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

Prepared for:
Lake Woodmoor Holdings, LLC
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

May 24, 2017

SF-17-015

PCD Project No. SF-17-___

May 24, 2017

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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Remove "City"

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 City/County for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparing this report.

Kiowa Engineering Corporation, 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 Holdings, LLC

Print Name: ______________________________________

Address: Lake Woodmoor Holdings, LLC
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. 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. 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.

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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 the Lake Woodmoor spillway (DP EX1). 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).

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.

Sub-basin OS-1 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, where it is conveyed southerly to Dirty Woman Creek.

Sub-basin OS-2 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.

Sub-basin OS-3 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 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 to a riprap-lined channel to outlet at Dirty Woman Creek.

Runoff leaving the subject site will be at or less than the existing (undeveloped) condition, so the development of the property will not adversely impact any improvements or drainageways downstream.

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) dated November 9, 1998. The FIRM showing the project site is included in the Appendix. A small portion of the northwest corner of the site is located within a FEMA regulated floodplain based on FIRM 08041C0276 F and as shown on the drainage plans. This portion of the site will remain undeveloped. 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 of the *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).

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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 capacity.

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 basins are 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 basins 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 basins 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 Basins A1 and B 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. Detention Basin A2 includes a grass-lined low flow channel to convey flow to its outlet structure. All detention basins will have a micropool and water quality orifice plate onto an outlet structure, an emergency spillway and a maintenance access trail. The detention basins will be private facilities 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 1.94 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-basins A-3 and A-4, and gutter flow south in Coronado Beach Drive to a swale along the north side of Lake Woodmoor Drive. This sub-basin also includes runoff from the backyards of the three lots west of Sub-basin A-5.1 and the detained runoff released from proposed Detention Basins A1 and A2. The combined runoff will be conveyed west in the Lake Woodmoor Drive north grass-lined swale (Swale 4) to the Lake Woodmoor spillway (DP 1).

<u>Sub-basin A-2</u> is approximately 1.47 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 2.28 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 2).

<u>Sub-basin A-4</u> is approximately 1.36 acres in area and is located south of Coronado Beach Drive between Sub-basins A-5 and A-7. 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 3). Captured runoff at DP 3 will combine with captured runoff from DP 2 and be conveyed in a 19-inch by 30-inch HERCP (24" RCP circular equivalent) to proposed Detention Basin A1 (DP 4).

<u>Sub-basin A-5</u> is approximately 0.18 acres in area and is located northeast of the intersection of Lake Woodmoor Drive and Coronado Beach Drive. This basin represents the area directly tributary to and including proposed Detention Basin A1.

<u>Sub-basin A-5.1</u> is approximately 1.18 acres in area and is located north of Lake Woodmoor Drive between Sub-basins A-1 and A-7. This basin represents the area directly tributary to proposed Detention Basin A2, including backyards, open space and a portion of the north half of Lake Woodmoor Drive. Grass-lined Swale 3 captures and conveys runoff to Detention Basin A2.

<u>Sub-basin A-6</u> 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 7).

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 to grass-lined Swale 2, and be conveyed south then 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 0.46 acres in area and is located south of Lake Woodmoor Drive and west of the private 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 the Lake Woodmoor spillway (DP 11).

Sub-basin B-2 is approximately 0.19 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 private access to Captiva Beach Lane (DP 10). 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 0.81 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 B.

Sub-basin B-4 is approximately 1.60 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 12). Runoff captured at DP 12 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 south in a 30-inch RCP through Sub-basin B-5.

Sub-basin B-5 is approximately 2.07 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 B (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).

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

Three full spectrum detention basins are planned for the proposed development:

Detention Basin A1. The required WQCV for a 40-hour drain time is 0.06 acre-feet. The required excess urban runoff volume (EURV) for a 72-hour drain time is 0.16 acre-feet. The storage volume required for detention is 0.35 acre-feet, which includes 0.32 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}=11.7 cfs (DP 6) 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 4.5 cfs for the 100-year storm event. A proposed 18-inch RCP equipped with a restrictor plate will convey runoff released from the detention basin to the north roadside swale along Lake Woodmoor Drive. If the outlet structure becomes plugged, a 38-foot wide emergency spillway will convey the runoff to the roadside swale.

Detention Basin A2. The required WQCV for a 40-hour drain time is 0.01 acre-feet. The required excess urban runoff volume (EURV) for a 72-hour drain time is 0.02 acre-feet. The storage volume required for detention is 0.08 acre-feet, which includes 0.07 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}=4.1 cfs (Basin A-5.1) 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 1.2 cfs for the 100-year storm event. A proposed 18-inch RCP equipped with a restrictor plate will convey runoff released from the detention basin to the north roadside swale along Lake Woodmoor Drive. If the outlet structure becomes plugged, a 15-foot wide emergency spillway will convey the runoff to the roadside swale.

Detention Basin B. The required WQCV for a 40-hour drain time is 0.06 acre-feet. The required excess urban runoff volume (EURV) for a 72-hour drain time is 0.15 acre-feet. The storage volume required for detention is 0.38 acre-feet, which includes 0.35 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}=13.3 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 5.4 cfs for the 100-year storm event. A proposed 18-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 22-foot wide emergency spillway will convey the runoff to the roadside swale.

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<u>Stormwater Quality</u>
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, the full spectrum Detention Basins A, A1 and B will also be used for stormwater quality treatment. A presedimentation forebay will be installed at each storm sewer outlet into the detention basins. Each 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 $15,720 per impervious acre. The current bridge fee associated with the Dirty Woman Creek Drainage Basin is $860 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 three extended detention basins. 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.

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Referenced Information
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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

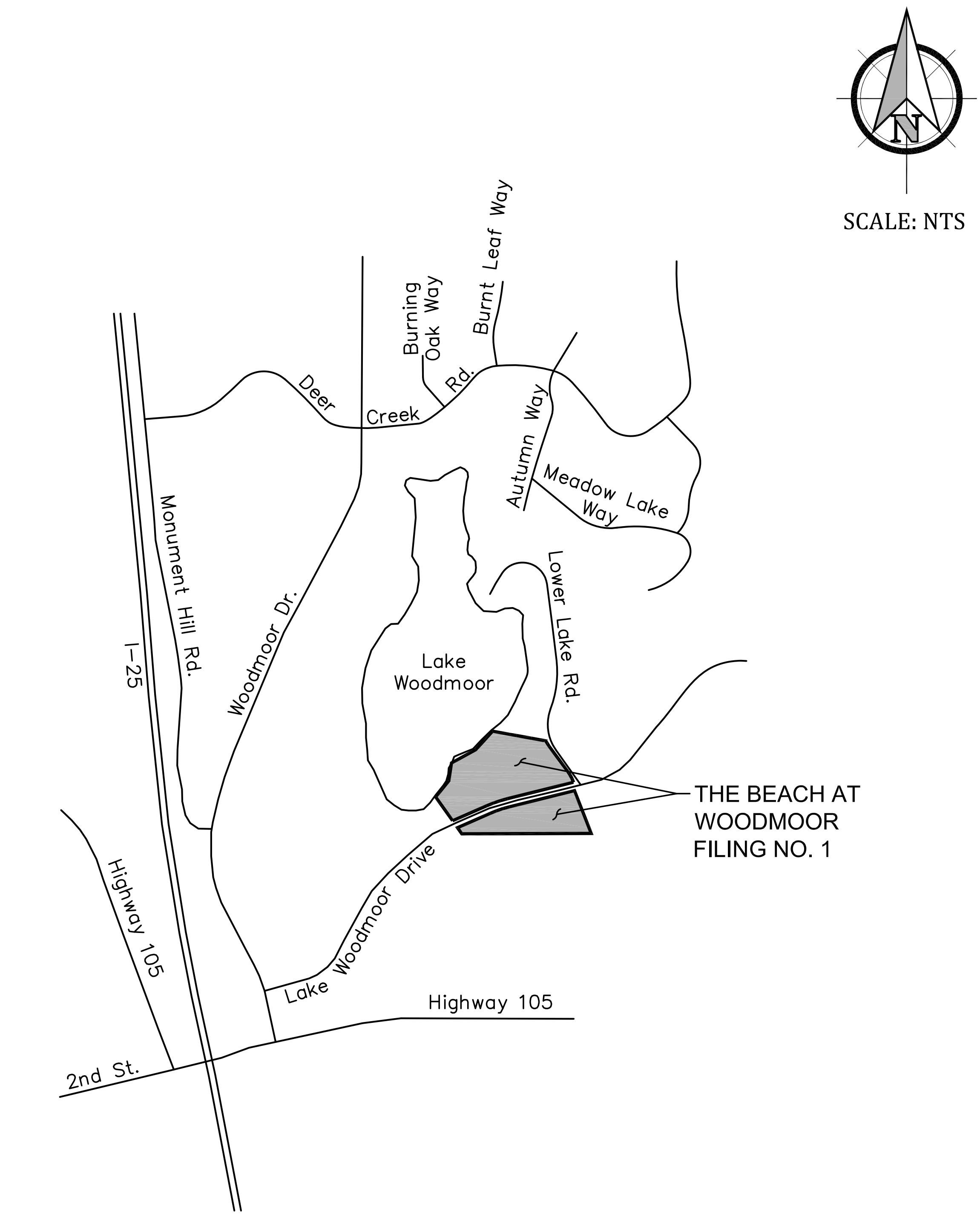


FIGURE 1

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

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

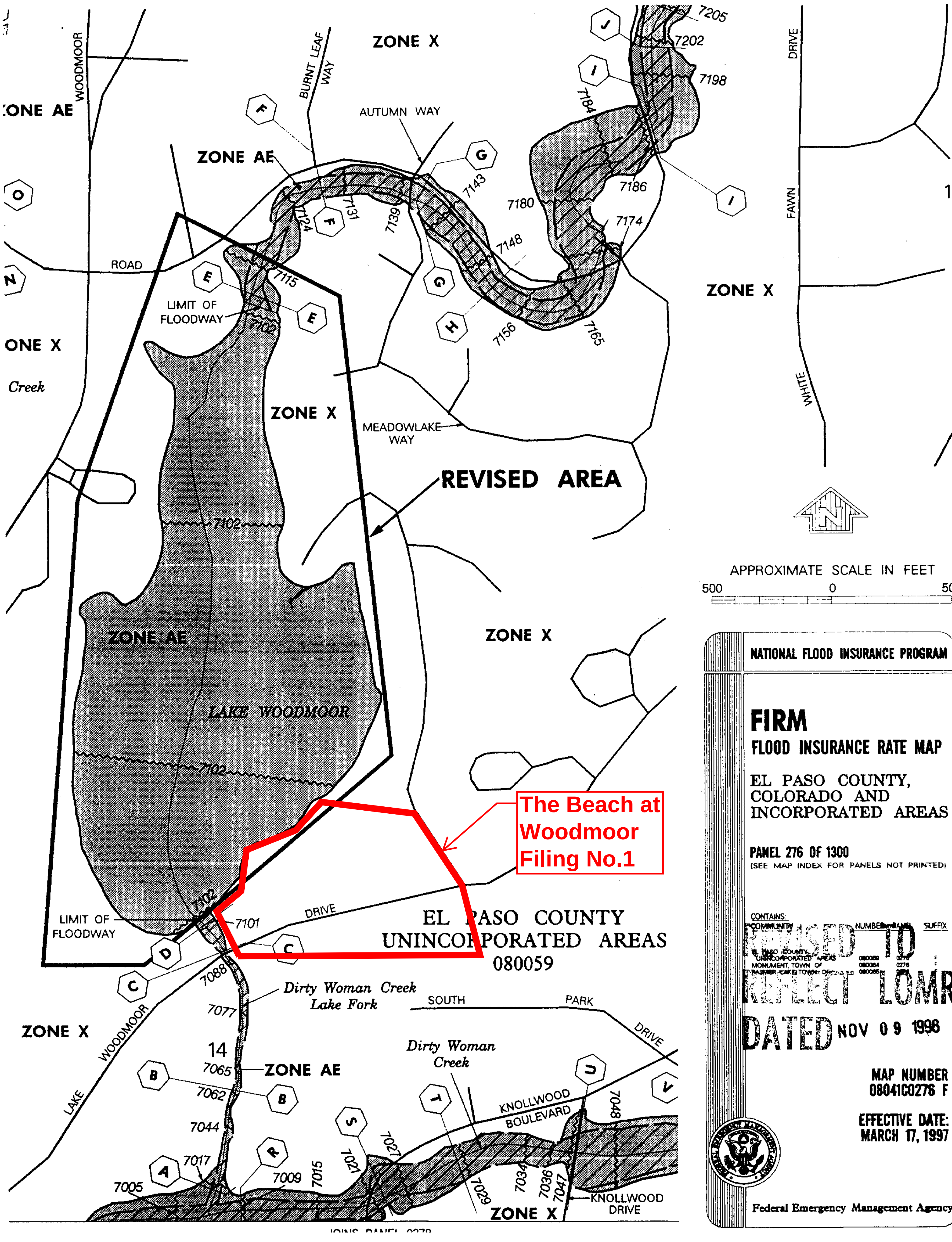
Pleasant

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
BURNT LEAF WAY
AUTUMN WAY
WOODMOOR
ROAD
LIMIT OF FLOODWAY
Creek
MEADOWLAKE WAY
REVISED AREA
LAKE WOODMOOR
ZONE X
The Beach at Woodmoor Filing No.1
DRIVE
EL PASO COUNTY
UNINCORPORATED AREAS
080059
Dirty Woman Creek
Lake Fork
SOUTH
PARK
DRIVE
LAKE WOODMOOR
Dirty Woman Creek
KNOLLWOOD BOULEVARD
KNOLLWOOD DRIVE
FAWN
WHITE
DRIVE
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)
REVISED TO REFLECT LOMR DATED NOV 09 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 =	$15,720 / ac	**Drainage Basin Fee =**	**$ 69,444.13**
Bridge Fee =	$860 / ac	**Bridge Fee =**	**$ 3,799.11**

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)	713	LF	$ 69.00	$ 49,197.00
24" Reinforced Concrete Pipe (RCP)	207	LF	$ 84.00	$ 17,388.00
30" Reinforced Concrete Pipe (RCP)	400	LF	$ 94.00	$ 37,600.00
19"x30" Horizontal Elliptical Reinforced Concrete Pipe (HERCP)	34	LF	$ 94.00	$ 3,196.00
Flared End Section (FES) RCP 18"	12	EA	$ 800.00	$ 9,600.00
Flared End Section (FES) HERCP 19"x30"	1	EA	$ 1,000.00	$ 1,000.00
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	1	EA	$ 3,270.00	$ 3,270.00
5' Dia. Storm Sewer Manhole, Slab Base, Depth < 15 feet	8	EA	$ 4,575.00	$ 36,600.00
Soil Riprap, d50 9" and 12"	96	CY	$ 98.00	$ 9,408.00
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)	7	EA	$ 300.00	$ 2,100.00
Concrete Collar	4	EA	$ 500.00	$ 2,000.00
Detention Outlet Structure	3	EA	$ 6,000.00	$ 18,000.00
Detention Emergency Spillway (incl. riprap and cutoff wall)	3	EA	$ 6,200.00	$ 18,600.00
Presedimentation Forebay	2	EA	$ 3,000.00	$ 6,000.00
Gravel Maintenance Access Trail	268	SY	$ 20.00	$ 5,360.00
Type II Bedding	15	CY	$ 35.00	$ 525.00
Detention Basin Seeding and Mulch	1.1	AC	$ 520.00	$ 572.00
			Estimated Storm Drainage Facilities Cost	**$ 247,917.21**
			Engineering 10%	$ 24,791.72
			Contingency 5%	$ 12,395.86
			Total Estimated Cost	**$ 285,104.79**

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	Area 1 Land Use			HI	Area 2 Land Use			GR	Area 3 Land Use			RO	Area 4 Land Use			CO	Area 5 Land Use			Basin % Imperv	Basin Runoff	
				% 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}
EX-1/EX1	271,474 sf	6.23ac	B	100%	0.39ac	6%	6%	2%	5.79ac	93%	2%	40%	0.05ac	1%	0%	90%		0%	0%	95%		0%	0%	**8.4%**	**0.13**	**0.40**
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	109,958 sf	2.52ac	B	100%	0.00ac	0%	0%	2%	2.52ac	100%	2%	40%		0%	0%	90%		0%	0%	95%		0%	0%	**2.0%**	**0.08**	**0.36**
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
Basin / Design Point	Contributing Basins	Area	C_5	Initial/Overland Time (t_i)			Travel Time (t_t)						Comp.		
				Length	Slope	t_i	Length	Slope	Land Type	Cv	Velocity	t_t	t_c		
EX-1/EX1		6.23ac	0.13	300lf	5.0%	18.1 min.	500lf	4.0%	GW	15	3.0 ft/sec	2.8 min.	20.9 min.		**20.9 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		2.52ac	0.08	120lf	8.8%	9.9 min.	630lf	2.5%	GW	15	2.4 ft/sec	4.4 min.	14.3 min.		**14.3 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		6.23 ac	0.13	0.40	20.9 min.	3.0 in/hr	5.1 in/hr	**2.4 cfs**	**12.5 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		2.52 ac	0.08	0.36	14.3 min.	3.6 in/hr	6.0 in/hr	**0.7 cfs**	**5.5 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	
				% 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 / DP 1	84,625 sf	1.94ac	B	100%	0.46ac	24%	24%	0%	0.39ac	20%	0%	42%	1.06ac	54%	23%	39%		0%	0%	40%	0.03ac	2%	1%	**47.1%**	**0.33**	**0.52**
A2	63,951 sf	1.47ac	B	100%	0.00ac	0%	0%	0%	1.37ac	93%	0%	42%		0%	0%	39%		0%	0%	40%	0.10ac	7%	3%	**2.8%**	**0.08**	**0.37**
A3 / DP 2	99,181 sf	2.28ac	B	100%	0.00ac	0%	0%	0%		0%	0%	42%	2.28ac	100%	42%	39%		0%	0%	40%		0%	0%	**41.9%**	**0.31**	**0.50**
A4 / DP 3	59,296 sf	1.36ac	B	100%	0.00ac	0%	0%	0%		0%	0%	42%	1.36ac	100%	42%	39%		0%	0%	40%		0%	0%	**41.9%**	**0.31**	**0.50**
A5 / DP 5	7,848 sf	0.18ac	B	100%	0.00ac	0%	0%	0%	0.18ac	100%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**0.0%**	**0.08**	**0.35**
A5.1	51,224 sf	1.18ac	B	100%	0.00ac	0%	0%	0%	0.60ac	51%	0%	42%	0.58ac	49%	21%	39%		0%	0%	40%		0%	0%	**20.6%**	**0.20**	**0.44**
A6 / DP 7	4,254 sf	0.10ac	B	100%	0.03ac	35%	35%	0%	0.06ac	65%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**34.5%**	**0.27**	**0.48**
A7	16,604 sf	0.38ac	B	100%	0.12ac	33%	33%	0%	0.26ac	67%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**32.7%**	**0.26**	**0.48**
B1	20,218 sf	0.46ac	B	100%	0.17ac	37%	37%	0%	0.29ac	63%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**37.3%**	**0.28**	**0.49**
B2 / DP 10	8,151 sf	0.19ac	B	100%	0.07ac	39%	39%	0%	0.11ac	61%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**38.9%**	**0.29**	**0.49**
B3	35,499 sf	0.81ac	B	100%	0.00ac	0%	0%	0%	0.50ac	62%	0%	42%		0%	0%	39%	0.31ac	38%	15%	40%		0%	0%	**14.8%**	**0.17**	**0.42**
B4 / DP 12	69,555 sf	1.60ac	B	100%	0.19ac	12%	12%	0%	0.64ac	40%	0%	42%		0%	0%	39%	0.77ac	48%	19%	40%		0%	0%	**30.4%**	**0.25**	**0.47**
B5	90,078 sf	2.07ac	B	100%	0.00ac	0%	0%	0%		0%	0%	42%		0%	0%	39%	2.07ac	100%	39%	40%		0%	0%	**38.8%**	**0.29**	**0.49**
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%	42%		0%	0%	39%		0%	0%	40%	0.03ac	8%	3%	**3.1%**	**0.09**	**0.37**
DP 4	A3, A4	3.64ac	B	100%	0.00ac	0%	0%	0%		0%	0%	42%	3.64ac	100%	42%	39%		0%	0%	40%		0%	0%	**41.9%**	**0.31**	**0.50**
DP 6	A3, A4, A5	3.82ac	B	100%	0.00ac	0%	0%	0%	0.18ac	5%	0%	42%	3.64ac	95%	40%	39%		0%	0%	40%		0%	0%	**39.9%**	**0.30**	**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	0.65ac	B	100%	0.25ac	38%	38%	0%	0.41ac	62%	0%	42%		0%	0%	39%		0%	0%	40%		0%	0%	**37.7%**	**0.29**	**0.49**
DP 13	OS6, B5	2.40ac	B	100%	0.00ac	0%	0%	0%	0.30ac	13%	0%	42%		0%	0%	39%	2.07ac	86%	34%	40%	0.03ac	1%	0%	**33.9%**	**0.27**	**0.48**
DP 14	OS6, B4, B5	3.99ac	B	100%	0.19ac	5%	5%	0%	0.95ac	24%	0%	42%		0%	0%	39%	2.84ac	71%	28%	40%	0.03ac	1%	0%	**32.5%**	**0.26**	**0.48**
DP 15	OS6, B3, B4, B5	4.81ac	B	100%	0.19ac	4%	4%	0%	1.45ac	30%	0%	42%		0%	0%	39%	3.15ac	65%	25%	40%	0.03ac	1%	0%	**29.5%**	**0.25**	**0.47**

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
Residential A Lots	US1	42%	0.24	0.31	0.37	0.43	0.47	0.50
Residential B Lots	US2	39%	0.23	0.29	0.35	0.42	0.46	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

