Watercourse slope (ft/ft)

Table RO-2

Type of Land Surf	Land Type	Cv
Grassed Waterway	GW	15
Heavy Meadow	HM	2.5
Nearly Bare Ground	NBG	10
Paved Area	PV	20
Riprap (Not Buried)	RR	6.5
Short Pasture/Lawns	SP	7
Tillage/Fields	TF	5

The Beach at Woodmoor Filing No. 1
Existing Condition Runoff Calculation

Basin / Design Point	Contributing Basins	Drainage Area	C_5	C_{100}	Time of Concentration	Rainfall Intensity i_5	Rainfall Intensity i_{100}	Runoff Q_5	Runoff Q_{100}	Basin / DP
EX-1/EX1		5.69 ac	0.11	0.38	21.2 min.	3.0 in/hr	5.0 in/hr	**1.8 cfs**	**11.0 cfs**	EX-1/EX1
EX-2		1.94 ac	0.09	0.37	9.2 min.	4.3 in/hr	7.1 in/hr	**0.8 cfs**	**5.2 cfs**	EX-2
EX-3/EX3		3.06 ac	0.12	0.39	13.9 min.	3.6 in/hr	6.1 in/hr	**1.4 cfs**	**7.4 cfs**	EX-3/EX3
OS-1		12.60 ac	0.35	0.45	19.0 min.	3.0 in/hr	5.4 in/hr	**13.2 cfs**	**30.5 cfs**	OS-1
OS-2		11.70 ac	0.35	0.45	15.4 min.	3.4 in/hr	5.8 in/hr	**13.9 cfs**	**30.8 cfs**	OS-2
OS-3		0.51 ac	0.11	0.39	8.4 min.	4.4 in/hr	7.4 in/hr	**0.2 cfs**	**1.4 cfs**	OS-3
DP OS13	OS1,OS2,OS3	24.81 ac	0.35	0.45	19.1 min.	3.2 in/hr	5.3 in/hr	**27.1 cfs**	**59.1 cfs**	DP OS13

Equations (taken from Fig 6-5, City of Colorado Springs DCM):

i_2=-1.19 ln(T_c) + 6.035
i_5=-1.50 ln(T_c) + 7.583
i_{10}=-1.75 ln(T_c) + 8.847
i_{25}=-2.00 ln(T_c) + 10.111
i_{50}=-2.25 ln(T_c) + 11.375
i_{100}=-2.52 ln(T_c) + 12.735

Q = CiA
Q = Peak Runoff Rate (cubic feet/second)
C = Runoff coef representing a ratio of peak runoff rate to ave rainfall intensity for a duration equal to the runoff time of concentration.
i = average rainfall intensity in inches per hour
A = Drainage area in acres

P1	Inches
WQCV	0.60 in
2 yr	1.19 in
5 yr	1.50 in
10 yr	1.75 in
25 yr	2.00 in
50 yr	2.25 in
100 yr	2.52 in

The Beach at Woodmoor Filing No. 1

Developed Condition Runoff Coeficient and Percent Impervious Calculation

Basin / DP	Basin or DP Area (DP contributing basins)		Soil Type	PV	Area 1 Land Use			LA	Area 2 Land Use			US1	Area 3 Land Use			US2	Area 4 Land Use			GR	Area 5 Land Use			Basin % Imperv	Basin Runoff Coeficient	
				% Imperv	Land Use Area	% Area	Comp Land Use % Imp	% Imperv	Land Use Area	% Area	Comp Land Use % Imp	% Imperv	Land Use Area	% Area	Comp Land Use % Imp	% Imperv	Land Use Area	% Area	Comp Land Use % Imp	% Imperv	Land Use Area	% Area	Comp Land Use % Imp		C_5	C_{100}
A1	30,400 sf	0.70ac	B	100%	0.08ac	11%	11%	0%	0.17ac	25%	0%	41%	0.41ac	59%	25%	37%		0%	0%	40%	0.03ac	5%	2%	37.5%	0.28	0.49
A2	90,950 sf	2.09ac	B	100%	0.00ac	0%	0%	0%	0.91ac	44%	0%	41%	1.06ac	51%	21%	37%		0%	0%	40%	0.11ac	5%	2%	23.2%	0.21	0.45
A3	90,730 sf	2.08ac	B	100%	0.00ac	0%	0%	0%		0%	0%	41%	2.08ac	100%	41%	37%		0%	0%	40%		0%	0%	41.4%	0.30	0.50
A4	57,260 sf	1.31ac	B	100%	0.00ac	0%	0%	0%		0%	0%	41%	1.31ac	100%	41%	37%		0%	0%	40%		0%	0%	41.4%	0.30	0.50
A5	96,700 sf	2.22ac	B	100%	0.30ac	14%	14%	0%	0.87ac	39%	0%	41%	1.05ac	47%	20%	37%		0%	0%	40%		0%	0%	33.3%	0.26	0.48
A6	4,250 sf	0.10ac	B	100%	0.03ac	35%	35%	0%	0.06ac	65%	0%	41%		0%	0%	37%		0%	0%	40%		0%	0%	34.6%	0.27	0.48
A7	16,600 sf	0.38ac	B	100%	0.12ac	33%	33%	0%	0.26ac	67%	0%	41%		0%	0%	37%		0%	0%	40%		0%	0%	32.7%	0.26	0.48
B1	10,680 sf	0.25ac	B	100%	0.12ac	50%	50%	0%	0.12ac	50%	0%	41%		0%	0%	37%		0%	0%	40%		0%	0%	50.4%	0.35	0.53
B2	20,480 sf	0.47ac	B	100%	0.23ac	49%	49%	0%	0.24ac	51%	0%	41%		0%	0%	37%		0%	0%	40%		0%	0%	49.2%	0.35	0.52
B3	33,520 sf	0.77ac	B	100%		0%	0%	0%	0.48ac	63%	0%	41%		0%	0%	37%	0.29ac	37%	14%	40%		0%	0%	13.9%	0.16	0.42
B4	54,900 sf	1.26ac	B	100%	0.13ac	10%	10%	0%	0.29ac	23%	0%	41%		0%	0%	37%	0.84ac	67%	25%	40%		0%	0%	35.2%	0.27	0.48
B5	93,340 sf	2.14ac	B	100%	0.00ac	0%	0%	0%		0%	0%	41%		0%	0%	37%	2.14ac	100%	37%	40%		0%	0%	37.3%	0.28	0.49
C1	10,570 sf	0.24ac	B	100%	0.08ac	32%	32%	0%	0.17ac	68%	0%	41%		0%	0%	37%		0%	0%	40%		0%	0%	31.9%	0.26	0.47
OS-4	466,608 sf	10.71ac	B																						0.35	0.45
OS-5	455,452 sf	10.46ac	B																						0.35	0.45
OS-6	14,347 sf	0.33ac	B	100%	0.00ac	0%	0%	0%	0.30ac	92%	0%	41%		0%	0%	37%		0%	0%	40%	0.03ac	8%	3%	3.1%	0.09	0.37
DP 4	A3, A4	3.40ac	B	100%	0.00ac	0%	0%	0%	0.00ac	0%	0%	41%	3.40ac	100%	41%	37%	0.00ac	0%	0%	40%	0.00ac	0%	0%	41.4%	0.30	0.50
DP 6	A3, A4, A5	5.62ac	B	100%	0.30ac	5%	5%	0%	0.87ac	15%	0%	41%	4.45ac	79%	33%	37%	0.00ac	0%	0%	40%	0.00ac	0%	0%	38.2%	0.29	0.49
DP 8	OS5, A6, A7	10.93ac	B																						0.35	0.45
DP 9	OS4, OS5, A6, A7	21.65ac	B																						0.35	0.45
DP 11	B1, B2	0.72ac	B	100%	0.35ac	50%	50%	0%	0.36ac	50%	0%	41%	0.00ac	0%	0%	37%	0.00ac	0%	0%	40%	0.00ac	0%	0%	49.6%	0.35	0.52
DP 12	A3,A4,A5,B1,B2	6.33ac	B	100%	0.66ac	10%	10%	0%	1.23ac	19%	0%	41%	4.45ac	70%	29%	37%	0.00ac	0%	0%	40%	0.00ac	0%	0%	39.5%	0.29	0.49
DP 13	OS6, B5	2.47ac	B	100%	0.00ac	0%	0%	0%	0.30ac	12%	0%	41%	0.00ac	0%	0%	37%	2.14ac	87%	32%	40%	0.03ac	1%	0%	32.8%	0.26	0.48
DP 14	OS6, B4, B5	3.73ac	B	100%	0.13ac	3%	3%	0%	0.59ac	16%	0%	41%	0.00ac	0%	0%	37%	2.99ac	80%	30%	40%	0.03ac	1%	0%	33.6%	0.27	0.48
DP 15	OS6,A3-A5,B1-B5	10.83ac	B	100%	0.79ac	7%	7%	0%	2.30ac	21%	0%	41%	4.45ac	41%	17%	37%	3.27ac	30%	11%	40%	0.03ac	0%	0%	35.6%	0.28	0.48
DP 15A	A3-A5,B1-B5	10.51ac	B	100%	0.79ac	7%	7%	0%	2.00ac	19%	0%	41%	4.45ac	42%	18%	37%	3.27ac	31%	12%	40%	0.00ac	0%	0%	36.7%	0.28	0.49

Basin Runoff Coefficient is based on UDFCD % Imperviousness Calculation								
Runoff Coefficients and Percents Impervious								
Hydrologic Soil Type:	**B**	**Runoff Coef Calc Method**						%Imp
Land Use	Abb	%	C_2	C_5	C_{10}	C_{25}	C_{50}	C_{100}
Commercial Area	CO	95%	0.79	0.81	0.83	0.85	0.87	0.88
Drives and Walks	DR	90%	0.71	0.73	0.75	0.78	0.80	0.81
Streets - Gravel (Packed)	GR	40%	0.23	0.30	0.36	0.42	0.46	0.50
Historic Flow Analysis	HI	2%	0.03	0.08	0.17	0.26	0.31	0.36
Lawns	LA	0%	0.02	0.08	0.15	0.25	0.30	0.35
Off-site flow-Undeveloped	OF	45%	0.26	0.32	0.38	0.44	0.48	0.51
Park	PA	7%	0.05	0.12	0.20	0.29	0.34	0.39
Playground	PL	13%	0.07	0.16	0.24	0.32	0.37	0.42
Streets - Paved	PV	####	0.89	0.90	0.92	0.94	0.95	0.96
Roofs	RO	90%	0.71	0.73	0.75	0.78	0.80	0.81
Residential A Lots	US1	41%	0.24	0.30	0.37	0.43	0.47	0.50
Residential B Lots	US2	37%	0.22	0.28	0.35	0.41	0.45	0.49

Weighted
%Imp
A
B
C
D

Equations (% Impervious Calculation):

$C_A = K_A+(1.31\ i^3 - 1.44\ i^2 + 1.135\ i - 0.12)$ [Eqn RO-6]

$C_{CD} = K_{CD}+(0.858\ i^3 - 0.786\ i^2 + 0.774\ i + 0.04)$ [Eqn RO-7]

$C_B = (C_A + C_{CD}) / 2$

I = % imperviousness/100 as a decimal (See Table RO-3)

C_A = Runoff coefficient for NRCS Type A Soils

C_B = Runoff coefficient for NRCS Type B Soils

C_{CD} = Runoff coefficient for NRCS Type C and D Soils

K_A = Correction Factor for NRCS Type A Soils - Table RO-4

K_A (2-yr)= 0

K_A (5-yr)= -0.08i + 0.09

K_A (10-yr)= -0.14i + 0.17

K_A (25-yr)= -0.19i + 0.24

K_A (50-yr)= -0.22i + 0.28

K_A (100-yr)= -0.25i + 0.32

K_{CD}=Correct Factor for NRCS Type C & D Soils-Table RO-4

K_{CD} (2-yr)= 0

K_{CD} (5-yr)= -0.10i + 0.11

K_{CD} (10-yr)= -0.18i + 0.21

K_{CD} (25-yr)= -0.28i + 0.33

K_{CD} (50-yr)= -0.33i + 0.40

K_{CD} (100-yr)= -0.39i + 0.46

North A Lot Interpolation

Lots/ac	%Impervious	5	100
4	0.40	0.30	0.50
4.15	0.41	0.31	0.50
8	0.65	0.45	0.6

South A Lot Interpolation

Lots/ac	%Impervious	5	100
4	0.41	0.30	0.50
4.07	0.42	0.31	0.50
8	0.65	0.45	0.59

B Lot Interpolation

Lots/ac	%Impervious	5	100
3	0.30	0.25	0.47
3.73	0.37	0.29	0.49
4	0.40	0.30	0.50

The Beach at Woodmoor Filing No. 1

Developed Condition Time of Concentration Calculation

Sub-Basin Data				Time of Concentration Estimate										Min. Tc in Urban		Final t_c
				Initial/Overland Time (t_i)			Travel Time (t_t)						Comp.	Tc Check (urban)		
Basin / Design Point	**Contributing Basins**	**Area**	**C_5**	Length	Slope	t_i	Length	Slope	Land Type	Cv	Velocity	t_t	t_c	Total Length	t_c Check	
A1		0.70ac	0.28	50lf	5.0%	6.2 min.	240lf	6.0%	SP	7	1.7 ft/sec	2.3 min.	8.5 min.	290lf	11.6 min.	**8.5 min.**
A2		2.09ac	0.21	50lf	10.0%	5.3 min.	330lf	15.0%	SP	7	2.7 ft/sec	2.0 min.	7.4 min.	380lf	12.1 min.	**7.4 min.**
A3		2.08ac	0.30	70lf	10.0%	5.7 min.	700lf	4.0%	PV	20	4.0 ft/sec	2.9 min.	8.6 min.	770lf	14.3 min.	**8.6 min.**
A4		1.31ac	0.30	70lf	4.0%	7.7 min.	700lf	4.0%	PV	20	4.0 ft/sec	2.9 min.	10.6 min.	770lf	14.3 min.	**10.6 min.**
A5		2.22ac	0.26	80lf	4.5%	8.3 min.	510lf	3.6%	GW	15	2.8 ft/sec	3.0 min.	11.3 min.	590lf	13.3 min.	**11.3 min.**
A6		0.10ac	0.27	60lf	2.5%	8.7 min.	90lf	6.7%	GW	15	3.9 ft/sec	0.4 min.	9.1 min.	150lf	10.8 min.	**9.1 min.**
A7		0.38ac	0.26	40lf	15.0%	3.9 min.	255lf	2.0%	GW	15	2.1 ft/sec	2.0 min.	5.9 min.	295lf	11.6 min.	**5.9 min.**
B1		0.25ac	0.35	50lf	2.5%	7.1 min.	180lf	2.5%	GW	15	2.4 ft/sec	1.3 min.	8.4 min.	230lf	11.3 min.	**8.4 min.**
B2		0.47ac	0.35	50lf	2.2%	7.5 min.	450lf	2.2%	GW	15	2.2 ft/sec	3.4 min.	10.9 min.	500lf	12.8 min.	**10.9 min.**
B3		0.77ac	0.16	50lf	4.0%	7.7 min.	190lf	8.0%	SP	7	2.0 ft/sec	1.6 min.	9.3 min.	240lf	11.3 min.	**9.3 min.**
B4		1.26ac	0.27	70lf	2.0%	10.0 min.	400lf	1.0%	PV	20	2.0 ft/sec	3.3 min.	13.4 min.	470lf	12.6 min.	**12.6 min.**
B5		2.14ac	0.28	70lf	2.0%	9.9 min.	400lf	1.0%	PV	20	2.0 ft/sec	3.3 min.	13.3 min.	470lf	12.6 min.	**12.6 min.**
C1		0.24ac	0.26	40lf	6.0%	5.4 min.	90lf	4.0%	GW	15	3.0 ft/sec	0.5 min.	5.9 min.	130lf	10.7 min.	**5.9 min.**
OS-4		10.71ac	0.35	230lf	8.0%	10.4 min.	1620lf	6.5%	GW	15	3.8 ft/sec	7.1 min.	17.5 min.	1850lf	20.3 min.	**17.5 min.**
OS-5		10.46ac	0.35	200lf	9.0%	9.3 min.	1730lf	11.5%	GW	15	5.1 ft/sec	5.7 min.	15.0 min.	1930lf	20.7 min.	**15.0 min.**
OS-6		0.33ac	0.09	50lf	6.0%	7.2 min.	185lf	6.0%	GW	15	3.7 ft/sec	0.8 min.	8.1 min.	235lf	11.3 min.	**8.1 min.**
DP 4	A3, A4	3.40ac	0.30	70lf	4.0%	7.7 min.	700lf	4.0%	PV	20	4.0 ft/sec	2.9 min.	10.6 min.	770lf	14.3 min.	**10.6 min.**
DP 6	A3, A4, A5	5.62ac	0.29	70lf	4.0%	7.8 min.	740lf	3.7%	PV	20	3.8 ft/sec	3.2 min.	11.0 min.	810lf	14.5 min.	**11.0 min.**
DP 8	OS5, A6, A7	10.93ac	0.35	200lf	9.0%	9.4 min.	1900lf	10.7%	GW	15	4.9 ft/sec	6.5 min.	15.8 min.	2100lf	21.7 min.	**15.8 min.**
DP 9	OS4, OS5, A6, A7	21.65ac	0.35	200lf	9.0%	9.4 min.	1900lf	10.7%	GW	15	4.9 ft/sec	6.5 min.	15.8 min.	2100lf	21.7 min.	**16.5 min.**
DP 11	B1, B2	0.72ac	0.35	50lf	2.2%	7.5 min.	760lf	2.5%	GW	15	2.4 ft/sec	5.3 min.	12.8 min.	810lf	14.5 min.	**12.8 min.**
DP 12	A3,A4,A5,B1,B2	6.33ac	0.29	70lf	4.0%	7.8 min.	800lf	3.6%	GW	15	2.8 ft/sec	4.7 min.	12.5 min.	870lf	14.8 min.	**12.5 min.**
DP 13	OS6, B5	2.47ac	0.26	70lf	2.0%	10.2 min.	400lf	1.0%	PV	20	2.0 ft/sec	3.3 min.	13.5 min.	470lf	12.6 min.	**12.6 min.**
DP 14	OS6, B4, B5	3.73ac	0.27	70lf	2.0%	10.1 min.	400lf	1.0%	PV	20	2.0 ft/sec	3.3 min.	13.5 min.	470lf	12.6 min.	**13.5 min.**
DP 15	OS6,A3-A5,B1-B5	10.83ac	0.28	70lf	1.9%	10.2 min.	630lf	2.7%	PV	20	3.3 ft/sec	3.2 min.	13.4 min.	700lf	13.9 min.	**14.3 min.**

