Run-on and Runoff Control System Plan

Public Service Company of Colorado, an Xcel Energy Company

Hayden Station Ash Disposal Facility
Project No. 98286

December 2018
Run-on and Runoff Control System Plan

prepared for

Public Service Company of Colorado,
an Xcel Energy Company
Hayden Station Ash Disposal Facility
Hayden, Colorado

Project No. 98286

December 2018

prepared by

Burns & McDonnell Engineering Company, Inc.
Denver, Colorado

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INDEX AND CERTIFICATION

Public Service Company of Colorado, an Xcel Energy Company
Run-on and Runoff Control System Plan
Project No. 98286

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Qualified Professional Engineer Certification

In accordance with 40 CFR Section 257.81(c)(5), I hereby certify, as a Professional Engineer in the State of Colorado, that the information in this document was assembled under my direct personal charge and that the information presented herein is accurate to the best of my knowledge. This plan is not intended or represented to be suitable for reuse by Public Service Company of Colorado, an Xcel Energy Company or others without specific verification or adaptation by the Engineer.

Bradley A. Coleman, PE
Colorado License No. 28723
Date: 12/20/2018
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<th>Abbreviation</th>
<th>Term/Phrase/Name</th>
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<tr>
<td>CCR landfill</td>
<td>Ash Disposal Facility</td>
</tr>
<tr>
<td>CCR</td>
<td>coal combustion residual</td>
</tr>
<tr>
<td>CFR</td>
<td>Code of Federal Regulations</td>
</tr>
<tr>
<td>HDPE</td>
<td>High density polyethylene</td>
</tr>
<tr>
<td>NDPES</td>
<td>National Pollutant Discharge Elimination System</td>
</tr>
<tr>
<td>PSCo</td>
<td>Public Service Company of Colorado, an Xcel Energy Company</td>
</tr>
<tr>
<td>RCRA</td>
<td>Resource Conservation and Recovery Act</td>
</tr>
<tr>
<td>USEPA</td>
<td>U.S. Environmental Protection Agency</td>
</tr>
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</table>
1.0 INTRODUCTION

This Run-on and Runoff Control System Plan has been prepared for the existing coal combustion residual (CCR) landfill located at the Public Service Company of Colorado (PSCo), an Xcel Energy Company, Hayden Station. Hayden Station is a 446-megawatt coal-fired, steam turbine power plant owned and operated by PSCo. The station is located at 13125 U.S. Highway 40, Hayden, Colorado 81639. The CCR landfill location is shown on Figure 1-1.

This Run-on and Run-off Control System Plan was prepared to comply with the requirements of the US Environmental Protection Agency’s (USEPA’s) CCR Rule for disposal of ash under Subtitle D of the Resource Conservation and Recovery Act (RCRA). The final rule was published in the Federal Register, Volume 80 Number 74 on April 17, 2015, and became effective on October 19, 2015.

1.1 Facility Description

The CCR landfill is located on Routt County Road 27 approximately one mile south of Colorado Highway 40 in Routt County, Colorado. A facility site plan is included as Figure 1-2. The CCR landfill is in an unincorporated portion of Routt County. The location is approximately 5 miles east of Hayden, Colorado and 20 miles west of Steamboat Springs, Colorado. The CCR landfill is generally located in the west half of Section 16 and the east half of Section 17, Township 6 North, Range 87 West of the 6th Principal Meridian, Routt County, Colorado. The service area for the CCR landfill is limited to the Hayden Station power plant. Wastes are not accepted from any other source.

The CCR landfill is an unlined ash monofill. The wastes accepted at the CCR landfill consist of coal ash, air emission control byproducts, water intake silt, excavation soils and coal impurities. The area inside the permitted boundary of the CCR landfill consists of approximately 154 acres of which approximately 136 acres is used for ash disposal and approximately 18 acres for storm water control structures, access roads, and borrow area.

1.2 Regulatory Requirements

40 Code of Federal Regulation (CFR) Section 257.81 requires PSCo to design, construct, operate, and maintain the following for the CCR landfill:

- A run-on control system to prevent flow onto the active portion of the CCR unit during the peak discharge from a 24-hour, 25-year storm.
• A runoff control system from the active portion of the CCR unit to collect and control at least the water volume resulting from a 24-hour, 25-year storm.

• A runoff control system designed to handle runoff so that it does not cause a discharge of pollutants to waters of the United States that is in violation of the requirements of the National Pollutant Discharge Elimination System (NPDES) under Section 402 of the Clean Water Act.

Furthermore, PSCo must prepare a run-on and runoff control systems plan documenting how the run-on and runoff systems meet these requirements. This plan shall be reviewed and amended and recertified as needed and at a minimum of every five years.
2.0 RUN-ON / RUNOFF CONTROLS FOR CCR LANDFILL

2.1 Site Setting
As shown on Figure 1-2, the CCR landfill has been constructed on the eastern side of a valley, abutting the western flank of an upland area. There are two streams near the CCR landfill. Sage Creek is located on the valley floor, directly west of the facility. Grassy Creek is located in an adjacent watershed on the eastern side of the upland area. Both drainages flow in a northerly direction toward the Yampa River, which is one mile north of the CCR landfill. The CCR landfill was constructed by placing ash behind earthen berms in horizontal lifts with the east side abutting against the hill side. Perimeter earthen berms are constructed in five-foot vertical increments on the other sides. The perimeter berm is constructed just ahead of ash placement such that all ash is contained within the perimeter berms and hillside.

Design Drawings are attached as Appendix A. Calculations of peak flow, channel sizing and shear resistance, and detention pond sizing are attached as Appendix B.

2.2 Storm Water Controls
The CCR landfill is located above the surrounding grades on all sides except the eastern portion, which abuts the hillside. The run-on controls consist of a diversion berm to intercept storm water run-on and direct the flow around the CCR landfill towards the southern detention pond as depicted in the Design Drawings in Appendix A. Benches constructed within the perimeter berms divert storm water runoff to channels constructed down the side slopes of the CCR landfill and direct channelized flow into detention ponds below the CCR landfill. Final storm water controls will be constructed concurrently with the final cover to direct flow to detention ponds located along the perimeter of the CCR landfill, outside of the ash disposal boundary. Storm water is and will be discharged from the detention ponds toward Sage Creek to the east via storm water controls along County Road 27 and the access road north of the CCR landfill. On-site runoff is characteristically intermittent. There are no other significant, natural surface water bodies within two miles of the CCR landfill.

2.2.1 Existing Benches
As of this writing, the exterior slopes on the west and southwest portion of the CCR landfill have been constructed and are well vegetated. Benches throughout the majority of this area are do not intercept free drainage off the slope above and route it laterally. They instead are sloped to allow drainage to continue on down over the slope below the benches. These slopes are routinely inspected for vegetation adequacy and signs of erosion and have not shown any over the last five years. Based on this, it was determined that grading the benches to create lateral drainage was not necessary and the runoff can continue to sheet flow.
off the side slopes without impacting the vegetated slopes. Channels G and H were constructed on the lower benches of the southwest perimeter berm area to capture runoff from the western portion of the CCR landfill and convey the flows to detention ponds along County Road 27 (see Drawing C002 in Appendix A).

### 2.2.2 Future Benches

Some current and all future perimeter berms include benches constructed to direct drainage perpendicular to the final cover side slopes. These benches are 20 feet wide with a minimum of five percent cross slope toward the CCR landfill to direct storm water to the interior of the bench. Benches will be grass-lined and have a minimum longitudinal slope of a half percent to direct flow toward downslope channels. Design details are included in Appendix A and the peak runoff calculations are included in Appendix B-1.

### 2.2.3 Drainage Channels

Downslope channels are constructed intermittently to collect and transport storm water runoff from the CCR landfill to detention ponds outside the CCR landfill boundary. Perimeter channels either are constructed or will be constructed along the north, east, and south boundaries of the CCR landfill to direct flow to the downslope channels. Channels will be lined with grass or an erosion mat adequate for flows and velocities associated with the 100-year, 24-hour storm event. Design details are included in Appendix A and the peak runoff calculations are included in Appendix B-1.

### 2.2.4 Detention Ponds

A total of four detention ponds will be constructed along the perimeter of the CCR landfill, outside of the limits of ash to received non-contact storm water from the downslope channels; two west of the CCR landfill (which are already constructed) and two north of the CCR landfill, which have not been constructed. Detention ponds are designed to treat and manage volumes associated with the 100-year, 24-hour storm event per Section 5.0 of the City of Steamboat Springs Engineering Standards (Steamboat Springs, 2007). Detention pond outlet structures will direct storm water from the detention ponds to existing storm water controls along County Road 27 and the CCR landfill access road. Design details are included in Appendix A. Detention pond storage calculations are included in Appendix B-2.

### 2.3 Contact Water Pond

A lined contact water pond was constructed at the CCR landfill to receive runoff from the active placement cells. The pond will remain in service for the life of the CCR landfill and will contain the contact water for use in dust control and moisture conditioning within the active cells. It has been sized to contain runoff from a 25-year, 24-hour storm from a 11-acre active cell area with 3.3 feet of freeboard.
The pond is lined using a composite liner system consisting of a geosynthetic clay liner overlain by a smooth 60-mil high density polyethylene (HDPE) geomembrane. The pond is sloped to the south at 0.5-percent and an HDPE sump is installed at the low point of the pond for placement of a pump to draw water or clean the pond. The contact water pond plan view and details are shown on Drawing C003 in Appendix B. Contact water pond storage calculations are included in Appendix B-3.

2.4 Site Development Plan

Site development plans have been generated to show progress of the CCR landfill construction through the remaining 20-year life of the CCR landfill. These drawings are representative of estimated conditions approximately every six to seven years based on average annual ash disposal volumes received to date.

As can be seen on the drawings, ash placement and perimeter berm construction in Phase 1 will focus on the northern portion of the CCR landfill until that area is filled even with the remainder of the CCR landfill. Then the surface of the CCR landfill will be brought up evenly until final ash grades are achieved. Stormwater run-on and runoff controls have been designed to accommodate these conditions from the current existing condition through final buildout.
3.0 REVISIONS

In accordance with 40 CFR Section 257.81(c)(2) and (4), the run-on and runoff control system plan amendments and updates will be prepared at a minimum of every five years.
4.0 REFERENCES

City of Steamboat Springs. 2007. *City of Steamboat Springs Engineering Standards*.


FIGURES
APPENDIX A – CCR LANDFILL DESIGN DRAWINGS
1. ASH PLACEMENT CELLS SHALL BE A MAXIMUM OF 10 ACRES. RUNOFF FROM ACTIVE CELLS SHALL BE COLLECTED AND CONVEYED TO THE CONTACT WATER POND. ONLY ONE CELL SHALL DRAIN TO CONTACT WATER POND AT ANY GIVEN TIME. OTHER CELLS SHALL BE COVERED WITH DAILY COVER AND RUNOFF SHALL BE DIRECTED TO STORMWATER CHANNELS.
NOTE:

1. THE HEIGHTS OF THE LOWER BERM SHOWED IN THESE EDOP DRAWINGS VARY AS NEEDED TO CREATE POSITIVE DRAINAGE TO STORMWATER FEATURES. LOWER BERM LENGTHS WILL VARY BETWEEN 60 AND 90 HORIZONTAL FEET AT A SLOPE OF 3H:1V. SUBSEQUENT BERMS SHALL BE CONSTRUCTED TO ACHIEVE A TOTAL LENGTH OF 60 FEET AT A SLOPE OF 3H:1V.