	%Impervious	5	100
4	0.41	0.30	0.50
4.15	0.42	0.31	0.50
8	0.65	0.45	0.59

South A Lot Interpolation

	%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

	%Impervious	5	100
4	0.41	0.30	0.50
3.64	0.39	0.29	0.49
8	0.65	0.45	0.59

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
Basin / Design Point	Contributing Basins	Area	C_5	Initial/Overland Time (t_i)			Travel Time (t_t)						Comp.	Tc Check (urban)		
				Length	Slope	t_i	Length	Slope	Land Type	Cv	Velocity	t_t	t_c	Total Length	t_c Check	
A1 / DP 1		1.94ac	0.33	40lf	2.0%	7.0 min.	540lf	4.2%	GW	15	3.1 ft/sec	2.9 min.	10.0 min.	580lf	13.2 min.	**10.0 min.**
A2		1.47ac	0.08	75lf	10.0%	7.5 min.	360lf	9.7%	GW	15	4.7 ft/sec	1.3 min.	8.8 min.	435lf	12.4 min.	**8.8 min.**
A3 / DP 2		2.28ac	0.31	100lf	3.6%	9.5 min.	725lf	3.2%	PV	20	3.6 ft/sec	3.4 min.	12.9 min.	825lf	14.6 min.	**12.9 min.**
A4 / DP 3		1.36ac	0.31	50lf	4.0%	6.5 min.	720lf	3.2%	PV	20	3.6 ft/sec	3.4 min.	9.8 min.	770lf	14.3 min.	**9.8 min.**
A5 / DP 5		0.18ac	0.08	100lf	3.1%	12.9 min.	532lf	2.6%	GW	15	2.4 ft/sec	3.7 min.	16.5 min.	632lf	13.5 min.	**13.5 min.**
A5.1		1.18ac	0.20	50lf	25.0%	4.0 min.	400lf	2.0%	GW	15	2.1 ft/sec	3.1 min.	7.1 min.	450lf	12.5 min.	**7.1 min.**
A6 / DP 7		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		0.46ac	0.28	50lf	5.0%	6.2 min.	580lf	4.1%	GW	15	3.0 ft/sec	3.2 min.	9.4 min.	630lf	13.5 min.	**9.4 min.**
B2 / DP 10		0.19ac	0.29	45lf	5.0%	5.8 min.	260lf	1.3%	GW	15	1.7 ft/sec	2.5 min.	8.3 min.	305lf	11.7 min.	**8.3 min.**
B3		0.81ac	0.17	50lf	5.0%	7.1 min.	225lf	4.9%	GW	15	3.3 ft/sec	1.1 min.	8.2 min.	275lf	11.5 min.	**8.2 min.**
B4 / DP 12		1.60ac	0.25	100lf	4.7%	9.3 min.	395lf	0.8%	PV	20	1.8 ft/sec	3.6 min.	12.9 min.	495lf	12.8 min.	**12.8 min.**
B5		2.07ac	0.29	100lf	1.9%	12.0 min.	395lf	0.8%	PV	20	1.8 ft/sec	3.6 min.	15.5 min.	495lf	12.8 min.	**12.8 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	3.64ac	0.31	100lf	3.6%	9.5 min.	725lf	3.2%	PV	20	3.6 ft/sec	3.4 min.	12.9 min.	825lf	14.6 min.	**12.9 min.**
DP 6	A3, A4, A5	3.82ac	0.30	100lf	3.1%	10.1 min.	532lf	2.6%	GW	15	2.4 ft/sec	3.7 min.	13.8 min.	632lf	13.5 min.	**13.5 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	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.	**18.2 min.**
DP 11	B1, B2	0.65ac	0.29	45lf	5.0%	5.8 min.	910lf	3.3%	GW	15	2.7 ft/sec	5.6 min.	11.4 min.	955lf	15.3 min.	**11.4 min.**
DP 13	OS6, B5	2.40ac	0.27	100lf	1.9%	12.3 min.	630lf	2.7%	PV	20	3.3 ft/sec	3.2 min.	15.5 min.	730lf	14.1 min.	**14.1 min.**
DP 14	OS6, B4, B5	3.99ac	0.26	100lf	1.9%	12.4 min.	630lf	2.7%	PV	20	3.3 ft/sec	3.2 min.	15.6 min.	730lf	14.1 min.	**15.0 min.**
DP 15	OS6, B3, B4, B5	4.81ac	0.25	100lf	1.9%	12.6 min.	630lf	2.7%	PV	20	3.3 ft/sec	3.2 min.	15.8 min.	730lf	14.1 min.	**15.0 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)

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 / DP 1		1.94 ac	0.33	0.52	10.0 min.	4.1 in/hr	6.9 in/hr	**2.7 cfs**	**6.9 cfs**	A1 / DP 1
A2		1.47 ac	0.08	0.37	8.8 min.	4.3 in/hr	7.3 in/hr	**0.5 cfs**	**3.9 cfs**	A2
A3 / DP 2		2.28 ac	0.31	0.50	12.9 min.	3.8 in/hr	6.3 in/hr	**2.6 cfs**	**7.2 cfs**	A3 / DP 2
A4 / DP 3		1.36 ac	0.31	0.50	9.8 min.	4.2 in/hr	7.0 in/hr	**1.7 cfs**	**4.7 cfs**	A4 / DP 3
A5 / DP 5		0.18 ac	0.08	0.35	13.5 min.	3.7 in/hr	6.2 in/hr	**0.05 cfs**	**0.4 cfs**	A5 / DP 5
A5.1		1.18 ac	0.20	0.44	7.1 min.	4.6 in/hr	7.8 in/hr	**1.1 cfs**	**4.1 cfs**	A5.1
A6 / DP 7		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 / DP 7
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		0.46 ac	0.28	0.49	9.4 min.	4.2 in/hr	7.1 in/hr	**0.6 cfs**	**1.6 cfs**	B1
B2 / DP 10		0.19 ac	0.29	0.49	8.3 min.	4.4 in/hr	7.4 in/hr	**0.2 cfs**	**0.7 cfs**	B2 / DP 10
B3		0.81 ac	0.17	0.42	8.2 min.	4.4 in/hr	7.4 in/hr	**0.6 cfs**	**2.6 cfs**	B3
B4 / DP 12		1.60 ac	0.25	0.47	12.8 min.	3.8 in/hr	6.3 in/hr	**1.5 cfs**	**4.7 cfs**	B4 / DP 12
B5		2.07 ac	0.29	0.49	12.8 min.	3.8 in/hr	6.3 in/hr	**2.3 cfs**	**6.4 cfs**	B5
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	3.64 ac	0.31	0.50	12.9 min.	3.8 in/hr	6.3 in/hr	**4.2 cfs**	**11.5 cfs**	DP 4
DP 6	A3, A4, A5	3.82 ac	0.30	0.49	13.5 min.	3.7 in/hr	6.2 in/hr	**4.2 cfs**	**11.7 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	18.2 min.	3.2 in/hr	5.4 in/hr	**24.3 cfs**	**52.9 cfs**	DP 9
DP 11	B1, B2	0.65 ac	0.29	0.49	11.4 min.	3.9 in/hr	6.6 in/hr	**0.7 cfs**	**2.1 cfs**	DP 11
DP 13	OS6, B5	2.40 ac	0.27	0.48	14.1 min.	3.6 in/hr	6.1 in/hr	**2.3 cfs**	**7.0 cfs**	DP 13
DP 14	OS6, B4, B5	3.99 ac	0.26	0.48	15.0 min.	3.5 in/hr	5.9 in/hr	**3.7 cfs**	**11.2 cfs**	DP 14
DP 15	OS6, B3, B4, B5	4.81 ac	0.25	0.47	15.0 min.	3.5 in/hr	5.9 in/hr	**4.2 cfs**	**13.3 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

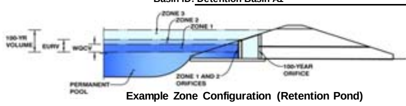
Detention Basins A1, A2 and B Calculations

Full Spectrum Detention Basins/Extended Detention Basins
Detention Volume Calculations
Emergency Spillway Calculations
Outlet Structure Calculations
Trash Rack and Safety Grate Sizing
Forebay Sizing and Trickle Channel Calculations

DETENTION BASIN STAGE-STORAGE TABLE BUILDER

Project: The Beach at Woodmoor Filing No. 1

Basin ID: Detention Basin A1



Example Zone Configuration (Retention Pond)

Required Volume Calculation

			Optional User Input 1-hr Precipitation	
Selected BMP Type =	**EDB**			
Watershed Area =	3.82	acres		
Watershed Length =	760	ft		
Watershed Slope =	0.032	ft/ft		
Watershed Imperviousness =	39.9%	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) =	0.057	acre-feet		
Excess Urban Runoff Volume (EURV) =	0.160	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.130	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.216	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.291	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.404	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.495	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.598	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.2 in.) =	0.823	acre-feet	3.20	inches
Approximate 2-yr Detention Volume =	0.123	acre-feet		
Approximate 5-yr Detention Volume =	0.193	acre-feet		
Approximate 10-yr Detention Volume =	0.212	acre-feet		
Approximate 25-yr Detention Volume =	0.225	acre-feet		
Approximate 50-yr Detention Volume =	0.260	acre-feet		
Approximate 100-yr Detention Volume =	0.325	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.057	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.103	acre-feet
Zone 3 Volume (User Defined - Zones 1 & 2) =	0.194	acre-feet
Total Detention Basin Volume =	0.354	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)
Micropool	--	0.00	--	--	--	24	0.001		
	--	0.27	--	--	--	24	0.001	6	0.000
	--	0.77	--	--	--	230	0.005	68	0.002
	--	1.27	--	--	--	1,420	0.033	468	0.011
	--	2.27	--	--	--	2,450	0.056	2,417	0.055
	--	3.27	--	--	--	3,230	0.074	5,257	0.121
	--	4.27	--	--	--	4,100	0.094	8,922	0.205
	--	5.27	--	--	--	5,100	0.117	13,522	0.310
	--	6.27	--	--	--	6,180	0.142	19,162	0.440
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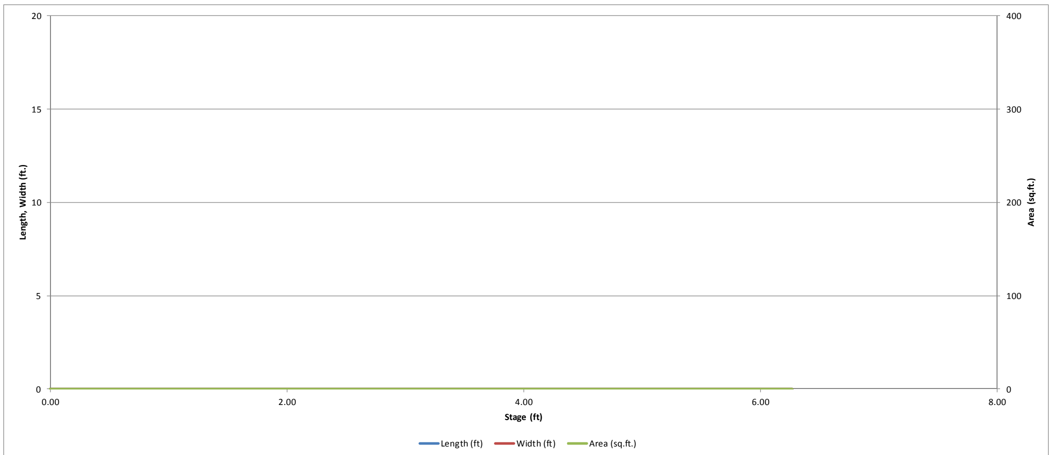
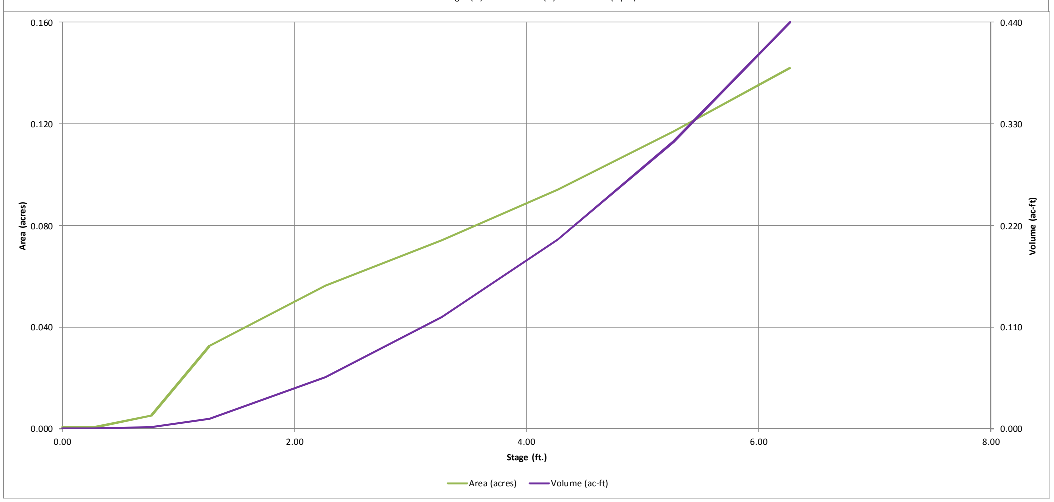
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

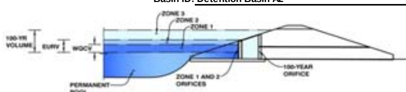




DETENTION BASIN STAGE-STORAGE TABLE BUILDER

Project: The Beach at Woodmoor Filing No. 1

Basin ID: Detention Basin A2



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	1.18	acres		
Watershed Length =	490	ft		
Watershed Slope =	0.019	ft/ft		
Watershed Imperviousness =	20.6%	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) =	0.012	acre-feet	Optional User Input 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.024	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.018	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.040	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.062	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.100	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.128	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.160	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.2 in.) =	0.228	acre-feet	3.20	inches
Approximate 2-yr Detention Volume =	0.017	acre-feet		
Approximate 5-yr Detention Volume =	0.034	acre-feet		
Approximate 10-yr Detention Volume =	0.039	acre-feet		
Approximate 25-yr Detention Volume =	0.042	acre-feet		
Approximate 50-yr Detention Volume =	0.053	acre-feet		
Approximate 100-yr Detention Volume =	0.072	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.012	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.013	acre-feet
Zone 3 Volume (User Defined - Zones 1 & 2) =	0.053	acre-feet
Total Detention Basin Volume =	0.077	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)
Micropool	--	0.00	--	--	--	16	0.000		
	--	0.17	--	--	--	16	0.000	3	0.000
	--	1.17	--	--	--	820	0.019	413	0.009
	--	2.17	--	--	--	2,860	0.066	2,261	0.052
	--	3.17	--	--	--	4,900	0.112	6,141	0.141
	--	4.17	--	--	--	7,000	0.161	12,091	0.278
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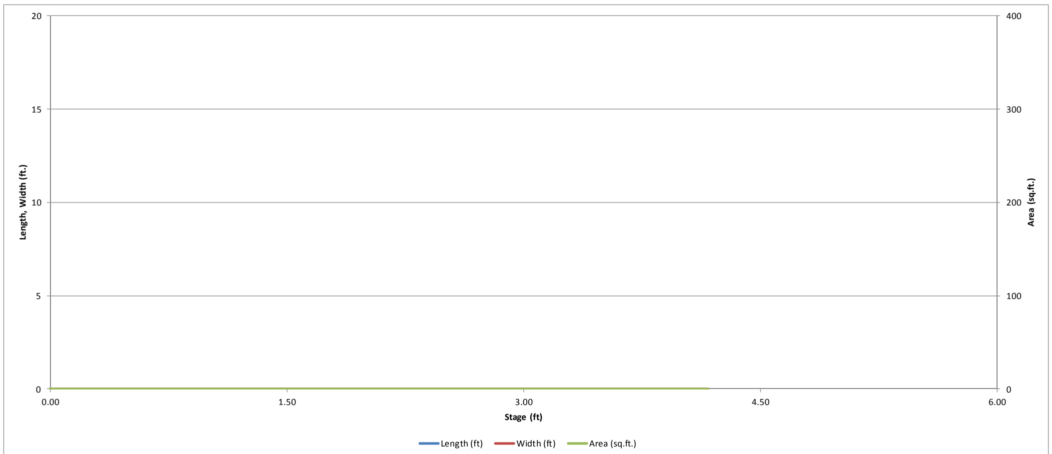
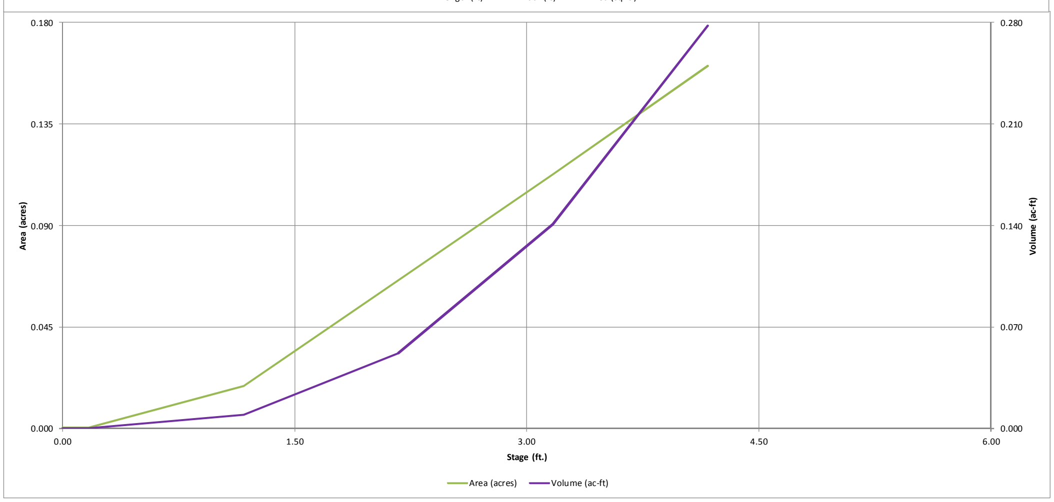
DETENTION BASIN STAGE-STORAGE TABLE BUILDER

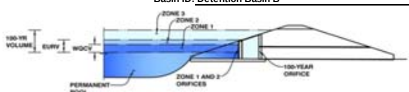




DETENTION BASIN STAGE-STORAGE TABLE BUILDER

Project: The Beach at Woodmoor Filing No. 1

Basin ID: Detention Basin B



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Selected BMP Type =	**EDB**			
Watershed Area =	4.81	acres		
Watershed Length =	800	ft		
Watershed Slope =	0.016	ft/ft		
Watershed Imperviousness =	29.5%	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) =	0.060	acre-feet	Optional User Input 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	0.145	acre-feet		
2-yr Runoff Volume (P1 = 1.19 in.) =	0.115	acre-feet	1.19	inches
5-yr Runoff Volume (P1 = 1.5 in.) =	0.214	acre-feet	1.50	inches
10-yr Runoff Volume (P1 = 1.75 in.) =	0.306	acre-feet	1.75	inches
25-yr Runoff Volume (P1 = 2 in.) =	0.454	acre-feet	2.00	inches
50-yr Runoff Volume (P1 = 2.25 in.) =	0.568	acre-feet	2.25	inches
100-yr Runoff Volume (P1 = 2.52 in.) =	0.699	acre-feet	2.52	inches
500-yr Runoff Volume (P1 = 3.2 in.) =	0.979	acre-feet	3.20	inches
Approximate 2-yr Detention Volume =	0.108	acre-feet		
Approximate 5-yr Detention Volume =	0.186	acre-feet		
Approximate 10-yr Detention Volume =	0.207	acre-feet		
Approximate 25-yr Detention Volume =	0.223	acre-feet		
Approximate 50-yr Detention Volume =	0.267	acre-feet		
Approximate 100-yr Detention Volume =	0.349	acre-feet		

Stage-Storage Calculation

Zone 1 Volume (WQCV) =	0.060	acre-feet
Zone 2 Volume (EURV - Zone 1) =	0.085	acre-feet
Zone 3 Volume (User Defined - Zones 1 & 2) =	0.234	acre-feet
Total Detention Basin Volume =	0.379	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)
Micropool	--	0.00	--	--	--	24	0.001		
	--	0.27	--	--	--	24	0.001	6	0.000
	--	0.77	--	--	--	250	0.006	72	0.002
	--	0.97	--	--	--	1,200	0.028	208	0.005
	--	1.27	--	--	--	2,360	0.054	731	0.017
	--	2.27	--	--	--	3,240	0.074	3,554	0.082
	--	3.27	--	--	--	4,100	0.094	7,224	0.166
	--	4.27	--	--	--	5,130	0.118	11,839	0.272
	--	5.27	--	--	--	6,500	0.149	17,654	0.405
	--	6.27	--	--	--	7,750	0.178	24,779	0.569
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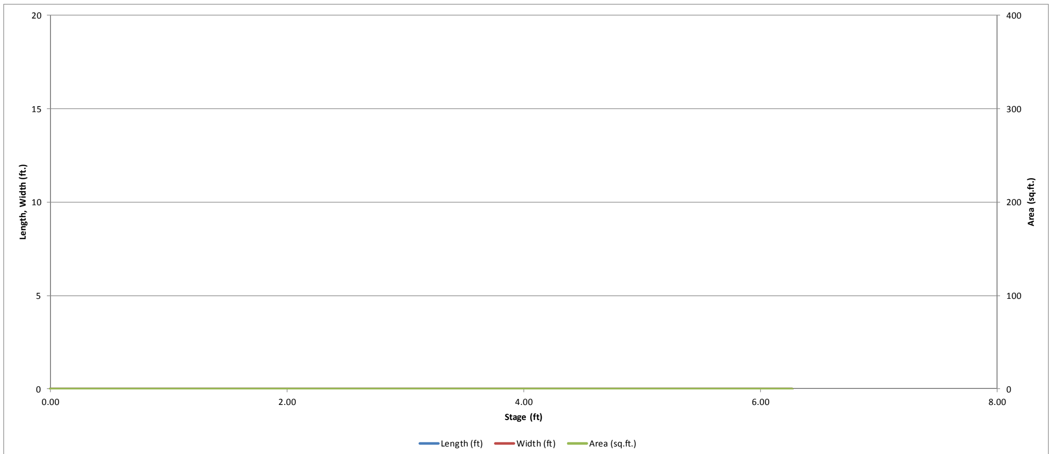
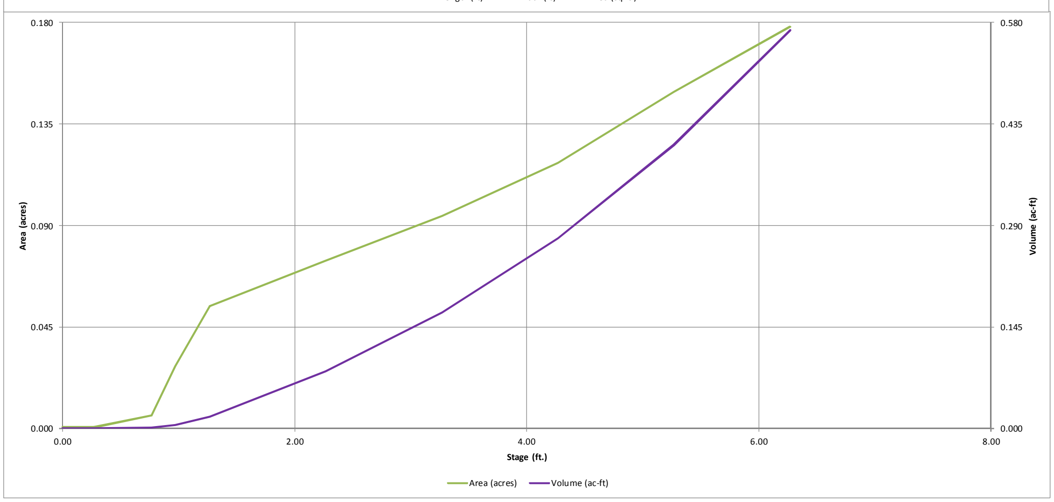
DETENTION BASIN STAGE-STORAGE TABLE BUILDER





UDFCD Detention Sizing

Detention Area	Total Acres	% Imperv.	Soil Group	100yr P_1	WQCV a	WQCV Z	WQCV Depth	WQCV Factor	WQCV Volume	EURV Depth	EURV Volume	K_{100}	V_{100}	Required Detention Volume V100+1/2WQCV
Detention Area A1	3.82 ac	39.9%	B	2.52in	1.0	1.0	0.18in	0.015	0.057ac-ft 2,490 cf	0.50in	0.160ac-ft 6,991 cf	1.020	0.32ac-ft 14,139 cf	**0.35 ac-ft** **15,384 cf**

WIR (Watershed Inches of Runoff) taken from Fig. EDB-2, Volume 3, Urban Storm Drainage Criteria Manual for the basin imperviousness shown.