Equations:

t_i (Overland) = $0.395(1.1\text{-}C_5)L^{0.5}\,S^{-0.333}$

C_5 = Runoff coefficient for 5-year

L = Length of overland flow (ft)

S = Slope of flow path (ft/ft)

tc Check = (L/180)+10 (Developed Cond. Only)

L = Overall Length

Velocity (Travel Time) = $CvS^{0.5}$

Cv = Conveyance Coef (see Table RO-2)

S = Watercourse slope (ft/ft)

Table RO-2

Type of Land Surface	Land Type	Cv
Grassed Waterway	GW	15
Heavy Meadow	HM	2.5
Nearly Bare Ground	NBG	10
Paved Area	PV	20
Riprap (Not Buried)	RR	6.5
Short Pasture/Lawns	SP	7
Tillage/Fields	TF	5

The Beach at Woodmoor Filing No. 1
Developed Condition Runoff Calculation

Basin / Design Point	Contributing Basins	Drainage Area	C_5	C_{100}	Time of Concentration	Rainfall Intensity i_5	Rainfall Intensity i_{100}	Runoff Q_5	Runoff Q_{100}	Basin / DP
A1		0.70 ac	0.28	0.49	8.5 min.	4.4 in/hr	7.3 in/hr	**0.9 cfs**	**2.5 cfs**	A1
A2		2.09 ac	0.21	0.45	7.4 min.	4.6 in/hr	7.7 in/hr	**2.1 cfs**	**7.3 cfs**	A2
A3		2.08 ac	0.30	0.50	8.6 min.	4.4 in/hr	7.3 in/hr	**2.8 cfs**	**7.6 cfs**	A3
A4		1.31 ac	0.30	0.50	10.6 min.	4.0 in/hr	6.8 in/hr	**1.6 cfs**	**4.4 cfs**	A4
A5		2.22 ac	0.26	0.48	11.3 min.	3.9 in/hr	6.6 in/hr	**2.3 cfs**	**7.0 cfs**	A5
A6		0.10 ac	0.27	0.48	9.1 min.	4.3 in/hr	7.2 in/hr	**0.1 cfs**	**0.3 cfs**	A6
A7		0.38 ac	0.26	0.48	5.9 min.	4.9 in/hr	8.2 in/hr	**0.5 cfs**	**1.5 cfs**	A7
B1		0.25 ac	0.35	0.53	8.4 min.	4.4 in/hr	7.4 in/hr	**0.4 cfs**	**1.0 cfs**	B1
B2		0.47 ac	0.35	0.52	10.9 min.	4.0 in/hr	6.7 in/hr	**0.6 cfs**	**1.6 cfs**	B2
B3		0.77 ac	0.16	0.42	9.3 min.	4.2 in/hr	7.1 in/hr	**0.5 cfs**	**2.3 cfs**	B3
B4		1.26 ac	0.27	0.48	12.6 min.	3.8 in/hr	6.3 in/hr	**1.3 cfs**	**3.9 cfs**	B4
B5		2.14 ac	0.28	0.49	12.6 min.	3.8 in/hr	6.3 in/hr	**2.3 cfs**	**6.6 cfs**	B5
C1		0.24 ac	0.26	0.47	5.9 min.	4.9 in/hr	8.3 in/hr	**0.3 cfs**	**1.0 cfs**	C1
OS-4		10.71 ac	0.35	0.45	17.5 min.	3.3 in/hr	5.5 in/hr	**12.3 cfs**	**26.6 cfs**	OS-4
OS-5		10.46 ac	0.35	0.45	15.0 min.	3.5 in/hr	5.9 in/hr	**12.9 cfs**	**27.8 cfs**	OS-5
OS-6		0.33 ac	0.09	0.37	8.1 min.	4.5 in/hr	7.5 in/hr	**0.1 cfs**	**0.9 cfs**	OS-6
DP 4	A3, A4	3.40 ac	0.30	0.50	10.6 min.	4.0 in/hr	6.8 in/hr	**4.2 cfs**	**11.5 cfs**	DP 4
DP 6	A3, A4, A5	5.62 ac	0.29	0.49	11.0 min.	4.0 in/hr	6.7 in/hr	**6.4 cfs**	**18.4 cfs**	DP 6
DP 8	OS5, A6, A7	10.93 ac	0.35	0.45	15.8 min.	3.4 in/hr	5.8 in/hr	**13.0 cfs**	**28.5 cfs**	DP 8
DP 9	OS4, OS5, A6, A7	21.65 ac	0.35	0.45	16.5 min.	3.4 in/hr	5.7 in/hr	**25.4 cfs**	**55.3 cfs**	DP 9
DP 11	B1, B2	0.72 ac	0.35	0.52	12.8 min.	3.8 in/hr	6.3 in/hr	**0.9 cfs**	**2.4 cfs**	DP 11
DP 12	A3,A4,A5,B1,B2	6.33 ac	0.29	0.49	12.5 min.	3.8 in/hr	6.4 in/hr	**7.1 cfs**	**19.9 cfs**	DP 12
DP 13	OS6, B5	2.47 ac	0.26	0.48	12.6 min.	3.8 in/hr	6.3 in/hr	**2.4 cfs**	**7.5 cfs**	DP 13
DP 14	OS6, B4, B5	3.73 ac	0.27	0.48	13.5 min.	3.7 in/hr	6.2 in/hr	**3.7 cfs**	**11.0 cfs**	DP 14
DP 15	OS6,A3-A5,B1-B5	10.83 ac	0.28	0.48	14.3 min.	3.6 in/hr	6.0 in/hr	**10.7 cfs**	**31.6 cfs**	DP 15

Equations (taken from Fig 6-5, City of Colorado Springs DCM):

i_2=-1.19 ln(T_c) + 6.035
i_5=-1.50 ln(T_c) + 7.583
i_{10}=-1.75 ln(T_c) + 8.847
i_{25}=-2.00 ln(T_c) + 10.111
i_{50}=-2.25 ln(T_c) + 11.375
i_{100}=-2.52 ln(T_c) + 12.735

Q = CiA
Q = Peak Runoff Rate (cubic feet/second)
C = Runoff coef representing a ratio of peak runoff rate to ave rainfall intensity for a duration equal to the runoff time of concentration.
i = average rainfall intensity in inches per hour
A = Drainage area in acres

P1	Inches
WQCV	0.60 in
2 yr	1.19 in
5 yr	1.50 in
10 yr	1.75 in
25 yr	2.00 in
50 yr	2.25 in
100 yr	2.52 in

APPENDIX A.1

Supporting Hydrologic Tables and Figures

Table 6-6. Runoff Coefficients for Rational Method

(Source: UDFCD 2001)

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

3.2 Time of Concentration

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

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

Table 6-7. Conveyance Coefficient, C_v

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

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

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

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

3.2.3 First Design Point Time of Concentration in Urban Catchments

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

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

Where:

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

L = waterway length (ft)

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

3.2.4 Minimum Time of Concentration

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

3.2.5 Post-Development Time of Concentration

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

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

Table 6-2. Rainfall Depths for Colorado Springs

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

Where Z= 6,840 ft/100

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

2.2 Design Storms

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

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

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

APPENDIX B

Detention Basin Calculations

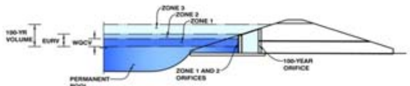
Full Spectrum Detention Basin/Extended Detention Basin
Emergency Spillway Calculations
Outlet Structure Calculations
Forebay Sizing and Trickle Channel Calculations

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)

Project: The Beach at Woodmoor Filing No. 1

Basin ID: Detention Basin A



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	10.51	acres		
Watershed Length =	700	ft		
Watershed Slope =	0.027	ft/ft		
Watershed Imperviousness =	36.70%	percent		
Percentage Hydrologic Soil Group A =	0.0%	percent		
Percentage Hydrologic Soil Group B =	100.0%	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) =	0.150	acre-feet	Optional User Override 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.402	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.315	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.441	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.639	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.990	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	1.225	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	1.533	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.2 in.) =	2.193	acre-feet	3.20	inches
Approximate 2-yr Detention Volume =	0.294	acre-feet		
Approximate 5-yr Detention Volume =	0.413	acre-feet		
Approximate 10-yr Detention Volume =	0.578	acre-feet		
Approximate 25-yr Detention Volume =	0.653	acre-feet		
Approximate 50-yr Detention Volume =	0.686	acre-feet		
Approximate 100-yr Detention Volume =	0.794	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.150	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.253	acre-feet
Zone 3 (100yr + 1 / 2 WQCV - Zones 1 & 2) =	0.466	acre-feet
Total Detention Basin Volume =	0.869	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	--	--	--	0	0.000		
	--	0.80	--	--	--	20	0.000	8	0.000
	--	1.30	--	--	--	1,500	0.034	373	0.009
	--	2.30	--	--	--	4,500	0.103	3,388	0.078
	--	3.30	--	--	--	5,500	0.126	8,388	0.193
	--	4.30	--	--	--	6,600	0.152	14,438	0.331
	--	5.30	--	--	--	7,600	0.174	21,538	0.494
	--	6.30	--	--	--	8,700	0.200	29,688	0.682
	--	7.30	--	--	--	9,800	0.225	38,938	0.894
	--	8.30	--	--	--	11,100	0.255	49,388	1.134
	--	9.30	--	--	--	12,400	0.285	61,138	1.404
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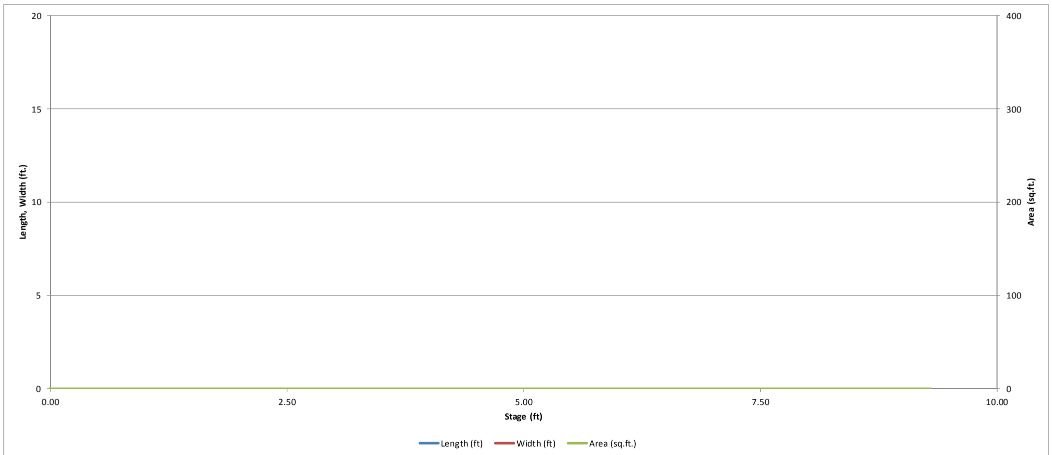
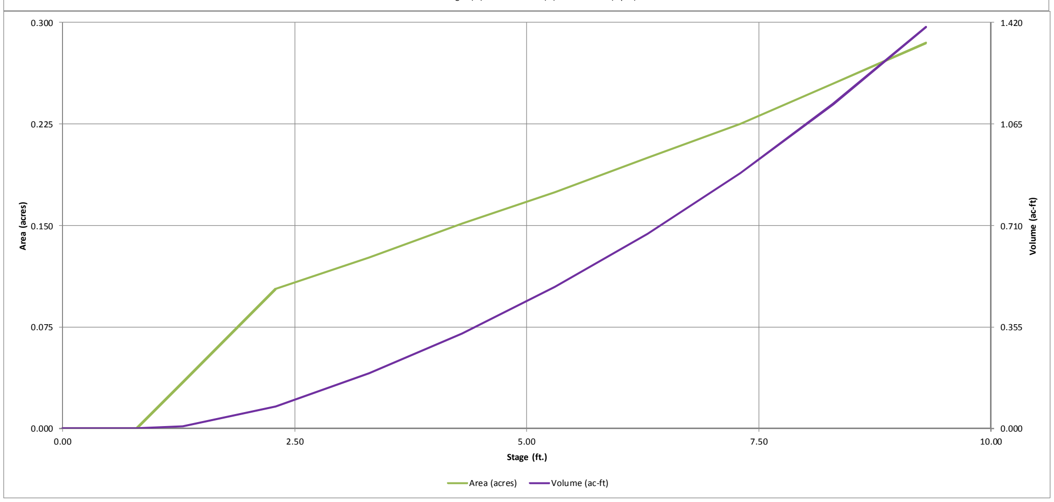
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

UD-Detention, Version 3.07 (February 2017)





Emergency Spillway Calculation

Detention Area	100-yr Flow	120% 100yr Flow	Water Surf Elev	Crest Elev	Crest Length	Z	C	Flow Depth (H)	**Calc'd Flow**	Check
A - DP 15	31.6 cfs	38.0 cfs	7,105.5	7,105.00	35 ft	3:1	3.0	0.50 ft	38 cfs	OK

Broad Crested Weir Equation (USDCM Eqn 12-20 and 12-21):

$Q = CLH^{1.5} + 2x((2/5)CZH^{5/2})$

H = Head above weir crest, in ft

C = Weir coefficient, C = 3.0 (most cases)

Z = Side slope (horizontal:vertical)

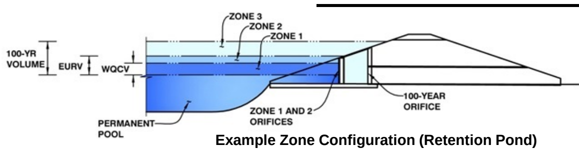
L = Length of weir at Crest, in ft. Not including sideslopes.

Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Project: The Beach at Woodmoor Filing No. 1

Basin ID: Detention Basin A



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.95	0.150	Orifice Plate
Zone 2 (EURV)	4.76	0.253	Orifice Plate
(100+1/2WQCV)	7.19	0.466	Weir&Pipe (Restrict)
		0.869	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 =	4.80	ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	N/A	inches
Orifice Plate: Orifice Area per Row =	N/A	inches

Calculated Parameters for Plate

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

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	1.60	3.20					
Orifice Area (sq. inches)	0.690	0.690	0.690					

	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 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	4.80	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	4.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	5.80	N/A	feet
Over Flow Weir Slope Length =	4.12	N/A	feet
Grate Open Area / 100-yr Orifice Area =	9.32	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	11.54	N/A	ft^2
Overflow Grate Open Area w/ Debris =	5.77	N/A	ft^2

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

	Zone 3 Restrictor	Not Selected	
Depth to Invert of Outlet Pipe =	0.50	N/A	ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter =	24.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	10.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	1.24	N/A	ft^2
Outlet Orifice Centroid =	0.48	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	1.40	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	7.30	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	35.00	feet
Spillway End Slopes =	3.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Calculated Parameters for Spillway

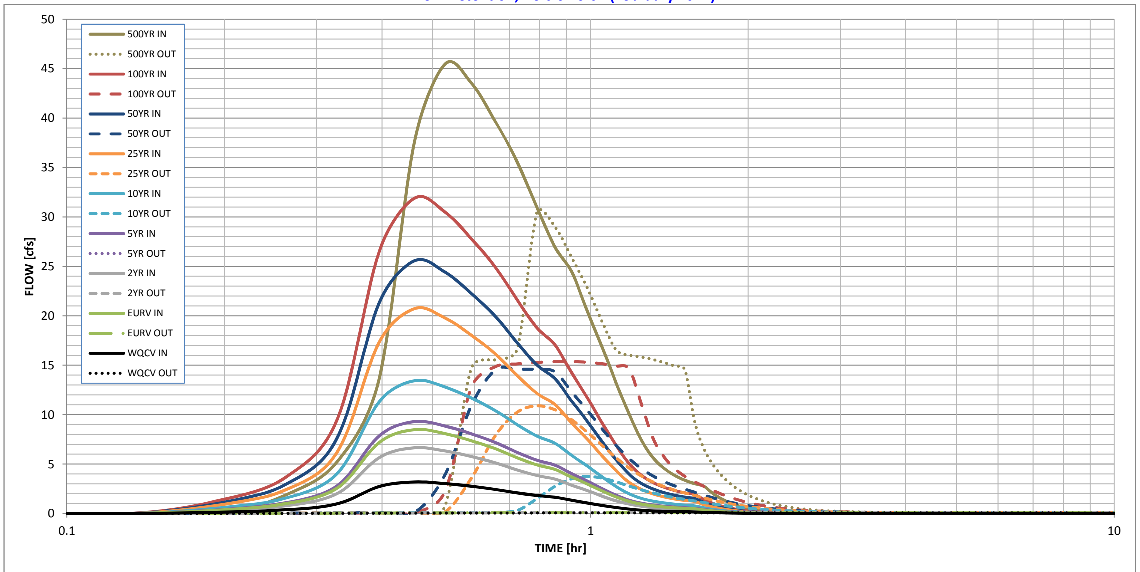
Spillway Design Flow Depth=	0.44	feet
Stage at Top of Freeboard =	8.74	feet
Basin Area at Top of Freeboard =	0.27	acres

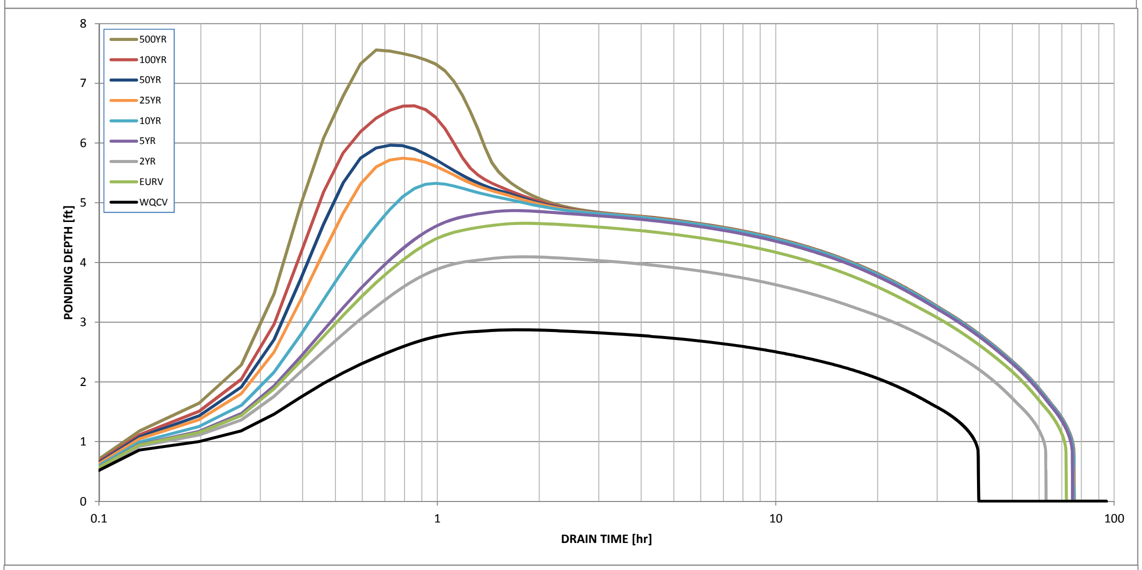
Routed Hydrograph Results

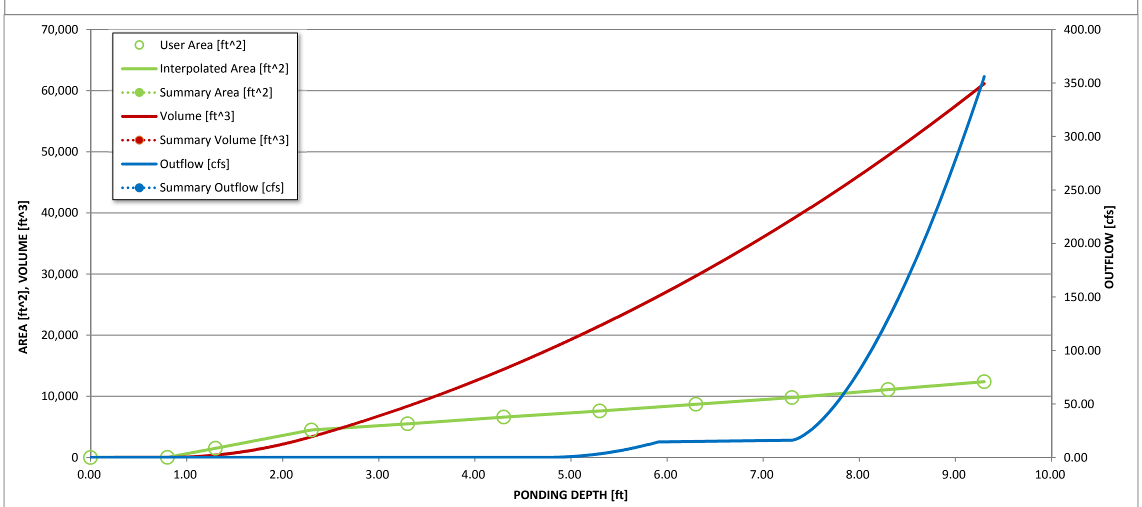
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	3.20
Calculated Runoff Volume (acre-ft) =	0.150	0.402	0.315	0.441	0.639	0.990	1.225	1.533	2.193
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.149	0.402	0.314	0.440	0.639	0.991	1.225	1.533	2.194
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.02	0.03	0.28	0.88	1.22	1.62	2.43
Predevelopment Peak Q (cfs) =	0.0	0.0	0.2	0.3	3.0	9.3	12.8	17.1	25.5
Peak Inflow Q (cfs) =	3.2	8.5	6.7	9.3	13.4	20.7	25.6	31.9	45.4
Peak Outflow Q (cfs) =	0.1	0.1	0.1	0.3	3.8	10.9	14.6	15.4	30.4
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.8	1.3	1.2	1.1	0.9	1.2
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.0	0.3	0.9	1.3	1.3	1.4
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	38	66	58	69	67	63	61	59	54
Time to Drain 99% of Inflow Volume (hours) =	39	70	61	73	73	71	70	69	66
Maximum Ponding Depth (ft) =	2.87	4.66	4.10	4.87	5.33	5.75	5.96	6.62	7.56
Area at Maximum Ponding Depth (acres) =	0.12	0.16	0.15	0.16	0.17	0.19	0.19	0.21	0.23
Maximum Volume Stored (acre-ft) =	0.140	0.386	0.300	0.420	0.498	0.574	0.615	0.747	0.951

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: 3.95 min

Hydrograph Constant: 1.266

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:03:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:07:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:11:51	0.15	0.38	0.30	0.42	0.60	0.91	1.11	1.38	1.95
0:15:48	0.39	1.02	0.80	1.11	1.60	2.46	3.02	3.76	5.33
0:19:45	1.00	2.62	2.06	2.86	4.12	6.32	7.77	9.66	13.68
0:23:42	2.74	7.19	5.66	7.87	11.31	17.35	21.34	26.53	37.55
0:27:39	3.19	8.49	6.66	9.29	13.43	20.72	25.57	31.90	45.44
0:31:36	3.03	8.09	6.34	8.86	12.81	19.80	24.44	30.52	43.53
0:35:33	2.76	7.36	5.77	8.06	11.66	18.03	22.25	27.78	39.62
0:39:30	2.44	6.56	5.13	7.19	10.42	16.13	19.94	24.92	35.59
0:43:27	2.09	5.64	4.41	6.19	8.99	13.97	17.28	21.62	30.97
0:47:24	1.83	4.93	3.85	5.40	7.84	12.15	15.03	18.79	26.93
0:51:21	1.65	4.46	3.48	4.89	7.10	11.02	13.63	17.04	24.40
0:55:18	1.34	3.66	2.85	4.02	5.86	9.13	11.31	14.18	20.36
0:59:15	1.08	2.98	2.31	3.27	4.78	7.48	9.29	11.67	16.80
1:03:12	0.81	2.28	1.76	2.50	3.68	5.80	7.23	9.11	13.19
1:07:09	0.59	1.68	1.29	1.85	2.74	4.36	5.45	6.91	10.07
1:11:06	0.43	1.22	0.95	1.35	1.99	3.15	3.97	5.05	7.42
1:15:03	0.34	0.95	0.74	1.05	1.54	2.43	3.04	3.85	5.63
1:19:00	0.28	0.78	0.61	0.86	1.27	1.99	2.49	3.15	4.57
1:22:57	0.24	0.67	0.52	0.73	1.07	1.69	2.11	2.66	3.86
1:26:54	0.21	0.59	0.46	0.64	0.94	1.48	1.85	2.33	3.37
1:30:51	0.19	0.53	0.41	0.58	0.85	1.33	1.66	2.09	3.02
1:34:48	0.18	0.49	0.38	0.54	0.78	1.23	1.53	1.92	2.77
1:38:45	0.13	0.36	0.28	0.39	0.57	0.90	1.12	1.41	2.05
1:42:42	0.10	0.26	0.20	0.29	0.42	0.66	0.82	1.03	1.49
1:46:39	0.07	0.19	0.15	0.21	0.31	0.48	0.60	0.76	1.10
1:50:36	0.05	0.14	0.11	0.15	0.23	0.36	0.45	0.56	0.81
1:54:33	0.04	0.10	0.08	0.11	0.16	0.26	0.32	0.41	0.59
1:58:30	0.02	0.07	0.05	0.08	0.12	0.18	0.23	0.29	0.42
2:02:27	0.02	0.05	0.04	0.05	0.08	0.13	0.16	0.21	0.31
2:06:24	0.01	0.03	0.02	0.04	0.05	0.09	0.11	0.14	0.21
2:10:21	0.01	0.02	0.01	0.02	0.03	0.05	0.07	0.09	0.13
2:14:18	0.00	0.01	0.01	0.01	0.02	0.03	0.04	0.05	0.07
2:18:15	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02	0.03
2:22:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
2:26:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:30:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:34:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:38:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:41:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:45:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:49:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:53:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:57:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:01:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:05:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:09:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:13:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:17:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:21:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:25:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:29:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:33:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:37:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:41:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:45:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:49:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:53:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:57:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:00:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:04:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:08:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:12:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:16:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:20:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:24:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:28:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:32:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:36:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:40:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:44:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

The Beach at Woodmoor Filing No. 1
Detention Calculations

Presedementation / Forebay Sizing

Forebay	100 Yr Flow	Detention WQCV	Total Req'd Forebay Vol 3.0% WQCV	Tributary Area	% Total Trib Area	Required Forebay Volume	Forebay Design Area	Forebay Design Depth	Forebay Design Volume	Discharge Design Flow 2.0% 100yr	Calc'd Open Width (1" min)	**Design Width**
DP 12	19.9cfs	6,515cf	195cf	6.33ac	62.9%	123cf	100sf	1.25-ft	125 cf	0.40 cfs	4.1-inch	**4.0-inch**
DP 14	11.0cfs			3.73ac	37.1%	72cf	60sf	1.25-ft	75 cf	0.22 cfs	3.6-inch	**3.5-inch**
					0.0%	---			---	---	---	
Totals		6,515cf	195cf	10.07ac	100.0%							

Opening Width Equation for Rectangular Opening

$L = Q / (CH^{1.5}) \times 12 + 0.2 \times H \times 12$ (UD-BMP Spreadsheet -- EDB tab)

C = 3.0

Forebay Overflow Calculation

Forebay	Water Surf Elev	Crest Elev	Crest Length	Flow Depth	**Calc'd Flow**
DP 12	100.25	100.0	6.0 ft	0.25 ft	2.3 cfs
DP 14	100.25	100.0	6.0 ft	0.25 ft	2.3 cfs
				---	---

Weir Equation:

$Q = CLH^{1.5}$

C = 3.0

C = Weir coefficient (dimensionless), C = 3.0 (most cases)

L = Length of weir at Crest, in ft. Not including sideslopes.

Trickle Channel Calculation

Location	100yr Flow	Req'd Flow 1.0% 100yr	Bottom Width	Flow Depth	Side Slope	Slope	Manning 'n'	Top Width	Flow Area	Wetted Perimeter	Hydraulic Radius	Flow Velocity	**Capacity**
Det A	31.6cfs	0.3cfs	2.0 ft	0.50 ft	0.0:1	0.5%	0.013	2.0 ft	1.00 sf	3.0 ft	0.33 ft	3.9 ft/sec	**3.9 cfs**

Equations:

Area (A) = $b(d)+zd^2$
- b = width
- d = depth

Perimeter (P) = $b+2d*(1+z^2)^{0.5}$
- z = side slope

Hydraulic Radius = A/P

Velocity = $(1.49/n)R_n^{2/3} S^{1/2}$
- S = Slope of the channel
- n = Manning's number
- R_n = Hydraulic Radius (Reynold's Number)

Flow = $(1.49/n)AR_n^{2/3} S^{1/2}$

APPENDIX B.1

Supporting Detention Basin Tables and Figures

beneficial if a project is being phased or when adequate land is not available to combine all of the elements in one facility.