2. CONTINUE PLACEMENT OF COMPACTED CLAY MATERIAL TO THE VERTICAL EXTENT OF ASH PLACEMENT.

3. DAILY COVER REQUIRED ON TOP DECK FOR AREAS NOT DRAINING TO THE CONTACT WATER POND.
<table>
<thead>
<tr>
<th>Channel Liner Requirements</th>
<th>Channel Liner</th>
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<tr>
<td>Channel (see C001)</td>
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</tr>
<tr>
<td>Channel (see C002)</td>
<td>Channel Liner</td>
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<tr>
<td>Channel (see C003)</td>
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<tr>
<td>Channel (see C004)</td>
<td>Channel Liner</td>
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<tr>
<td>Channel (see C005)</td>
<td>Channel Liner</td>
</tr>
<tr>
<td>Channel (see C006)</td>
<td>Channel Liner</td>
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</tbody>
</table>

**NOTES:**
1. CMP END SECTION SHALL BE A STANDARD "TACK ON" APRON THAT WILL FIT THE SPECIFIED PIPE DIAMETER. MATERIAL, DIMENSIONS, AND INSTALLATIONS SHALL MEET CDOT SPECIFICATIONS.
2. ALL PONDS SHALL HAVE A BOTTOM SLOPE OF 0.5% OR GREATER TOWARD THE OUTLET STRUCTURE.
3. STORMRAX™ SHALL HAVE SUPPORTING T CROSS BARS WITH FASTENERS ATTACHED TO TRASH RACK.
4. RIPRIP FILTER STONE SHALL BE PLACED AGAINST THE PIPE TO AN ELEVATION JUST BELOW THE TOP ROW ALL AROUND PIPE. INSTALL AND MAINTAIN PIPE AS PLUMB AND TRUE.
5. FLEXMAT™ CHANNEL LINING SHALL EXTEND THE FULL WIDTH OF THE CHANNEL TO THE TOP OF THE CHANNEL.

**OUTLET STRUCTURE DETAILS:**

<table>
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<tr>
<th>Name</th>
<th>Invert Elevation</th>
<th>Diameter (D1)</th>
<th>Height (L)</th>
<th>No. of Rows</th>
<th>Top of Stand Pipe Elevation</th>
<th>Diameter (D2)</th>
<th>Gravel</th>
<th>Material</th>
<th>Invert Out</th>
<th>Spillway Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOPD1</td>
<td>6485.0 ft</td>
<td>24&quot;</td>
<td>0.0 ft</td>
<td>6</td>
<td>5485.0 ft</td>
<td>32&quot;</td>
<td>CMP</td>
<td>6483.0</td>
<td>6487.0</td>
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</table>

**TOPSHEET 0205, C012**
### Scale for Microfilming

- **Inches**: Scale
- **Millimeters**: Scale

### Design Drawings

**Project Details**
- **Name**: Hayden Station Ash Disposal Facility
- **Design Drawings**: 98286
- **File Name**: 98286C012DET3.DWG
- **Copyright**: © Burns & McDonnell Engineering Company, Inc.

**NOTES**:
1. All pipes shall be installed in accordance with CDOT specifications.
2. Contractor shall excavate unsuitable material and replace with material as specified by the engineer. As an alternative and at the discretion of the engineer, the trench bottom may be stabilized using a geotextile material.
3. CMP end section shall be a standard "TACK ON" APRON that will fit the specified pipe diameter. Material, dimensions, and installations shall meet CDOT specifications.

### Culvert Details

<table>
<thead>
<tr>
<th>Name</th>
<th>Invert In</th>
<th>Invert Out</th>
<th>Type</th>
<th>Length</th>
<th>Quantity</th>
<th>Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert 1</td>
<td>6608.9'</td>
<td>6608.6'</td>
<td>CMP</td>
<td>30.0'</td>
<td>3</td>
<td>13&quot; x 17&quot; elliptical</td>
</tr>
<tr>
<td>Culvert 2</td>
<td>6645.9'</td>
<td>6644.9'</td>
<td>CMP</td>
<td>100.0'</td>
<td>1</td>
<td>30&quot; Ø</td>
</tr>
<tr>
<td>Culvert 3</td>
<td>6627.2'</td>
<td>6626.2'</td>
<td>CMP</td>
<td>63.0'</td>
<td>1</td>
<td>24&quot; Ø</td>
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</table>

### Spillway Details

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<tr>
<th>Outlet Structure</th>
<th>Width of Bottom Flow (W1)</th>
<th>Width of Spillway Elevation (W2)</th>
<th>Spillway Elevation</th>
<th>Top of Pond Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pond 1</td>
<td>27</td>
<td>27</td>
<td>6487.7'</td>
<td>6485.7'</td>
</tr>
<tr>
<td>Pond 2</td>
<td>11</td>
<td>10</td>
<td>6692.8'</td>
<td>6691.8'</td>
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</tbody>
</table>

**NOTES:**
- **Final Backfill**: 2' min.
- **Embedment Bedding**: Suitable foundation (see Note 2)
- **Suitable Foundation**: Not to scale
- **SPILLWAY SECTION**: See diagrams
- **SPILLWAY ELEVATION**: See Table
- **SPILLWAY ELEVATION (SEE TABLE)**: Reinforced pad with #5 @ 12" O.C. each way @ 3" clear from top of concrete, typ.
- **SPILLWAY ELEVATION**: See Table
- **SPILLWAY SECTION**: Not to scale
- **SPILLWAY SECTION**: See Diagram
- **SPILLWAY SECTION**: See Diagram
- **SPILLWAY SECTION**: See Diagram

**DESIGNER**:
- **M. Gonzales**: 303-721-9292
- **B. Coleman**: 303-721-9292

**REV**:
- **C010**: 0

**MATERIALS**:
- **Culverts**: FLEXAMAT™ with turf reinforcement mat underlay
- **Concrete Cutoff Wall**: 3' min.
- **Daylight with Existing Grade**: W1 (See Table)
- **Top of Pond Elevation (See Outlet Structure Table)**
- **Top of Pond Elevation**: 2'
- **Top of Pond Elevation**: 1'
- **Top of Channel**: CMP (see Note 1)
- **Typical Pipe Bedding**: CMP (see Note 1)
- **Springline**: Not to scale
- **Concrete Cutoff Wall**: 3' min.

**HAYDEN, COLORADO**

**Hayden Station Ash Disposal Facility**

**DETAILS 3**
APPENDIX B – DESIGN CALCULATIONS
APPENDIX B-1 – PEAK RUNOFF
Purpose: The purpose of this calculation package is to calculate the peak flows for the drainage basins shown on Figure 1 at Hayden during the 100-year, 24 hour storm event.

Background: The Hayden ash disposal facilities (ADF) design being completed as part of the engineering, design and operations plan (EDOP). Stormwater on the ADF is currently conveyed through existing drainage channels. Through each phase of development additional channels will be constructed to accommodate peak flows from all ADF drainage basins.

Methodology: The SCS Runoff Curve Number Method, in accordance with the NRCS TR-55 Manual and Steamboat Springs Drainage Criteria will be used to estimate peak runoff flows for each drainage area shown on Figure 1 after final cover operations have been completed.

Notes:  

<table>
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<th>Notes</th>
<th>0 = Data Input Cell</th>
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<tbody>
<tr>
<td></td>
<td>0 = Calculated/Reference Cell</td>
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</tbody>
</table>

Attachments:  
1. Drainage Area Map (Figure 1)  
2. City of Steamboat Springs Engineering Standards. (2007). (Attachment 1)  

Conclusions:

<table>
<thead>
<tr>
<th>Basin ID</th>
<th>Area (Acre)</th>
<th>Peak Flows (cfs)</th>
<th>T_e (hr)</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>20.7</td>
<td>19.0</td>
<td>0.40</td>
</tr>
<tr>
<td>A1</td>
<td>2.9</td>
<td>2.7</td>
<td>0.40</td>
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<tr>
<td>B</td>
<td>30.2</td>
<td>33.5</td>
<td>0.27</td>
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<tr>
<td>B1</td>
<td>21.7</td>
<td>28.4</td>
<td>0.19</td>
</tr>
<tr>
<td>C</td>
<td>14.1</td>
<td>12.2</td>
<td>0.48</td>
</tr>
<tr>
<td>C1</td>
<td>11.4</td>
<td>10.5</td>
<td>0.39</td>
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<tr>
<td>C2</td>
<td>2.1</td>
<td>2.4</td>
<td>0.26</td>
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<tr>
<td>D</td>
<td>80.5</td>
<td>52.6</td>
<td>0.72</td>
</tr>
<tr>
<td>D1</td>
<td>37.9</td>
<td>22.0</td>
<td>0.88</td>
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<td>D2</td>
<td>16.0</td>
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<td>0.42</td>
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<tr>
<td>D3</td>
<td>11.5</td>
<td>12.5</td>
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<tr>
<td>Bench</td>
<td>9.0</td>
<td>8.8</td>
<td>0.35</td>
</tr>
</tbody>
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Prepared By: Mark Gonzales, EIT  Date: 10/10/2017  
Checked By: Joshua Lee, PE  Date: 1/15/2018  
Approved By:  Date: 3/21/18
Calculation by: MDG     date: 10/10/2017
Checked by: JLL     date: 1/15/2018

### Weighted Curve Number

Weighted Curve Number, $CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}}$

```
<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
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<tbody>
<tr>
<td>C</td>
<td>Herbaceous- Fair</td>
<td>81</td>
<td>20.7</td>
<td>1,680</td>
</tr>
</tbody>
</table>
```

$CN = \frac{20.74 \times 1,680}{20.74} = 81$

### Time of Concentration

#### Sheet Flow

- Elevation range: 6,695 to 6,681 ft (From AutoCAD)
- $\Delta = 14$ ft
- Flow Length, $L = 300$ ft (From AutoCAD)
- Slope, $s = 4.67\%$
- Flow Resistance Coefficient, $k = 0.150$
- Sheet Flow travel time, $t_i = 17.72$ min
- $t_i = 0.30$ hr

#### Channelized Flow

- Grassed Waterways
- Elevation range: 6,681 to 6,626 ft (From AutoCAD)
- $\Delta = 55$ ft
- Watercourse Slope, $S_w = 0.045$ ft/ft
- Conveyance Coefficient, $C_v = 15$
- Flow Length, $L = 1,233$ feet
- Flow Length, $L = 1,233$ feet
- Channelized Flow time, $t_t = 6.49$ min
- $t_t = 0.11$ hr