WIR = Depth = $a*(0.91*I^3 - 1.19*I^2 + 0.78*I)$

I = % Impervious   a = Drain Time

a (40hr) = 1.0	Extended Detention Basin
a (24hr) = 0.9	
a (12hr) = 0.8	

WQCV Factor (Water Quality Capture Volume) = (WIR/12) x Z

Z = Volume Factor

Z (Extended Detention Basin) = 1.0   2015 USDCM

Grate: Steel = 490 lbs/cf or 1/4" thick steel = 10.20 lbs/sf

V_{100}
V_{100}+1/2WQCV
V_{100}+WQCV
V_{100}+1/2EURV

$EURV_k$ = Depth = Excess Urban Runoff Volume in watershed inshes (K = A, B or CD)

$EURV_A = 1.68\ i^{1.28}$ (USDCM, Eqn 12-1)

$EURV_B$ = 1.36 i1.08 (USDCM, Eqn 12-2)

$EURV_{CD}$ = 1.20 i1.08 (USDCM, Eqn 12-3)

Required Detention Storage Volume (Vx) = Kx A (Equation SO-1)

$K_2=P_1((0.968i^{1.458})A\%+(0.964i^{1.183})B\%+(0.962i^{1.104})CD\%)$

$K_5=P_1((0.973i^{1.368})A\%+(0.900i^{1.098}+0.082i^{0.098})B\%+(0.795i^{1.226}+0.159i^{0.226})CD\%)$

$K_{10}=P_1((0.988i^{1.237})A\%+(0.751i^{1.254}+0.174i^{0.254})B\%+(0.630i^{1.371}+0.248i^{0.371})CD\%)$

$K_{100}=P_1((0.728i^{1.258}+0.150i^{0.258})A\%+(0.364i^{1.286}+0.381i^{0.286})B\%+(0.306i^{1.286}+0.402i^{0.286})CD\%)$

K_x=(in inches)(USDCM, Eqn 12-4 and UDFCD Runoff and Detention Storage Volumes Memo 2015-03-26)

Recommended Release Rate = 90% of Predevelopment Flow

Equation for the 100yr detention does not match the current simplified equation in UDCM Vol 2 (Equation 12-4). Typical comment for Detention A1, A2, B

Detention Basin Earthwork

Elevation	Area (A)	Avg. Area	Volume	Depth	Cumulative Volume		Elev.
7104.73	24sf	Lowest Orifice			**0cf**	**0.00ac-ft**	7104.73
7105	24sf	24sf	6cf	0.27 ft	**6cf**	**0.00ac-ft**	7105
7105.5	230sf	127sf	64cf	0.77 ft	**70cf**	**0.00ac-ft**	7105.5
7106	1,420sf	825sf	413cf	1.27 ft	**482cf**	**0.01ac-ft**	7106
7107	2,450sf	1,935sf	1,935cf	2.27 ft	**2,417cf**	**0.06ac-ft**	7107
7108	3,230sf	2,840sf	2,840cf	3.27 ft	**5,257cf**	**0.12ac-ft**	7108
7109	4,100sf	3,665sf	3,665cf	4.27 ft	**8,922cf**	**0.20ac-ft**	7109
7110	5,100sf	4,600sf	4,600cf	5.27 ft	**13,522cf**	**0.31ac-ft**	7110
7111	6,180sf	5,640sf	5,640cf	6.27 ft	**19,162cf**	**0.44ac-ft**	7111
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Average End Area Formula: V = (A1+A2)/2 x Elev Difference

				Depth
Lowest Orifice =			7104.73 ft	0.00 ft
WQCV =	2,490 cf	0.06 ac-ft	7107.03 ft	2.30 ft
EURV =	6,991 cf	0.16 ac-ft	7108.42 ft	3.69 ft
100yr Volume =	14,139 cf	0.32 ac-ft	7110.11 ft	5.38 ft
100yr Volume + 1/2 WQCV =	15,384 cf	0.35 ac-ft	7110.33 ft	5.60 ft
Detention Freeboard Depth =			0.02 ft	
Spillway Crest =	15,496 cf	0.36 ac-ft	7110.35 ft	5.62 ft
Spillway 100yr Flow Depth =	16,906 cf	0.39 ac-ft	7110.60 ft	5.87 ft
Spillway Freeboard Depth =			0.40 ft	
Top of Berm =	19,162 cf	0.44 ac-ft	7111.00 ft	6.27 ft

Since the outlet structure design tab of the UD-Detention was used, why is this excel worksheet included? This seems to be a redundant calculation.

Spillway to be 1 ft above the 100yr flow depth.

The Beach at Woodmoor Filing No. 1
Detention Area Calculations

UDFCD Detention Sizing

Detention Area	Total Acres	% Imperv.	Soil Group	100yr P_1	WQCV a	WQCV Z	WQCV Depth	WQCV Factor	WQCV Volume	EURV Depth	EURV Volume	K_{100}	V_{100}	Required Detention Volume V100+1/2WQCV
Detention Area A2	1.18 ac	20.6%	B	2.52in	1.0	1.0	0.12in	0.010	0.012ac-ft 506 cf	0.25in	0.024ac-ft 1,058 cf	0.731	0.07ac-ft 3,133 cf	**0.08 ac-ft** **3,386 cf**

WIR (Watershed Inches of Runoff) taken from Fig. EDB-2, Volume 3, Urban Storm Drainage Criteria Manual for the basin imperviousness shown.

WIR = Depth = $a*(0.91*I^3 - 1.19*I^2 + 0.78*I)$

I = % Impervious   a = Drain Time

a (40hr) = 1.0	Extended Detention Basin
a (24hr) = 0.9	
a (12hr) = 0.8	

WQCV Factor (Water Quality Capture Volume) = (WIR/12) x Z

Z = Volume Factor

Z (Extended Detention Basin) = 1.0   2015 USDCM

Grate: Steel = 490 lbs/cf or 1/4" thick steel = 10.20 lbs/sf

$EURV_k$ = Depth = Excess Urban Runoff Volume in watershed inshes (K = A, B or CD)

V100 — $EURV_A = 1.68\ i^{1.28}$ (USDCM, Eqn 12-1)

V100+1/2WQCV — $EURV_B$ = 1.36 i1.08 (USDCM, Eqn 12-2)

V100+WQCV — $EURV_{CD}$ = 1.20 i1.08 (USDCM, Eqn 12-3)

V100+1/2EURV — Required Detention Storage Volume (Vx) = Kx A (Equation SO-1)

$K_2=P_1((0.968i^{1.458})A\%+(0.964i^{1.183})B\%+(0.962i^{1.104})CD\%)$

$K_5=P_1((0.973i^{1.368})A\%+(0.900i^{1.098}+0.082i^{0.098})B\%+(0.795i^{1.226}+0.159i^{0.226})CD\%)$

$K_{10}=P_1((0.988i^{1.237})A\%+(0.751i^{1.254}+0.174i^{0.254})B\%+(0.630i^{1.371}+0.248i^{0.371})CD\%)$

$K_{100}=P_1((0.728i^{1.258}+0.150i^{0.258})A\%+(0.364i^{1.286}+0.381i^{0.286})B\%+(0.306i^{1.286}+0.402i^{0.286})CD\%)$

K_x=(in inches)(USDCM, Eqn 12-4 and UDFCD Runoff and Detention Storage Volumes Memo 2015-03-26)

Recommended Release Rate = 90% of Predevelopment Flow

Detention Basin Earthwork

Elevation	Area (A)	Avg. Area	Volume	Depth	Cumulative Volume		Elev.
7115.83	16sf	Lowest Orifice			**0cf**	**0.00ac-ft**	7115.83
7116	16sf	16sf	3cf	0.17 ft	**3cf**	**0.00ac-ft**	7116
7117	820sf	418sf	418cf	1.17 ft	**421cf**	**0.01ac-ft**	7117
7118	2,860sf	1,840sf	1,840cf	2.17 ft	**2,261cf**	**0.05ac-ft**	7118
7119	4,900sf	3,880sf	3,880cf	3.17 ft	**6,141cf**	**0.14ac-ft**	7119
7120	7,000sf	5,950sf	5,950cf	4.17 ft	**12,091cf**	**0.28ac-ft**	7120
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Average End Area Formula: V = (A1+A2)/2 x Elev Difference

			Elevation	Depth
Lowest Orifice =			7115.83 ft	0.00 ft
WQCV =	506 cf	0.01 ac-ft	7117.05 ft	1.22 ft
EURV =	1,058 cf	0.02 ac-ft	7117.35 ft	1.52 ft
100yr Volume =	3,133 cf	0.07 ac-ft	7118.22 ft	2.39 ft
100yr Volume + 1/2 WQCV =	3,386 cf	0.08 ac-ft	7118.29 ft	2.46 ft
Detention Freeboard Depth =			0.41 ft	
Spillway Crest =	4,977 cf	0.11 ac-ft	7118.70 ft	2.87 ft
Spillway 100yr Flow Depth =	6,141 cf	0.14 ac-ft	7119.00 ft	3.17 ft
Spillway Freeboard Depth =			1.00 ft	
Top of Berm =	12,091 cf	0.28 ac-ft	7120.00 ft	4.17 ft

UDFCD Detention Sizing

Detention Area	Total Acres	% Imperv.	Soil Group	100yr P_1	WQCV					EURV		K_{100}	V_{100}	**Required Detention Volume**
					a	Z	Depth	Factor	Volume	Depth	Volume			**V100+1/2WQCV**
Detention Area B	4.81 ac	29.5%	B	2.52in	1.0	1.0	0.15in	0.012	0.060ac-ft 2,617 cf	0.36in	0.146ac-ft 6,353 cf	0.868	0.35ac-ft 15,156 cf	**0.38 ac-ft** **16,464 cf**

WIR (Watershed Inches of Runoff) taken from Fig. EDB-2, Volume 3, Urban Storm Drainage Criteria Manual for the basin imperviousness shown.

WIR = Depth = a*(0.91*I^3 -1.19*I^2 +0.78*I)

I = % Impervious   a = Drain Time

a (40hr) = 1.0	Extended Detention Basin
a (24hr) = 0.9	
a (12hr) = 0.8	

WQCV Factor (Water Quality Capture Volume) = (WIR/12) x Z

Z = Volume Factor

Z (Extended Detention Basin) = 1.0   2015 USDCM

Grate: Steel = 490 lbs/cf or 1/4" thick steel = 10.20 lbs/sf

V_{100}
V_{100}+1/2WQCV
V_{100}+WQCV
V_{100}+1/2EURV

$EURV_k$ = Depth = Excess Urban Runoff Volume in watershed inshes (K = A, B or CD)

$EURV_A = 1.68\ i^{1.28}$ (USDCM, Eqn 12-1)

$EURV_B$ = 1.36 i1.08 (USDCM, Eqn 12-2)

$EURV_{CD}$ = 1.20 i1.08 (USDCM, Eqn 12-3)

Required Detention Storage Volume (Vx) = Kx A (Equation SO-1)

$K_2=P_1((0.968i^{1.458})A\%+(0.964i^{1.183})B\%+(0.962i^{1.104})CD\%)$

$K_5=P_1((0.973i^{1.368})A\%+(0.900i^{1.098}+0.082i^{0.098})B\%+(0.795i^{1.226}+0.159i^{0.226})CD\%)$

$K_{10}=P_1((0.988i^{1.237})A\%+(0.751i^{1.254}+0.174i^{0.254})B\%+(0.630i^{1.371}+0.248i^{0.371})CD\%)$

$K_{100}=P_1((0.728i^{1.258}+0.150i^{0.258})A\%+(0.364i^{1.286}+0.381i^{0.286})B\%+(0.306i^{1.286}+0.402i^{0.286})CD\%)$

K_x=(in inches)(USDCM, Eqn 12-4 and UDFCD Runoff and Detention Storage Volumes Memo 2015-03-26)

Recommended Release Rate = 90% of Predevelopment Flow

Detention Basin Earthwork

Elevation	Area (A)	Avg. Area	Volume	Depth	**Cumulative Volume**		**Elev.**
7097.73	24sf	Lowest Orifice			**0cf**	**0.00ac-ft**	7097.73
7098	24sf	24sf	6cf	0.27 ft	**6cf**	**0.00ac-ft**	7098
7098.5	250sf	137sf	69cf	0.77 ft	**75cf**	**0.00ac-ft**	7098.5
7098.7	1,200sf	725sf	145cf	0.97 ft	**220cf**	**0.01ac-ft**	7098.7
7099	2,360sf	1,780sf	534cf	1.27 ft	**754cf**	**0.02ac-ft**	7099
7100	3,240sf	2,800sf	2,800cf	2.27 ft	**3,554cf**	**0.08ac-ft**	7100
7101	4,100sf	3,670sf	3,670cf	3.27 ft	**7,224cf**	**0.17ac-ft**	7101
7102	5,130sf	4,615sf	4,615cf	4.27 ft	**11,839cf**	**0.27ac-ft**	7102
7103	6,500sf	5,815sf	5,815cf	5.27 ft	**17,654cf**	**0.41ac-ft**	7103
7104	7,750sf	7,125sf	7,125cf	6.27 ft	**24,779cf**	**0.57ac-ft**	7104
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Average End Area Formula: V = (A1+A2)/2 x Elev Difference

				Depth
Lowest Orifice =			7097.73 ft	0.00 ft
WQCV =	2,617 cf	0.06 ac-ft	7099.67 ft	1.94 ft
EURV =	6,353 cf	0.15 ac-ft	7100.76 ft	3.03 ft
100yr Volume =	15,156 cf	0.35 ac-ft	7102.51 ft	4.78 ft
100yr Volume + 1/2 WQCV =	16,464 cf	0.38 ac-ft	7102.71 ft	4.98 ft
Detention Freeboard Depth =			0.09 ft	
Spillway Crest =	17,015 cf	0.39 ac-ft	7102.80 ft	5.07 ft
Spillway 100yr Flow Depth =	19,079 cf	0.44 ac-ft	7103.20 ft	5.47 ft
Spillway Freeboard Depth =			0.80 ft	
Top of Berm =	24,779 cf	0.57 ac-ft	7104.00 ft	6.27 ft

Spillway to be 1 ft above the 100yr flow depth.

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
A1	11.6 cfs	13.9 cfs	7,110.6	7,110.35	38 ft	3:1	3.0	0.25 ft	14 cfs	OK
A2	4.1 cfs	4.9 cfs	7,119.0	7,118.7	15 ft	3:1	3.0	0.30 ft	8 cfs	OK
B	13.3 cfs	16.0 cfs	7,103.2	7,102.8	22 ft	3:1	3.0	0.40 ft	17 cfs	OK

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

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

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

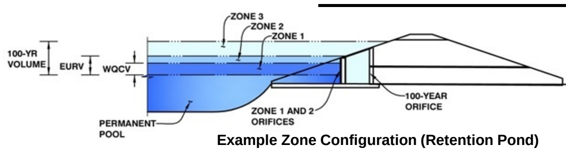
L = Length of weir at Crest, in ft. Not including sideslopes.

H = Head above weir crest, in ft

Z = Side slope (horizontal:vertical)

Detention Basin Outlet Structure Design

Project: The Beach at Woodmoor Filing No. 1
Basin ID: Detention Basin A1



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.30	0.057	Orifice Plate
Zone 2 (EURV)	3.77	0.103	Orifice Plate
Zone 3 (User)	5.63	0.194	Weir&Pipe (Restrict)
		0.354	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	3.77	ft (relative to bottom of basin at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	14.10	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	1.26	2.51					
Orifice Area (sq. inches)	0.31	0.31	0.65					

	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	N/A	N/A	ft (relative to bottom of basin 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 =	3.77	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	6.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	6.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area / total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	4.77	N/A	feet
Over Flow Weir Slope Length =	6.08	N/A	feet
Grate Open Area / 100-yr Orifice Area =	41.93	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	17.03	N/A	ft^2
Overflow Grate Open Area with Debris =	8.52	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 =	0.33	N/A	ft (distance below bottom of basin at Stage = 0 ft)
Outlet Pipe Diameter =	18.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	5.05		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	0.41	N/A	ft^2
Outlet Orifice Centroid =	0.25	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	1.12	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	5.64	ft (relative to bottom of basin at Stage = 0 ft)
Spillway Crest Length =	38.00	feet
Spillway End Slopes =	3.00	H:V
Freeboard above Max Water Surface =	0.40	feet

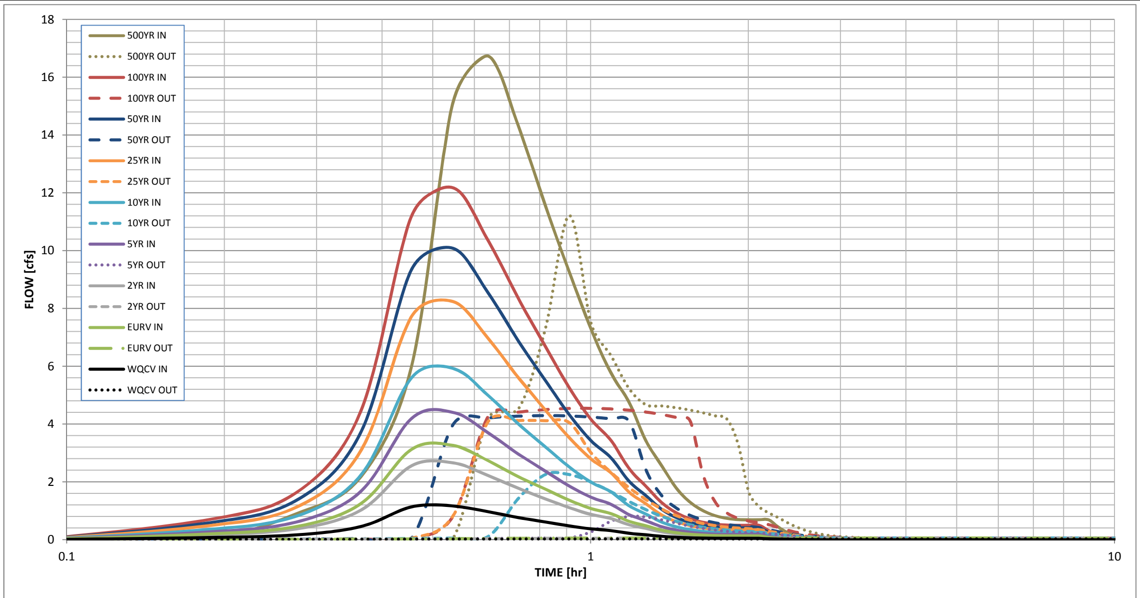
Calculated Parameters for Spillway

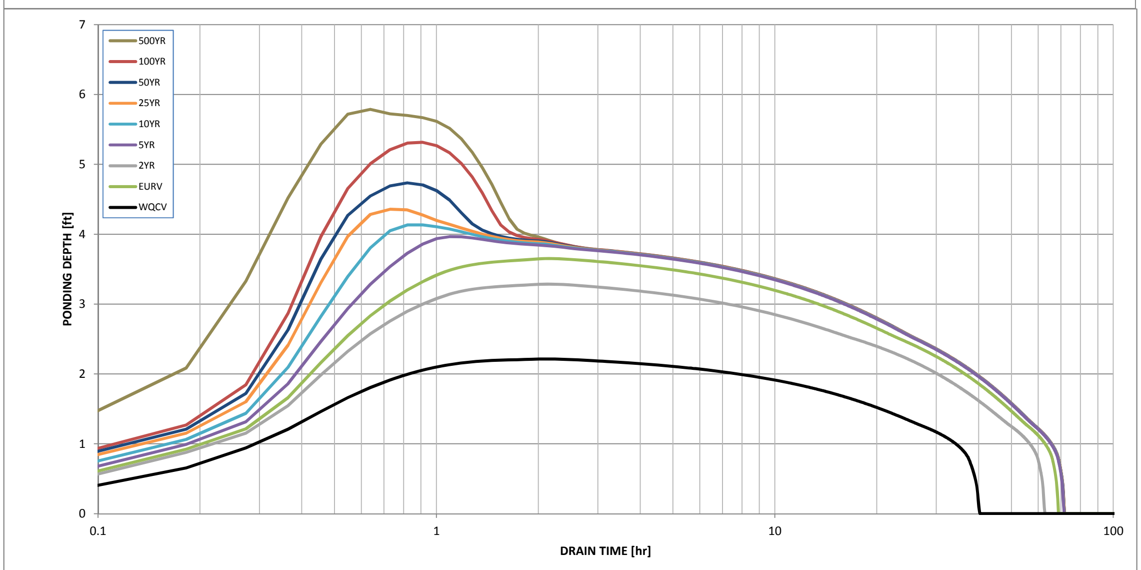
Spillway Design Flow Depth=	0.22	feet
Stage at Top of Freeboard =	6.26	feet
Basin Area at Top of Freeboard =	0.14	acres

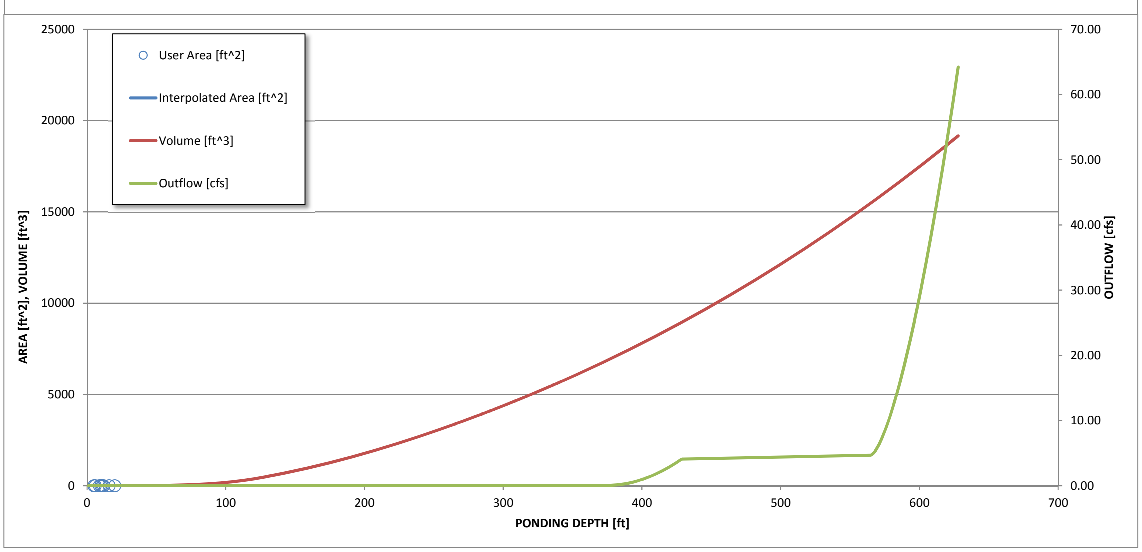
Routed Hydrograph Results

Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	1.19	1.50	1.75	2.00	2.25	2.52	3.20
Calculated Runoff Volume (acre-ft) =	0.057	0.160	0.130	0.216	0.291	0.404	0.495	0.598	0.823
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.056	0.159	0.130	0.215	0.290	0.404	0.494	0.598	0.822
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.17	0.35	0.80	1.03	1.32	1.87
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.7	1.3	3.1	4.0	5.1	7.2
Peak Inflow Q (cfs) =	1.2	3.3	2.7	4.4	5.9	8.2	10.1	12.2	16.7
Peak Outflow Q (cfs) =	0.0	0.1	0.1	0.8	2.3	4.1	4.3	4.5	11.2
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.2	1.7	1.3	1.1	0.90	1.6
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.0	0.1	0.2	0.2	0.3	0.3
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) =	40	69	63	72	72	72	72	72	72
Time to Drain 99% of Inflow Volume (hours) =	**41**	69	63	72	72	72	72	72	72
Maximum Ponding Depth (ft) =	2.21	3.65	3.29	3.97	4.13	4.36	4.74	5.31	5.79
Area at Maximum Ponding Depth (acres) =	0.05	0.08	0.07	0.09	0.09	0.10	0.10	0.12	0.13
Maximum Volume Stored (acre-ft) =	0.052	0.150	0.121	0.177	0.192	0.212	0.251	0.315	0.373