4.1.1 Flood Control Volume

UDFCD has developed empirical equations for estimating the total required storage volume that can be applied to on-site, multi-level ponds or to on-site or sub-regional FSD ponds. The empirical equations include:

$$V_i = K_i A \qquad \text{Equation 13-1}$$

For NRCS soil types B, C and D.

$$K_{100} = (1.78 \cdot I - 0.002\, I^2 - 3.56) / 900 \qquad \text{Equation 13-2}$$

$$K_5 = (0.77 \cdot I - 2.65) / 1{,}000 \qquad \text{Equation 13-3}$$

For NRCS soil Type A:

$$K_{100A} = (-0.00005501 \cdot I^2 + 0.030148 \cdot I - 0.12) / 12 \qquad \text{Equation 13-4}$$

Where:

V_i = required volume, with i= year storm, acre-feet

K_i = empirical volume coefficient, with i= year storm

i = return period for storm event, years

I = fully developed tributary basin imperviousness, %

A = tributary drainage basin area, acres

These equations can be applied to calculate the total detention storage for drainage basins up to about 130 acres. When more than one soil type or land use is present in the drainage basin, the storage volume must be weighted by the proportionate areas of each soil type and/or land use. For FSDs, the EURV need not be added to this volume. See UDFCD Manual Volume 2, Storage Chapter for a full description of this method.

4.1.2 EURV

UDFCD has developed empirical equations for estimating the EURV portion of the storage volume that can be applied to on-site, sub-regional or regional FSD ponds.

The empirical equations are as follows:

For NRCS Soil Group A:

$$EURV_A = 1.1\,(2.0491(I/100) - 0.1113) \qquad \text{Equation 13-5}$$

For NRCS Soil Group B:

$$EURV_B = 1.1\,(1.2846(I/100) - 0.0461) \qquad \text{Equation 13-6}$$

For NRCS Soil Group C/D:

$$EURV_{CD} = 1.1\ (1.1381(I/100) - 0.0339)$$ Equation 13-7

Where:

$EURV_K$ = Excess Urban Runoff Volume in watershed inches, K=A, B or C/D soil group

I = drainage basin imperviousness, %

These equations apply to all FSDs and the EURV need not be added to the flood control volume or to the WQCV. When more than one soil type or land use is present in the drainage basin, the EURV must be weighted by the proportionate areas of each soil type and/or land use. If hydrologic routing is used to size the flood control volume, the EURV remains the same as calculated by these equations and is included in the pond's stage/storage configuration for modeling.

4.1.3 Initial Surcharge Volume

The initial surcharge volume is at least 0.3 percent of the WQCV and should be 4- to 12-inches deep. The initial surcharge volume is included in the WQCV and does not increase the required total storage volume.

4.1.4 Design Worksheets

The Full Spectrum Worksheet in the UD-Detention Spreadsheet performs all of these calculations for the standard designs. For multi-level ponds, the flood control volumes are calculated for the two design storm frequencies: the major storm and the minor storm.

4.2 Allowable Release Rates

Allowable release rates from detention facilities vary with the type of facility and with the storage volume type, as follows:

- **Flood Storage Volume**: The flood storage release rates are determined by the allowable release rates that are intended to approximate storm event runoff rates from the undeveloped upstream drainage basin.

- **EURV**: The EURV release rate is determined based on a72-hour drain time. The purpose of this slow release rate is to mitigate the impacts of increased runoff volumes due to development by reducing the potential for downstream erosion.

- **WQCV**: The WQCV release rate is determined based on a 40-hour drain time for extended detention basins. The purpose of this slow release rate is to provide time for pollutants to settle, The WQCV is incorporated into the EURV and works with it to release less erosive flows. The method for determining this design rate is described in Chapter 3 of Volume 2 of this Manual.

4.2.1 Flood Storage Release Rates

Allowable releases rates from the flood storage element of detention may be based on generalized average unit runoff rates or estimates of pre-development runoff rates. Allowable unit release rates (cfs/ac) may be used for any type of detention, however, when a hydrograph routing method is applied (for regional or

Safety Grates

Safety grates are intended to keep people and animals from inadvertently entering a storm drain. They are sometimes required even when debris entering a storm drain is not a concern. The grate on top of the outlet drop box is considered a safety grate and should be designed accordingly. The danger associated with outlet structures is the potential associated with pinning a person or animal to unexposed outlet pipe or grate. See the *Culverts and Bridges* chapter of Volume 2 of this manual for design criteria related to safety grates.

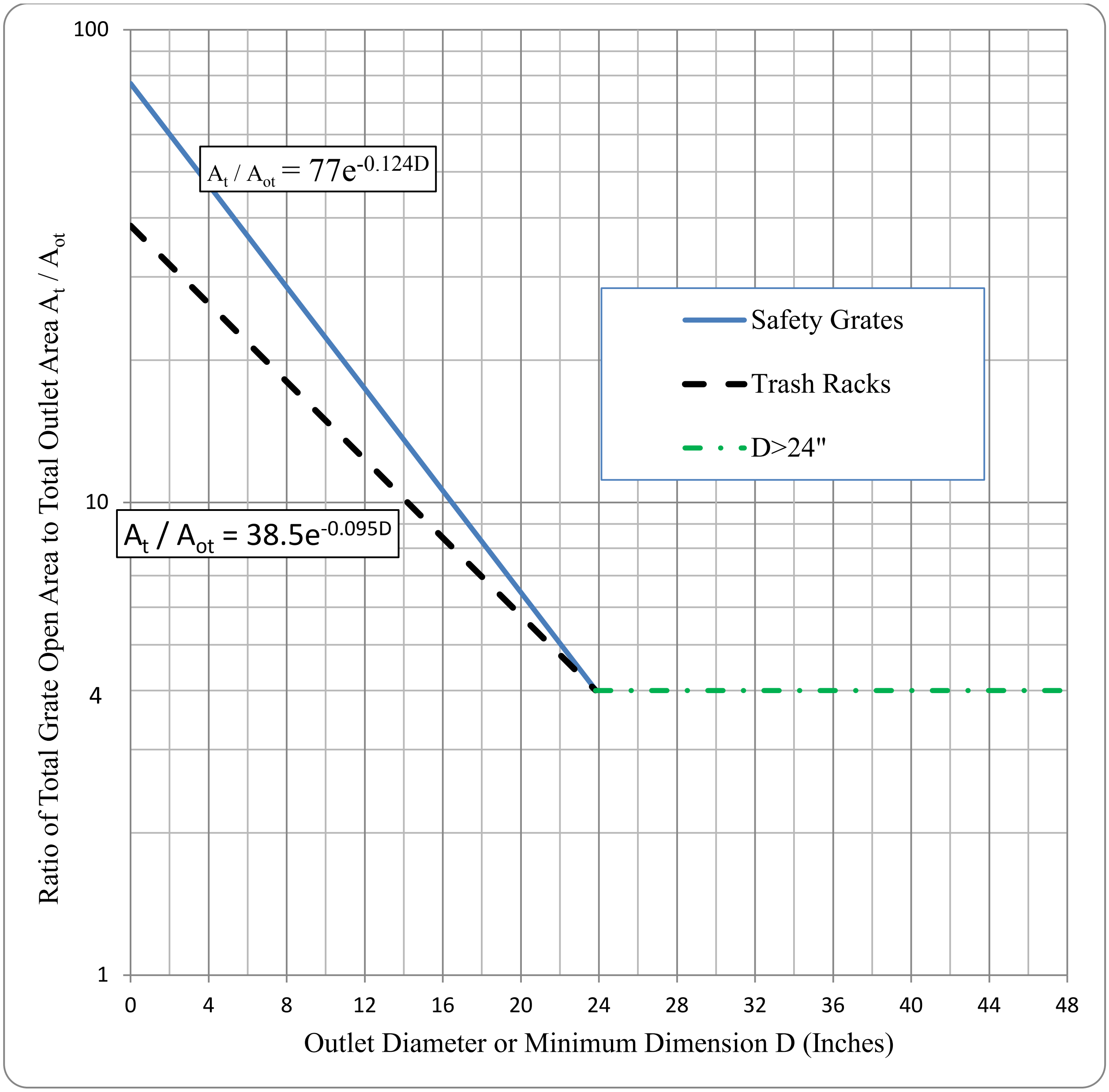


Figure OS-1. Trash Rack Sizing

Table OS-2. Thickness of steel water quality plate

Steel plate thickness (in inches) based on design depth and span of plate										
	Head (feet)									
Span (feet)	3	4	5	6	7	8	9	10	11	12
1	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875
2	0.1875	0.2500	0.2500	0.2500	0.2500	0.2500	0.2500	0.2500	0.2500	0.2500
3	0.2500	0.2500	0.3750	0.3750	0.3750	0.3750	0.3750	0.3750	0.3750	0.5000
4	0.2500	0.3750	0.3750	0.3750	0.3750	0.5000	0.5000	0.5000	0.5000	0.5000

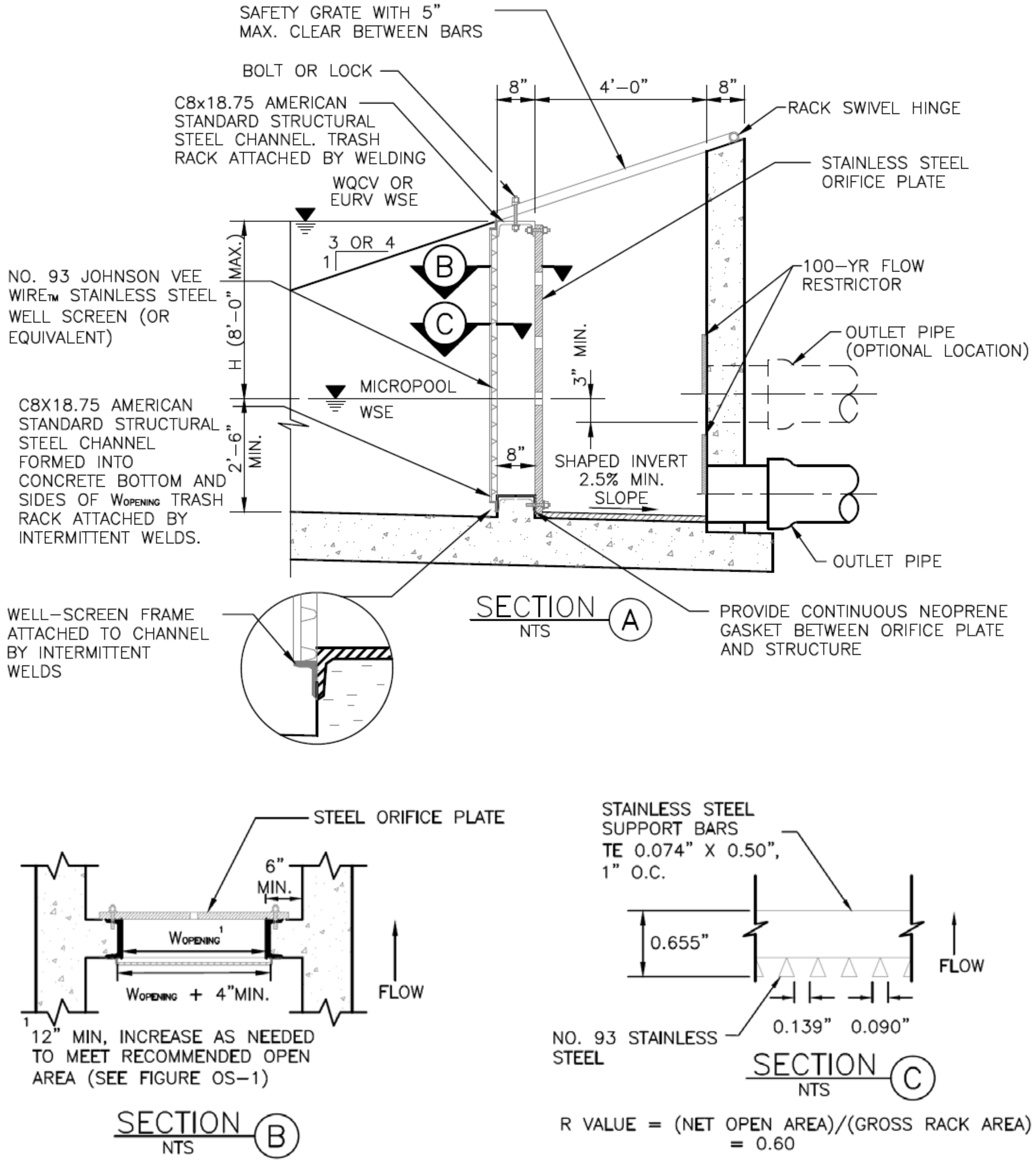
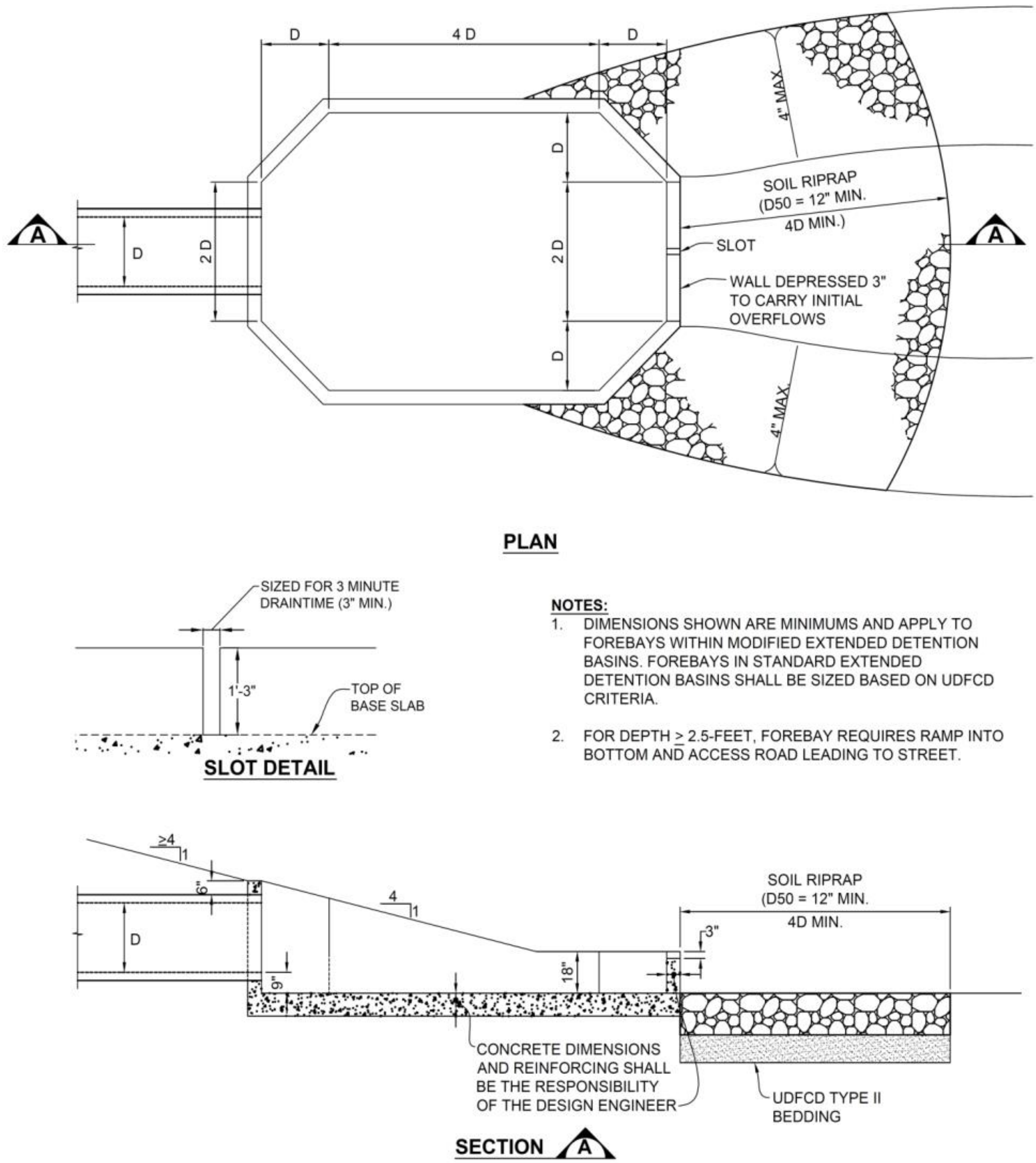


Figure OS-5. Typical outlet structure with well screen trash rack

Figure 13-9. Concept for Integral Forebay at Pipe Outfall



NOTES:

1. DIMENSIONS SHOWN ARE MINIMUMS AND APPLY TO FOREBAYS WITHIN MODIFIED EXTENDED DETENTION BASINS. FOREBAYS IN STANDARD EXTENDED DETENTION BASINS SHALL BE SIZED BASED ON UDFCD CRITERIA.
2. FOR DEPTH ≥ 2.5-FEET, FOREBAY REQUIRES RAMP INTO BOTTOM AND ACCESS ROAD LEADING TO STREET.