Time of Concentration, $T_c = t_i + T_o$

### Peak Discharge

- Drainage Area, $A = 0.0324$ mi$^2$ (See Above)
- Rainfall Distribution Type
- Storm Frequency
- Storm Rainfall Depth, $P = 2.80$ in (See Attachment 1-4)
- Potential max retention after runoff begins, $S = (1000/CN) - 10$, in
- Initial abstraction, $I_a = 0.2S$, in
- Depth of runoff over entire watershed, $Q = (P - I_a)^2 / [(P - I_a) + S]$, in
- Initial abstraction / Rainfall Depth, $I_a / P = 0.17$
- Unit peak discharge, $Q_u = 505$ csm/in
- Pond and swamp adjustment factor, $F_p$
- $F_p = 1.0$

$Q_p = \text{Total Discharge from Watershed} = Q_u A Q F_p$, cfs

$Q_p = 19.0$ cfs
### Drainage Basin ID

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Herbacious- Fair</td>
<td>81</td>
<td>2.9</td>
<td>236</td>
</tr>
</tbody>
</table>

#### Calculations

### Time of Concentration

- **Sheet Flow**
  - Elevation range: 6,668 ft to 6,648 ft
  - Δ = 20 ft
  - Flow Length, L = 64 ft
  - Slope, s = 31.25%
  - Flow Resistance Coefficient, k = 0.150
  - Sheet Flow travel time, \( t_i = \frac{1.8 \times (1.1 - K) \times L^5}{S^{1/3}} \)
  - \( t_i = 4.34 \) min
  - \( t_i = 0.07 \) hr

- **Channelized Flow**
  - Elevation range: 6,648 ft to 6,640 ft
  - Watercourse Slope, \( s_w = 0.006 \) ft/ft
  - Conveyance Coefficient, \( C_v = 15 \)
  - Velocity, \( V = \frac{1.15 \times L}{1,353} \) ft/sec
  - Flow Length, L = 1,353 ft
  - Channelized Flow Time, \( t_t = \frac{L}{V \times 60} \)
  - \( t_t = 19.55 \) min
  - \( t_t = 0.33 \) hr

- **Total Time of Concentration**, \( T_c = T_i + T_o \)
  - \( T_c = 0.40 \) hr

### Peak Discharge

- **Drainage Area**, \( A = 0.0045 \) mi²
- **Rainfall Distribution Type**
- **Storm Frequency**
- **Storm Rainfall Depth**, \( P = 2.80 \) in
- **Initial abstraction**, \( I_a = 0.25 S \), in
- **Potential max retention after runoff begins**,
  - \( S = (1000/CN) - 10 \), in
  - Initial abstraction, \( I_a = 0.25 \), in
- **Depth of runoff over entire watershed**, \( Q = (P - I_t)^2 / [(P - I_t) + S] \), in
  - Initial abstraction / Rainfall Depth, \( I_t / P = 0.17 \)
- **Unit peak discharge**, \( Q_u = 505 \) csm/in
- **Pond and swamp adjustment factor**, \( F_p = 1.0 \)

\[ Q_p = \text{Total Discharge from Watershed} = Q_u A Q F_p \text{, cfs} \]

See Attachment 1, Attachment 1-1

\( CN = 81 \)
Hyden Station Ash Disposal Facility
PEAK RUNOFF CALCULATIONS

Calculation by: MDG  date: 10/10/2017
Checked by: JLL       date: 1/15/2018

Drainage Basin ID  B

<table>
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<tr>
<th>Hydrologic Soil Group</th>
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<th>Curve Number</th>
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<tr>
<td>C</td>
<td>Herbacious- Fair</td>
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<td>30.2</td>
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<tr>
<td></td>
<td>Total</td>
<td></td>
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<td>2,450</td>
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</table>

See Attachment 1, Attachment 1-1

Weighted Curve Number, \(CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}}\)

\(CN = 81\)

**Time of Concentration**

- Sheet Flow elevation range
- Elevation difference, \(\Delta\)
- Flow Length, \(L\)
- Slope, \(s = \Delta \text{ efov} / \text{length}\)
- Flow Resistance Coefficient, \(k\)
- Sheet Flow travel time, \(t_i = \frac{1.8 \times (1.1 - K) x L^5}{S^{1/3}}\)

**Upstream**

- \(6,624\) ft
- \(84\%\)
- \(300\) ft
- \(28.00\%\)
- \(9.75\) min
- \(0.16\) hr

**Downstream**

- \(6,540\) ft
- \(15\)
- \(3.62\) ft/sec
- \(1,444\) ft
- \(6.65\) min
- \(0.11\) hr

**Conveyance Coefficient, \(C_v\)**

\(Grassed Waterways\)

- Watercourse Slope, \(S_w\)
- Velocity, \(V = C_v \times S_w^{1/2}\)
- Flow Length, \(L\)
- Channelized Flow time, \(t_t = \frac{L}{V \times 60}\)

**Time of Concentration, \(T_c = T_t + T_o\)**

- \(0.27\) hr

**Peak Discharge**

- Drainage Area, \(A, \text{ mi}^2\)
- Storm Frequency
- Storm Rainfall Depth, \(P, \text{ in}\)
- Potential max retention after runoff begins, \(S = \frac{(1000/CN) - 10}{\text{in}}\)
- Initial abstraction, \(I_a = 0.2S, \text{ in}\)
- Depth of runoff over entire watershed, \(Q = \frac{(P - I_t)^2}{[P - I_a + S]}\)
- Initial abstraction / Rainfall Depth, \(I_a / P\)
- Unit peak discharge, \(Q_p, \text{ csm/in}\)
- Pond and swamp adjustment factor, \(F_p\)

\(Q_p = \text{Total Discharge from Watershed} = Q_a \times A \times Q_p \times F_p, \text{ cfs}\)

\(Q_p = 33.5\)
### Drainage Basin ID

#### Hydrologic Soil Group

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
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<tbody>
<tr>
<td>C</td>
<td>Herbaceous-Fair</td>
<td>81</td>
<td>21.7</td>
<td>1,762</td>
</tr>
</tbody>
</table>

**Totals**

- **Area (ac)**: 21.7
- **Curve Number x Area**: 1,762

See Attachment 1, Attachment 1-1

#### Weighted Curve Number

\[ CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}} \]

- **CN**: 81

### Time of Concentration

#### Sheet Flow

- **Elevation range**: 6,663 to 6,643 ft (From AutoCAD)
- **Elevation difference, \( \Delta \)**: 20 ft
- **Flow Length, \( L \)**: 68 ft (From AutoCAD)
- **Slope, \( s \)**: 29.41%
- **Flow Resistance Coefficient, \( k \)**: 0.150
- **Sheet Flow travel time, \( t_i \)**: 4.57 min
- **Sheet Flow travel time, \( t_i \)**: 0.08 hr

#### Channelized Flow

- **Elevation range**: 6,643 to 6,456 feet (From AutoCAD)
- **Elevation difference, \( \Delta \)**: 187 ft
- **Watercourse Slope, \( s_w \)**: 0.095 ft/ft
- **Conveyance Coefficient, \( C_v \)**: 15
- **Velocity, \( V \)**: 4.62 ft/sec (From AutoCAD)
- **Flow Length, \( L \)**: 1,960 feet
- **Channelized Flow time, \( t_t \)**: 7.10 min
- **Channelized Flow time, \( t_t \)**: 0.12 hr

**Time of Concentration, \( T_c = t_i + T_o \)**

- **\( T_c \)**: 0.19 hr

### Peak Discharge

#### Drainage Area, \( A \)

\[ A = \frac{0.0340}{1} \]  

See Attachment 2-1

#### Storm Frequency

\[ P = 2.80 \]  

See Attachment 1-4

#### Storm Rainfall Depth, \( S \)

\[ S = 2.35 \]  

See Attachment 1-4

#### Initial abstraction, \( I_a \)

\[ I_a = 0.47 \]  

See Attachment 1-5

#### Potential max retention after runoff begins, \( S = (1000/CN) - 10 \), in

\[ S = 2.35 \]  

See Attachment 1-5

#### Initial abstraction / Rainfall Depth, \( I_a / P \)

\[ I_a / P = 0.17 \]  

See Attachment 2-2

#### Unit peak discharge, \( Q_u \)

\[ Q_u = 720 \]  

See Attachment 2-2

#### Pond and swamp adjustment factor, \( F_p \)

\[ F_p = 1.0 \]  

See Attachment 2-3

#### \( Q_P \)

\[ Q_P = \text{Total Discharge from Watershed} = Q_u A Q F_p, \text{ cfs} \]

\[ Q_P = 28.4 \]  

See Attachment 2-3
### Drainage Basin ID

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
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<tr>
<td>C</td>
<td>Herbaceous- Fair</td>
<td>81</td>
<td>14.1</td>
<td>1,144</td>
</tr>
</tbody>
</table>

Totals => 14.1 x 1,144

Weighted Curve Number, CN = Curve Number x Area / Total Area

CN = 81

### Time of Concentration

**Sheet Flow**

- **Elevation range**: 6,694–6,679 ft (From AutoCAD)
- **Elevation difference, Δ**: 15 ft
- **Flow Length, L**: 300 ft (From AutoCAD)
- **Slope, s**: 5.00%
- **Flow Resistance Coefficient, k**: 0.150 (See Attachment 1-2)
- **Sheet Flow travel time, t_s**: 17.32 min
- **Sheet Flow time, t_s**: 0.29 hr

**Channelized Flow**

- **Elevation range**: 6,679–6,641 ft (From AutoCAD)
- **Elevation difference, Δ**: 38 ft
- **Watercourse Slope, S_w**: 0.023 ft/ft (See Attachment 1-3)
- **Conveyance Coefficient, C_v**: 15
- **Velocity, V**: 2.30 ft/sec
- **Flow Length, L**: 1,622 feet (From AutoCAD)
- **Channelized Flow Time, t_c**: 11.77 min
- **Channelized Flow time, t_c**: 0.20 hr

**Time of Concentration, T_c = t_s + T_0**

- **T_c**: 0.48 hr

### Peak Discharge

**Drainage Area, A, mi²**

- **A**: 0.0221 (See Above)

**Rainfall Distribution Type**

- **P**: 2.80
- **S**: 2.35
- **I_a**: 0.47
- **Q**: 1.16
- **I_a/P**: 0.17
- **Q_a**: 475
- **F_p**: 1.0

**Depth of runoff over entire watershed, Q = (P - I_a)^2 / [(P - I_a) + S]**

- **Q**: 12.2 (See Attachment 2-3)
### Calculation by: MDG  
**date:** 10/10/2017

### Checked by: JLL  
**date:** 1/15/2018

#### Drainage Basin ID

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
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<td>Herbaceous-Fair</td>
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<td>11.4</td>
<td>919</td>
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</table>

**Totals**: 11.4 x 919

See Attachment 1, Attachment 1-1

---

**Weighted Curve Number**, $CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}}$

<table>
<thead>
<tr>
<th></th>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation range</td>
<td>6,694 ft</td>
<td>6,679 ft</td>
</tr>
<tr>
<td>$\Delta$, ft</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Flow Length, L</td>
<td>300 ft</td>
<td>300 ft</td>
</tr>
<tr>
<td>Slope, $s$, %</td>
<td>5.00</td>
<td>5.00</td>
</tr>
<tr>
<td>Flow Resistance Coefficient, k</td>
<td>0.150</td>
<td>0.150</td>
</tr>
<tr>
<td>Sheet Flow travel time, $t_i$, min</td>
<td>17.32 min</td>
<td>17.32 min</td>
</tr>
<tr>
<td>$t_i$, hr</td>
<td>0.29</td>
<td>0.29</td>
</tr>
</tbody>
</table>