Detention Basin Outlet Structure Design







Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs

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

Time Interval	
5.47 min	
Hydrograph Constant	0.915

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:05:28	0.00	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.03
0:10:56	0.07	0.19	0.16	0.25	0.34	0.46	0.55	0.66	0.88
0:16:24	0.16	0.45	0.37	0.60	0.80	1.11	1.34	1.61	2.18
0:21:52	0.45	1.25	1.02	1.67	2.23	3.07	3.72	4.45	6.01
0:27:20	1.14	3.13	2.56	4.20	5.61	7.73	9.35	11.21	15.16
0:32:47	1.16	3.26	2.66	4.40	5.92	8.24	10.08	12.18	16.73
0:38:15	0.96	2.73	2.22	3.69	4.99	6.96	8.53	10.34	14.29
0:43:43	0.76	2.18	1.77	2.95	4.00	5.59	6.86	8.33	11.55
0:49:11	0.61	1.75	1.42	2.37	3.21	4.49	5.51	6.68	9.26
0:54:39	0.47	1.36	1.11	1.85	2.51	3.52	4.32	5.25	7.28
1:00:07	0.37	1.08	0.87	1.47	1.99	2.79	3.42	4.16	5.77
1:05:35	0.32	0.90	0.73	1.22	1.66	2.32	2.84	3.44	4.75
1:11:03	0.22	0.63	0.51	0.86	1.17	1.64	2.02	2.46	3.42
1:16:31	0.17	0.48	0.39	0.65	0.88	1.24	1.52	1.85	2.56
1:21:59	0.11	0.33	0.27	0.45	0.61	0.85	1.05	1.28	1.78
1:27:26	0.08	0.24	0.20	0.33	0.45	0.62	0.77	0.93	1.29
1:32:54	0.06	0.19	0.15	0.25	0.35	0.49	0.60	0.72	1.00
1:38:22	0.05	0.16	0.13	0.21	0.29	0.40	0.49	0.60	0.82
1:43:50	0.05	0.14	0.11	0.19	0.26	0.36	0.44	0.53	0.73
1:49:18	0.05	0.13	0.11	0.18	0.25	0.34	0.42	0.51	0.70
1:54:46	0.05	0.13	0.11	0.18	0.24	0.34	0.41	0.50	0.69
2:00:14	0.05	0.13	0.11	0.18	0.24	0.34	0.41	0.50	0.69
2:05:42	0.05	0.13	0.11	0.18	0.24	0.34	0.41	0.50	0.69
2:11:10	0.03	0.08	0.07	0.11	0.15	0.21	0.26	0.32	0.44
2:16:38	0.02	0.05	0.04	0.06	0.09	0.12	0.15	0.19	0.26
2:22:05	0.01	0.03	0.02	0.04	0.05	0.07	0.09	0.10	0.15
2:27:33	0.00	0.01	0.01	0.02	0.03	0.04	0.05	0.06	0.08
2:33:01	0.00	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.04
2:38:29	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01
2:43:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:49:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2:54:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:00:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:05:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:11:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:16:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:22:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:27:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:33:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:38:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:44:04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:49:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:55:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:00:28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:05:56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:11:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:16:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:22:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:27:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:33:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:38:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:44:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:49:39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:55:07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:00:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:06:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:11:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:16:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:22:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:27:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:33:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:38:50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:44:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:49:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:55:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:00:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:06:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:11:37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:17:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:22:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:28:01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:33:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Outlet Structure Major Storm Grate/Box Calculation

Detention Area	100-yr Flow	120% 100yr Flow	Water Surf Elev	Crest Elev	Calc'd Crest Length	d (rad)	Flow Depth	**Calc'd Flow**	**Crest Length Used***
A1	11.7 cfs		10.25	8.22	0.0 ft	2.63	2.03 ft	67.4 cfs	**4.0 ft**

Weir Equation:

$Q = CLH^{1.5} + CH^{5/2} \tan(d/2)$

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

C = 3.0

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

H = Depth of flow over the crest, in ft

d = Angle of triangle weir portion (radians)

d = 2.63 radians for 4:1 side slope

d = 2.49 radians for 3:1 side slope

Outlet Struct Capacity = Inlet Capacity Calculation at the depth to the spillway crest plus flow depth.

*Weir calculation shows that sides alone have enough capacity to convey 100-yr flow, but bars for grate will result in a longer required weir length: 32 bars x 1/4" thick = 8 in. or 0.67 ft. So, 0.0 ft (calc'd crest length) + 0.67 ft (additional length needed for bars) = 0.67 ft. Then apply a 50% debris clogging factor: 0.67 ft x 2 = 1.33 ft. However, 1.33 ft is an impractical structure width, so use 4.0 ft as a minimum crest length. Now check the maximum velocity through the grate using a 4.0 ft crest length:

Check Major Storm Grate Conditions

Maximum velocity through grate = 2.0 ft/sec

Q_{100} = 11.7 cfs

Open area of grate required = Q_{100} / V_{max} = 11.7 cfs / 2.0 ft/sec = 5.85 sf

Grate has an 70% open area (area of bars = 30% of total grate area)

Total grate area required = 5.85 sf / 0.70 = 8.4 sf

Actual outlet structure opening for grate (from weir calculation) = 4 ft x 4 ft = **16 sf** (16 sf > 8.4 sf, so 4' crest length is okay)

16 sf / 4 ft = 4.0 ft min. crest length, use **4.0 ft**

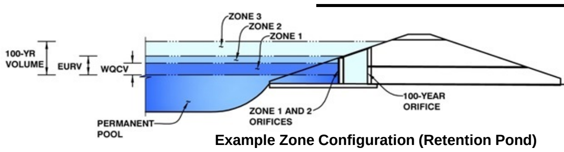
Check velocity through grate using outlet structure opening of 4.0 ft x 4.0 ft = 16 sf:

Actual velocity through grate = 11.7 cfs / (16 sf x 0.70) = **1.0 ft/sec** (1.0 ft/sec < 2.0 ft/sec, okay)

Therefore, use **4.0 ft** for crest length.

Detention Basin Outlet Structure Design

Project: The Beach at Woodmoor Filing No. 1
Basin ID: Detention Basin A2



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	1.27	0.012	Orifice Plate
Zone 2 (EURV)	1.66	0.013	Orifice Plate
Zone 3 (User)	2.52	0.053	Weir&Pipe (Restrict)
		0.077	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	1.66	ft (relative to bottom of basin 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	1.00						
Orifice Area (sq. inches)	0.11	0.11						

	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	N/A	N/A	ft (relative to bottom of basin 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 =	1.67	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	8.00	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 =	2.17	N/A	feet
Over Flow Weir Slope Length =	4.03	N/A	feet
Grate Open Area / 100-yr Orifice Area =	71.73	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	11.29	N/A	ft^2
Overflow Grate Open Area with Debris =	5.64	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 =	0.33	N/A	ft (distance below bottom of basin at Stage = 0 ft)
Outlet Pipe Diameter =	18.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	2.60		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	0.16	N/A	ft^2
Outlet Orifice Centroid =	0.13	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	0.78	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	2.87	ft (relative to bottom of basin at Stage = 0 ft)
Spillway Crest Length =	15.00	feet
Spillway End Slopes =	3.00	H:V
Freeboard above Max Water Surface =	1.00	feet

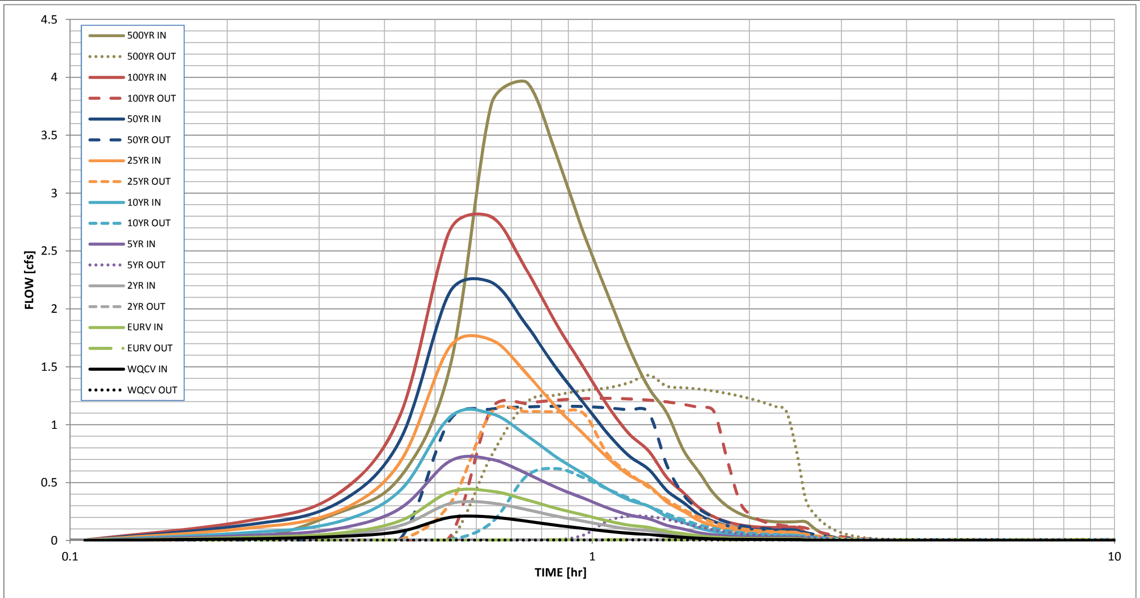
Calculated Parameters for Spillway

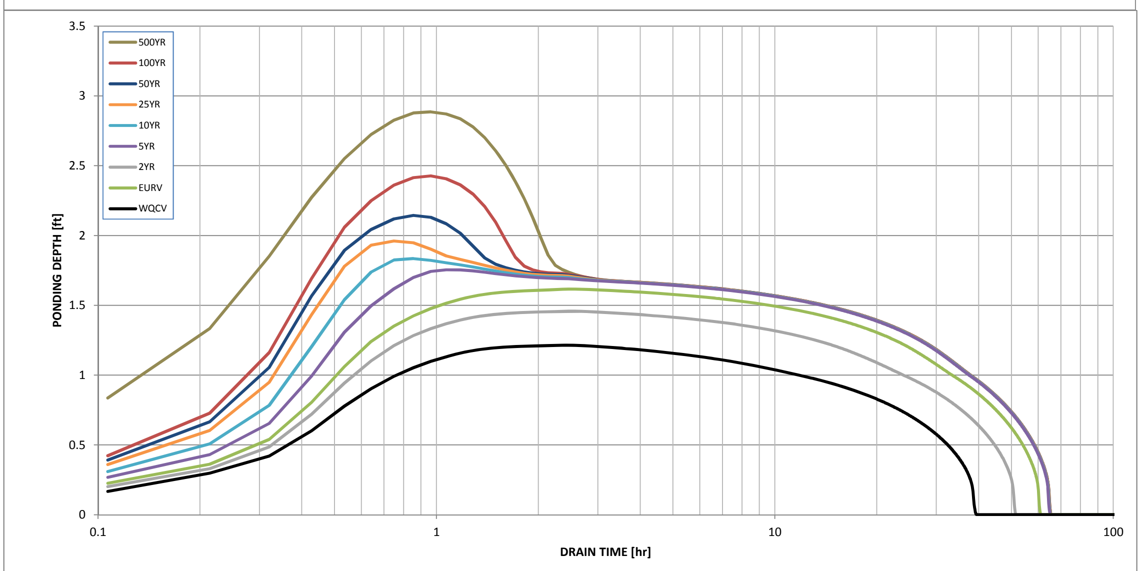
Spillway Design Flow Depth=	0.15	feet
Stage at Top of Freeboard =	4.02	feet
Basin Area at Top of Freeboard =	0.15	acres

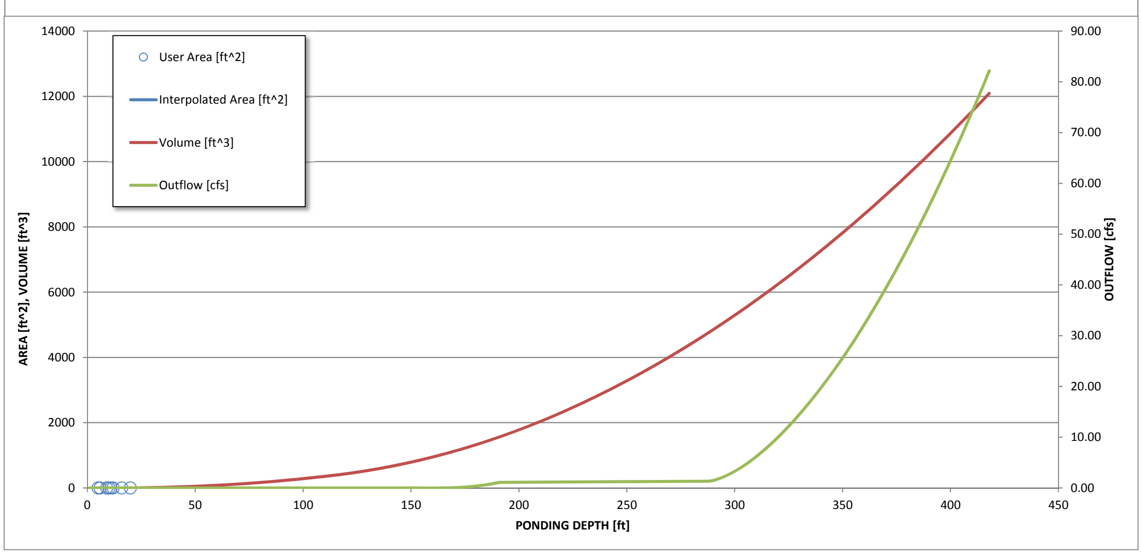
Routed Hydrograph Results

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) =	0.012	0.024	0.018	0.040	0.062	0.100	0.128	0.160	0.228
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.012	0.024	0.018	0.040	0.062	0.099	0.128	0.160	0.227
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.15	0.30	0.69	0.89	1.15	1.63
Predevelopment Peak Q (cfs) =	0.0	0.0	0.0	0.2	0.4	0.8	1.1	1.4	1.9
Peak Inflow Q (cfs) =	0.2	0.4	0.3	0.7	1.1	1.7	2.2	2.8	4.0
Peak Outflow Q (cfs) =	0.0	0.0	0.0	0.2	0.6	1.1	1.2	1.2	1.4
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.2	1.8	1.4	1.1	0.90	0.7
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.0	0.1	0.1	0.1	0.1	0.1
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	40	61	52	65	65	65	65	65	66
Time to Drain 99% of Inflow Volume (hours) =	**40**	61	52	65	65	66	66	66	66
Maximum Ponding Depth (ft) =	1.21	1.62	1.46	1.75	1.84	1.96	2.14	2.43	2.89
Area at Maximum Ponding Depth (acres) =	0.02	0.04	0.03	0.05	0.05	0.06	0.06	0.08	0.10
Maximum Volume Stored (acre-ft) =	0.010	0.022	0.017	0.028	0.032	0.039	0.050	0.070	0.110

Detention Basin Outlet Structure Design







Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs

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

Time Interval	Hydrograph Constant
6.41 min	0.780

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
0:06:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
0:12:49	0.01	0.03	0.02	0.04	0.07	0.10	0.13	0.16	0.23
0:19:14	0.03	0.06	0.05	0.10	0.15	0.24	0.31	0.38	0.54
0:25:38	0.08	0.17	0.13	0.27	0.43	0.67	0.86	1.07	1.50
0:32:03	0.20	0.42	0.32	0.69	1.07	1.68	2.15	2.68	3.77
0:38:28	0.20	0.43	0.32	0.70	1.10	1.73	2.23	2.80	3.96
0:44:52	0.17	0.35	0.27	0.58	0.91	1.44	1.86	2.34	3.32
0:51:17	0.13	0.28	0.21	0.46	0.72	1.15	1.48	1.87	2.66
0:57:42	0.10	0.22	0.17	0.37	0.58	0.92	1.19	1.50	2.13
1:04:06	0.08	0.17	0.13	0.28	0.45	0.72	0.93	1.17	1.67
1:10:31	0.06	0.13	0.10	0.22	0.35	0.57	0.73	0.92	1.32
1:16:55	0.05	0.11	0.09	0.19	0.30	0.48	0.61	0.77	1.10
1:23:20	0.04	0.08	0.06	0.13	0.21	0.33	0.43	0.54	0.78
1:29:45	0.03	0.06	0.05	0.10	0.16	0.25	0.33	0.41	0.59
1:36:09	0.02	0.04	0.03	0.07	0.11	0.17	0.22	0.28	0.40
1:42:34	0.01	0.03	0.02	0.05	0.08	0.13	0.16	0.21	0.30
1:48:58	0.01	0.02	0.02	0.04	0.06	0.10	0.13	0.16	0.23
1:55:23	0.01	0.02	0.01	0.03	0.05	0.08	0.11	0.13	0.19
2:01:48	0.01	0.02	0.01	0.03	0.05	0.07	0.10	0.12	0.17
2:08:12	0.01	0.02	0.01	0.03	0.04	0.07	0.09	0.12	0.16
2:14:37	0.01	0.02	0.01	0.03	0.04	0.07	0.09	0.11	0.16
2:21:02	0.01	0.02	0.01	0.03	0.04	0.07	0.09	0.11	0.16
2:27:26	0.01	0.02	0.01	0.03	0.04	0.07	0.09	0.11	0.16
2:33:51	0.01	0.01	0.01	0.02	0.03	0.04	0.05	0.07	0.10
2:40:15	0.00	0.01	0.00	0.01	0.02	0.02	0.03	0.04	0.06
2:46:40	0.00	0.00	0.00	0.01	0.01	0.01	0.02	0.02	0.03
2:53:05	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.02
2:59:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
3:05:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:12:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:18:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:25:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:31:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:37:57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:44:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:50:46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:57:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:03:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:10:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:16:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:22:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:29:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:35:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:42:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:48:28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:54:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:01:17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:07:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:14:06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:20:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:26:55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:33:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:39:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:46:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:52:34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:58:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:05:23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:11:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:18:12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:24:37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:31:02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:37:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:43:51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:50:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:56:40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:03:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:09:29	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:15:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:22:18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:28:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:35:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7:41:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Outlet Structure Major Storm Grate/Box Calculation

Detention Area	100-yr Flow	120% 100yr Flow	Water Surf Elev	Crest Elev	Calc'd Crest Length	d (rad)	Flow Depth	**Calc'd Flow**	**Crest Length Used***
A2	4.1 cfs		8.29	7.34	0.0 ft	2.63	0.95 ft	10.1 cfs	**4.0 ft**

Weir Equation:

$Q = CLH^{1.5} + CH^{5/2} \tan(d/2)$

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

C = 3.0

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

H = Depth of flow over the crest, in ft

d = Angle of triangle weir portion (radians)

d = 2.63 radians for 4:1 side slope

d = 2.49 radians for 3:1 side slope

Outlet Struct Capacity = Inlet Capacity Calculation at the depth to the spillway crest plus flow depth.

*Weir calculation shows that sides alone have enough capacity to convey 100-yr flow, but bars for grate will result in a longer required weir length: 32 bars x 1/4" thick = 8 in. or 0.67 ft. So, 0.0 ft (calc'd crest length) + 0.67 ft (additional length needed for bars) = 0.67 ft. Then apply a 50% debris clogging factor: 0.67 ft x 2 = 1.33 ft. However, 1.33 ft is an impractical structure width, so use 4.0 ft as a minimum crest length. Now check the maximum velocity through the grate using a 4.0 ft crest length:

Check Major Storm Grate Conditions

Maximum velocity through grate = 2.0 ft/sec

Q_{100} = 4.1 cfs

Open area of grate required = Q_{100} / V_{max} = 4.1 cfs / 2.0 ft/sec = 2.05 sf

Grate has an 70% open area (area of bars = 30% of total grate area)

Total grate area required = 2.05 sf / 0.70 = 3.0 sf

Actual outlet structure opening for grate (from weir calculation) = 4 ft x 4 ft = **16 sf** (16 sf > 3.0 sf, so 4' crest length is okay)

16 sf / 4 ft = 4.0 ft min. crest length, use **4.0 ft**

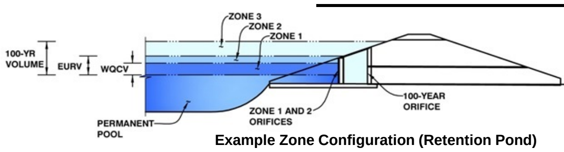
Check velocity through grate using outlet structure opening of 4.0 ft x 4.0 ft = 16 sf:

Actual velocity through grate = 4.1 cfs / (16 sf x 0.70) = **0.4 ft/sec** (0.4 ft/sec < 2.0 ft/sec, okay)

Therefore, use **4.0 ft** for crest length.