Figure 13-11. Concept for Outlet Structure with Flared Wingwalls and Handrail
(Integral Micropool Shown)

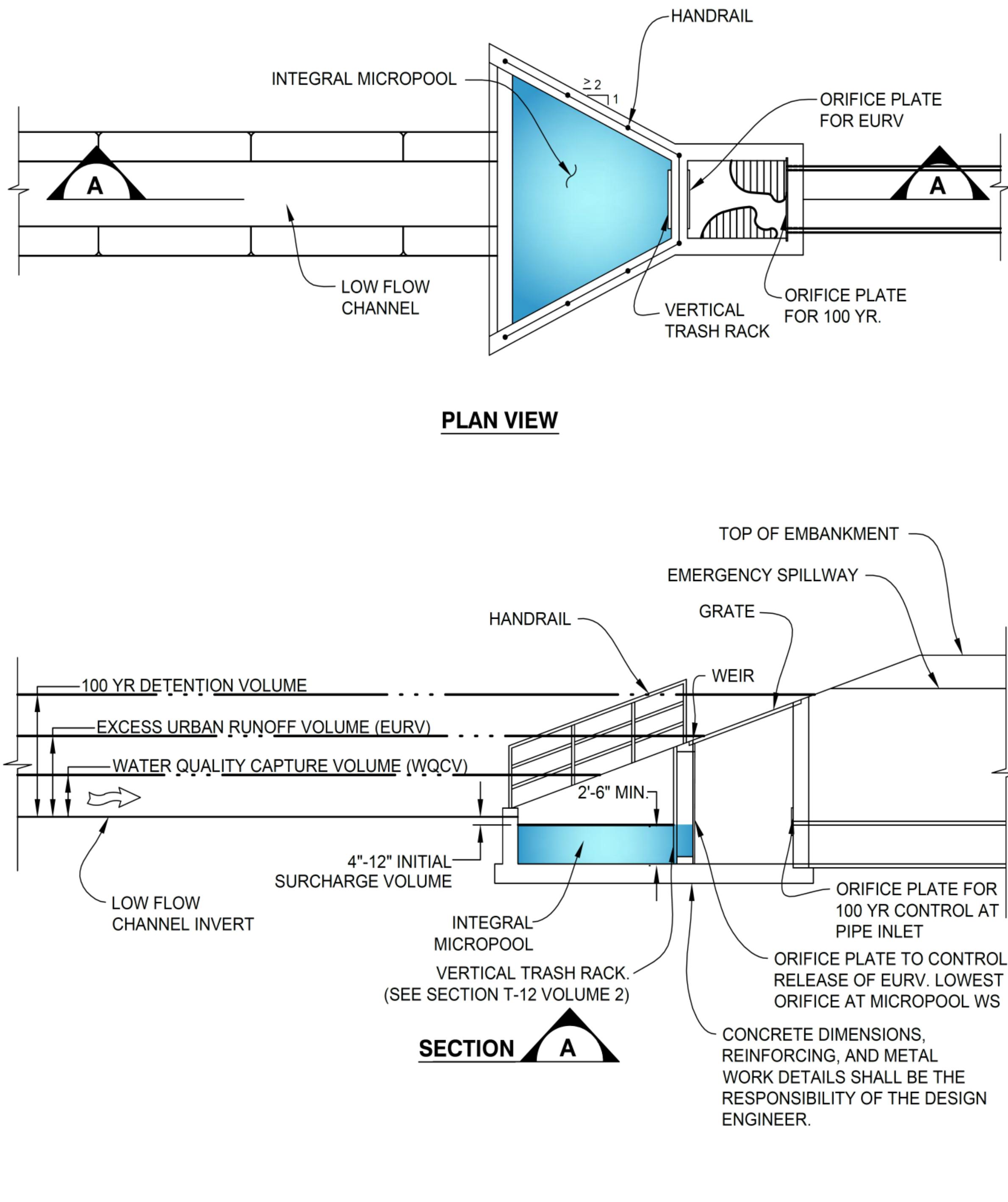


Figure 13-12c. Emergency Spillway Protection

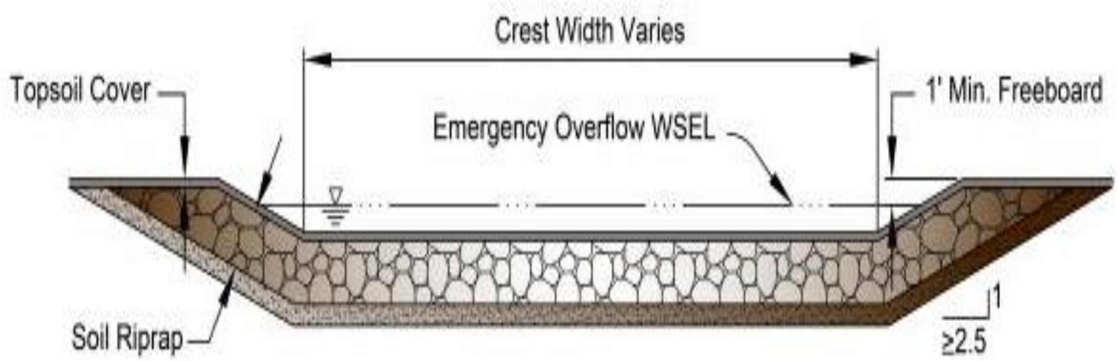
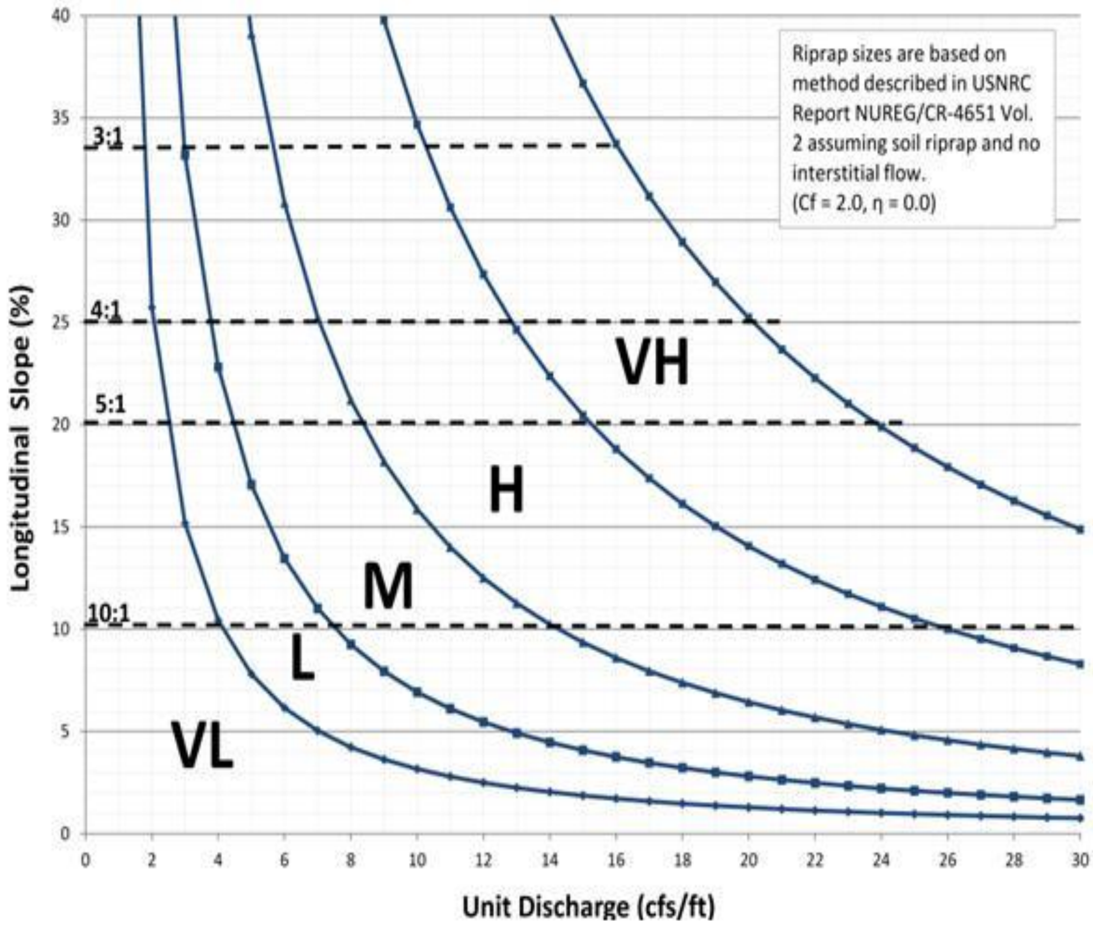


Figure 13-12d. Riprap Types for Emergency Spillway Protection



APPENDIX C

Hydraulic Calculations

Street Capacity Calculations – UD Inlet
Inlet Capacity Calculations – UD Inlet and Hand Calcs
Pipe Sizing Calculations
Pipe Outlet Erosion Protection Calculations
Swale Capacity Calculations

Inlet Management

Worksheet Protected

INLET NAME	Inlet 1	Inlet 2	Inlet 3	Inlet 4
Site Type (Urban or Rural)	URBAN	URBAN	URBAN	URBAN
Inlet Application (Street or Area)	STREET	STREET	STREET	STREET
Hydraulic Condition	On Grade	On Grade	In Sump	In Sump
Inlet Type	CDOT Type R Curb Opening	CDOT Type R Curb Opening	CDOT Type R Curb Opening	CDOT Type R Curb Opening

USER-DEFINED INPUT

User-Defined Design Flows

Minor Q_{Known} (cfs)	2.8	1.6	1.3	2.4
Major Q_{Known} (cfs)	7.6	4.4	3.9	7.5

Bypass (Carry-Over) Flow from Upstream

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

Watershed Characteristics

Subcatchment Area (acres)				
Percent Impervious				
NRCS Soil Type				

Watershed Profile

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

Minor Storm Rainfall Input

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

Major Storm Rainfall Input

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

CALCULATED OUTPUT

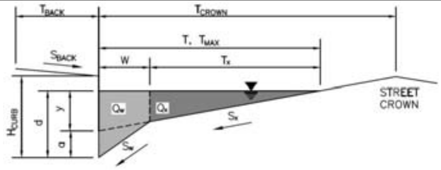
Minor Total Design Peak Flow, Q (cfs)	**2.8**	**1.6**	**1.3**	**2.4**
Major Total Design Peak Flow, Q (cfs)	**7.6**	**4.4**	**3.9**	**7.5**
Minor Flow Bypassed Downstream, Q_b (cfs)	0.0	0.0	N/A	N/A
Major Flow Bypassed Downstream, Q_b (cfs)	0.0	0.0	N/A	N/A

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

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

Project: **The Beach at Woodmoor Filing No. 1**

Inlet ID: **Inlet 1**



Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =		ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =		ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =		
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	13.2	ft
Gutter Width	W =	1.17	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.145	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.039	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} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.16	3.16	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.76	1.76	inches
Water Depth at Gutter Flowline	d =	4.92	4.92	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	12.0	12.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.304	0.304	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	7.7	7.7	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	3.4	3.4	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**11.0**	**11.0**	**cfs**
Flow Velocity within the Gutter Section	V =	3.0	3.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	1.2	1.2	

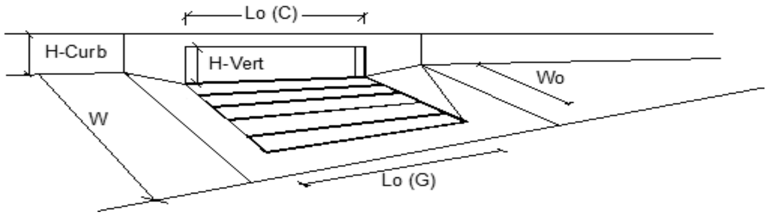
Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\,TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	0.129	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\,TH}$	$Q_{X\,TH}$ =	7.7	66.9	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	7.7	53.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	3.4	9.9	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	11.1	62.9	cfs
Average Flow Velocity Within the Gutter Section	V =	3.0	4.6	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	1.2	3.3	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	1.00	0.49	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**11.1**	**30.7**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =	4.92	6.62	inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =	0.01	1.71	inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Spread Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**11.0**	**30.7**	**cfs**

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

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

INLET ON A CONTINUOUS GRADE



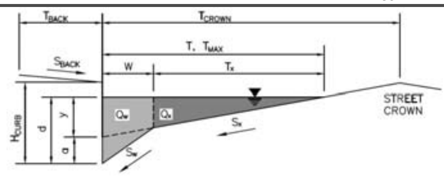
Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	15.00	15.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C_f-C =	0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity'		MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)	**Q_o =**	**2.8**	**7.6**	cfs
Water Spread Width	T =	7.2	11.3	ft
Water Depth at Flowline (outside of local depression)	d =	3.5	4.5	inches
Water Depth at Street Crown (or at T_{MAX})	d_{CROWN} =	0.0	0.0	inches
Ratio of Gutter Flow to Design Flow	E_o =	0.566	0.360	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	1.2	4.9	cfs
Discharge within the Gutter Section W	Q_w =	1.6	2.7	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	0.24	0.34	sq ft
Velocity within the Gutter Section W	V_W =	6.6	8.2	fps
Water Depth for Design Condition	d_{LOCAL} =	6.5	7.5	inches
Grate Analysis (Calculated)		MINOR	MAJOR	
Total Length of Inlet Grate Opening	L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow	$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition		MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Interception Capacity	Q_i =	N/A	N/A	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet	GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet	GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins	V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow	R_f =	N/A	N/A	
Interception Rate of Side Flow	R_x =	N/A	N/A	
Actual Interception Capacity	**Q_a =**	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)	**Q_b =**	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)		MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)	S_e =	0.212	0.142	ft/ft
Required Length L_T to Have 100% Interception	L_T =	7.29	14.57	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	7.29	14.57	ft
Interception Capacity	Q_i =	2.8	7.6	cfs
Under Clogging Condition		MINOR	MAJOR	
Clogging Coefficient	CurbCoef =	1.31	1.31	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog =	0.04	0.04	
Effective (Unclogged) Length	L_e =	13.03	13.03	ft
Actual Interception Capacity	**Q_a =**	**2.8**	**7.6**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	**Q_b =**	**0.0**	**0.0**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**2.8**	**7.6**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**0.0**	**0.0**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**100**	**100**	**%**

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

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

Project: **The Beach at Woodmoor Filing No. 1**

Inlet ID: **Inlet 2**



Gutter Geometry (Enter data in the blue cells)				
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =			ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =			ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =			
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00		inches
Distance from Curb Face to Street Crown	T_{CROWN} =	13.2		ft
Gutter Width	W =	1.17		ft
Street Transverse Slope	S_X =	0.020		ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.145		ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.039		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} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