See Attachment 1-2

---

**Time of Concentration**

<table>
<thead>
<tr>
<th></th>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation range</td>
<td>6,679 ft</td>
<td>6,645 ft</td>
</tr>
<tr>
<td>$\Delta$, ft</td>
<td>34</td>
<td>34</td>
</tr>
<tr>
<td>Watercourse Slope, $S_w$</td>
<td>0.034 ft/ft</td>
<td>0.034 ft/ft</td>
</tr>
<tr>
<td>Conveyance Coefficient, $C_v$</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Velocity, $V$, ft/sec</td>
<td>2.78 ft/sec</td>
<td>2.78 ft/sec</td>
</tr>
<tr>
<td>Flow Length, L</td>
<td>990 ft</td>
<td>990 ft</td>
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<tr>
<td>Channelized Flow Time, $t_t$, min</td>
<td>5.94 min</td>
<td>5.94 min</td>
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<tr>
<td>$t_t$, hr</td>
<td>0.10</td>
<td>0.10</td>
</tr>
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</table>

**Grassed Waterways**

**Time of Concentration**, $T_c = T_t + T_o$

**Peak Discharge**

<table>
<thead>
<tr>
<th></th>
<th>A = 0.0177</th>
<th>P = 2.80</th>
<th>S = 2.35</th>
<th>I$_a$ = 0.47</th>
<th>Q = 1.16</th>
<th>I$_a$/P = 0.17</th>
<th>Q$_b$ = 510</th>
<th>F$_o$ = 1.0</th>
<th>Q$_p$ = 10.5</th>
</tr>
</thead>
</table>

**A =** Drainage Area, $\text{mi}^2$

**P =** Rainfall Distribution Type

**S =** Storm Frequency

**I$_a$ =** Storm Rainfall Depth, P, in

**Q =** Potential max retention after runoff begins, $S = \frac{(1000/CN) - 10}{\text{in}}$, in

**I$_a$/P =** Initial abstraction / Rainfall Depth, $I_a / P$

**Q$_b$ =** Unit peak discharge, $Q_a \times \text{csm/in}$

**F$_o$ =** Pond and swamp adjustment factor, $F_p$

**Q$_p$ =** Total Discharge from Watershed = $Q_a \times A \times F_p$, cfs

See Attachment 2-1, Attachment 1-4, Attachment 1-5, Attachment 1-5, Attachment 2-2, Attachment 2-3
Drainage Basin ID: C2

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
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</thead>
<tbody>
<tr>
<td>C</td>
<td>81</td>
<td>2.1</td>
<td>169</td>
</tr>
</tbody>
</table>

Totals: CN = 81

Weighted Curve Number, CN = Curve Number x Area / Total Area

**Time of Concentration**

Sheet Flow elevation range

- Upstream: 6,691 ft
- Downstream: 6,681 ft

Elevation difference, Δ

- Upstream: 10 ft
- Downstream: 205 ft

Flow Length, L

- Upstream: 205 ft
- Downstream: 10 ft

Slope, s = Δ elev / length

- Upstream: 4.88%
- Downstream: 0.150

Flow Resistance Coefficient, k

- Upstream: 0.150
- Downstream: See Attachment 1-2

Sheet Flow travel time,

\[ t_s = \frac{1.8 \times (1.1 - K)x L^5}{S^{1/3}} \]

- Upstream: 14.44 min
- Downstream: 0.24 hr

Channelized Flow elevation range

- Upstream: 6,681 ft
- Downstream: 6,649 ft

Elevation difference, Δ

- Upstream: 32 ft
- Downstream: 0.104 ft/ft

Watercourse Slope, S_w

- Upstream: 0.104
- Downstream: See Attachment 1-3

Conveyance Coefficient, C_v

- Upstream: 15
- Downstream: See Attachment 1-3

Velocity, V = C_v x S_w^0.5

- Upstream: 4.83 ft/sec
- Downstream: See Attachment 1-3

Flow Length, L

- Upstream: 308 ft
- Downstream: (From AutoCAD)

Channelized Flow Time,

\[ t_c = \frac{L}{V \times 60} \]

- Upstream: 1.06 min
- Downstream: 0.02 hr

Time of Concentration, T_c = T_t + T_o

\[ T_c = 0.26 \text{ hr} \]

**Peak Discharge**

Drainage Area, A, mi^2

- A = 0.0033

Rainfall Distribution Type

- See Attachment 2-1

Storm Frequency

- See Attachment 1-4

Storm Rainfall Depth, P, in

- P = 2.80

Initial abstraction, I_a = 0.2S, in

- I_a = 2.35

Initial abstraction / Rainfall Depth, I_a / P

- I_a / P = 0.47

Potential max retention after runoff begins, S = (1000/CN) - 10, in

- S = 2.35

Initial abstraction / Rainfall Depth, I_a / P

- I_a / P = 0.47

Unit peak discharge, Q_u csm/in

- Q_u = 1.16

Pond and swamp adjustment factor, F_p

- F_p = 0.17

Q_p = Total Discharge from Watershed = Q_u A Q F_p, cfs

- Q_p = 2.4
### Curve Number Table

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
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<tbody>
<tr>
<td>C</td>
<td>Herbacious- Fair</td>
<td>81</td>
<td>83.9</td>
<td>6,796</td>
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</table>

**Totals (for Drainage Basin ID D)**

<p>| | | |</p>
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<th></th>
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</thead>
<tbody>
<tr>
<td><strong>Weighted Curve Number, CN</strong></td>
<td><strong>81</strong></td>
<td><strong>6,796</strong></td>
</tr>
</tbody>
</table>

### Time of Concentration

**Sheet Flow**

- Elevation range: 6,650 - 6,636 ft (From AutoCAD)
- Elevation difference, Δ: 14 ft
- Flow Length, L: 300 ft (From AutoCAD)
- Slope, s = Δ elev / length
- Flow Resistance Coefficient, k: 0.150
- Sheet Flow travel time, \( t_s = \frac{1.8 \times (1.1 - K) \times L^5}{S^{1/3}} \)
- \( t_s = 17.72 \text{ min} \)
  
**Downstream**

- Elevation range: 6,636 - 6,472 feet (From AutoCAD)
- Elevation difference, Δ: 164 ft
- Watercourse Slope, \( S_w = 0.045 \text{ ft/ft} \)
- Conveyance Coefficient, \( C_v = 15 \) Grassed Waterways
- Velocity, \( V = C_v \times S_w^{0.5} \)
- Flow Length, L: 3,613 feet (From AutoCAD)
- Channelized Flow Time, \( t_t = \frac{L}{V \times 60} \)
- \( t_t = 18.84 \text{ min} \)
  
**Combined Time of Concentration**

\[ T_c = t_s + t_t \]

\[ T_c = 17.72 \text{ min} + 18.84 \text{ min} = 36.56 \text{ min} = 0.61 \text{ hr} \]

### Peak Discharge

**Drainage Area, A, mi²**

- \( A = 0.1311 \) (See Above)

**Rainfall Distribution Type**

- See Attachment 2-1

**Storm Frequency**

- See Attachment 1-4

**Storm Rainfall Depth, P, in**

- \( P = 2.80 \) (See Attachment 1-4)

**Potential max retention after runoff begins, S = (1000/CN) - 10, in**

- See Attachment 1-5

**Initial abstraction, I_a = 0.2S, in**

- See Attachment 1-5

**Depth of runoff over entire watershed, Q = (P - I_a)^2 / [(P - I_a) + S], in**

- See Attachment 2-2

**Initial abstraction / Rainfall Depth, I_a / P**

- \( I_a / P = 0.17 \) (See Attachment 2-2)

**Unit peak discharge, Q_u csm/in**

- See Attachment 2-3

**Pond and swamp adjustment factor, F_p**

- \( F_p = 1.0 \) (See Attachment 2-3)

**Q_p = Total Discharge from Watershed = Q_u A Q_{p} c f s**

- \( Q_p = 54.8 \)
Drainage Basin ID: D1

<table>
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<th>Cover Type</th>
<th>Curve Number</th>
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<tbody>
<tr>
<td>C</td>
<td>Herbaceous- Fair</td>
<td>81</td>
<td>41.3</td>
<td>3,345</td>
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</table>

See Attachment 1, Attachment 1-1

Weighted Curve Number, $CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}}$

$CN = 81$

**Time of Concentration**

Sheet Flow elevation range
Elevation difference, $\Delta$
Flow Length, $L$
Slope, $s = \frac{\Delta \text{ elev}}{\text{length}}$
Flow Resistance Coefficient, $k$
Sheet Flow travel time, $t_i = \frac{1.8 \times (1.1 - K) \times L^5}{S^{1/3}}$

Upstream  | Downstream
---|---
6,650 | 6,636

(From AutoCAD)

$\Delta = 14$ ft
$L = 300$ ft
$s = 4.67\%$
$k = 0.150$
$t_i = 17.72$ min

See Attachment 1-2

$\Delta = 0.150$
$t_i = 0.30$ hr

See Attachment 1-2

Channelized Flow elevation range
Elevation difference, $\Delta$
Watercourse Slope, $S_w$
Conveyance Coefficient, $C_v$
Velocity, $V = C_v \times S_w^{0.5}$
Flow Length, $L$
Channelized Flow Time, $t_t = \frac{L}{V \times 60}$

Grassed Waterways

Upstream  | Downstream
---|---
6,636 | 6,603 ft

(From AutoCAD)

$\Delta = 33$ ft
$s = 0.015$ ft/ft
$C_v = 15$
$V = 1.81$ ft/sec
$L = 2,274$ feet

(From AutoCAD)

$t_t = 20.97$ min
$t_t = 0.35$ hr

(From AutoCAD)

$T_c = 0.64$ hr

**Peak Discharge**

Drainage Area, $A$, mi$^2$
Rainfall Distribution Type
Storm Frequency
Storm Rainfall Depth, $P$, in
Potential max retention after runoff begins, $S = (1000/CN) - 10$, in
Initial abstraction, $I_a = 0.2S$, in
Depth of runoff over entire watershed, $Q = (P - I_a)^2 / [(P - I_a) + S]$, in
Initial abstraction / Rainfall Depth, $I_a / P$
Unit peak discharge, $Q_u$, csm/in
Pond and swamp adjustment factor, $F_p$

$Q_p = \text{Total Discharge from Watershed} = Q_u \times A \times F_p$, cfs

See Attachment 2-1

$A = 0.0645$ (See Above)

See Attachment 1-4

$P = 2.80$ (See Attachment 1-4)

$S = 2.35$ (See Attachment 1-5)

$I_a = 0.47$ (See Attachment 1-5)

$Q = 1.16$ (See Attachment 1-5)

$I_a / P = 0.17$ (See Attachment 2-2)

$Q_u = 320$ (See Attachment 2-2)