Detention Basin Outlet Structure Design

Project: The Beach at Woodmoor Filing No. 1
Basin ID: Detention Basin B



Example Zone Configuration (Retention Pond)

	Stage (ft)	Zone Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	1.97	0.060	Orifice Plate
Zone 2 (EURV)	3.05	0.085	Orifice Plate
Zone 3 (User)	5.10	0.234	Weir&Pipe (Restrict)
		0.379	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Orifice Plate =	3.05	ft (relative to bottom of basin at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing =	11.60	inches
Orifice Plate: Orifice Area per Row =	0.33	sq. inches (diameter = 5/8 inch)

Calculated Parameters for Plate

WQ Orifice Area per Row =	2.292E-03	ft^2
Elliptical Half-Width =	N/A	feet
Elliptical Slot Centroid =	N/A	feet
Elliptical Slot Area =	N/A	ft^2

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

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	1.02	2.03					
Orifice Area (sq. inches)	0.33	0.33	0.33					

	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 bottom of basin at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	N/A	N/A	ft (relative to bottom of basin 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 =	3.05	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	3.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	7.00	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 =	5.38	N/A	feet
Over Flow Weir Slope Length =	7.38	N/A	feet
Grate Open Area / 100-yr Orifice Area =	40.81	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	20.66	N/A	ft^2
Overflow Grate Open Area with Debris =	10.33	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 =	0.33	N/A	ft (distance below bottom of basin at Stage = 0 ft)
Outlet Pipe Diameter =	18.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	5.92		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	0.51	N/A	ft^2
Outlet Orifice Centroid =	0.29	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	1.22	N/A	radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	5.10	ft (relative to bottom of basin at Stage = 0 ft)
Spillway Crest Length =	22.00	feet
Spillway End Slopes =	3.00	H:V
Freeboard above Max Water Surface =	0.80	feet

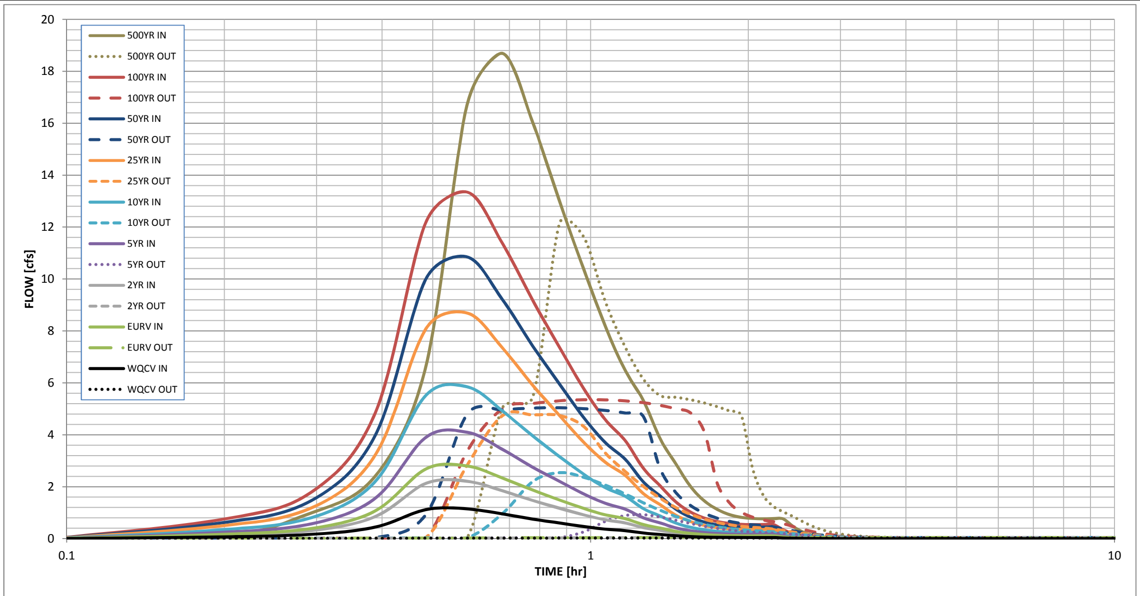
Calculated Parameters for Spillway

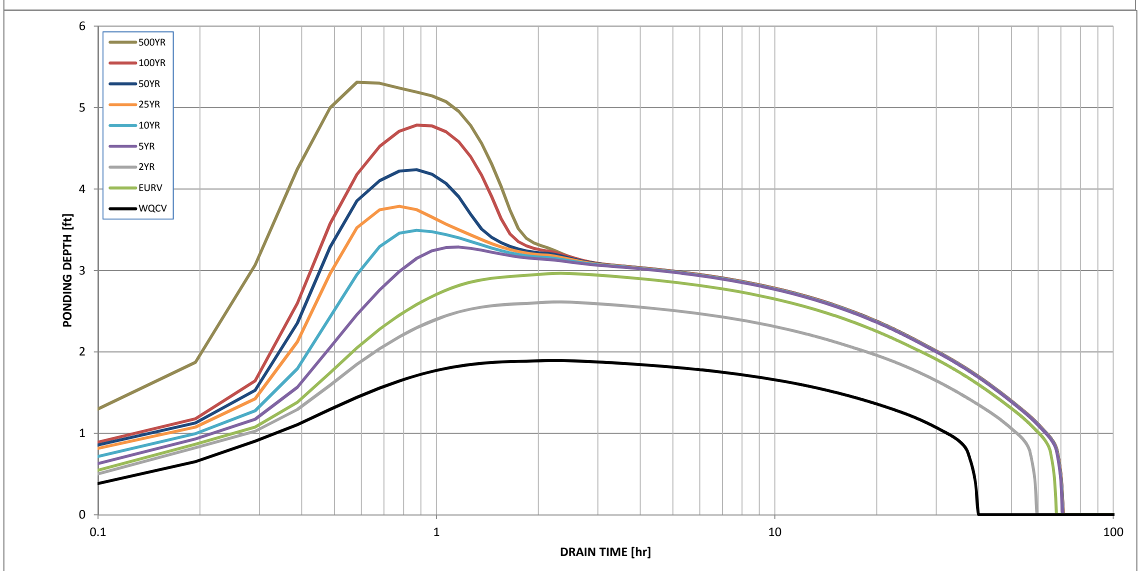
Spillway Design Flow Depth=	0.33	feet
Stage at Top of Freeboard =	6.23	feet
Basin Area at Top of Freeboard =	0.18	acres

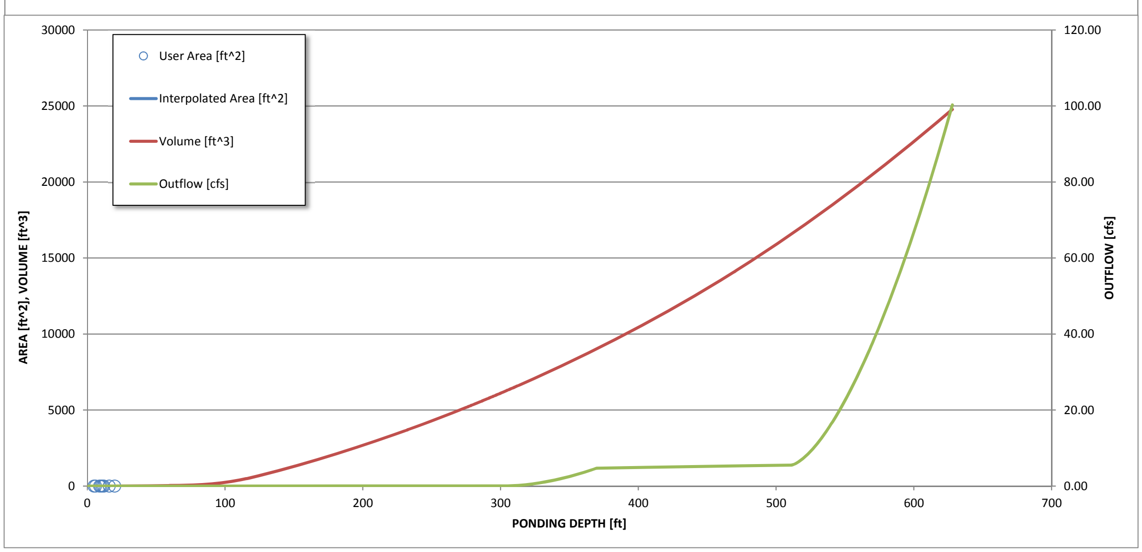
Routed Hydrograph Results

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) =	0.060	0.145	0.115	0.214	0.306	0.454	0.568	0.699	0.979
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.059	0.145	0.115	0.213	0.305	0.453	0.567	0.698	0.979
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.16	0.32	0.74	0.96	1.23	1.74
Predevelopment Peak Q (cfs) =	0.0	0.0	0.1	0.8	1.5	3.6	4.6	5.9	8.4
Peak Inflow Q (cfs) =	1.2	2.8	2.2	4.1	5.8	8.7	10.9	13.4	18.7
Peak Outflow Q (cfs) =	0.0	0.0	0.0	0.9	2.5	4.8	5.0	5.4	12.2
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	1.2	1.6	1.3	1.1	0.90	1.4
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.0	0.1	0.2	0.2	0.3	0.3
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) =	40	68	60	71	71	71	71	71	71
Time to Drain 99% of Inflow Volume (hours) =	**40**	68	60	71	71	71	71	71	71
Maximum Ponding Depth (ft) =	1.89	2.96	2.61	3.29	3.49	3.79	4.24	4.78	5.31
Area at Maximum Ponding Depth (acres) =	0.07	0.09	0.08	0.09	0.10	0.11	0.12	0.13	0.15
Maximum Volume Stored (acre-ft) =	0.055	0.138	0.108	0.167	0.187	0.217	0.267	0.336	0.411

Detention Basin Outlet Structure Design







Detention Basin Outlet Structure Design

Outflow Hydrograph Workbook Filename:

Storm Inflow Hydrographs

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

Time Interval	5.82 min
Hydrograph Constant	0.859

SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0:05:49	0.00	0.01	0.00	0.01	0.01	0.02	0.02	0.02	0.03
0:11:38	0.07	0.16	0.13	0.24	0.33	0.48	0.59	0.72	0.97
0:17:28	0.16	0.39	0.31	0.56	0.79	1.16	1.44	1.75	2.42
0:23:17	0.45	1.07	0.85	1.56	2.20	3.22	3.98	4.84	6.65
0:29:06	1.12	2.69	2.13	3.91	5.52	8.09	10.01	12.19	16.77
0:34:55	1.15	2.80	2.21	4.10	5.85	8.69	10.85	13.35	18.69
0:40:44	0.95	2.34	1.84	3.44	4.92	7.34	9.21	11.37	16.02
0:46:33	0.75	1.86	1.47	2.75	3.95	5.90	7.41	9.17	12.97
0:52:23	0.60	1.50	1.18	2.21	3.17	4.74	5.95	7.36	10.39
0:58:12	0.47	1.17	0.92	1.72	2.48	3.71	4.67	5.78	8.18
1:04:01	0.37	0.92	0.72	1.36	1.96	2.94	3.70	4.58	6.48
1:09:50	0.31	0.77	0.61	1.14	1.63	2.44	3.06	3.78	5.33
1:15:39	0.22	0.54	0.43	0.80	1.16	1.74	2.19	2.71	3.85
1:21:28	0.17	0.41	0.32	0.61	0.87	1.31	1.64	2.04	2.88
1:27:18	0.11	0.28	0.22	0.42	0.60	0.90	1.14	1.41	2.01
1:33:07	0.08	0.21	0.16	0.31	0.44	0.66	0.83	1.03	1.45
1:38:56	0.06	0.16	0.13	0.24	0.34	0.51	0.64	0.80	1.13
1:44:45	0.05	0.13	0.10	0.20	0.28	0.42	0.53	0.66	0.93
1:50:34	0.05	0.12	0.09	0.18	0.25	0.38	0.47	0.58	0.82
1:56:23	0.05	0.12	0.09	0.17	0.24	0.36	0.45	0.56	0.78
2:02:13	0.05	0.11	0.09	0.17	0.24	0.36	0.44	0.55	0.77
2:08:02	0.05	0.11	0.09	0.17	0.24	0.36	0.44	0.55	0.77
2:13:51	0.05	0.11	0.09	0.17	0.24	0.36	0.44	0.55	0.77
2:19:40	0.03	0.07	0.05	0.10	0.15	0.22	0.28	0.35	0.50
2:25:29	0.02	0.04	0.03	0.06	0.09	0.13	0.16	0.21	0.29
2:31:18	0.01	0.02	0.02	0.03	0.05	0.07	0.09	0.12	0.17
2:37:08	0.00	0.01	0.01	0.02	0.03	0.04	0.05	0.06	0.09
2:42:57	0.00	0.01	0.00	0.01	0.01	0.02	0.02	0.03	0.04
2:48:46	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01
2:54:35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:00:24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:06:13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:12:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:17:52	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:23:41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:29:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:35:19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:41:08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:46:58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:52:47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3:58:36	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:04:25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:10:14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:16:03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:21:53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:27:42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:33:31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:39:20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:45:09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:50:59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:56:48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:02:37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:08:26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:14:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:20:04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:25:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:31:43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:37:32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:43:21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:49:10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5:54:59	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:00:49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:06:38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:12:27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:18:16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:24:05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:29:54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:35:44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:41:33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:47:22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:53:11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6:59:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Outlet Structure Major Storm Grate/Box Calculation

Detention Area	100-yr Flow	120% 100yr Flow	Water Surf Elev	Crest Elev	Calc'd Crest Length	d (rad)	Flow Depth	**Calc'd Flow**	**Crest Length Used***
B	13.3 cfs		102.56	100.58	0.0 ft	2.63	1.98 ft	63.3 cfs	**4.0 ft**

Weir Equation:

$Q = CLH^{1.5} + CH^{5/2} \tan(d/2)$

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

C = 3.0

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

H = Depth of flow over the crest, in ft

d = Angle of triangle weir portion (radians)

d = 2.63 radians for 4:1 side slope

d = 2.49 radians for 3:1 side slope

Outlet Struct Capacity = Inlet Capacity Calculation at the depth to the spillway crest plus flow depth.

*Weir calculation shows that sides alone have enough capacity to convey 100-yr flow, but bars for grate will result in a longer required weir length: 32 bars x 1/4" thick = 8 in. or 0.67 ft. So, 0.0 ft (calc'd crest length) + 0.67 ft (additional length needed for bars) = 0.67 ft. Then apply a 50% debris clogging factor: 0.67 ft x 2 = 1.33 ft. However, 1.33 ft is an impractical structure width, so use 4.0 ft as a minimum crest length. Now check the maximum velocity through the grate using a 4.0 ft crest length:

Check Major Storm Grate Conditions

Maximum velocity through grate = 2.0 ft/sec

Q_{100} = 13.3 cfs

Open area of grate required = Q_{100} / V_{max} = 13.3 cfs / 2.0 ft/sec = 6.65 sf

Grate has an 70% open area (area of bars = 30% of total grate area)

Total grate area required = 6.65 sf / 0.70 = 9.5 sf

Actual outlet structure opening for grate (from weir calculation) = 4 ft x 4 ft = **16 sf** (16 sf > 9.5 sf, so 4' crest length is okay)

16 sf / 4 ft = 4.0 ft min. crest length, use **4.0 ft**

Check velocity through grate using outlet structure opening of 4.0 ft x 4.0 ft = 16 sf:

Actual velocity through grate = 13.3 cfs / (16 sf x 0.70) = **1.2 ft/sec** (1.2 ft/sec < 2.0 ft/sec, okay)

Therefore, use **4.0 ft** for crest length.

The Beach at Woodmoor Filing No. 1
Drainage Structure Calculations

Grate	Safety Grate or Trash Rack	Type of Grate (see below)	R Value		Outlet Diameter or Min. Dimension	A_{ot} Total Outlet/ Orifice Area	A_t/A_{ot}	Minimum Gross Grate Area
			Table	User Input				
A1-1	Trash	WS	0.60		0.7-in	0.0085sf	36.07	0.51sf
A1-2	Safety	Other	N/A	0.70	5.1-in	0.41sf	40.78	23.89sf
A2-1	Trash	WS	0.60		0.4-in	0.0015sf	37.15	0.09sf
A2-2	Safety	Other	N/A	0.70	2.6-in	0.1600sf	55.61	12.71sf
B-1	Trash	WS	0.60		0.7-in	0.0081sf	36.07	0.49sf
B-2	Safety	Other	N/A	0.70	6.0-in	0.5200sf	36.59	27.18sf

A_t / A_{ot} = Ratio of Total Grate Open Area to Total Outlet Area (taken from UDSCM Fig OS-1: Trash Rack Sizing)

A_t = Total Grate Open Area (R-Value x Grate Area) (Example: 1'Wx6'H Well Screen=1'x6'x0.60=3.6ft^2)

A_{ot} = Total Outlet Area (Example: If orifice plate includes 3-1"dia holes A_{ot}=2.356in^2=0.016ft^2)

Safety Grate: $A_t / A_{ot} = 77e^{-0.124D}$ -- (Outlet Diameter or Minimum Dimension less than 24-inches)

Trash Rack: $A_t / A_{ot} = 38.5e^{-0.095D}$ (Outlet Diameter or Minimum Dimension less than 24-inches)

Outlet Diameter is orifice plate hole size of pipe out of structure

Safety
Trash

Minimum Gross Grate Area: Calculated from outside dimension of grate

R Value = Net Open Area / Gross Rack Area

Type of Grate	Abbreviation	R-Value
Bar Grate 2" O.C. Cross Rods	BG 2	0.71
Bar Grate 4" O.C. Cross Rods	BG 4	0.77
Well Screen	WS	0.60
Other	Other	

<u>Grate A1-1</u>: 0.51sf / **2.5' high** = 0.204ft (2.5in). Use **12" wide opening** as a minimum.

<u>Grate A1-2</u>: 23.89sf / **4' wide opening** = 5.97ft min. for length. Use **6'-0" length**.

<u>Grate A2-1</u>: 0.09sf / **1.08' high** = 0.0833ft (1.0in). Use **12" wide opening** as a minimum.

<u>Grate A2-2</u>: 12.71sf / **4' wide opening** = 3.18ft min. for length. Use **4'-0" length** as a minimum.

<u>Grate B-1</u>: 0.49sf / **2.5' high** = 0.20ft (2.4in). Use **12" wide opening** as a minimum.

<u>Grate B-2</u>: 27.18sf / **4' wide opening** = 6.8ft min. for length. Use **7'-0" length**.

<u>WQ plate thickness</u> from UDFCD Vol. 3, Table OS-2: for Head, use from spillway water surface elev. to bottom of WQ plate elev.:
For Detention Basin A1: Elev. 10.5 - Elev. 2.56 = 7.94ft, use 8ft with a span of 1ft. Table OS-2 says to use a 3/16" thick WQ plate, but use a **1/4" thick steel WQ plate** for min. thickness.

For Detention Basin A2: Elev. 19.0 - Elev. 13.66 = 5.34ft, use 6ft with a span of 1ft. Table OS-2 says to use a 3/16" thick WQ plate, but use a **1/4" thick steel WQ plate** for min. thickness.

For Detention Basin B: Elev. 103.0 - Elev. 95.56 = 7.44ft, use 8ft with a span of 1ft. Table OS-2 says to use a 3/16" thick WQ plate, but use a **1/4" thick steel WQ plate** for min. thickness.

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**
DB A1	11.5cfs	2,490cf	75cf	3.64ac	100.0%	75cf	86sf	1.25-ft	108 cf	0.23 cfs	3.7-inch	**3.7-inch**
DB B	11.2cfs	2,612cf	78cf	3.99ac	100.0%	78cf	86sf	1.25-ft	108 cf	0.22 cfs	3.6-inch	**3.6-inch**

Opening Width Equation for Rectangular Opening

$L = Q / (CH^{1.5}) \times 12 + 0.2xHx12$ (UD-BMP Spreadsheet -- EDB tab)

C = 3.0

Presedementation / Forebay Sizing (Check)

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	Drain Time 5.0 min.	**Opening Width (1" min)**	**Opening Flow**	Check
DB A1	11.5cfs	2,490cf	75cf	3.64ac	100.0%	75cf	86sf	1.25-ft	108 cf	0.29 cfs	**3.7-in**	**1.29 cfs**	OK
DB B	11.2cfs	2,612cf	78cf	3.99ac	100.0%	78cf	86sf	1.25-ft	108 cf	0.29 cfs	**3.6-in**	**1.26 cfs**	OK

Weir Equation:

$Q = CLH^{1.5}$

C = 3.0

Forebay Overflow Calculation

Forebay	Water Surf Elev	Crest Elev	Crest Length	Flow Depth	**Calc'd Flow**
DB A1	100.25	100.0	6.0 ft	0.25 ft	2.3 cfs
DB B	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 A1	11.7cfs	0.1cfs	2.0 ft	0.33 ft	0.0:1	0.5%	0.013	2.0 ft	0.66 sf	2.7 ft	0.25 ft	3.2 ft/sec	**2.1 cfs**
Det B	13.3cfs	0.1cfs	2.0 ft	0.33 ft	0.0:1	0.5%	0.013	2.0 ft	0.66 sf	2.7 ft	0.25 ft	3.2 ft/sec	**2.1 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:

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

For NRCS soil types B, C and D.

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

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

For NRCS soil Type A:

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

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:

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

For NRCS Soil Group B:

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

For NRCS Soil Group C/D:

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

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.