		Minor Storm	Major Storm	
Maximum Capacity for 1/2 Street based On Allowable Spread				
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.16	3.16	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.76	1.76	inches
Water Depth at Gutter Flowline	d =	4.92	4.92	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	12.0	12.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.304	0.304	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	7.7	7.7	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	3.4	3.4	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**11.0**	**11.0**	**cfs**
Flow Velocity within the Gutter Section	V =	3.0	3.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	1.2	1.2	

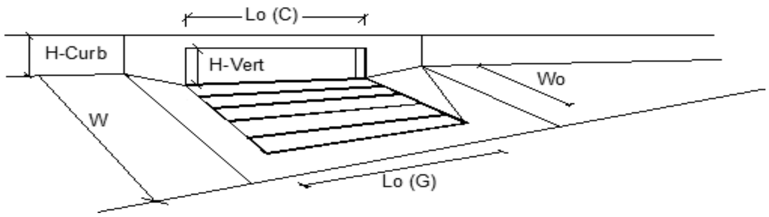
		Minor Storm	Major Storm	
Maximum Capacity for 1/2 Street based on Allowable Depth				
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\,TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	0.129	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\,TH}$	$Q_{X\,TH}$ =	7.7	66.9	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	7.7	53.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	3.4	9.9	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	11.1	62.9	cfs
Average Flow Velocity Within the Gutter Section	V =	3.0	4.6	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	1.2	3.3	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	1.00	0.49	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**11.1**	**30.7**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =	4.92	6.62	inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =	0.01	1.71	inches

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Spread Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	**Q_{allow} =**	**11.0**	**30.7**	**cfs**

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

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

INLET ON A CONTINUOUS GRADE



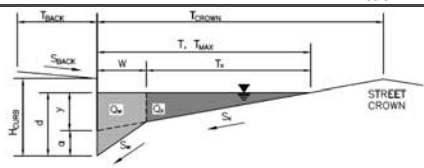
Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')		a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)		No =	1	1	
Length of a Single Unit Inlet (Grate or Curb Opening)		L_o =	10.00	10.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)		W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)		C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)		C_f-C =	0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity'			MINOR	MAJOR	
Design Discharge for Half of Street (from Sheet *Inlet Management*)		**Q_o =**	**1.6**	**4.4**	cfs
Water Spread Width		T =	5.3	8.9	ft
Water Depth at Flowline (outside of local depression)		d =	3.0	3.9	inches
Water Depth at Street Crown (or at T_{MAX})		d_{CROWN} =	0.0	0.0	inches
Ratio of Gutter Flow to Design Flow		E_o =	0.719	0.462	
Discharge outside the Gutter Section W, carried in Section T_x		Q_x =	0.5	2.4	cfs
Discharge within the Gutter Section W		Q_w =	1.2	2.0	cfs
Discharge Behind the Curb Face		Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W		A_W =	0.20	0.28	sq ft
Velocity within the Gutter Section W		V_W =	5.9	7.3	fps
Water Depth for Design Condition		d_{LOCAL} =	6.0	6.9	inches
Grate Analysis (Calculated)			MINOR	MAJOR	
Total Length of Inlet Grate Opening		L =	N/A	N/A	ft
Ratio of Grate Flow to Design Flow		$E_{o\text{-}GRATE}$ =	N/A	N/A	
Under No-Clogging Condition			MINOR	MAJOR	
Minimum Velocity Where Grate Splash-Over Begins		V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow		R_f =	N/A	N/A	
Interception Rate of Side Flow		R_x =	N/A	N/A	
Interception Capacity		Q_i =	N/A	N/A	cfs
Under Clogging Condition			MINOR	MAJOR	
Clogging Coefficient for Multiple-unit Grate Inlet		GrateCoef =	N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet		GrateClog =	N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet		L_e =	N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins		V_o =	N/A	N/A	fps
Interception Rate of Frontal Flow		R_f =	N/A	N/A	
Interception Rate of Side Flow		R_x =	N/A	N/A	
Actual Interception Capacity		**Q_a =**	**N/A**	**N/A**	cfs
Carry-Over Flow = Q_o-Q_a (to be applied to curb opening or next d/s inlet)		**Q_b =**	**N/A**	**N/A**	cfs
Curb or Slotted Inlet Opening Analysis (Calculated)			MINOR	MAJOR	
Equivalent Slope S_e (based on grate carry-over)		S_e =	0.263	0.176	ft/ft
Required Length L_T to Have 100% Interception		L_T =	4.95	9.98	ft
Under No-Clogging Condition			MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)		L =	4.95	9.98	ft
Interception Capacity		Q_i =	1.6	4.4	cfs
Under Clogging Condition			MINOR	MAJOR	
Clogging Coefficient		CurbCoef =	1.25	1.25	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet		CurbClog =	0.06	0.06	
Effective (Unclogged) Length		L_e =	8.75	8.75	ft
Actual Interception Capacity		**Q_a =**	**1.6**	**4.4**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a		**Q_b =**	**0.0**	**0.0**	cfs
Summary			MINOR	MAJOR	
Total Inlet Interception Capacity		**Q =**	**1.6**	**4.4**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)		**Q_b =**	**0.0**	**0.0**	**cfs**
Capture Percentage = Q_a/Q_o =		**C% =**	**100**	**100**	**%**

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

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

Project: The Beach at Woodmoor Filing No. 1

Inlet ID: Inlet 3



Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =		ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =		ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =		
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	13.2	ft
Gutter Width	W =	1.17	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.145	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	

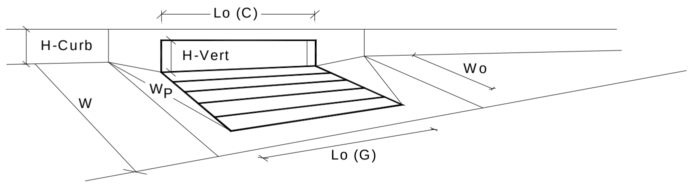
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.16	3.16	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.76	1.76	inches
Water Depth at Gutter Flowline	d =	4.92	4.92	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	12.0	12.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.304	0.304	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\,TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	0.129	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\,TH}$	$Q_{X\,TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		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



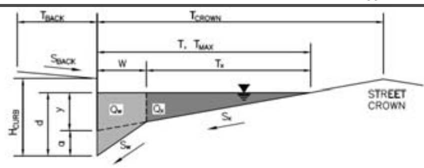
		MINOR	MAJOR	
Design Information (Input) CDOT Type R Curb Opening				
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (outside of local depression)	Ponding Depth =	4.9	8.5	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 =	1.17	1.17	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	
Grate Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	N/A	N/A	cfs
Interception with Clogging	Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	N/A	N/A	cfs
Interception with Clogging	Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	N/A	N/A	cfs
Interception with Clogging	Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)	Q_{Grate} =	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb Opening as a Weir (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{wi} =	3.0	10.2	cfs
Interception with Clogging	Q_{wa} =	2.7	9.1	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)		MINOR	MAJOR	
Interception without Clogging	Q_{oi} =	8.9	11.5	cfs
Interception with Clogging	Q_{oa} =	8.0	10.4	cfs
Curb Opening Capacity as Mixed Flow		MINOR	MAJOR	
Interception without Clogging	Q_{mi} =	4.8	10.1	cfs
Interception with Clogging	Q_{ma} =	4.3	9.1	cfs
Resulting Curb Opening Capacity (assumes clogged condition)	Q_{Curb} =	**2.7**	**9.1**	**cfs**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	5.00	5.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	T =	13.2	28.2	ft.>T-Crown
Resultant Flow Depth at Street Crown	d_{CROWN} =	0.0	3.6	inches
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} =	0.24	0.54	ft
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	0.63	1.00	
Curb Opening Performance Reduction Factor for Long Inlets	RF_{Curb} =	1.00	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets	RF_{Grate} =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**2.7**	**9.1**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)	$Q_{PEAK\ REQUIRED}$ =	1.3	3.9	cfs

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

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

Project: The Beach at Woodmoor Filing No. 1

Inlet ID: Inlet 4



Gutter Geometry (Enter data in the blue cells)				
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =		ft	
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =		ft/ft	
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =			
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches	
Distance from Curb Face to Street Crown	T_{CROWN} =	13.2	ft	
Gutter Width	W =	1.17	ft	
Street Transverse Slope	S_X =	0.020	ft/ft	
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.145	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		

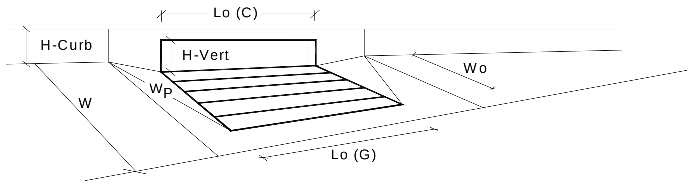
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

Maximum Capacity for 1/2 Street based On Allowable Spread		Minor Storm	Major Storm	
Water Depth without Gutter Depression (Eq. ST-2)	y =	3.16	3.16	inches
Vertical Depth between Gutter Lip and Gutter Flowline (usually 2")	d_C =	2.0	2.0	inches
Gutter Depression (d_C - (W * S_x * 12))	a =	1.76	1.76	inches
Water Depth at Gutter Flowline	d =	4.92	4.92	inches
Allowable Spread for Discharge outside the Gutter Section W (T - W)	T_X =	12.0	12.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.304	0.304	
Discharge outside the Gutter Section W, carried in Section T_X	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_T - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Maximum Flow Based On Allowable Spread	**Q_T =**	**SUMP**	**SUMP**	**cfs**
Flow Velocity within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity times Gutter Flowline Depth	V*d =	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\ TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	0.129	
Theoretical Discharge outside the Gutter Section W, carried in Section $T_{X\ TH}$	$Q_{X\ TH}$ =	0.0	0.0	cfs
Actual Discharge outside the Gutter Section W, (limited by distance T_{CROWN})	Q_X =	0.0	0.0	cfs
Discharge within the Gutter Section W (Q_d - Q_X)	Q_W =	0.0	0.0	cfs
Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)	Q_{BACK} =	0.0	0.0	cfs
Total Discharge for Major & Minor Storm (Pre-Safety Factor)	Q =	0.0	0.0	cfs
Average Flow Velocity Within the Gutter Section	V =	0.0	0.0	fps
V*d Product: Flow Velocity Times Gutter Flowline Depth	V*d =	0.0	0.0	
Slope-Based Depth Safety Reduction Factor for Major & Minor (d $\geq$ 6") Storm	R =	SUMP	SUMP	
Max Flow Based on Allowable Depth (Safety Factor Applied)	**Q_d =**	**SUMP**	**SUMP**	**cfs**
Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)	d =			inches
Resultant Flow Depth at Street Crown (Safety Factor Applied)	d_{CROWN} =			inches

		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



Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')		a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	4.9	8.5	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 =	1.17	1.17	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	
Grate Flow Analysis (Calculated)			MINOR	MAJOR	
Clogging Coefficient for Multiple Units		Coef =	N/A	N/A	
Clogging Factor for Multiple Units		Clog =	N/A	N/A	
Grate Capacity as a Weir (based on Modified HEC22 Method)			MINOR	MAJOR	
Interception without Clogging		Q_{wi} =	N/A	N/A	cfs
Interception with Clogging		Q_{wa} =	N/A	N/A	cfs
Grate Capacity as a Orifice (based on Modified HEC22 Method)			MINOR	MAJOR	
Interception without Clogging		Q_{oi} =	N/A	N/A	cfs
Interception with Clogging		Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow			MINOR	MAJOR	
Interception without Clogging		Q_{mi} =	N/A	N/A	cfs
Interception with Clogging		Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)		**Q_{Grate} =**	**N/A**	**N/A**	**cfs**
Curb Opening Flow Analysis (Calculated)			MINOR	MAJOR	
Clogging Coefficient for Multiple Units		Coef =	1.00	1.00	
Clogging Factor for Multiple Units		Clog =	0.10	0.10	
Curb Opening as a Weir (based on Modified HEC22 Method)			MINOR	MAJOR	
Interception without Clogging		Q_{wi} =	3.0	10.2	cfs
Interception with Clogging		Q_{wa} =	2.7	9.1	cfs
Curb Opening as an Orifice (based on Modified HEC22 Method)			MINOR	MAJOR	
Interception without Clogging		Q_{oi} =	8.9	11.5	cfs
Interception with Clogging		Q_{oa} =	8.0	10.4	cfs
Curb Opening Capacity as Mixed Flow			MINOR	MAJOR	
Interception without Clogging		Q_{mi} =	4.8	10.1	cfs
Interception with Clogging		Q_{ma} =	4.3	9.1	cfs
Resulting Curb Opening Capacity (assumes clogged condition)		**Q_{Curb} =**	**2.7**	**9.1**	**cfs**
Resultant Street Conditions			MINOR	MAJOR	
Total Inlet Length		L =	5.00	5.00	feet
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)		T =	13.2	28.2	ft.>T-Crown
Resultant Flow Depth at Street Crown		d_{CROWN} =	0.0	3.6	inches
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} =	0.24	0.54	ft
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	0.63	1.00	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	1.00	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets		RF_{Grate} =	N/A	N/A	
			MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)		**Q_a =**	**2.7**	**9.1**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)		$Q_{PEAK\ REQUIRED}$ =	2.4	7.5	cfs

KIOWA ENGINEERING CORPORATION

JOB 16059 - THE BEACH AT WOODMOOR FIL. No. 1

SHEET NO. 1 OF 1

CALCULATED BY CJC DATE 10/27/17

CHECKED BY DATE

SCALE

TYPE C INLET CAPACITIES

USE CITY'S DCM VOL. 1, FIG. 8-10

INLET 5: BASINS A-6 AND A-7

$Q_5 = 0.6$ CFS

$Q_{100} = 1.8$ CFS

FIG. 8-10 ALREADY INCLUDES A REDUCTION FACTOR, BUT WILL ALSO ASSUME A 50% CLOGGING FACTOR:

AT $Q_5 = 0.6 \times 2 = 1.2$ CFS, $Y \sim$ 1.6 in. (0.13 FT.) [ONE STD. GRATE]

AT $Q_{100} = 1.8 \times 2 = 3.6$ CFS, $Y \sim$ 3.3 in. (0.28 FT.)