$F_p = 1.0$ (See Attachment 2-3)

$Q_p = 24.0$
Hayden Station Ash Disposal Facility  
PEAK RUNOFF CALCULATIONS  

<table>
<thead>
<tr>
<th>Curve Number</th>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
</tr>
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<tbody>
<tr>
<td>C</td>
<td>Herbaceous- Fair</td>
<td>81</td>
<td>16.0</td>
<td>1,292</td>
</tr>
</tbody>
</table>

**Weighted Curve Number, CN = Curve Number x Area / Total Area**

CN = [Calculation]

**Time of Concentration**

**Sheet Flow elevation range**

Elevation difference, Δ
Flow Length, L
Slope, s = Δ elev / length
Flow Resistance Coefficient, k
Sheet Flow travel time,

\[ t_f = \frac{1.8 \times (1.1 - K) \times L^5}{S^{1/3}} \]

**Upstream**

<table>
<thead>
<tr>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,689</td>
<td>6,675</td>
</tr>
</tbody>
</table>

(From AutoCAD)

**Channelized Flow elevation range**

Elevation difference, Δ
Watercourse Slope, S_w
Conveyance Coefficient, C_v
Velocity, V = C_v x S_w^{0.5}
Flow Length, L
Channelized Flow Time,

\[ t_t = \frac{L}{V \times 60} \]

**Upstream**

<table>
<thead>
<tr>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,675</td>
<td>6,608</td>
</tr>
</tbody>
</table>

(From AutoCAD)

**Peak Discharge**

Drainage Area, A, mi²
Rainfall Distribution Type
Storm Frequency
Storm Rainfall Depth, P, in
Potential max retention after runoff begins, S = (1000/CN) - 10, in
Initial abstraction, I_a = 0.2S, in
Depth of runoff over entire watershed, Q = (P - I_a)^2 / [(P - I_a) + S], in
Initial abstraction / Rainfall Depth, I_a / P
Unit peak discharge, Q_u, csm/in
Pond and swamp adjustment factor, F_p

\[ Q_p = \text{Total Discharge from Watershed} = Q_u \times A \times F_p, \text{ cfs} \]

[Attachment 1, Attachment 1-1]
## Drainage Basin ID

**D3**

### Curve Number

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Herbaceous-Fair</td>
<td>81</td>
<td>11.5</td>
<td>931</td>
</tr>
</tbody>
</table>

**Totals**

|               | 11.5 | 931 |

Weighted Curve Number, CN = Curve Number x Area / Total Area

CN = **81**

### Time of Concentration

#### Sheet Flow

- **Elevation range**
- **Elevation difference, Δ**: 6 ft
- **Flow Length, L**: 110 ft
- **Slope, s**: 5.45%
- **Flow Resistance Coefficient, k**: 0.150
- **Sheet Flow travel time, t_s**: 10.19 min
- **Sheet Flow travel time, t_s** (hr): 0.17

#### Channelized Flow

- **Elevation range**: 6,681 ft to 6,675 ft
- **Elevation difference, Δ**: 6 ft
- **Watercourse Slope, S_w**: 0.050 ft/ft
- **Conveyance Coefficient, C_v, Grassed Waterways**: 15
- **Velocity, V**: 3.36 ft/sec
- **Flow Length, L**: 1,436 feet
- **Channelized Flow Time, t_c**: 7.13 min
- **Channelized Flow Time, t_c** (hr): 0.12
- **Time of Concentration, T_c**: 0.29 hr

### Peak Discharge

#### Rainfall Distribution Type

- **Type**: II

#### Storm Frequency

- **P**: 100 years
- **S**: 2.80 in
- **I_a**: 0.47 in/hr
- **Q**: 1.16 cfs
- **I_a / P**: 0.17
- **Q_o**: 600 cfs
- **F_o**: 1.0

**Q_p** = Total Discharge from Watershed = Q_o * A * Q_p

**Q_p** = 12.5 cfs
Hayden Station Ash Disposal Facility
PEAK RUNOFF CALCULATIONS

Calculation by: MDG
date: 10/10/2017

Checked by: JLL
date: 1/15/2018

Drainage Basin ID

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Cover Type</th>
<th>Curve Number</th>
<th>Area (ac)</th>
<th>Curve Number x Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Herbacious- Fair</td>
<td>81</td>
<td>9.0</td>
<td>728</td>
</tr>
</tbody>
</table>

Totals: CN = 9.0 x 728

See Attachment 1, Attachment 1-1

Weighted Curve Number, CN = Curve Number x Area / Total Area

<table>
<thead>
<tr>
<th>Time of Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet Flow elevation range</td>
</tr>
</tbody>
</table>
Elevation difference, Δ
Flow Length, L
Slope, s = Δ elev / length
Flow Resistance Coefficient, k
Sheet Flow travel time, \( t_i = \frac{1.8 \times (1.1 - K)x L^5}{S^{1/3}} \)

<table>
<thead>
<tr>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,695</td>
<td>6,680</td>
</tr>
<tr>
<td>Δ = 15 ft</td>
<td>(From AutoCAD)</td>
</tr>
<tr>
<td>L = 300 ft</td>
<td>(From AutoCAD)</td>
</tr>
<tr>
<td>s = 5.00%</td>
<td></td>
</tr>
<tr>
<td>k = 0.150</td>
<td>See Attachment 1-2</td>
</tr>
<tr>
<td>( t_i = 17.32 ) min</td>
<td></td>
</tr>
<tr>
<td>( t_i = 0.29 ) hr</td>
<td></td>
</tr>
</tbody>
</table>

| Channelized Flow elevation range |
Elevation difference, Δ
Watercourse Slope, \( S_w \)
Conveyance Coefficient, \( C_v \)
Grassed Waterways
Velocity, \( V = C_v x S_w^{0.5} \)
Flow Length, L
Channelized Flow Time, \( t_t = \frac{L}{V \times 60} \)

<table>
<thead>
<tr>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,680</td>
<td>6,650</td>
</tr>
<tr>
<td>Δ = 30 ft</td>
<td>(From AutoCAD)</td>
</tr>
<tr>
<td>s = 0.042 ft/ft</td>
<td>(From AutoCAD)</td>
</tr>
<tr>
<td>( C_v = 15 )</td>
<td>See Attachment 1-3</td>
</tr>
<tr>
<td>( V = 3.07 ) ft/sec</td>
<td>See Attachment 1-3</td>
</tr>
<tr>
<td>L = 718 feet</td>
<td>(From AutoCAD)</td>
</tr>
<tr>
<td>( t_t = 3.90 ) min</td>
<td></td>
</tr>
<tr>
<td>( t_t = 0.07 ) hr</td>
<td></td>
</tr>
</tbody>
</table>

Time of Concentration, \( T_c = T_i + T_o \)

<table>
<thead>
<tr>
<th>Peak Discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Area, A, ( \text{mi}^2 )</td>
</tr>
<tr>
<td>Rainfall Distribution Type</td>
</tr>
<tr>
<td>Storm Frequency</td>
</tr>
<tr>
<td>Storm Rainfall Depth, P, in</td>
</tr>
<tr>
<td>Potential max retention after runoff begins, ( S = (1000/CN) - 10 ), in</td>
</tr>
<tr>
<td>Initial abstraction, ( I_a = 0.2S ), in</td>
</tr>
<tr>
<td>Depth of runoff over entire watershed, ( Q = (P - I_a)^2 / [(P - I_a) + S] ), in</td>
</tr>
<tr>
<td>Initial abstraction / Rainfall Depth, ( I_s / P )</td>
</tr>
<tr>
<td>Unit peak discharge, ( Q_u \text{ csm/in} )</td>
</tr>
<tr>
<td>Pond and swamp adjustment factor, ( F_p )</td>
</tr>
</tbody>
</table>

\( Q_P = \text{Total Discharge from Watershed} = Q_u \times A \times F_p \), cfs

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A = 0.0140</td>
<td>(See Above)</td>
</tr>
<tr>
<td>P = 2.80</td>
<td>See Attachment 2-1</td>
</tr>
<tr>
<td>S = 2.35</td>
<td>See Attachment 1-4</td>
</tr>
<tr>
<td>I_s = 0.47</td>
<td>See Attachment 1-4</td>
</tr>
<tr>
<td>Q = 1.16</td>
<td>See Attachment 1-5</td>
</tr>
<tr>
<td>( I_s / P = 0.17 )</td>
<td>See Attachment 1-5</td>
</tr>
<tr>
<td>( Q_o = 540 )</td>
<td>See Attachment 2-2</td>
</tr>
<tr>
<td>( F_o = 1.0 )</td>
<td>See Attachment 2-2</td>
</tr>
</tbody>
</table>

\( Q_o = 8.8 \)
SECTION 5.0
DRAINAGE CRITERIA

SEE SEPARATE DOCUMENT

Effective September, 2007

City of Steamboat Springs
Department of Public Works
124 10th Street
Steamboat Springs, CO 80487
(970) 879-2060

Prepared By:
WRC Engineering, Inc.
950 S Cherry Street, Suite 404
Denver, CO 80246
### Table 5.6.5 Additional Runoff Curve Numbers for Arid and Semiarid Rangelands

<table>
<thead>
<tr>
<th>Cover Type</th>
<th>Hydrologic Condition</th>
<th>Runoff Curve Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Herbaceous – mixture of grass, weeds and low-growing brush, with brush the minor element</td>
<td>Poor</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>62</td>
</tr>
<tr>
<td>Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush</td>
<td>Poor</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
</tr>
<tr>
<td>Pinyon-juniper – pinyon, juniper, or both; grass understory</td>
<td>Poor</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>41</td>
</tr>
<tr>
<td>Sage-grass – sage with an understory of grass</td>
<td>Poor</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>35</td>
</tr>
<tr>
<td>Desert shrub – major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, paloverde, mesquite, and cactus</td>
<td>Poor</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>49</td>
</tr>
</tbody>
</table>


#### 5.6.2.3.3 Sub-watershed Sizing

The determination of the peak rate of runoff at a given design point is affected by the number of sub-watersheds within a larger watershed. Typically, the more sub-watersheds that are used to define a larger watershed, the more representative the resulting peak flow is of actual runoff conditions. The improved predictive capability of multiple sub-watersheds is due to better homogeneity of the sub-watershed characteristics, as compared to analysis of the watershed with no sub-watersheds. Recommended guidelines are:

- For watersheds up to 100 acres in size, the maximum sub-watershed size should be approximately 20 acres.
- For watersheds over 100 acres in size, increasingly larger sub-watersheds may be used as long as the land use and surface characteristics within each sub-watershed are homogeneous. In addition, the sub-watershed sizing should be consistent with the level of detail needed to determine peak flow rates at various design points within a given watershed.