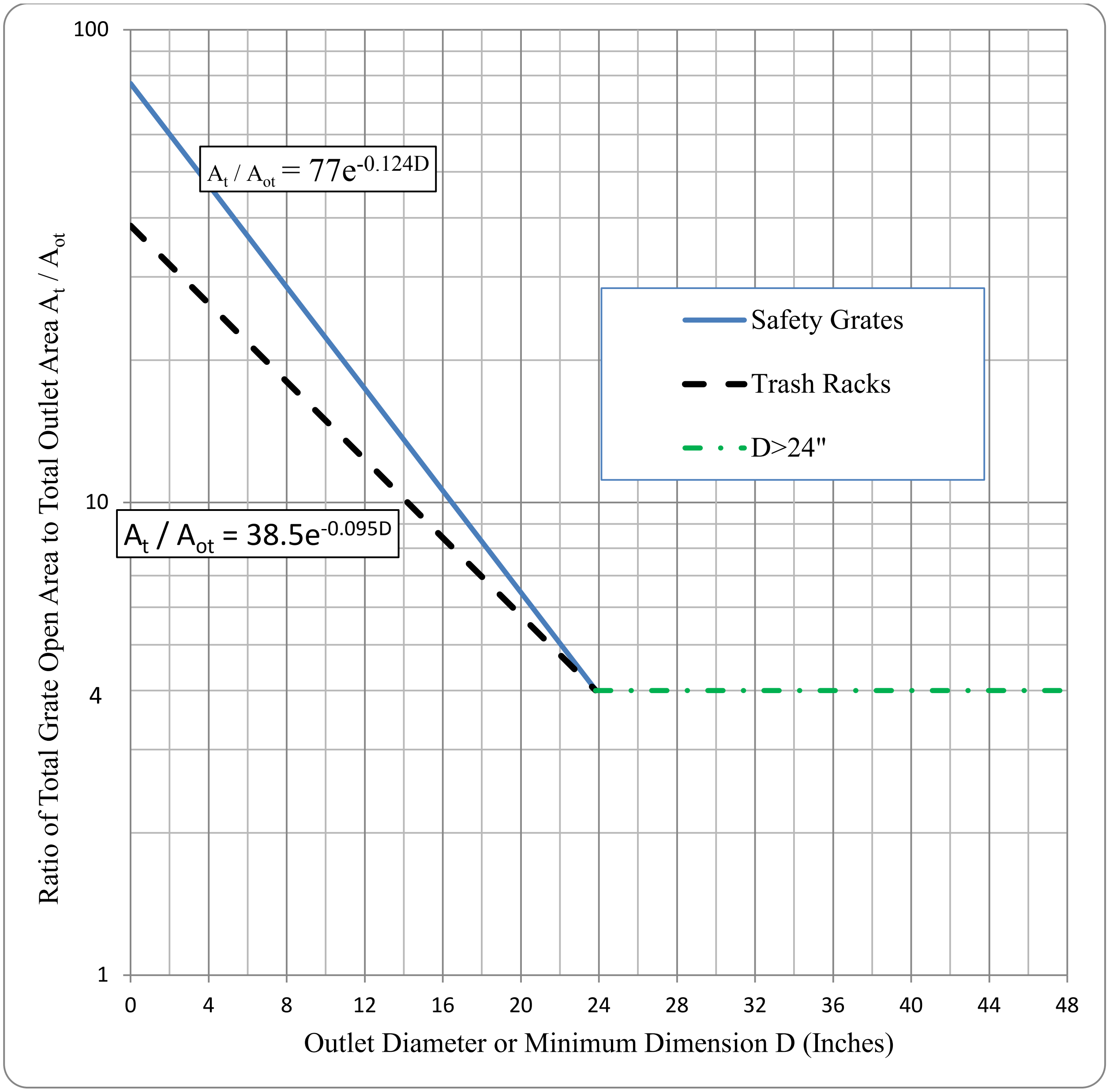


Figure OS-1. Trash Rack Sizing

Table OS-2. Thickness of steel water quality plate

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

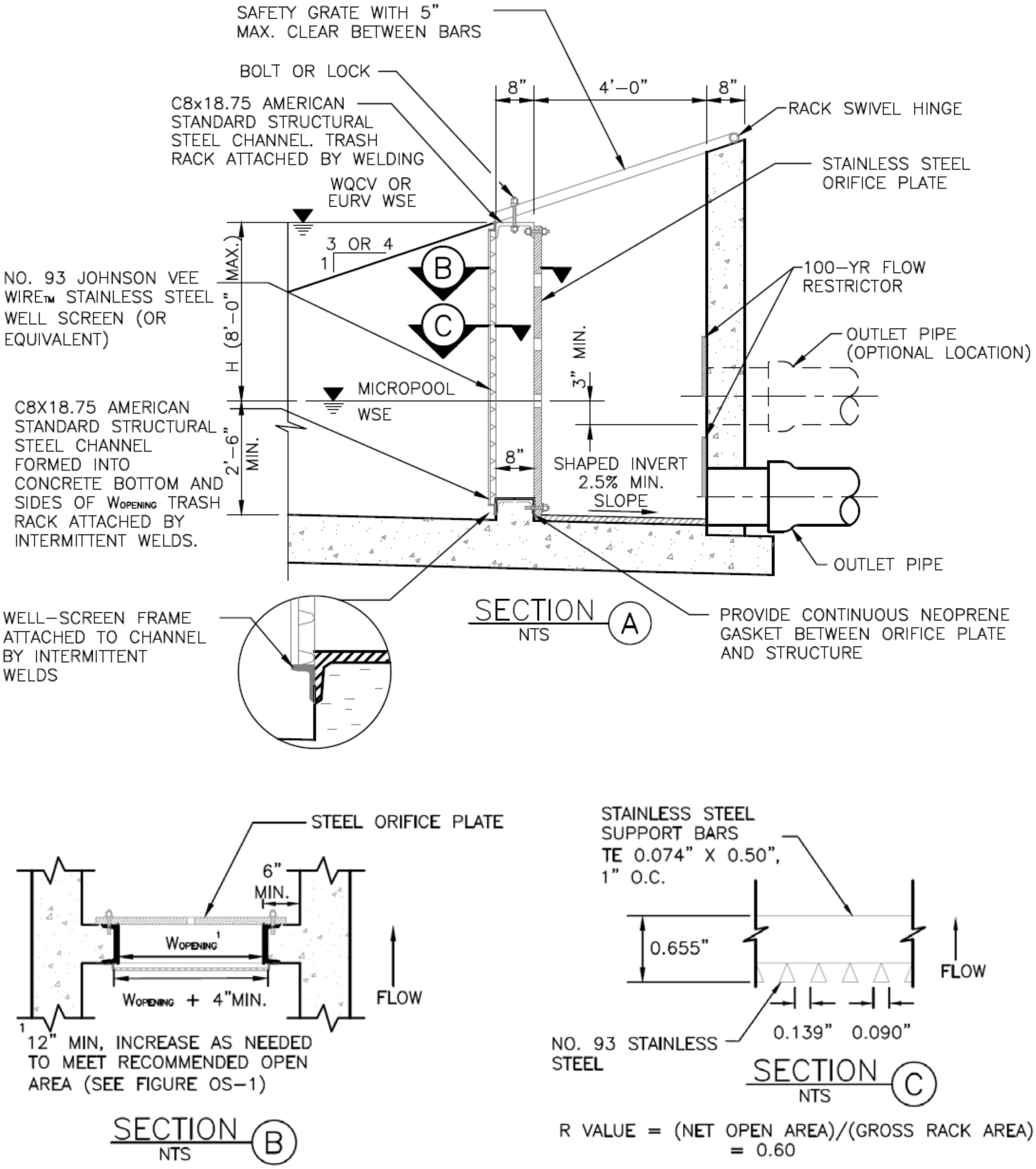


Figure OS-5. Typical outlet structure with well screen trash rack

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

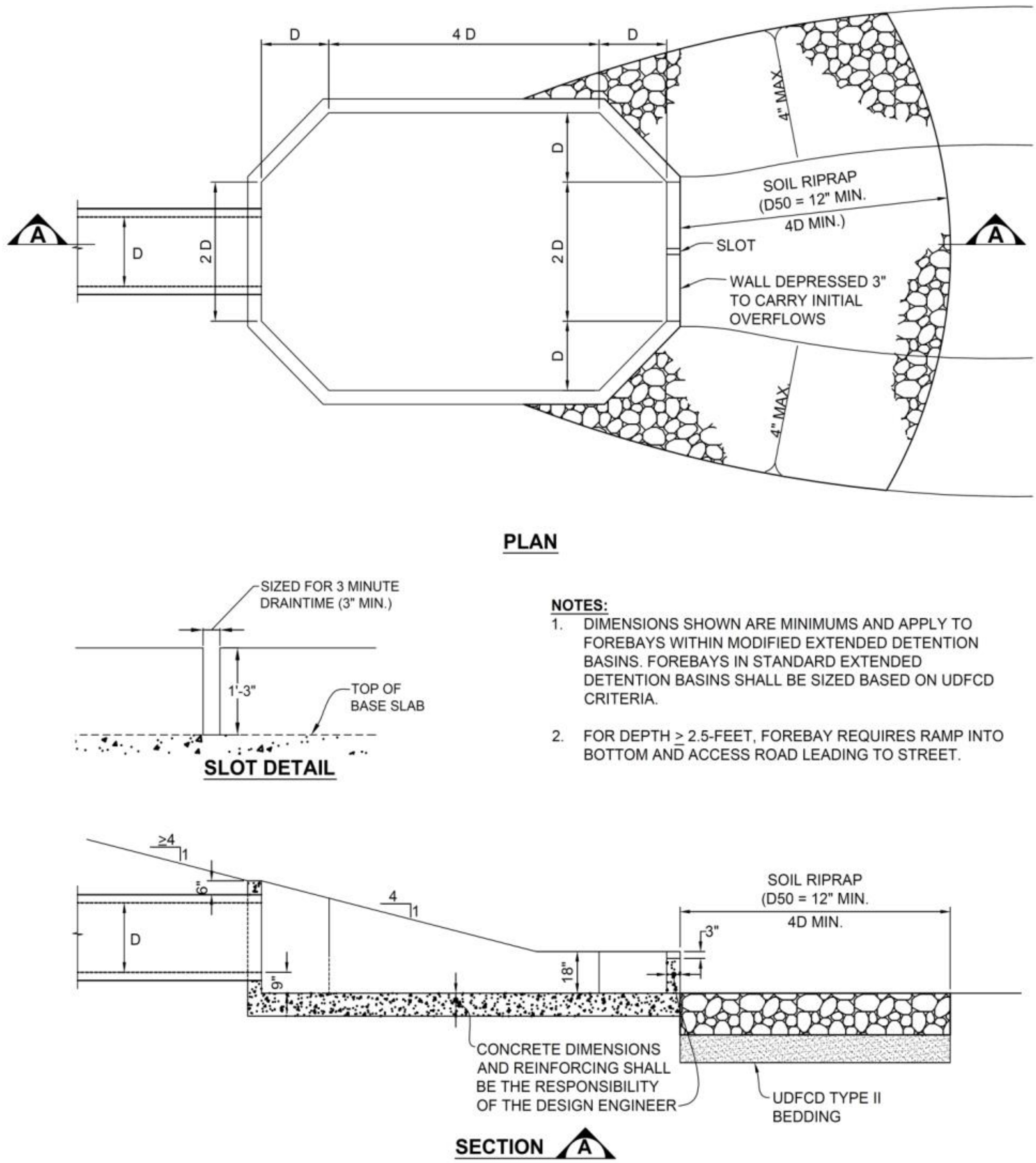
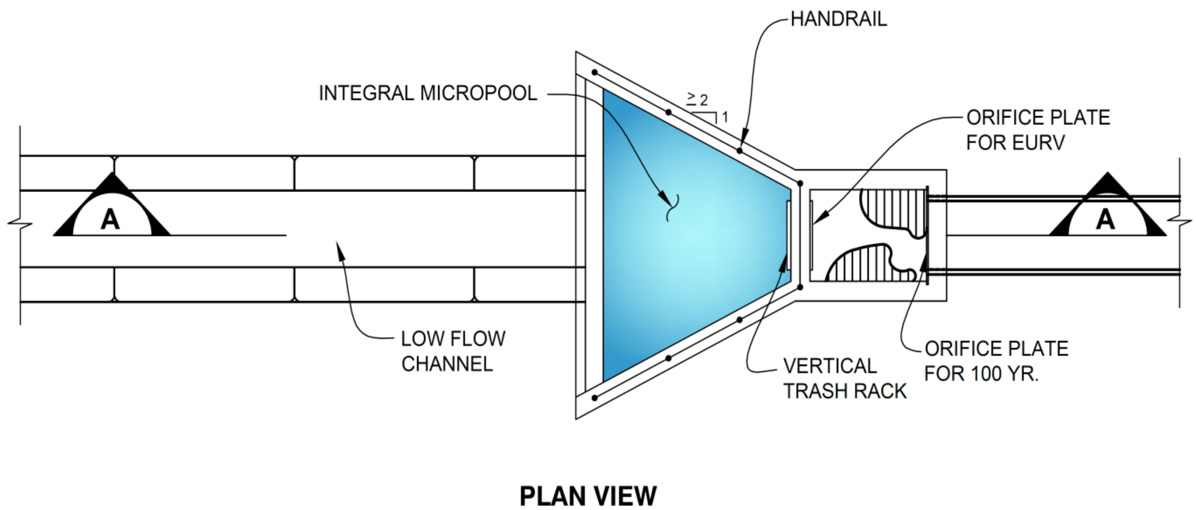


Figure 13-11. Concept for Outlet Structure with Flared Wingwalls and Handrail
(Integral Micropool Shown)



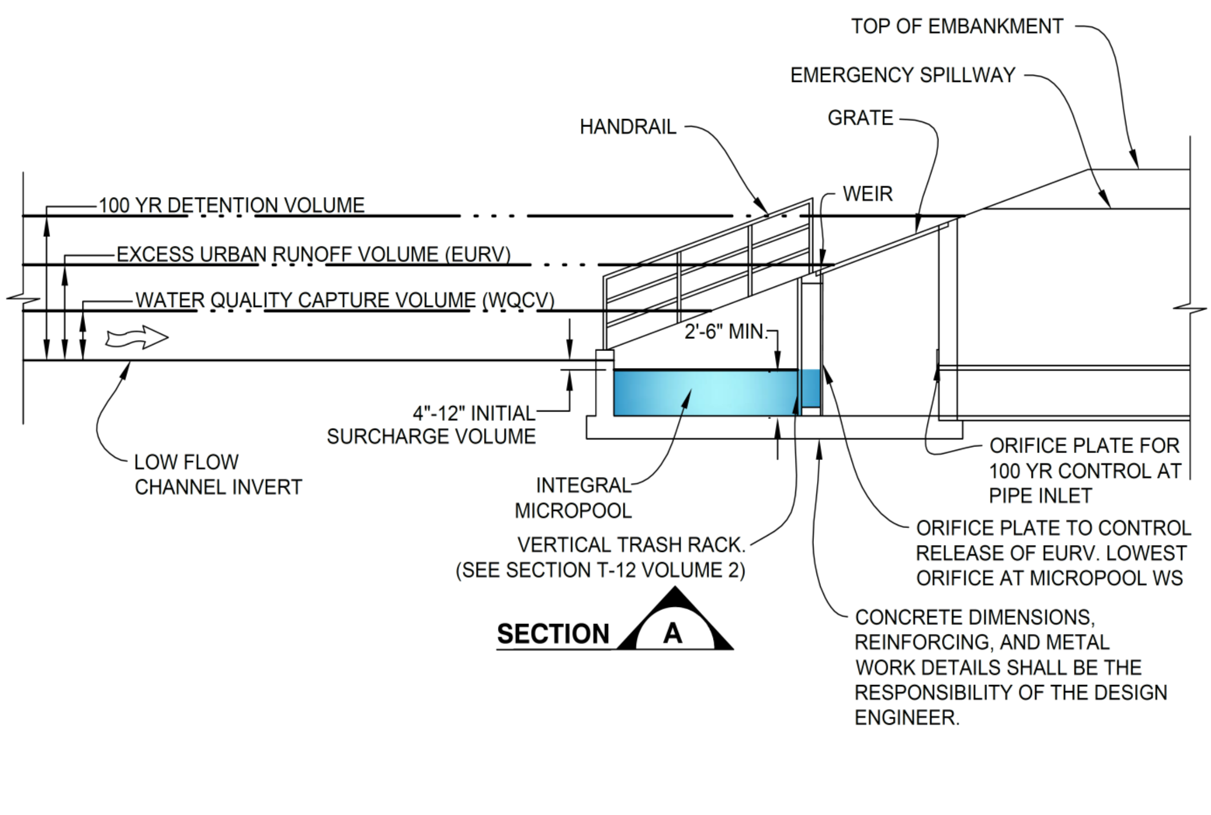


Figure 13-12c. Emergency Spillway Protection

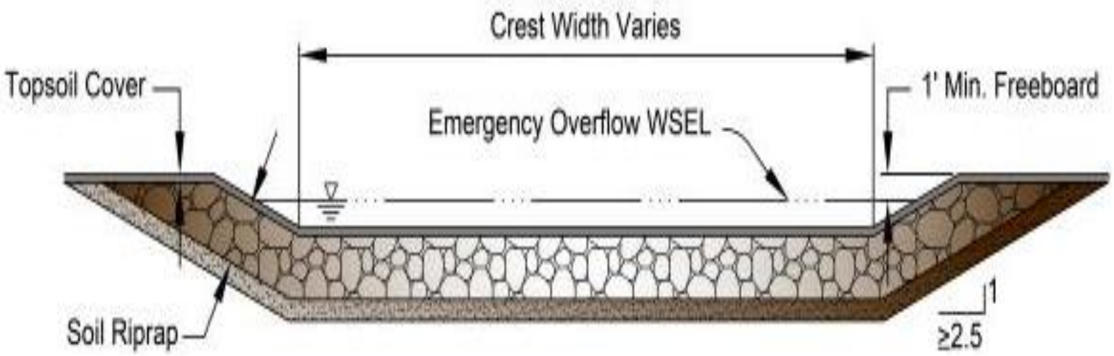
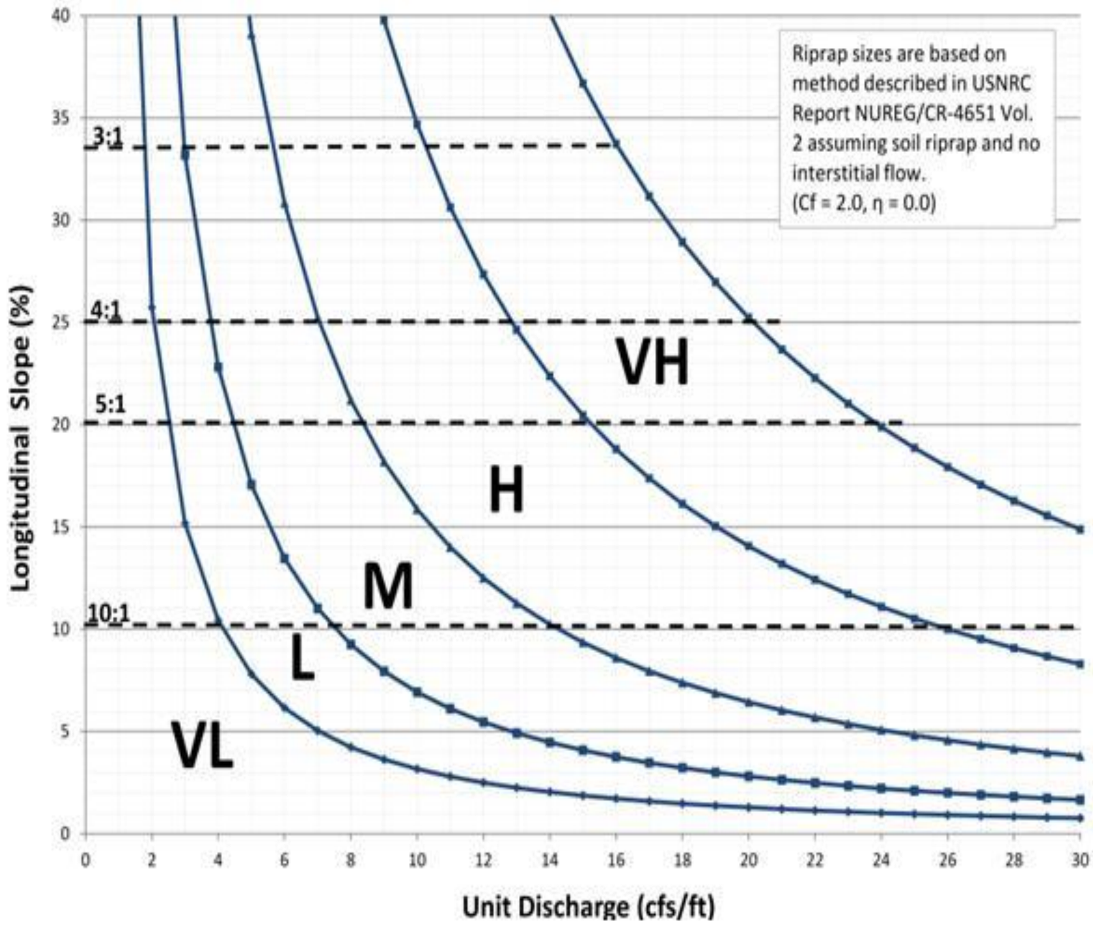


Figure 13-12d. Riprap Types for Emergency Spillway Protection



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)	2.6	1.7	1.5	2.3
Major Q_{Known} (cfs)	7.2	4.7	5.6	5.6

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

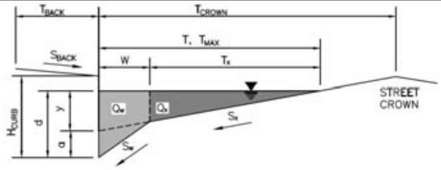
Minor Total Design Peak Flow, Q (cfs)	**2.6**	**1.7**	**1.5**	**2.3**
Major Total Design Peak Flow, Q (cfs)	**7.2**	**4.7**	**5.6**	**5.6**
Minor Flow Bypassed Downstream, Q_b (cfs)	0.0	0.0	N/A	N/A
Major Flow Bypassed Downstream, Q_b (cfs)	0.0	0.1	N/A	N/A

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	

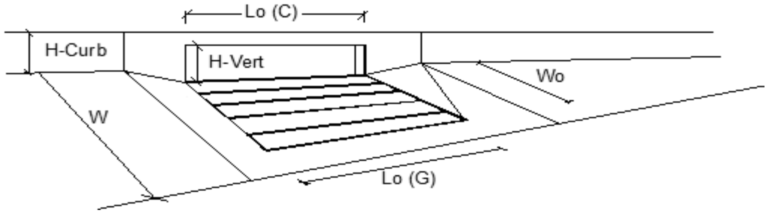
Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\,TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	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



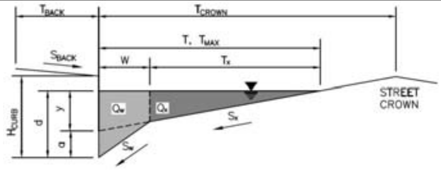
Design Information (Input)		MINOR	MAJOR	
Type of Inlet (CDOT Type R Curb Opening)	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a Single Unit 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 =**	**2.6**	**7.2**	cfs
Water Spread Width	T =	6.9	11.0	ft
Water Depth at Flowline (outside of local depression)	d =	3.4	4.4	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 =	0.585	0.369	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	1.1	4.6	cfs
Discharge within the Gutter Section W	Q_w =	1.5	2.7	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	0.23	0.33	sq ft
Velocity within the Gutter Section W	V_W =	6.5	8.1	fps
Water Depth for Design Condition	d_{LOCAL} =	6.4	7.4	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 =	0.218	0.145	ft/ft
Required Length L_T to Have 100% Interception	L_T =	6.92	14.04	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	6.92	14.04	ft
Interception Capacity	Q_i =	2.6	7.2	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 =**	**2.6**	**7.2**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	**Q_b =**	**0.0**	**0.0**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**2.6**	**7.2**	**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

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	

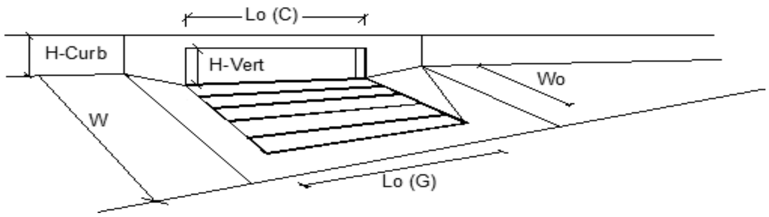
Maximum Capacity for 1/2 Street based on Allowable Depth		Minor Storm	Major Storm	
Theoretical Water Spread	T_{TH} =	13.2	28.2	ft
Theoretical Spread for Discharge outside the Gutter Section W (T - W)	$T_{X\,TH}$ =	12.0	27.0	ft
Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. ST-7)	E_O =	0.303	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



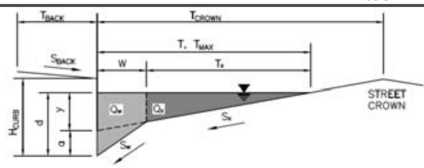
		MINOR	MAJOR	
Design Information (Input) CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	1	1	
Length of a 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 =**	**1.7**	**4.7**	cfs
Water Spread Width	T =	5.5	9.2	ft
Water Depth at Flowline (outside of local depression)	d =	3.1	4.0	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 =	0.703	0.449	
Discharge outside the Gutter Section W, carried in Section T_x	Q_x =	0.5	2.6	cfs
Discharge within the Gutter Section W	Q_w =	1.2	2.1	cfs
Discharge Behind the Curb Face	Q_{BACK} =	0.0	0.0	cfs
Flow Area within the Gutter Section W	A_W =	0.20	0.29	sq ft
Velocity within the Gutter Section W	V_W =	5.9	7.4	fps
Water Depth for Design Condition	d_{LOCAL} =	6.1	7.0	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 =	0.257	0.172	ft/ft
Required Length L_T to Have 100% Interception	L_T =	5.16	10.45	ft
Under No-Clogging Condition		MINOR	MAJOR	
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L_T)	L =	5.16	10.00	ft
Interception Capacity	Q_i =	1.7	4.7	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 =**	**1.7**	**4.6**	cfs
Carry-Over Flow = $Q_{b(GRATE)}$-Q_a	**Q_b =**	**0.0**	**0.1**	cfs
Summary		MINOR	MAJOR	
Total Inlet Interception Capacity	**Q =**	**1.7**	**4.6**	**cfs**
Total Inlet Carry-Over Flow (flow bypassing inlet)	**Q_b =**	**0.0**	**0.1**	**cfs**
Capture Percentage = Q_a/Q_o =	**C% =**	**100**	**99**	**%**

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	

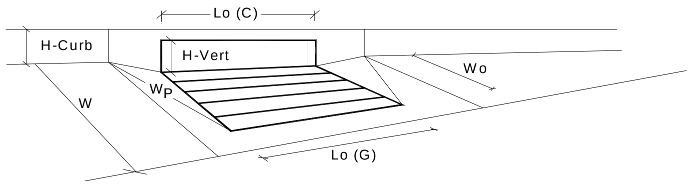
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