DEPTH AVAIL. = 7123.8 - 7123.5 = 0.3 FT., OKAY

* THEREFORE, USE A TYPE C INLET WITH ONE STANDARD GRATE

INLET 6: BASIN A-5, CARRY-OVER FLOW FROM BASIN A-4

$Q_5 = 2.3$ CFS + 0 CFS = 2.3 CFS

$Q_{100} = 7.0$ CFS + 0 CFS = 7.0 CFS

FOR A 50% CLOGGING FACTOR,

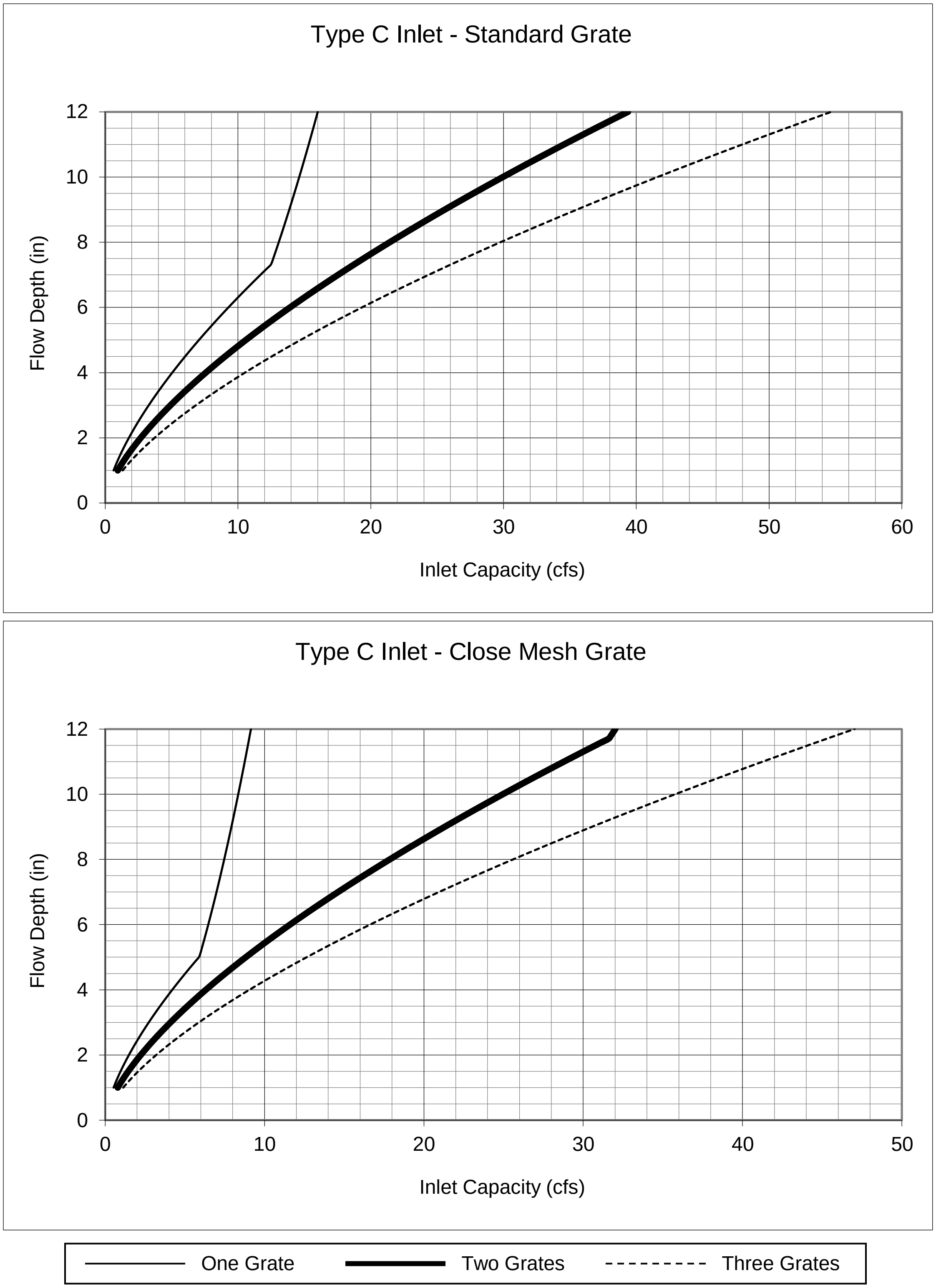
AT $Q_5 = 2.3 \times 2 = 4.6$ CFS, $Y \sim$ 3.8 in. (0.32 FT.) [ONE STD. GRATE]

AT $Q_{100} = 7.0 \times 2 = 14.0$ CFS, $Y \sim$ 9.2 in. (0.77 FT.)

DEPTH AVAIL. = 7104.0 - 7102.7 = 1.3 FT., OKAY

* THEREFORE, USE A TYPE C INLET WITH ONE STANDARD GRATE

Figure 8-10. Inlet Capacity Chart Sump Conditions, Area (Type C) Inlet



Notes:
1. The standard inlet parameters must apply to use these charts.

The Beach at Woodmoor Filing No. 1
Pipe Diameter Calculations

Pipe #	100yr Flow	Design Flow	Contributing Flows	Manning 'n'	Pipe Slope	Calculated Pipe Diameter	**Pipe Diameter**	Minimum Slope of Pipe	Full Pipe Flow Velocity	Head above Pipe Flowline	H	Pipe Inlet Control Capacity	**Mannings Pipe Capacity**	Capacity Check
S1	7.6 cfs	7.6 cfs	Basin A3	0.013	**1.0%**	16-inch	**18-inch**	0.52%	6.0 ft/sec	2.2 ft	1.5 ft	11.1 cfs	**10.5 cfs**	OK
S2	11.5 cfs	11.5 cfs	DP 4	0.013	**0.5%**	21-inch	**24-inch**	0.26%	5.1 ft/sec	2.5 ft	1.5 ft	20.1 cfs	**16.0 cfs**	OK
S3	18.4 cfs	18.4 cfs	DP 6	0.013	**1.0%**	22-inch	**24-inch**	0.66%	7.2 ft/sec	5.8 ft	4.8 ft	35.9 cfs	**22.7 cfs**	OK
S4	19.9 cfs	19.9 cfs	DP 12	0.013	**1.0%**	23-inch	**24-inch**	0.78%	7.2 ft/sec	2.5 ft	1.5 ft	20.1 cfs	**22.7 cfs**	OK
S4A	2.4 cfs	2.4 cfs	DP 11	0.013	**1.0%**	10-inch	**18-inch**	0.05%	6.0 ft/sec	3.5 ft	2.8 ft	15.3 cfs	**10.5 cfs**	OK
S5	0.3 cfs	0.3 cfs	Basin A6	0.013	**1.0%**	5-inch	**18-inch**	0.00%	6.0 ft/sec	3.5 ft	2.8 ft	15.3 cfs	**10.5 cfs**	OK
S6	26.6 cfs	26.6 cfs	Basin OS-4	0.013	**1.0%**	26-inch	**30-inch**	0.42%	8.4 ft/sec	3.5 ft	2.3 ft	38.4 cfs	**41.1 cfs**	OK
S7	26.6 cfs	26.6 cfs	Basin OS-4	0.013	**1.4%**	24-inch	**24-inch**	1.39%	8.5 ft/sec	3.7 ft	2.7 ft	26.9 cfs	**26.8 cfs**	OK
S8	27.8 cfs	27.8 cfs	Basin OS-5	0.013	**2.8%**	21-inch	**24-inch**	1.51%	12.0 ft/sec	3.9 ft	2.9 ft	27.9 cfs	**37.8 cfs**	OK
S9	28.5 cfs	28.5 cfs	DP 8	0.013	**1.4%**	25-inch	**30-inch**	0.48%	9.9 ft/sec	3.7 ft	2.5 ft	40.1 cfs	**48.7 cfs**	OK
S10	55.3 cfs	55.3 cfs	DP 9	0.013	**1.9%**	30-inch	**30-inch**	1.82%	11.5 ft/sec	9.9 ft	8.7 ft	75.3 cfs	**56.7 cfs**	OK
S11	1.6 cfs	1.6 cfs	Basin B2	0.013	**1.0%**	9-inch	**18-inch**	0.02%	6.0 ft/sec	3.0 ft	2.3 ft	13.8 cfs	**10.5 cfs**	OK
S12	3.9 cfs	3.9 cfs	Basin B4	0.013	**1.0%**	12-inch	**18-inch**	0.14%	6.0 ft/sec	3.3 ft	2.5 ft	14.6 cfs	**10.5 cfs**	OK
S13	11.0 cfs	11.0 cfs	DP 14	0.013	**1.2%**	18-inch	**18-inch**	1.10%	6.5 ft/sec	3.3 ft	2.5 ft	14.6 cfs	**11.5 cfs**	OK
S14	15.4 cfs	15.4 cfs	DB A	0.013	**1.0%**	21-inch	**24-inch**	0.46%	7.2 ft/sec	5.0 ft	4.0 ft	32.8 cfs	**22.7 cfs**	OK
Ex. 30"	28.5 cfs	28.5 cfs	DP 8	0.013	**1.8%**	23-inch	**30-inch**	0.48%	11.2 ft/sec	3.4 ft	2.2 ft	37.5 cfs	**55.2 cfs**	OK

Equations:

Pipe Dia=$((2.16Qn)/(S^{0.5}))^{0.375}$
- Q = Discharge in cubic feet per second
- n = Manning's roughness coefficient
  - RCP=0.013, CMP=0.024, HDPE (smooth)=0.012
- S = Slope of the pipe
- R_h = Hydraulic Radius

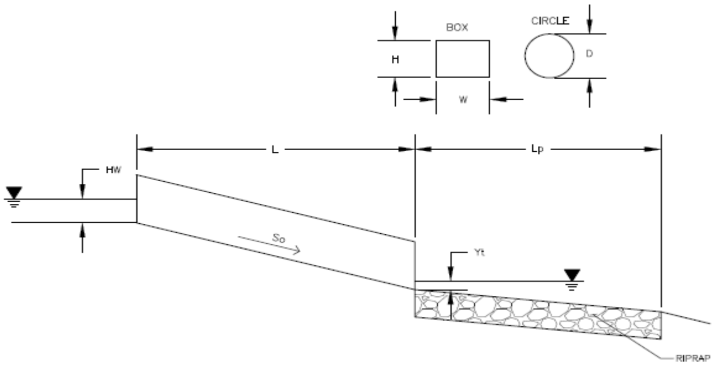
Flow Velocity = $(1.49/n)R_h^{2/3} S^{1/2}$
Pipe Capacity = $(1.49/n)AR_h^{2/3} S^{1/2}$
- A = Cross-sectional area of pipe
- A=p $(D^2/4)$
- D = Inside Diameter of Pipe

$R_h = A_w/W_p$
- $A_w = p(d^2/4)$
  - A_w = Water Cross Sectional Area
  - d = Water (Flow) Depth Within Pipe
- W_p = pd (For Capacity Calculation)
  - W_p=Wetted Perimeter of Pipe

Orifice Equation:
- $Q = CA(2gH)^{0.5}$
  - C = Orifice coefficient (dimensionless)
  - C = 0.65
  - A = Cross-sectional area of opening, in sf
  - g = Gravitational accel constant, 32.2 ft/sec^2
  - H = Head above centerline of pipe, ft

Determination of Culvert Headwater and Outlet Protection

Project: The Beach at Woodmoor Filing No. 1
Basin ID: 18" RCP, Pipe No. S11



Soil Type:
Choose One:
(●) Sandy
( ) Non-Sandy

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

Design Discharge	Q =	1.6	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)		Square End Projection	
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7112	ft
Outlet Elevation **OR** Slope	So =	0.025	ft/ft
Culvert Length	L =	80	ft
Manning's Roughness	n =	0.013	
Bend Loss Coefficient	k_b =	0	
Exit Loss Coefficient	k_x =	1	
Tailwater Surface Elevation	Elev Y_t =	7110.5	ft
Max Allowable Channel Velocity	V =	5	ft/s

Required Protection (Output):

Tailwater Surface Height	Y_t =	0.50	ft
Flow Area at Max Channel Velocity	A_t =	0.32	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	1.45	
Sum of All Losses Coefficients	k_s =	2.95	ft
Culvert Normal Depth	Y_n =	0.31	ft
Culvert Critical Depth	Y_c =	0.48	ft
Tailwater Depth for Design	d =	0.99	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	0.91	ft
Expansion Factor	1/(2*tan(Θ)) =	6.70	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	0.58	$ft^{0.5}$/s
Froude Number	Fr =	2.24	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.55	
Inlet Control Headwater	HW_I =	0.65	ft
Outlet Control Headwater	HW_O =	-0.97	
Design Headwater Elevation	**HW =**	**7,112.65**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**0.44**	
Minimum Theoretical Riprap Size	d_{50} =	1	in
Nominal Riprap Size	d_{50} =	6	in
UDFCD Riprap Type	**Type =**	**VL**	
Length of Protection	L_p =	**5**	**ft**
Width of Protection	**T =**	**3**	**ft**

The Beach at Woodmoor Filing No. 1

Swale Capacity Calculations

Description	Design Point	Design Flow	Bottom Width	Channel Side Slope		Flow Depth	Channel Slope	Manning "n"	Top Width	Channel Area	Wetted Perimeter	Hydraulic Radius	Flow Velocity	Channel Flow Capacity	Swale Lining
				Left	Right										
Swale 1	A6	0.3 cfs	0.0 ft	4:1	4:1	0.20 ft	7.2%	0.035	1.6 ft	0.16 sf	1.6 ft	0.10 ft	2.4 ft/sec	**0.4 cfs**	Grass
Swale 3	A5	7.0 cfs	0.0 ft	4:1	4:1	0.70 ft	3.1%	0.035	5.6 ft	1.96 sf	5.8 ft	0.34 ft	3.6 ft/sec	**7.2 cfs**	Grass
Swale 5	B2	1.6 cfs	0.0 ft	4:1	4:1	0.42 ft	2.3%	0.035	3.4 ft	0.71 sf	3.5 ft	0.20 ft	2.2 ft/sec	**1.6 cfs**	Grass
Swale 6	11	2.4 cfs	0.0 ft	4:1	4:1	0.46 ft	3.3%	0.035	3.7 ft	0.85 sf	3.8 ft	0.22 ft	2.8 ft/sec	**2.4 cfs**	Grass

Equations:

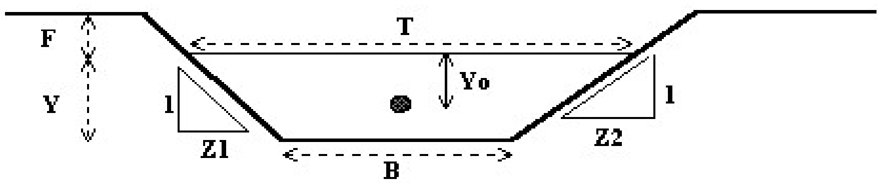
Area (A) = $b(d)+zd^2$
b = width
d = depth

Perimeter (P) = $b+2d*(1+z^2)^{0.5}$
z = side slope
Hydraulic Radius = A/P

Velocity = $(1.49/n)R_n^{2/3}\,S^{1/2}$
S = Slope of the channel
n = Manning's number
R_n = Hydraulic Radius (Reynold's Number)
Flow = $(1.49/n)AR_n^{2/3}\,S^{1/2}$

Normal Flow Analysis - Trapezoidal Channel

Project: **The Beach at Woodmoor Filing No. 1**
Channel ID: **Swale 7**



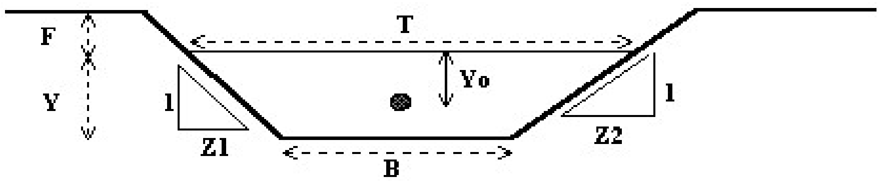
Design Information (Input)			
Channel Invert Slope	So =	0.0705	ft/ft
Manning's n	n =	0.050	
Bottom Width	B =	16.00	ft
Left Side Slope	Z1 =	0.02	ft/ft
Right Side Slope	Z2 =	0.05	ft/ft
Freeboard Height	F =	0.00	ft
Design Water Depth	Y =	**0.50**	ft

Normal Flow Condtion (Calculated)			
Discharge	**Q =**	**38.37**	cfs
Froude Number	**Fr =**	**1.19**	
Flow Velocity	**V =**	**4.79**	fps
Flow Area	A =	8.01	sq ft
Top Width	T =	16.04	ft
Wetted Perimeter	P =	17.00	ft
Hydraulic Radius	R =	0.47	ft
Hydraulic Depth	D =	0.50	ft
Specific Energy	Es =	0.86	ft
Centroid of Flow Area	Yo =	0.25	ft
Specific Force	Fs =	0.48	kip

This bottom width does not reflect the existing swale 7 called out on the drainage map.