#### 5.6.2.3.4 Parameters

For the basin portion of the HEC-HMS model, the designer shall use SCS methodology unless site conditions specifically indicate some other method. A brief discussion shall be submitted as part of the required drainage studies indicating the various methodologies and parameters that were utilized in the basin model including, but not limited to, loss, transform, baseflow, imperviousness, curve numbers, and initial abstraction.
The time of concentration for both urban and non-urban areas is calculated as follows:

\[ t_c = t_i + t_t \]  

(5.6.1)

Where:
- \( t_c \) = Time of concentration (min)
- \( t_i \) = Initial, Inlet, or overland flow time (min)
- \( t_t \) = Travel time in the ditch, channel, gutter, storm drain, etc. (min)

The initial or overland flow time, \( t_i \), may be calculated using the following equation:

\[ t_i = 1.8 \left(1.1 - K\right) \frac{L_o^{1/2}}{S^{1/3}} \]  

(5.6.2)

Where:
- \( t_i \) = Initial or Overland Flow Time (min)
- \( K \) = Flow Resistance Coefficient
- \( L_o \) = Length of Overland Flow, (ft, 300-ft maximum)
- \( S \) = Average Watershed Slope (percent)

Equation 5.6.2 was originally developed for use with the Rational Formula method. The 5-year runoff coefficient, \( C_5 \), presented in Table 5.6.1 is recommended for the flow resistance coefficient, \( K \).

**Table 5.6.1 Design Runoff Coefficients**

<table>
<thead>
<tr>
<th>Percentage Imperviousness</th>
<th>Runoff Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2-yr</td>
</tr>
<tr>
<td>0%</td>
<td>0.04</td>
</tr>
<tr>
<td>5%</td>
<td>0.08</td>
</tr>
<tr>
<td>10%</td>
<td>0.11</td>
</tr>
<tr>
<td>15%</td>
<td>0.14</td>
</tr>
<tr>
<td>20%</td>
<td>0.17</td>
</tr>
<tr>
<td>25%</td>
<td>0.20</td>
</tr>
<tr>
<td>30%</td>
<td>0.22</td>
</tr>
<tr>
<td>35%</td>
<td>0.25</td>
</tr>
<tr>
<td>40%</td>
<td>0.28</td>
</tr>
<tr>
<td>45%</td>
<td>0.31</td>
</tr>
<tr>
<td>50%</td>
<td>0.34</td>
</tr>
<tr>
<td>55%</td>
<td>0.37</td>
</tr>
<tr>
<td>60%</td>
<td>0.41</td>
</tr>
<tr>
<td>65%</td>
<td>0.45</td>
</tr>
<tr>
<td>70%</td>
<td>0.49</td>
</tr>
<tr>
<td>75%</td>
<td>0.54</td>
</tr>
<tr>
<td>80%</td>
<td>0.60</td>
</tr>
<tr>
<td>85%</td>
<td>0.66</td>
</tr>
<tr>
<td>90%</td>
<td>0.73</td>
</tr>
<tr>
<td>95%</td>
<td>0.80</td>
</tr>
<tr>
<td>100%</td>
<td>0.89</td>
</tr>
</tbody>
</table>
The overland flow length, \( L_o \), is generally defined as the length over which the flow characteristics appear as sheet flow or very shallow flow in broad, grassed swales. Changes in land slope, surface characteristics, and small drainage ditches or gullies will tend to force the overland flow into a combined flow condition, which results in higher flow velocities and shorter travel times. The initial flow time in both urban and non-urban areas shall be limited to the time to travel a distance of 300 feet.

For watersheds longer than 300 feet, the travel time, \( t_t \), must be added to the overland flow time. Travel time can be calculated using Manning's equation and the hydraulic properties of the storm drain, gutter, swale, ditch, or channel or can be approximated from Equation 5.6.3 and Table 5.6.2:

\[
V = C_v S_w^{0.5} \tag{5.6.3}
\]

Where:
- \( V \) = Velocity, fps
- \( S_w \) = watercourse slope, ft/ft
- \( C_v \) = Conveyance coefficient

The minimum conveyance coefficient, \( C_v \), that shall be used for a developed site shall be 7.0, corresponding to short pasture and lawns.

**Table 5.6.2 Travel Time Conveyance Coefficients**

<table>
<thead>
<tr>
<th>Land Surface</th>
<th>Conveyance Coefficient, ( C_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy meadow</td>
<td>2.5</td>
</tr>
<tr>
<td>Tillage/Field</td>
<td>5.0</td>
</tr>
<tr>
<td>Short pasture and lawns</td>
<td>7.0</td>
</tr>
<tr>
<td>Nearly bare ground</td>
<td>10</td>
</tr>
<tr>
<td>Grassed waterways</td>
<td>15</td>
</tr>
<tr>
<td>Paved areas and shallow swales</td>
<td>20</td>
</tr>
</tbody>
</table>

The time of concentration is then the sum of the initial flow time \( t_i \) and the travel time, \( t_t \). The minimum recommended \( t_c \) for non-urban watersheds is 10 minutes.

5.6.2.1.2 Urbanized Watersheds

Overland flow in urbanized watersheds can occur from the back of the lot to the street, in parking lots, in landscape areas, or within park areas and can be calculated using the procedure described for non-urbanized watersheds. Travel time, \( t_t \), to the first design point or inlet is often determined based on the conveyance coefficient for paved areas and shallow swales, but can be estimated using Manning’s equation.

The time of concentration for the first design point in an urbanized watershed using this procedure should not exceed the time of concentration calculated using Equation 5.6.4, which was developed using rainfall/runoff data collected in urbanized regions (USDCM,1969).

\[
t_c = \frac{L}{180} + 10 \tag{5.6.4}
\]
5.5.1 **INTRODUCTION**

Presented in this Section are design rainfall data for the minor and major storm events. These data are used to determine storm runoff peak flows and volumes in conjunction with the runoff models described in Section 5.6, Storm Runoff. All hydrologic analyses for Steamboat Springs shall utilize the rainfall data presented herein for calculating storm runoff.

5.5.2 **RAINFALL ANALYSIS**

For the City of Steamboat Springs, 24-hour point precipitation values are provided in Table 5.5.1 for various recurrence interval storms. These data are from the NOAA Atlas 2, Volume III, 1973.

**Table 5.5.1 24-Hour Point Rainfall Values for Steamboat Springs**

<table>
<thead>
<tr>
<th>Recurrence Interval</th>
<th>24-Hour Precipitation Depths (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>1.3</td>
</tr>
<tr>
<td>5-year</td>
<td>1.7</td>
</tr>
<tr>
<td>10-year</td>
<td>1.9</td>
</tr>
<tr>
<td>25-year</td>
<td>2.4</td>
</tr>
<tr>
<td>50-year</td>
<td>2.6</td>
</tr>
<tr>
<td>100-year</td>
<td>2.8</td>
</tr>
</tbody>
</table>

5.5.3 **INTENSITY-DURATION-FREQUENCY CURVES FOR RATIONAL METHOD**

Rainfall intensities as a function of storm duration and recurrence interval are provided in Table 5.5.2 and Figure 5.5.1. These values were taken or derived from the values in the NOAA Atlas 2, Volume III, 1973.

**Table 5.5.2 Intensity-Duration-Frequency Values**

<table>
<thead>
<tr>
<th>Storm Duration (min)</th>
<th>Precipitation Intensity – Steamboat Springs (inches/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2-year Recurrence</td>
</tr>
<tr>
<td>5</td>
<td>2.04</td>
</tr>
<tr>
<td>10</td>
<td>1.56</td>
</tr>
<tr>
<td>15</td>
<td>1.32</td>
</tr>
<tr>
<td>30</td>
<td>0.92</td>
</tr>
<tr>
<td>60</td>
<td>0.58</td>
</tr>
</tbody>
</table>
5.6.2.3.1 CN Determination

If the SCS Method is specified for use in the basin model portion of the HEC-HMS model, a soil-cover curve number (CN) is used for computing excess precipitation. The curve number CN is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition.

The soil group is determined from published soil maps for the area, which correlate each soil name with the soil group. Land use and treatment class are determined during field visits or from aerial photographs. Procedures for determining land use and treatment class are found in Chapter 8 of the National Engineering Handbook, Section 4 (SCS, 1985). An antecedent moisture condition II (AMC-II) is recommended for the City of Steamboat Springs.

Having determined the soil group, land use and treatment class, and the antecedent moisture condition, CN values can be determined from Table 5.6.4 below. Additionally, for undeveloped areas, Table 5.6.5 may be used if the site meets any of the more specific cover types listed there. In watersheds with varying land use, a composite CN may also be calculated directly from imperviousness estimates using the following equation.

\[
CN = 98*\text{Imp} + X*(1-\text{Imp})
\]  

(5.6.6)

Where:

- \(\text{Imp}\) = Imperviousness as a decimal
- \(X\) = Adjustment factor based on NRCS Soil Type

<table>
<thead>
<tr>
<th>NRCS Soil Type</th>
<th>Adjustment Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>39</td>
</tr>
<tr>
<td>B</td>
<td>61</td>
</tr>
<tr>
<td>C</td>
<td>74</td>
</tr>
<tr>
<td>D</td>
<td>80</td>
</tr>
</tbody>
</table>

5.6.2.3.2 Losses

Once the curve number is determined, precipitation loss can be determined by first calculating the soil moisture storage deficit and then the initial abstraction using the equations below. The HEC-HMS model will also calculate loss and accumulated runoff when the initial abstraction is entered into the loss tab when using the SCS methodology.

\[
Q = \frac{(P - IA)^2}{(P - IA) + S}
\]  

(5.6.7)

\[
S = \frac{1,000}{CN} - 10
\]  

(5.6.8)

\[
IA = 0.2 \times S
\]  

(5.6.9)

Where:

- \(Q\) = Accumulated Excess (in)
- \(P\) = Accumulated Rainfall Depth (in)
- \(IA\) = Initial abstraction (in)
- \(S\) = Currently Available Soil Moisture Storage Deficit (in)
- \(CN\) = SCS Curve number
Urban Hydrology for Small Watersheds

TR-55
Rainfall data sources
This section lists the most current 24-hour rainfall data published by the National Weather Service (NWS) for various parts of the country. Because NWS Technical Paper 40 (TP-40) is out of print, the 24-hour rainfall maps for areas east of the 105th meridian are included here as figures B-3 through B-8. For the area generally west of the 105th meridian, TP-40 has been superseded by NOAA Atlas 2, the Precipitation-Frequency Atlas of the Western United States, published by the National Ocean and Atmospheric Administration.

East of 105th meridian

West of 105th meridian

Alaska

Hawaii

Puerto Rico and Virgin Islands
Exhibit 4-II  Unit peak discharge ($q_u$) for NRCS (SCS) type II rainfall distribution

- Basin A - 505 csm/in
- Sub Basin A1 - 505 csm/in
- Basin B - 610 csm/in
- Sub Basin B1 - 720 csm/in
- Sub Basin C2 - 625 csm/in
- Sub Basin B2 - 720 csm/in
- Sub Basin C1 - 510 csm/in
- Basin C - 475 csm/in
- Sub Basin D2 - 490 csm/in
- Basin D - 360 csm/in
- Sub Basin D1 - 320 csm/in
- Sub Basin D3 - 600 csm/in
- Sub Basin C - 510 csm/in
- Sub Basin C1 - 505 csm/in
- Sub Basin A1 - 505 csm/in
- Sub Basin Bench - 540 csm/in

Unit peak discharge ($q_u$) (csm/in)

Time of concentration ($T_c$) (hours)
Table 4-2

<table>
<thead>
<tr>
<th>Percentage of pond and swamp areas</th>
<th>Adjustment factor (F_p)</th>
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**Limitations**

The Graphical method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular Hydrograph method (chapter 5). Use TR-20 if the watershed is very complex or a higher degree of accuracy is required.