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



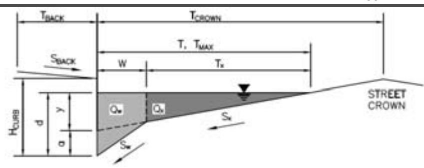
Design Information (Input)		MINOR	MAJOR	
Type of Inlet — CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Water Depth at Flowline (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)	$Q_{PEAK\ REQUIRED}$ =	1.5	5.6	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	

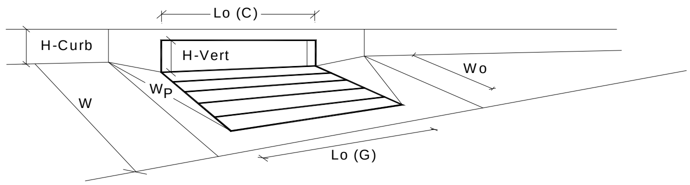
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	13.2	13.2	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.9	8.5	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☑	check = yes

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)		MINOR	MAJOR	
Type of Inlet — CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from '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)	$Q_{PEAK\ REQUIRED}$ =	2.3	5.6	cfs

KIOWA ENGINEERING CORPORATION

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

SHEET NO. 1 OF 1

CALCULATED BY CJC DATE 5/22/17

CHECKED BY ______ DATE ______

SCALE ______

TYPE C INLET CAPACITY

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

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 :

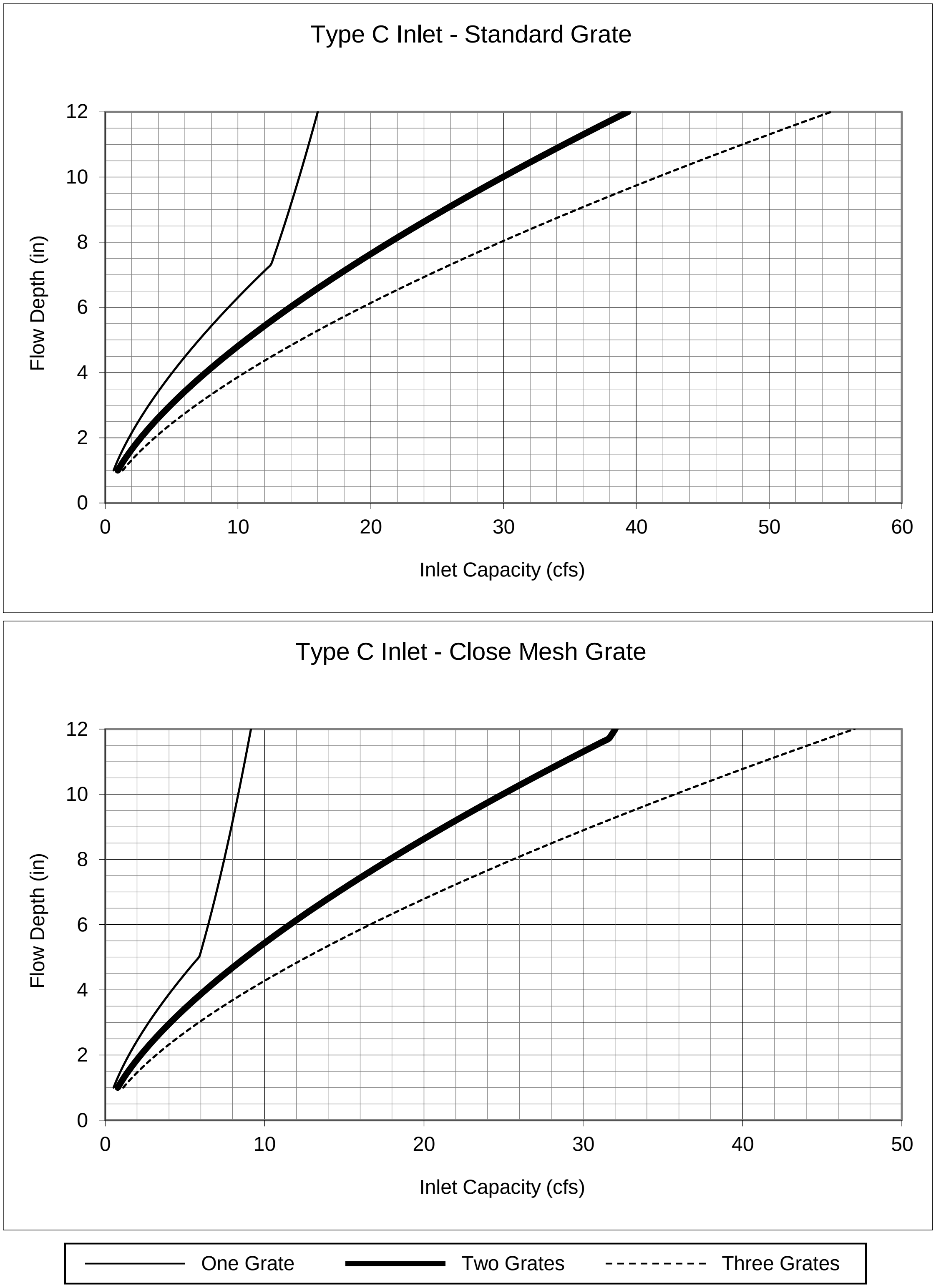
AT Q_5 = 0.6 × 2 = 1.2 CFS, Y ~ 1.6 in. (0.13 FT.) [ONE STD. GRATE]

AT Q_{100} = 1.8 × 2 = 3.6 CFS, Y ~ 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

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[1]**	Minimum Slope of Pipe	Full Pipe Flow Velocity	Head above Pipe Flowline	H	Pipe Inlet Control Capacity	**Mannings Pipe Capacity**	Capacity Check
S1	7.2 cfs	7.2 cfs	A3 / DP 2	0.013	**1.0%**	16-inch	**18-inch**	0.47%	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	11.7 cfs	11.7 cfs	DB A1	0.013	**1.0%**	19-inch	**18-inch**	1.24%	6.0 ft/sec	5.8 ft	5.1 ft	20.7 cfs	**10.5 cfs**	Pressure
S4	6.9 cfs	6.9 cfs	A1 / DP1	0.013	**1.0%**	15-inch	**18-inch**	0.43%	6.0 ft/sec	2.5 ft	1.8 ft	12.2 cfs	**10.5 cfs**	OK
S5	0.3 cfs	0.3 cfs	A6 / DP 7	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	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	52.9 cfs	52.9 cfs	OS-4, DP 8	0.013	**1.7%**	30-inch	**30-inch**	1.66%	10.9 ft/sec	14.9 ft	13.7 ft	94.6 cfs	**53.6 cfs**	OK
S8	26.6 cfs	26.6 cfs	OS-4	0.013	**1.4%**	24-inch	**24-inch**	1.38%	8.5 ft/sec	3.7 ft	2.7 ft	26.9 cfs	**26.8 cfs**	OK
S9	52.9 cfs	52.9 cfs	DP 9	0.013	**1.7%**	30-inch	**30-inch**	1.66%	10.9 ft/sec	13.6 ft	12.4 ft	90.0 cfs	**53.6 cfs**	OK
S10	0.7 cfs	0.7 cfs	B2 / DP 10	0.013	**1.0%**	7-inch	**18-inch**	0.00%	6.0 ft/sec	3.5 ft	2.8 ft	15.3 cfs	**10.5 cfs**	OK
S11	4.7 cfs	4.7 cfs	B4 / DP 12	0.013	**1.0%**	13-inch	**18-inch**	0.20%	6.0 ft/sec	3.3 ft	2.5 ft	14.6 cfs	**10.5 cfs**	OK
S12	11.2 cfs	11.2 cfs	DP 14	0.013	**1.2%**	18-inch	**18-inch**	1.14%	6.5 ft/sec	3.3 ft	2.5 ft	14.6 cfs	**11.5 cfs**	OK
S13	13.3 cfs	13.3 cfs	DB B	0.013	**1.0%**	20-inch	**18-inch**	1.60%	6.0 ft/sec	5.2 ft	4.4 ft	19.4 cfs	**10.5 cfs**	Pressure
S14	2.1 cfs	2.1 cfs	DP 11	0.013	**1.0%**	10-inch	**18-inch**	0.04%	6.0 ft/sec	3.5 ft	2.8 ft	15.3 cfs	**10.5 cfs**	OK
S15	4.1 cfs	4.1 cfs	DB A2	0.013	**1.0%**	13-inch	**18-inch**	0.15%	6.0 ft/sec	2.8 ft	2.1 ft	13.2 cfs	**10.5 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

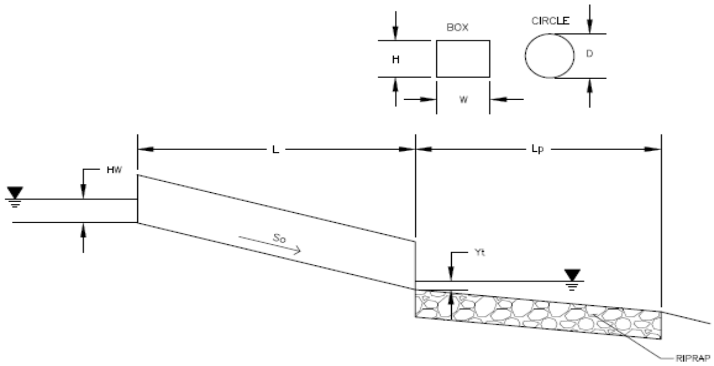
Orifice Equation:

$Q = CA(2gH)^{0.5}$
- 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

[1] Pipe #S2 will be a 19"x30" HERCP (24" circular equivalent).

Determination of Culvert Headwater and Outlet Protection

Project: The Beach at Woodmoor Filing No. 1
Basin ID: 18" RCP, Pipe No. S3



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

Design Information (Input):

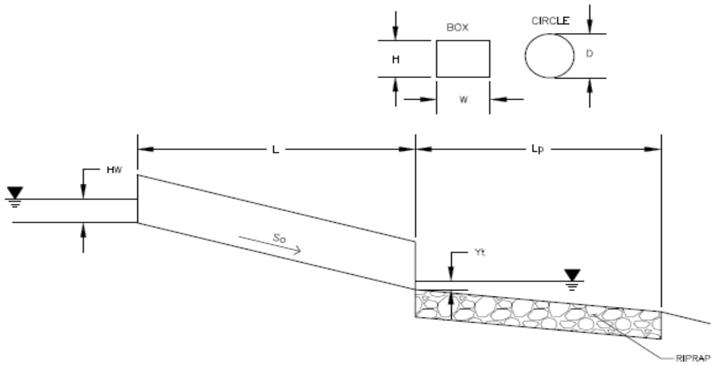
Design Discharge	Q =	11.7	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)	Square End with Headwall		
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7104.4	ft
Outlet Elevation **OR** Slope	So =	0.01	ft/ft
Culvert Length	L =	47	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 =	7104.25	ft
Max Allowable Channel Velocity	V =	5	ft/s

Required Protection (Output):

Tailwater Surface Height	Y_t =	0.32	ft
Flow Area at Max Channel Velocity	A_t =	2.34	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	0.85	
Sum of All Losses Coefficients	k_s =	2.35	ft
Culvert Normal Depth	Y_n =	1.13	ft
Culvert Critical Depth	Y_c =	1.30	ft
Tailwater Depth for Design	d =	1.40	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	-	ft
Expansion Factor	1/(2*tan(Θ)) =	1.24	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	4.25	$ft^{0.5}$/s
Froude Number	Fr =	-	**Pressure flow!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.21	
Inlet Control Headwater	HW_I =	2.73	ft
Outlet Control Headwater	HW_O =	2.53	ft
Design Headwater Elevation	**HW =**	**7,107.13**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**1.82**	**HW/D > 1.5!**
Minimum Theoretical Riprap Size	d_{50} =	11	in
Nominal Riprap Size	d_{50} =	12	in
UDFCD Riprap Type	**Type =**	**M**	
Length of Protection	**L_p =**	**8**	**ft**
Width of Protection	**T =**	**8**	**ft**

Determination of Culvert Headwater and Outlet Protection

Project: **The Beach at Woodmoor Filing No. 1**
Basin ID: **18" RCP, Pipe No. S4**



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

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

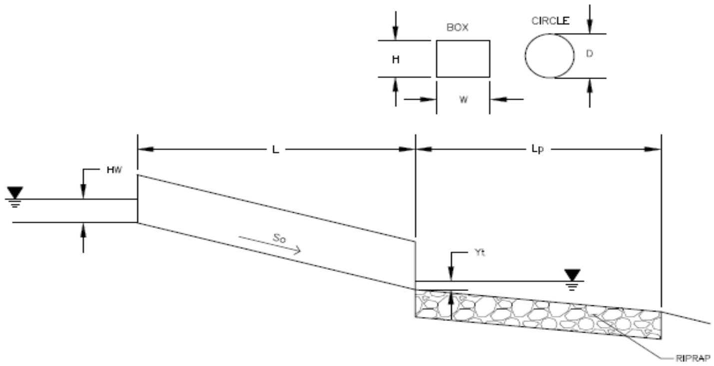
Design Discharge	Q =	6.9	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)		Square End Projection	
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7104	ft
Outlet Elevation **OR** Slope	So =	0.01	ft/ft
Culvert Length	L =	90	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 =	7103.6	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 =	1.38	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	1.63	
Sum of All Losses Coefficients	k_s =	3.13	ft
Culvert Normal Depth	Y_n =	0.89	ft
Culvert Critical Depth	Y_c =	1.02	ft
Tailwater Depth for Design	d =	1.26	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	1.19	ft
Expansion Factor	1/(2*tan(Θ)) =	5.36	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	2.50	$ft^{0.5}$/s
Froude Number	Fr =	1.31	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.42	
Inlet Control Headwater	HW_I =	1.63	ft
Outlet Control Headwater	HW_O =	1.10	ft
Design Headwater Elevation	**HW =**	**7,105.63**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**1.09**	
Minimum Theoretical Riprap Size	d_{50} =	4	in
Nominal Riprap Size	d_{50} =	6	in
UDFCD Riprap Type	**Type =**	**VL**	
Length of Protection	**L_p =**	**7**	**ft**
Width of Protection	**T =**	**3**	**ft**

Determination of Culvert Headwater and Outlet Protection

Project: The Beach at Woodmoor Filing No. 1
Basin ID: 18" RCP, Pipe No. S5



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

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

	Design Discharge	Q =	0.3	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 =	7128.5	ft
	Outlet Elevation **OR** Slope	Elev OUT =	7127.8	ft
	Culvert Length	L =	70	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 =	7128.3	ft
	Max Allowable Channel Velocity	V =	5	ft/s

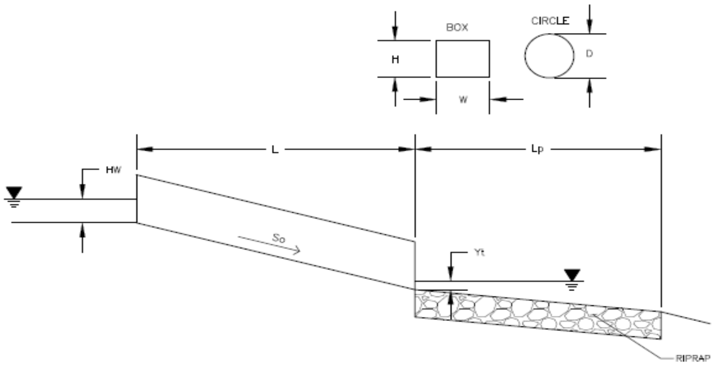
Required Protection (Output):

Tailwater Surface Height	Y_t =	0.50	ft
Flow Area at Max Channel Velocity	A_t =	0.06	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	1.27	
Sum of All Losses Coefficients	k_s =	2.77	ft
Culvert Normal Depth	Y_n =	0.17	ft
Culvert Critical Depth	Y_c =	0.20	ft
Tailwater Depth for Design	d =	0.85	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	0.84	ft
Expansion Factor	1/(2*tan(Θ)) =	6.70	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	0.11	ft$^{0.5}$/s
Froude Number	Fr =	1.34	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.60	
Inlet Control Headwater	HW_I =	0.36	ft
Outlet Control Headwater	HW_O =	0.15	
Design Headwater Elevation	**HW =**	**7,128.86**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**0.24**	
Minimum Theoretical Riprap Size	d_{50} =	0	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**

Determination of Culvert Headwater and Outlet Protection

Project: The Beach at Woodmoor Filing No. 1

Basin ID: 18" RCP, Pipe No. S10



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

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

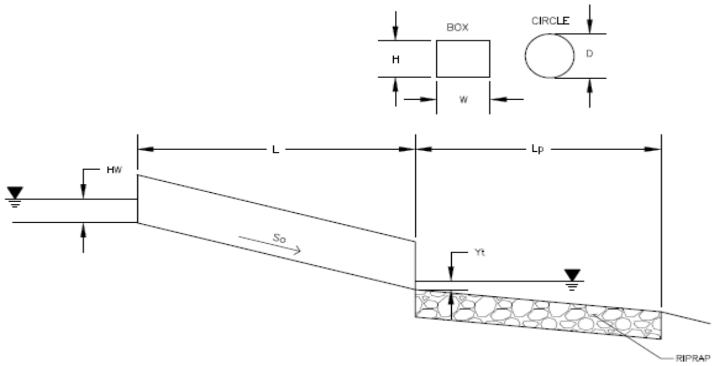
Design Discharge	Q =	0.7	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)		Square End Projection	
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7122.5	ft
Outlet Elevation **OR** Slope	So =	0.01	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 =	7122.2	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 =	0.14	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 =	0.26	ft
Culvert Critical Depth	Y_c =	0.31	ft
Tailwater Depth for Design	d =	0.91	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	0.88	ft
Expansion Factor	1/(2*tan(Θ)) =	6.70	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	0.25	ft$^{0.5}$/s
Froude Number	Fr =	1.40	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.57	
Inlet Control Headwater	HW_I =	0.43	ft
Outlet Control Headwater	HW_O =	0.11	
Design Headwater Elevation	**HW =**	**7,122.93**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**0.29**	
Minimum Theoretical Riprap Size	d_{50} =	0	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**

Determination of Culvert Headwater and Outlet Protection

Project: **The Beach at Woodmoor Filing No. 1**
Basin ID: **18" RCP, Pipe No. S13**



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

Design Information (Input):

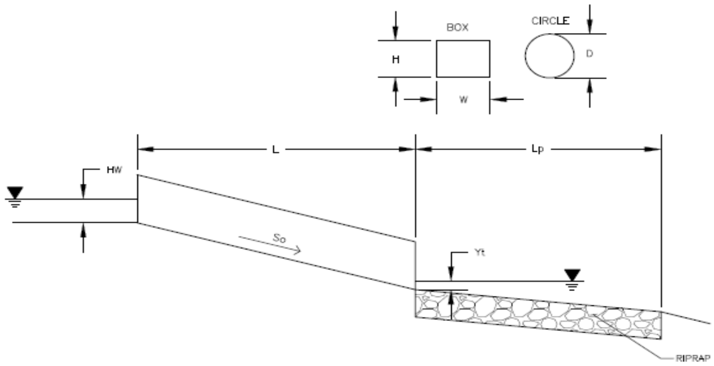
Design Discharge	Q =	13.3	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)	Square End with Headwall		
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7097.4	ft
Outlet Elevation **OR** Slope	So =	0.01	ft/ft
Culvert Length	L =	41	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 =	7097.5	ft
Max Allowable Channel Velocity	V =	5	ft/s

Required Protection (Output):

Tailwater Surface Height	Y_t =	0.51	ft
Flow Area at Max Channel Velocity	A_t =	2.66	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	0.74	
Sum of All Losses Coefficients	k_s =	2.24	ft
Culvert Normal Depth	Y_n =	0.97	ft
Culvert Critical Depth	Y_c =	1.36	ft
Tailwater Depth for Design	d =	1.43	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	-	ft
Expansion Factor	1/(2*tan(Θ)) =	2.00	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	4.83	$ft^{0.5}/s$
Froude Number	Fr =	-	**Pressure flow!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.34	
Inlet Control Headwater	HW_I =	3.24	ft
Outlet Control Headwater	HW_O =	2.99	
Design Headwater Elevation	**HW =**	**7,100.64**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**2.16**	**HW/D > 1.5!**
Minimum Theoretical Riprap Size	d_{50} =	7	in
Nominal Riprap Size	d_{50} =	9	in
UDFCD Riprap Type	**Type =**	**L**	
Length of Protection	**L_p =**	**8**	**ft**
Width of Protection	**T =**	**6**	**ft**

Determination of Culvert Headwater and Outlet Protection

Project: **The Beach at Woodmoor Filing No. 1**
Basin ID: **18" RCP, Pipe No. S14**



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

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

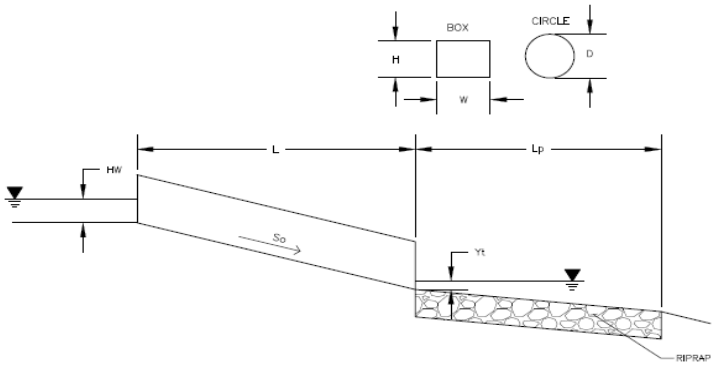
Design Discharge	Q =	2.1	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)	Square End Projection		
Box Culvert:		**OR**	
Barrel Height (Rise) in Feet	Height (Rise) =		ft
Barrel Width (Span) in Feet	Width (Span) =		ft
Inlet Edge Type (Choose from pull-down list)			
Number of Barrels	No =	1	
Inlet Elevation	Elev IN =	7100.5	ft
Outlet Elevation **OR** Slope	So =	0.01	ft/ft
Culvert Length	L =	33	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 =	7100.7	ft
Max Allowable Channel Velocity	V =	5	ft/s

Required Protection (Output):

Tailwater Surface Height	Y_t =	0.53	ft
Flow Area at Max Channel Velocity	A_t =	0.42	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	0.60	
Sum of All Losses Coefficients	k_s =	2.10	ft
Culvert Normal Depth	Y_n =	0.45	ft
Culvert Critical Depth	Y_c =	0.55	ft
Tailwater Depth for Design	d =	1.02	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	0.98	ft
Expansion Factor	1/(2*tan(Θ)) =	6.70	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	0.76	$ft^{0.5}/s$
Froude Number	Fr =	1.43	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.54	
Inlet Control Headwater	HW_I =	0.77	ft
Outlet Control Headwater	HW_O =	0.74	ft
Design Headwater Elevation	**HW =**	**7,101.27**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**0.51**	
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**

Determination of Culvert Headwater and Outlet Protection

Project: The Beach at Woodmoor Filing No. 1
Basin ID: 18" RCP, Pipe No. S15



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

Supercritical Flow! Using Da to calculate protection type.