Normal Flow Analysis - Trapezoidal Channel

Project: The Beach at Woodmoor Filing No. 1
Channel ID: Swale 7



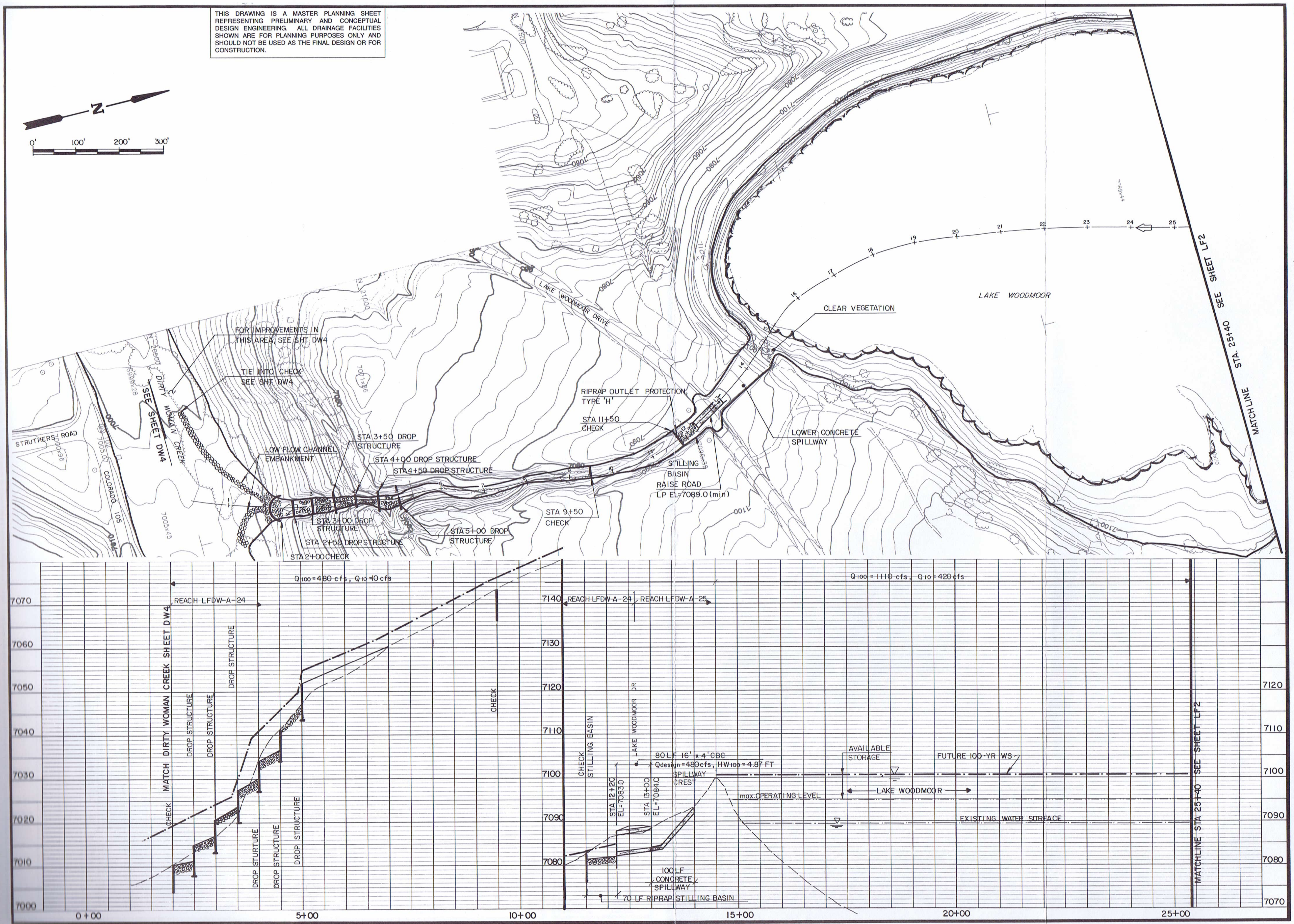
Design Information (Input)			
Channel Invert Slope	So =	0.0705	ft/ft
Manning's n	n =	0.050	
Bottom Width	B =	0.50	ft
Left Side Slope	Z1 =	0.05	ft/ft
Right Side Slope	Z2 =	0.05	ft/ft
Freeboard Height	F =	0.00	ft
Design Water Depth	Y =	**0.75**	ft

Normal Flow Condtion (Calculated)			
Discharge	**Q =**	**1.10**	cfs
Froude Number	**Fr =**	**0.57**	
Flow Velocity	**V =**	**2.72**	fps
Flow Area	A =	0.40	sq ft
Top Width	T =	0.58	ft
Wetted Perimeter	P =	2.00	ft
Hydraulic Radius	R =	0.20	ft
Hydraulic Depth	D =	0.70	ft
Specific Energy	Es =	0.86	ft
Centroid of Flow Area	Yo =	0.37	ft
Specific Force	Fs =	0.01	kip

APPENDIX D

Referenced Information

Excerpt from Dirty Woman and Crystal Creeks Drainage Basin Planning Study



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

Dirty Woman and Crystal Creeks Drainage Basin Planning Study

PRELIMINARY DESIGN

Lake Fork Dirty Woman Creek
Sta. 0+00 to Sta. 25+40

El Paso County Department of Public Works Stormwater Management Division

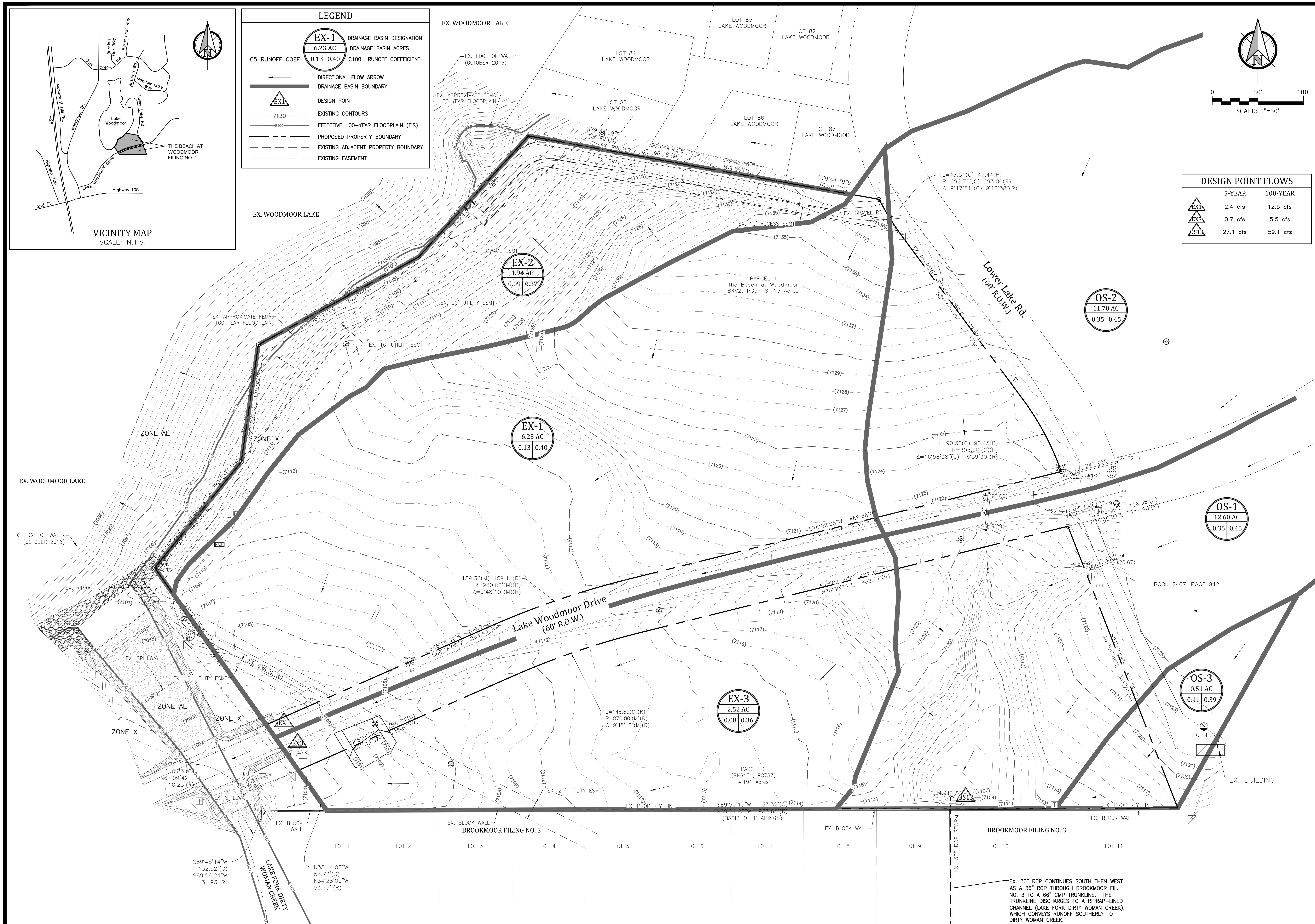
Project No. 91-07-17
Date: 1/93
Design: AWMc
Drawn: EAK
Check: RNW
Revisions:

LF1

APPENDIX E

Existing and Proposed Drainage Plans

Sheet DP1 - Existing Condition Drainage Plan
Sheet DP2 - Developed Condition Drainage Plan



DESIGN POINT FLOWS

	5-YEAR	100-YEAR
EX1	2.4 cfs	12.5 cfs
EX3	0.7 cfs	5.5 cfs
OS13	27.1 cfs	59.1 cfs

Celebrating 30 years
Kiowa Engineering Corporation
1604 South 21st Street
Colorado Springs, Colorado 80904
(719)630-7342

The Beach at Woodmoor Filing No. 1

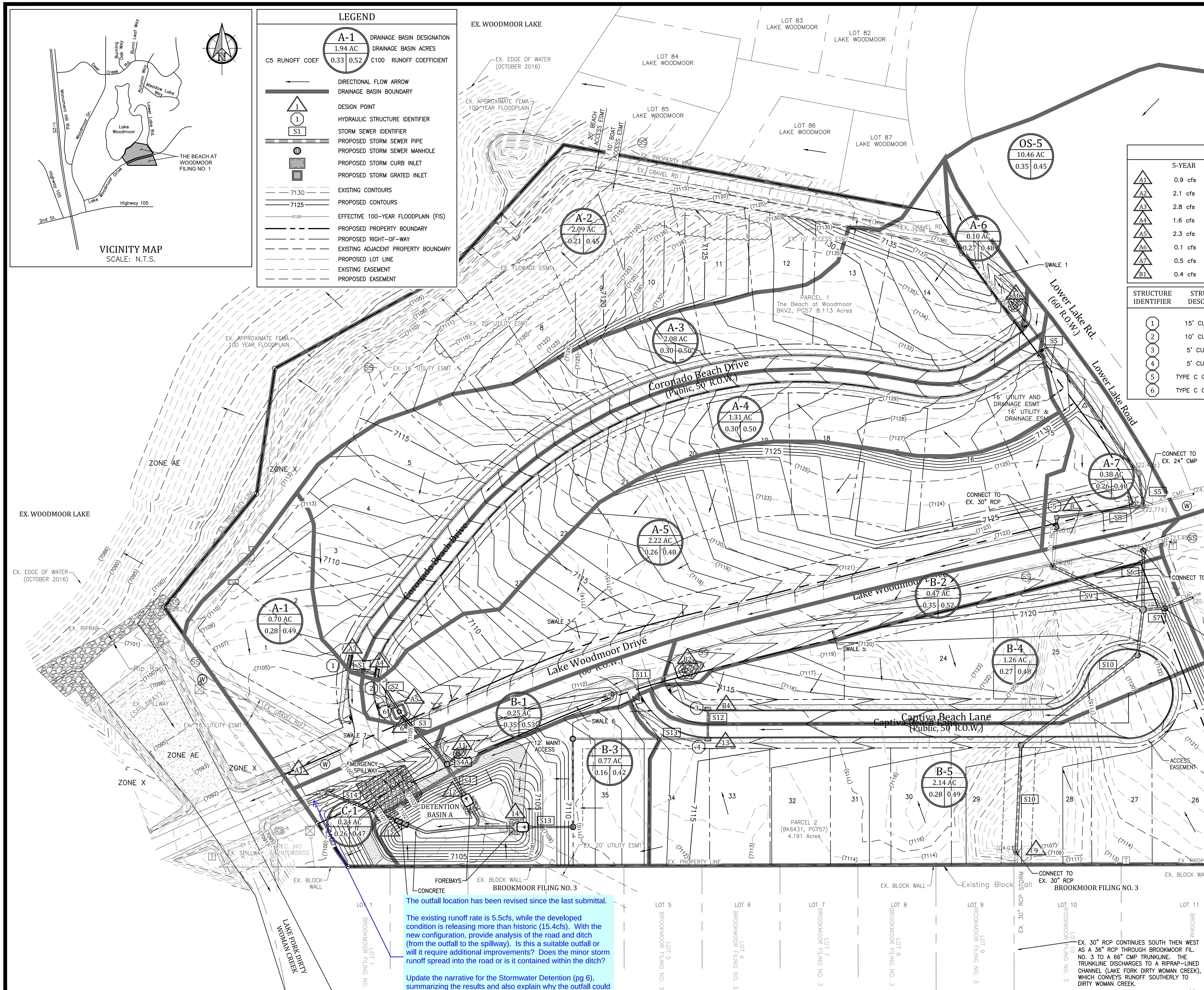
Existing Condition Drainage Plan

El Paso County, Colorado

Project No.:	16059
Date:	January 16, 2018
Design:	JAK
Drawn:	JAK
Check:	CJC
Revisions:	

SHEET

DP1



The outfall location has been revised since the last submittal.

The existing runoff rate is 5.5cfs, while the developed condition is releasing more than historic (15.4cfs). With the new configuration, provide analysis of the road and ditch (from the outfall to the spillway). Is this a suitable outfall or will it require additional improvements? Does the minor storm runoff spread into the road or is it contained within the ditch?

Update the narrative for the Stormwater Detention (pg 6), summarizing the results and also explain why the outfall could not directly go to the spillway per the previous design. Is it because the adjacent parcel would not permit the improvement through their property?

DESIGN POINT FLOWS

Design Point	5-YEAR	100-YEAR
A1	0.9 cfs	2.5 cfs
A2	2.1 cfs	7.3 cfs
A3	2.8 cfs	7.6 cfs
A4	1.6 cfs	4.4 cfs
A5	2.3 cfs	7.0 cfs
A6	0.1 cfs	0.3 cfs
A7	0.5 cfs	1.5 cfs
B1	0.4 cfs	1.0 cfs
B2	0.6 cfs	1.6 cfs
B3	0.5 cfs	2.3 cfs
B4	1.3 cfs	3.9 cfs
B5	2.3 cfs	6.6 cfs
C1	0.3 cfs	1.0 cfs

STRUCTURE IDENTIFIER	STRUCTURE DESCRIPTION
1	15' CURB INLET
2	10' CURB INLET
3	5' CURB INLET
4	5' CURB INLET
5	TYPE C GRATED INLET
6	TYPE C GRATED INLET

STRUCTURE IDENTIFIER	STRUCTURE DESCRIPTION
S1	18" RCP
S2	24" RCP
S3	24" RCP
S4	24" RCP
S4A	18" RCP
S5	18" RCP
S6	30" RCP
S7	24" RCP
S8	24" RCP
S9	30" RCP
S10	30" RCP
S11	18" RCP
S12	18" RCP
S13	18" RCP
S14	24" RCP

DETENTION BASIN A DATA

	WATER SURFACE ELEV. (FT)	REQUIRED VOLUME	RELEASE RATE
WQCV	7100.6	0.15 AC-FT	
EURV	7102.5	0.40 AC-FT	
100-YR	7104.5	0.79 AC-FT	15.4 CFS
100-YR + ½ WQCV	7104.9	0.87 AC-FT	15.4 CFS
SPILLWAY CREST ELEVATION: 7105.0			
TOP OF EMBANKMENT MINIMUM ELEVATION: 7106.5			

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The Beach at Woodmoor Filing No. 1

Developed Condition Drainage Plan

El Paso County, Colorado

Project No.:	16059
Date:	January 16, 2018
Design:	JAK
Drawn:	JAK
Check:	CJC
Revisions:	

SHEET

DP2

16059 Drainage Plan.dwg/Jan 16, 2018