Design Information (Input):

Design Discharge	Q =	4.1	cfs
Circular Culvert:			
Barrel Diameter in Inches	D =	18	inches
Inlet Edge Type (Choose from pull-down list)		Square End with Headwall	
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 =	7115.5	ft
Outlet Elevation **OR** Slope	So =	0.01	ft/ft
Culvert Length	L =	47	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 =	7115.2	ft
Max Allowable Channel Velocity	V =	5	ft/s

Required Protection (Output):

Tailwater Surface Height	Y_t =	0.17	ft
Flow Area at Max Channel Velocity	A_t =	0.82	ft^2
Culvert Cross Sectional Area Available	A =	1.77	ft^2
Entrance Loss Coefficient	k_e =	0.50	
Friction Loss Coefficient	k_f =	0.85	
Sum of All Losses Coefficients	k_s =	2.35	ft
Culvert Normal Depth	Y_n =	0.65	ft
Culvert Critical Depth	Y_c =	0.78	ft
Tailwater Depth for Design	d =	1.14	ft
Adjusted Diameter **OR** Adjusted Rise	D_a =	1.07	ft
Expansion Factor	1/(2*tan(Θ)) =	2.11	
Flow/Diameter$^{2.5}$ **OR** Flow/(Span * Rise$^{1.5}$)	Q/D^2.5 =	1.49	$ft^{0.5}/s$
Froude Number	Fr =	1.40	**Supercritical!**
Tailwater/Adjusted Diameter **OR** Tailwater/Adjusted Rise	Yt/D =	0.16	
Inlet Control Headwater	HW_I =	1.14	ft
Outlet Control Headwater	HW_O =	0.86	
Design Headwater Elevation	**HW =**	**7,116.64**	**ft**
Headwater/Diameter OR Headwater/Rise Ratio	**HW/D =**	**0.76**	
Minimum Theoretical Riprap Size	d_{50} =	9	in
Nominal Riprap Size	d_{50} =	12	in
UDFCD Riprap Type	**Type =**	**M**	
Length of Protection	**L_p =**	**8**	**ft**
Width of Protection	**T =**	**6**	**ft**

The Beach at Woodmoor Filing No. 1
Swale Capacity Calculations

Description	Design Point	Design Flow	Bottom Width	Channel Side Slope Left	Channel Side Slope Right	Flow Depth	Channel Slope	Manning "n"	Top Width	Channel Area	Wetted Perimeter	Hydraulic Radius	Flow Velocity	Channel Flow Capacity
Swale 1	7	0.3 cfs	0.0 ft	4:1	4:1	1.00 ft	7.2%	0.035	8.0 ft	4.00 sf	8.2 ft	0.49 ft	7.1 ft/sec	**28.2 cfs**
Swale 2	Basins A-6, A-7	1.8 cfs	0.0 ft	3:1	3:1	1.00 ft	2.8%	0.035	6.0 ft	3.00 sf	6.3 ft	0.47 ft	4.3 ft/sec	**13.0 cfs**
Swale 3	Basin A-5.1	4.1 cfs	0.0 ft	4:1	4:1	1.00 ft	1.9%	0.035	8.0 ft	4.00 sf	8.2 ft	0.49 ft	3.6 ft/sec	**14.5 cfs**
Swale 4	1	6.9 cfs	0.0 ft	3:1	3:1	1.00 ft	3.7%	0.035	6.0 ft	3.00 sf	6.3 ft	0.47 ft	5.0 ft/sec	**14.9 cfs**
Swale 5	10	0.7 cfs	0.0 ft	6:1	6:1	0.50 ft	1.4%	0.035	6.0 ft	1.50 sf	6.1 ft	0.25 ft	2.0 ft/sec	**3.0 cfs**
Swale 6	11	2.1 cfs	0.0 ft	3:1	3:1	1.00 ft	3.4%	0.035	6.0 ft	3.00 sf	6.3 ft	0.47 ft	4.8 ft/sec	**14.3 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 = $(1.49/n)AR_n^{2/3}S^{1/2}$

Add a footnote or column identifying the swale lining. What type of lining are they? See EPC DCM Table 10-4 for permissible velocity.

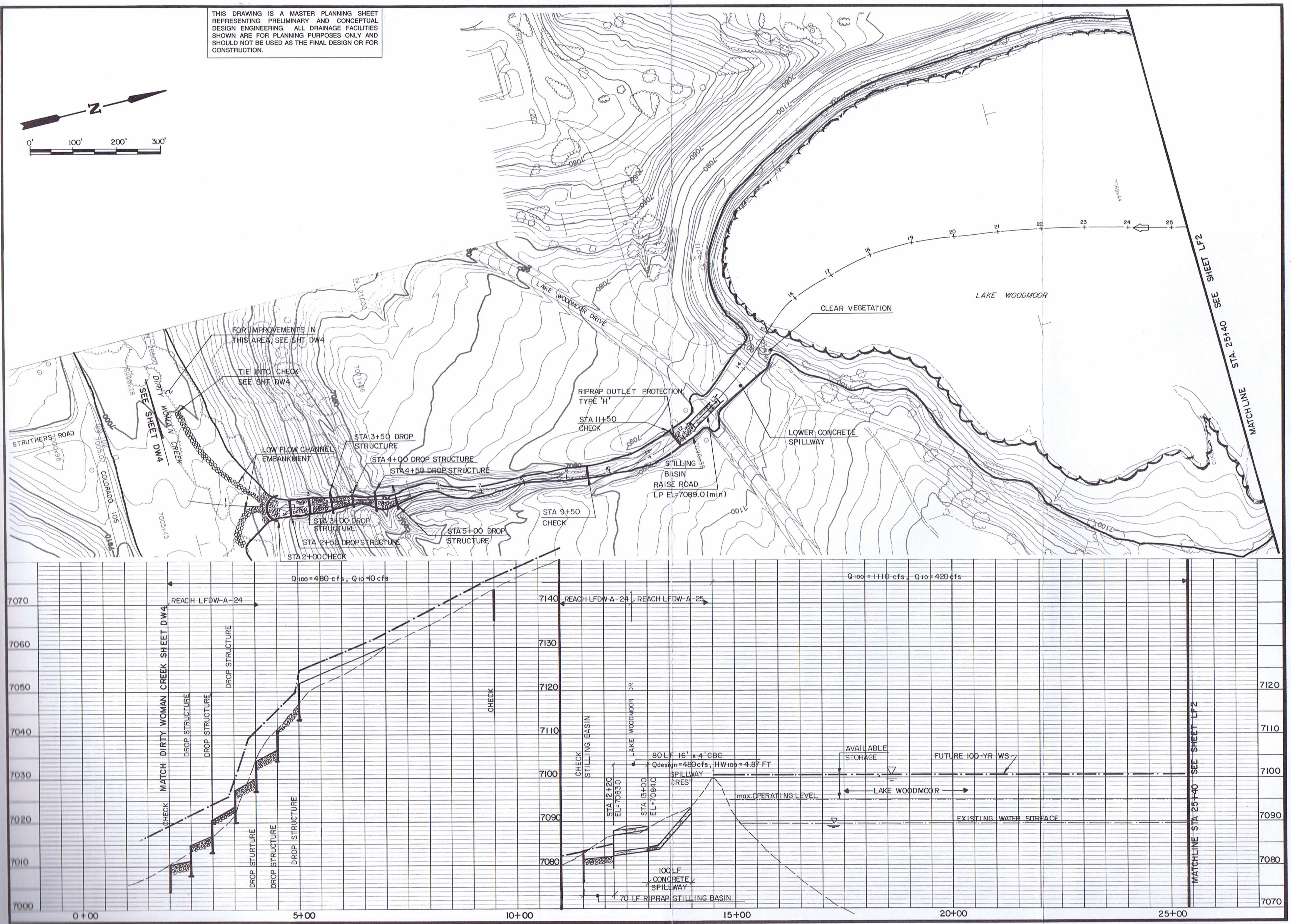
At least swale 1 exceeds the permissible velocity for the channel slope. See EPC DCM Table 10-4.

APPENDIX D

Referenced Information

Excerpt from Dirty Woman and Crystal Creeks Drainage Basin Planning Study

THIS DRAWING IS A MASTER PLANNING SHEET REPRESENTING PRELIMINARY AND CONCEPTUAL DESIGN ENGINEERING. ALL DRAINAGE FACILITIES SHOWN ARE FOR PLANNING PURPOSES ONLY AND SHOULD NOT BE USED AS THE FINAL DESIGN OR FOR CONSTRUCTION.



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

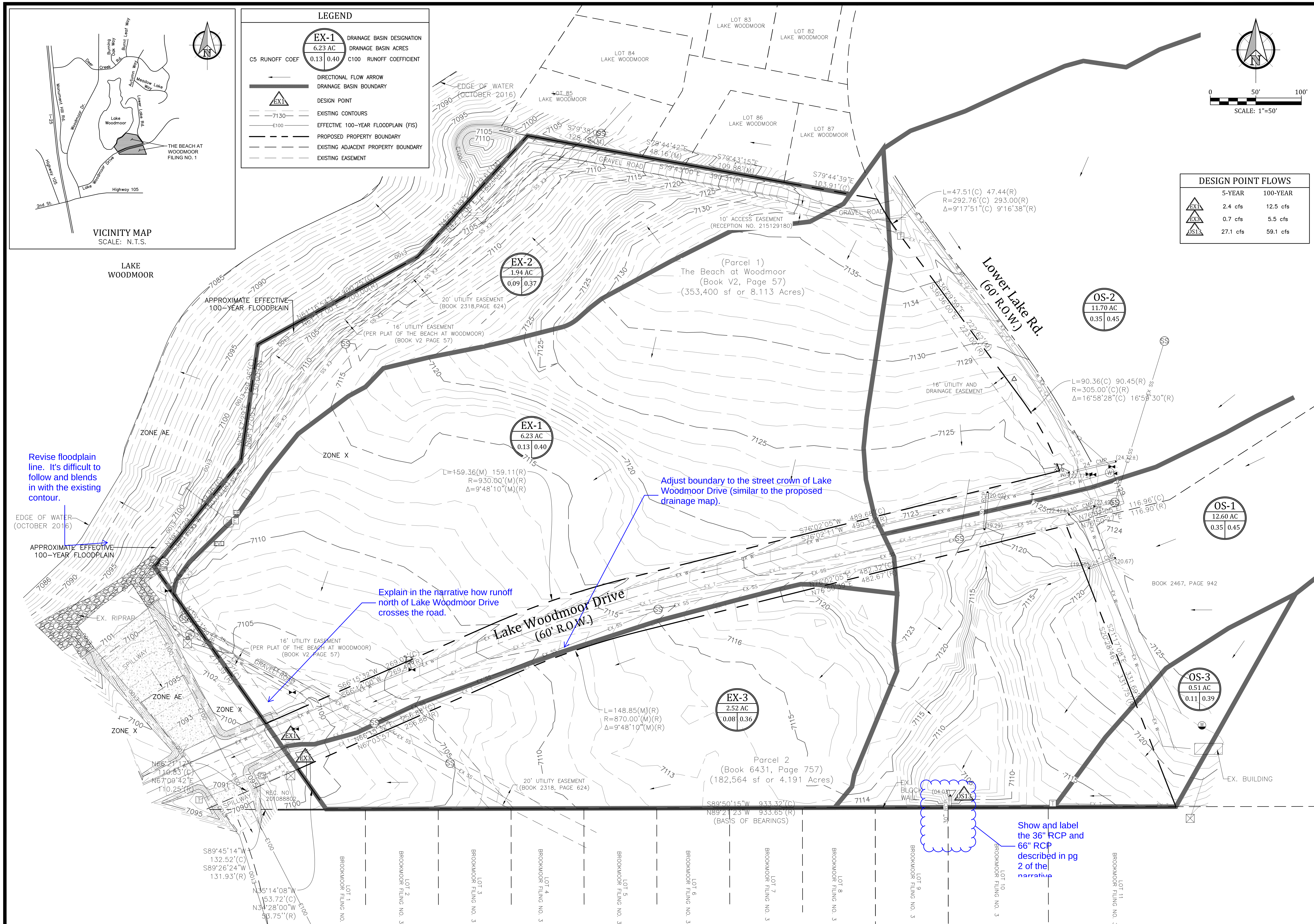
Project No.	91-07-17
Date:	1/93
Design:	AWMc
Drawn:	EAK
Check:	RNW
Revisions:	

LF1

APPENDIX E

Existing and Proposed Drainage Plans

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



LEGEND

EX-1	DRAINAGE BASIN DESIGNATION
6.23 AC	DRAINAGE BASIN ACRES
C5 RUNOFF COEF 0.13 / 0.40 C100	RUNOFF COEFFICIENT
←	DIRECTIONAL FLOW ARROW
▬	DRAINAGE BASIN BOUNDARY
△ EX1	DESIGN POINT
—7130—	EXISTING CONTOURS
—E100—	EFFECTIVE 100–YEAR FLOODPLAIN (FIS)
— — —	PROPOSED PROPERTY BOUNDARY
— — —	EXISTING ADJACENT PROPERTY BOUNDARY
— — —	EXISTING EASEMENT

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

Revise floodplain line. It's difficult to follow and blends in with the existing contour.

Adjust boundary to the street crown of Lake Woodmoor Drive (similar to the proposed drainage map).

Explain in the narrative how runoff north of Lake Woodmoor Drive crosses the road.

Show and label the 36" RCP and 66" RCP described in pg 2 of the narrative

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The Beach at Woodmoor Filing No. 1

Existing Condition Drainage Plan

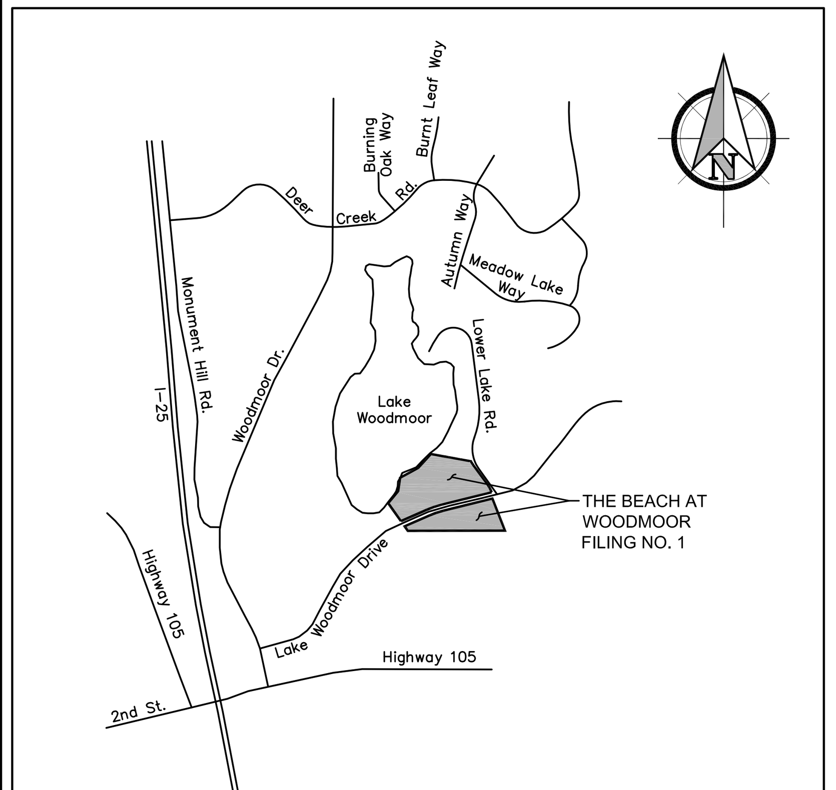
El Paso County, Colorado

Project No.:	16059
Date:	May 24, 2017
Design:	JAK
Drawn:	JAK
Check:	CJC
Revisions:	

SHEET

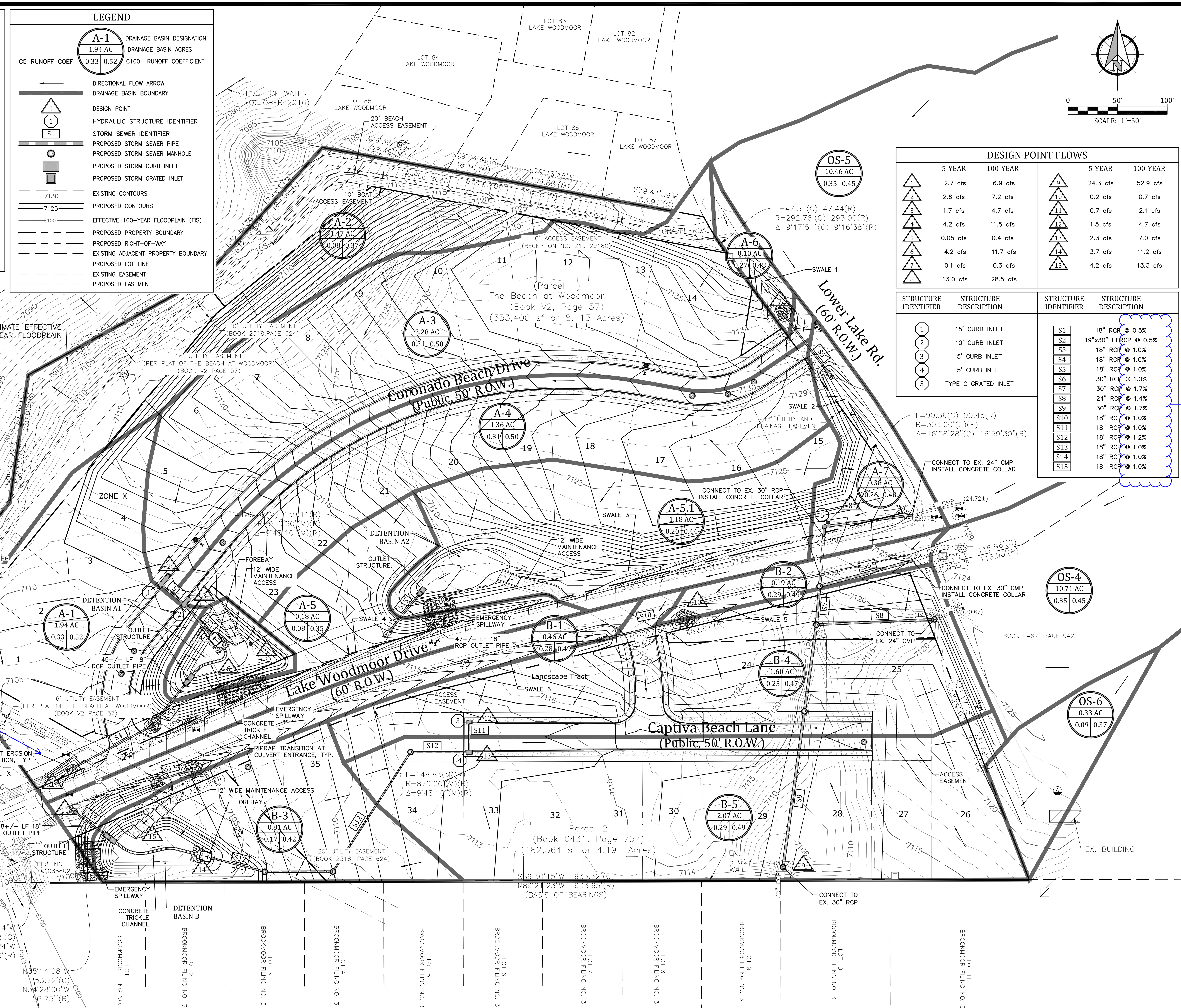
DP1

16059 Drainage Plan.dwg/May 22, 2017



LEGEND

A-1	DRAINAGE BASIN DESIGNATION
1.94 AC	DRAINAGE BASIN ACRES
C5 RUNOFF COEF 0.33 / 0.52 C100	RUNOFF COEFFICIENT
←	DIRECTIONAL FLOW ARROW
	DRAINAGE BASIN BOUNDARY
△ 1	DESIGN POINT
⬡ 1	HYDRAULIC STRUCTURE IDENTIFIER
S1	STORM SEWER IDENTIFIER
	PROPOSED STORM SEWER PIPE
	PROPOSED STORM SEWER MANHOLE
	PROPOSED STORM CURB INLET
	PROPOSED STORM GRATED INLET
—7130—	EXISTING CONTOURS
—7125—	PROPOSED CONTOURS
—E100—	EFFECTIVE 100-YEAR FLOODPLAIN (FIS)
	PROPOSED PROPERTY BOUNDARY
	PROPOSED RIGHT-OF-WAY
	EXISTING ADJACENT PROPERTY BOUNDARY
	PROPOSED LOT LINE
	EXISTING EASEMENT
	PROPOSED EASEMENT



DETENTION BASIN A1 DATA	WATER SURFACE ELEV. (FT)	REQUIRED VOLUME	RELEASE RATE
WQCV	7107.0	0.06 AC–FT	
EURV	7108.4	0.16 AC–FT	
100–YR	7110.1	0.32 AC–FT	4.5 CFS
100–YR + ½ WQCV	7110.3	0.35 AC–FT	4.5 CFS
SPILLWAY CREST ELEVATION: 7110.35			
TOP OF EMBANKMENT MINIMUM ELEVATION: 7111.0			

DETENTION BASIN A2 DATA	WATER SURFACE ELEV. (FT)	REQUIRED VOLUME	RELEASE RATE
WQCV	7117.0	0.01 AC–FT	
EURV	7117.4	0.02 AC–FT	
100–YR	7118.2	0.07 AC–FT	1.2 CFS
100–YR + ½ WQCV	7118.3	0.08 AC–FT	1.2 CFS
SPILLWAY CREST ELEVATION: 7118.7			
TOP OF EMBANKMENT MINIMUM ELEVATION: 7120.0			

DETENTION BASIN B DATA	WATER SURFACE ELEV. (FT)	REQUIRED VOLUME	RELEASE RATE
WQCV	7099.7	0.06 AC–FT	
EURV	7100.8	0.15 AC–FT	
100–YR	7102.5	0.35 AC–FT	5.4 CFS
100–YR + ½ WQCV	7012.7	0.38 AC–FT	5.4 CFS
SPILLWAY CREST ELEVATION: 7102.8			
TOP OF EMBANKMENT MINIMUM ELEVATION: 7104.0			

DESIGN POINT FLOWS

Design Point	5-YEAR	100-YEAR	Design Point	5-YEAR	100-YEAR
1	2.7 cfs	6.9 cfs	9	24.3 cfs	52.9 cfs
2	2.6 cfs	7.2 cfs	10	0.2 cfs	0.7 cfs
3	1.7 cfs	4.7 cfs	11	0.7 cfs	2.1 cfs
4	4.2 cfs	11.5 cfs	12	1.5 cfs	4.7 cfs
5	0.05 cfs	0.4 cfs	13	2.3 cfs	7.0 cfs
6	4.2 cfs	11.7 cfs	14	3.7 cfs	11.2 cfs
7	0.1 cfs	0.3 cfs	15	4.2 cfs	13.3 cfs
8	13.0 cfs	28.5 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

STRUCTURE IDENTIFIER	STRUCTURE DESCRIPTION
S1	18" RCP @ 0.5%
S2	19"x30" HERCP @ 0.5%
S3	18" RCP @ 1.0%
S4	18" RCP @ 1.0%
S5	18" RCP @ 1.0%
S6	30" RCP @ 1.0%
S7	30" RCP @ 1.7%
S8	24" RCP @ 1.4%
S9	30" RCP @ 1.7%
S10	18" RCP @ 1.0%
S11	18" RCP @ 1.0%
S12	18" RCP @ 1.2%
S13	18" RCP @ 1.0%
S14	18" RCP @ 1.0%
S15	18" RCP @ 1.0%

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The Beach at Woodmoor Filing No. 1

Developed Condition Drainage Plan

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Project No.:	16059
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Check:	CJC
Revisions:	

SHEET

DP2

16059 Drainage Plan.dwg/May 22, 2017