- The watershed must be hydrologically homogeneous, that is, describable by one CN. Land use, soils, and cover are distributed uniformly throughout the watershed.
- The watershed may have only one main stream or, if more than one, the branches must have nearly equal T_c’s.
- The method cannot perform valley or reservoir routing.
- The F_p factor can be applied only for ponds or swamps that are not in the T_c flow path.
- Accuracy of peak discharge estimated by this method will be reduced if I_a / P values are used that are outside the range given in exhibit 4. The limiting I_a / P values are recommended for use.
- This method should be used only if the weighted CN is greater than 40.
- When this method is used to develop estimates of peak discharge for both present and developed conditions of a watershed, use the same procedure for estimating T_c.
- T_c values with this method may range from 0.1 to 10 hours.

**Example 4-1**

Compute the 25-year peak discharge for the 250-acre watershed described in examples 2-2 and 3-1. Figure 4-2 shows how worksheet 4 is used to compute q_p as 345 cfs.
**Purpose:** The purpose of this calculation package is to evaluate the existing and design detention ponds for the peak flows from the drainage basins shown on Figure 1 at Hayden during the 100-year, 24-hour storm event.

**Background:** The Hayden ash disposal facilities (ADF) design being completed as part of the engineering, design and operations plan (EDOP). Stormwater on the ADF is currently conveyed through existing drainage channels to detention ponds. Through each phase of development additional channels and ponds will be constructed to accommodate peak flows from all ADF drainage basins. Detention Ponds 1 and 2 have already been built as part of ADF operations. Pond 1 does not have an outlet structure. The pond is pumped when needed, and has an emergency spillway for extreme events. Pond 1 is evaluated using the stage storage available from the current geometry and an outlet structure and spillway designed to control the peak flows from the ADF drainage basins. Pond 2 features an outlet structure and spillway. Pond 2 is evaluated using the stage storage from the current geometry and as built survey for the outlet structure and spillway dimensions and elevations.

**Methodology:** Detention ponds were evaluated using the Urban Drainage and Flood Control District (UDFCD) Detention Basin Design Workbook, UD-Detention, Version 3.07. The workbook uses Modified Puls method for outlet routing. A stage storage relationship for each pond was input into the workbook and the outlet structure was sized using rows of small diameter holes for control of smaller storm events and an overflow wier and circular opening for the 100-year, 24-hour storm event. The discharge pipe was evaluated using UDFCD Culvert Hydraulics Workbook Version 3.05 to calculate normal and critical flow conditions in the pipe.

**References:**
1. Drainage Area Map (Figure 1)
3. Urban Drainage and Flood Control District (UDFCD) Detention Basin Design Workbook, UD-Detention v3.07
4. Urban Drainage and Flood Control District (UDFCD) Culvert Hydraulics Workbook, UD-Culvert v3.05

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<th>Conclusions</th>
<th>Pond ID</th>
<th>Drainage Area (acres)</th>
<th>Peak Inflow, 100-year (cfs)</th>
<th>Standpipe Diameter (in)</th>
<th>Standpipe Height (feet)</th>
<th>Perforation Diameter (in)</th>
<th>Number of Perforation Rows</th>
<th>Discharge Orifice Size (in)</th>
<th>Peak Outflow (cfs)</th>
<th>100-year Ponding Depth (feet)</th>
<th>Spillway Height (feet)</th>
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The construction of an outlet structure and new spillway in Detention Pond 1 per the details on the EDOP Drawings will provide control of the peak flows from the ADF. Pond 2 will function as constructed for the ADF final conditions with the addition of a row of perforations at the invert of the structure to allow for complete drainage within 120 hours. Ponds 3 and 4 will provide adequate storage to control peak flows from the ADF.
### Required Volume Calculation

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### Stage-Storage Calculation

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<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Area (ft²)</th>
<th>Area (acres)</th>
<th>Volume (ft³)</th>
<th>Volume (ac-ft)</th>
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Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Detention Basin Outlet Structure Design

Example Zone Configuration (Retention Pond)

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

Calculations Parameters for Underdrain

- Underdrain Orifice Insert Depth = N/A ft (distance below the filtration media surface)
- Underdrain Orifice Diameter = N/A inches
- Depth at top of Zone using Orifice Plate = 2.00 ft (relative to basin bottom at Stage = 0 ft)
- Orifice Plate: Orifice Vertical Spacing = 1.77 sq. inches (diameter = 1-1/2 inches)
- Orifice Plate: Orifice Area per Row = 1.77 sq. inches
- Inflow Hydrograph Volume (acre-ft)
- Calculated Runoff Volume (acre-ft)
- Maximum Volume Stored (acre-ft)
- Predevelopment Peak Q (cfs)
- Peak Outflow Q (cfs)
- One-Hour Rainfall Depth (in)
- Calculated Runoff Volume (acre-ft)

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

Calculations Parameters for Overflow Weir

- Overflow Weir Front Edge Height, Ho = 2.00 ft
- Freeboard above Max Water Surface = 0.50 feet
- Over Flow Weir Slope = 0.00 H:V (enter zero for flat grate)
- Spillway Invert Stage = 3.00 ft (relative to basin bottom at Stage = 0 ft)
- Spillway End Slopes = 1.00 H:V
- Basin Area at Top of Freeboard = 0.63 acres

User Input: Overflow Grate Open Area % = 80% %

Calculations Parameters for Overflow Grate

- Overflow Grate Open Area w/ Debris = N/A ft
- Overflow Grate Open Area w/o Debris = N/A ft

User Input: Vertical Orifice (Circular or Rectangular)

Calculations Parameters for Vertical Orifice

- Vertical Orifice Diameter = N/A inches
- Invert of Vertical Orifice = N/A ft (relative to basin bottom at Stage = 0 ft)
- Vertical Orifice Area = N/A ft

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

Calculations Parameters for Outlet Pipe w/ Flow Restriction Plate

- Outlet Orifice Area = 2.41 ft
- Outlet Orifice Centroid = 0.88 feet
- Half-Central Angle of Restrictor Plate on Pipe = N/A radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Calculations Parameters for Spillway

- Spillway Invert Stage = 3.00 ft (relative to basin bottom at Stage = 0 ft)
- Stage at Top of Freeboard = 4.19 feet
- Basin Area at Top of Freeboard = 0.63 acres

Routed Hydrograph Results

- WQCV
- EURV
- 2 Year
- 5 Year
- 10 Year
- 25 Year
- 50 Year
- 100 Year
- 500 Year

OPTIONAL: Override Runoff Volume (acre-ft)

- Inflow Hydrograph Volume (acre-ft)
- Predicted Unit Peak Flow, q (cfs/acre)
- Predicted Peak Q (cfs)
- Peak Inflow Q (cfs)
- Peak Outflow Q (cfs)
- Ratio Peak Outflow to Predicted Q

Structure Controlling Flow:

- Max Velocity through Grade 1 (fps)
- Max Velocity through Grade 2 (fps)
- Time to Drain 97% of Inflow Volume (hours)
- Time to Drain 99% of Inflow Volume (hours)
- Maximum Ponding Depth (ft)
- Area at Maximum Ponding Depth (acres)
- Maximum Volume Stored (acre-ft)

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

Calculations Parameters for Plate

- WQCV Orifice Area per Row = 1.229-02 ft
- Elliptical Half-Width = N/A ft
- Elliptical Slot Centroid = N/A feet
- Elliptical Slot Area = N/A ft

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

Calculations Parameters for Plate

- Orifice Plate: Orifice Vertical Spacing = N/A inches
- Elliptical Slot Centroid = N/A feet

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

Calculations Parameters for Plate

- Stage of Orifice Centroid (ft)
- Orifice Area (sq. inches)

User Input: Vertical Orifice (Circular or Rectangular)

Calculations Parameters for Vertical Orifice

- Vertical Orifice Area = N/A ft
- Vertical Orifice Centroid = N/A feet

User Input: Vertical Orifice (Circular or Rectangular)

Calculations Parameters for Vertical Orifice

- Vertical Orifice Area = N/A ft
- Vertical Orifice Centroid = N/A feet

User Input: Overdrain Orifice Diameter = N/A inches

Calculations Parameters for Underdrain

- Underdrain Orifice Area + N/A ft
- Underdrain Orifice Centroid + N/A feet

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

Calculations Parameters for Underdrain

- Underdrain Orifice Insert Depth = N/A ft (distance below the filtration media surface)
- Underdrain Orifice Diameter = N/A inches
- Depth at top of Zone using Orifice Plate = 2.00 ft (relative to basin bottom at Stage = 0 ft)
- Orifice Plate: Orifice Vertical Spacing = 1.77 sq. inches (diameter = 1-1/2 inches)
- Orifice Plate: Orifice Area per Row = 1.77 sq. inches
- Inflow Hydrograph Volume (acre-ft)
- Calculated Runoff Volume (acre-ft)
- Maximum Volume Stored (acre-ft)
- Predevelopment Peak Q (cfs)
- Peak Outflow Q (cfs)
- One-Hour Rainfall Depth (in)
- Calculated Runoff Volume (acre-ft)

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

Calculations Parameters for Overflow Weir

- Overflow Weir Front Edge Height, Ho = 2.00 ft
- Freeboard above Max Water Surface = 0.50 feet
- Over Flow Weir Slope = 0.00 H:V (enter zero for flat grate)
- Spillway Invert Stage = 3.00 ft (relative to basin bottom at Stage = 0 ft)
- Spillway End Slopes = 1.00 H:V
- Basin Area at Top of Freeboard = 0.63 acres

Routed Hydrograph Results

- WQCV
- EURV
- 2 Year
- 5 Year
- 10 Year
- 25 Year
- 50 Year
- 100 Year
- 500 Year

OPTIONAL: Override Runoff Volume (acre-ft)

- Inflow Hydrograph Volume (acre-ft)
- Predicted Unit Peak Flow, q (cfs/acre)
- Predicted Peak Q (cfs)
- Peak Inflow Q (cfs)
- Peak Outflow Q (cfs)
- Ratio Peak Outflow to Predicted Q

Structure Controlling Flow:

- Max Velocity through Grade 1 (fps)
- Max Velocity through Grade 2 (fps)
- Time to Drain 97% of Inflow Volume (hours)
- Time to Drain 99% of Inflow Volume (hours)
- Maximum Ponding Depth (ft)
- Area at Maximum Ponding Depth (acres)
- Maximum Volume Stored (acre-ft)
Design Information (Input):

Circular Culvert: Barrel Diameter in Inches
D = 24 inches
Intake Edge Type (choose from pull-down list)
Square End with Headwall

OR:

Box Culvert: Barrel Height (Rise) in Feet
Height (Rise) = ft.
Barrel Width (Span) in Feet
Width (Span) = ft.
Intake Edge Type (choose from pull-down list)
1.5 : 1 Bevel w/ 90 Deg. Headwall
Number of Barrels
No = 1
Inlet Elevation at Culvert Invert
Inlet Elev = 6454 ft. elev.
Outlet Elevation at Culvert Invert OR Slope of Culvert (ft v./ft h.)
Outlet Elev = 6453 ft. elev.
Culvert Length in Feet
L = 50 ft.
Manning’s Roughness
n = 0.024
Bend Loss Coefficient
Kb = 0
Exit Loss Coefficient
Kx = 1

Design Information (calculated):

Entrance Loss Coefficient
Ke = 0.50
Friction Loss Coefficient
Kf = 2.10
Sum of All Loss Coefficients
Ks = 3.60
Orifice Inlet Condition Coefficient
Cd = 0.86
Minimum Energy Condition Coefficient
KElow = -0.0342

Calculations of Culvert Capacity (output):

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<th>Culvert Inlet-Control</th>
<th>Culvert Outlet-Control</th>
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Processing Time: 01.21 Seconds
Required Volume Calculation

Selected BMP Type = EDB

Watershed Area = 83.90 acres
Watershed Length = 4,500 ft
Watershed Slope = 0.020 ft/ft
Watershed Imperviousness = 2.00% percent
Percentage Hydrologic Soil Group A = 0.0% percent
Percentage Hydrologic Soil Group B = 0.0% percent
Percentage Hydrologic Soil Groups C/D = 100.0% percent
Desired WQCV Drain Time = 40.0 hours
Location for 1-hr Rainfall Depths = User Input
Water Quality Capture Volume (WQCV) = 0.106 acre-feet
Excess Urban Runoff Volume (EURV) = 0.123 acre-feet
2-yr Runoff Volume (P1 = 0.48 in.) = 0.039 acre-feet
5-yr Runoff Volume (P1 = 0.66 in.) = 0.257 acre-feet
10-yr Runoff Volume (P1 = 0.82 in.) = 0.926 acre-feet
25-yr Runoff Volume (P1 = 1.24 in.) = 0.533 acre-feet
50-yr Runoff Volume (P1 = 1.44 in.) = 0.535 acre-feet
100-yr Runoff Volume (P1 = 1.97 in.) = 0.885 acre-feet
Approximate 2-yr Detention Volume = 0.037 acre-feet
Approximate 5-yr Detention Volume = 0.250 acre-feet
Approximate 10-yr Detention Volume = 0.443 acre-feet
Approximate 25-yr Detention Volume = 0.533 acre-feet
Approximate 50-yr Detention Volume = 0.536 acre-feet
Approximate 100-yr Detention Volume = 0.885 acre-feet

Stage-Storage Calculation

Zone 1 Volume (WQCV) = 0.106 acre-feet
Zone 2 Volume (100-year - Zone 1) = 0.779 acre-feet
Select Zone 3 Storage Volume (Optional) = 0.886 acre-feet
Total Detention Basin Volume = 0.886 acre-feet
Initial Surcharge Volume (ISV) = user ft³
Initial Surcharge Depth (ISD) = user ft
Total Available Detention Depth (H aval) = user ft
Depth of Trickle Channel (H TC) = user ft
Slope of Trickle Channel (S TC) = user H:V
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 ft²
Surcharge Volume Length (L ISV) = user ft
Surcharge Volume Width (W ISV) = user ft
Depth of Basin Floor (H ISV) = user ft
Length of Basin Floor (LISV) = user ft
Width of Basin Floor (W ISV) = user ft
Area of Basin Floor (A ISV) = user ft²
Volume of Basin Floor (V ISV) = user ft³
Depth of Main Basin (H MBA) = user ft
Length of Main Basin (L MBA) = user ft
Width of Main Basin (W MBA) = user ft
Area of Main Basin (A MBA) = user ft²
Volume of Main Basin (V MBA) = user ft³
Calculated Total Basin Volume (V total) = user acre-feet

Stage - Storage Description | Stage (ft) | Optimal Override Stage (ft) | Length (ft) | Width (ft) | Area (ft²) | Optional Override Area (ft²) | Area (acres) | Volume (ft³) | Volume (ac-ft)
--- | --- | --- | --- | --- | --- | --- | --- | --- | ---
Top of Micropool | -- | 0.00 | -- | -- | -- | 22 | 0.001 | -- | --
-- | 0.43 | -- | -- | -- | -- | 3,563 | 0.082 | 725 | 0.017
-- | 1.43 | -- | -- | -- | -- | 13,570 | 0.312 | 2,901 | 0.017
-- | 2.43 | -- | -- | -- | -- | 15,418 | 0.354 | 23,830 | 0.547
-- | 3.43 | -- | -- | -- | -- | 17,338 | 0.398 | 40,208 | 0.923
-- | 4.43 | -- | -- | -- | -- | 19,390 | 0.444 | 58,557 | 1.344
-- | 5.43 | -- | -- | -- | -- | 21,469 | 0.493 | 78,985 | 1.813
-- | 6.43 | -- | -- | -- | -- | 23,745 | 0.545 | 101,806 | 2.333
-- | 7.43 | -- | -- | -- | -- | 26,106 | 0.599 | 126,531 | 2.905

Stage-Storage Table Builder

Depth increment = 0.2 ft

<table>
<thead>
<tr>
<th>Stage-Storage Description</th>
<th>Stage (ft)</th>
<th>Optimal Override Stage (ft)</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Area (ft²)</th>
<th>Area (acres)</th>
<th>Volume (ft³)</th>
<th>Volume (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Micropool</td>
<td>--</td>
<td>0.00</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>22</td>
<td>0.001</td>
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</tr>
<tr>
<td>--</td>
<td>0.43</td>
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<td>3,563</td>
<td>0.082</td>
<td>725</td>
<td>0.017</td>
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<tr>
<td>--</td>
<td>1.43</td>
<td>--</td>
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<td>--</td>
<td>--</td>
<td>13,570</td>
<td>0.312</td>
<td>2,901</td>
<td>0.017</td>
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<td>2.43</td>
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<td>15,418</td>
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<tr>
<td>--</td>
<td>3.43</td>
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<td>--</td>
<td>17,338</td>
<td>0.398</td>
<td>40,208</td>
<td>0.923</td>
</tr>
<tr>
<td>--</td>
<td>4.43</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>19,390</td>
<td>0.444</td>
<td>58,557</td>
<td>1.344</td>
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<tr>
<td>--</td>
<td>5.43</td>
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<td>--</td>
<td>--</td>
<td>21,469</td>
<td>0.493</td>
<td>78,985</td>
<td>1.813</td>
</tr>
<tr>
<td>--</td>
<td>6.43</td>
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<td>--</td>
<td>--</td>
<td>--</td>
<td>23,745</td>
<td>0.545</td>
<td>101,806</td>
<td>2.333</td>
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<tr>
<td>--</td>
<td>7.43</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>26,106</td>
<td>0.599</td>
<td>126,531</td>
<td>2.905</td>
</tr>
</tbody>
</table>

UD-Detention_v3.07 - Pond 2.xlsm, Basin
UD-Detention, Version 3.07 (February 2017)
3/21/2018, 10:28 AM
Detention Basin Outlet Structure Design

### Calculated Parameters for Underdrain
- **Underdrain Orifice**:
  - Invert of Lowest Orifice: 3.66 ft (relative to basin bottom at Stage = 0 ft)
  - Depth at top of Zone using Orifice Plate: 3.66 ft (relative to basin bottom at Stage = 0 ft)
  - Orifice Plate: Orifice Area per Row: 0.78 sq. inches (diameter = 1 inch)
  - Orifice Vertical Spacing: N/A inches
  - Orifice Area (sq. inches): 0.78

### Calculated Parameters for Plate
- **Orifice Plate**:
  - Orifice Area per Row: 0.78 sq. inches
  - Elliptical Slot Area: N/A ft²
- **Overflow Grate**:
  - Open Area %: 80%

### Calculated Parameters for Spillway
- **Spillway Crest Length**: 5.00 feet
- **Stage at Top of Freeboard**: 10.23 feet

### Calculated Parameters for Overflow Weir
- **Overflow Weir Front Edge Length**: 6.28 feet
- **Horiz. Length of Weir Sides**: 0.00 feet
- **Overflow Grate Open Area %**: 80%
- **Debris Clogging %**: 50%

### Calculated Parameters for Outlet Pipe
- **Outlet Orifice Diameter**: N/A inches
- **Outlet Orifice Centroid**: N/A feet

### Calculated Parameters for Vertical Orifice
- **Vertical Orifice Diameter**: inches
- **Invert of Vertical Orifice**: ft (relative to basin bottom at Stage = 0 ft)
- **Depth at top of Zone using Vertical Orifice**: ft (relative to basin bottom at Stage = 0 ft)

### Calculated Parameters for Flow Restriction Plate
- **Circular Orifice**:
  - Diameter: 24.00 inches
  - Invert of Orifice: 0.00 ft (relative to basin bottom at Stage = 0 ft)
- **Rectangular Orifice**:
  - Width: 0.00 ft

### Calculated Parameters for Overflow Weir Grate
- **Grate Open Area %**: 80%
- **Grate Open Area / 100-yr Orifice Area**: Should be > 1

### Calculated Parameters for Spillway End Slopes
- **Spillway Invert Stage**: 7.48 ft (relative to basin bottom at Stage = 0 ft)
- **Spillway Design Flow Depth**: 1.75 feet

### Table: Example Zone Configuration (Retention Pond)
<table>
<thead>
<tr>
<th>Zone</th>
<th>Stage (ft)</th>
<th>Zone Volume (ac-ft)</th>
<th>Outlet Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1 (WQCV)</td>
<td>3.34</td>
<td>0.779</td>
<td>WorkPipe (Circular)</td>
</tr>
<tr>
<td>Zone 2 (100-year)</td>
<td>1.02</td>
<td>0.106</td>
<td>Orifice Plate</td>
</tr>
</tbody>
</table>

### routed Hydrograph Results
- **WQCV**
  - One-Hour Rainfall Depth (in): 0.53
  - Calculated Runoff Volume (acre-ft): 0.106
  - OPTIONAL Override Runoff Volume (acre-ft): 0.105

- **EYRC**
  - Peak Inflow Volume (ft³): 0.00
  - Peak Inflow Volume (cfs): 0.00

- **Predevelopment Unit Peak Flow, q (cfs/acre)**
  - Peak Inflow Q (cfs): 0.00
  - Peak Inflow Q (cfs): 0.00

- **Ratio Peak Outflow to Predevelopment**
  - Ratio Peak Outflow: 0.00

- **Max Velocity through Grate 1 (fps)**
  - Maximum Volume Stored (acre-ft): 0.00
- **Max Velocity through Grate 2 (fps)**
  - Time to Drain 91% of Inflow Volume (hours): 38
- **Time to Drain 99% of Inflow Volume (hours)**
  - Maximum Volume Stored (acre-ft): 0.00

- **Area at Maximum Ponding Depth (acres)**
  - Time to Drain 91% of Inflow Volume (hours): 38
- **Maximum Ponding Depth (ft)**
  - Area at Maximum Ponding Depth (acres): 0.00

### Detention Basin Outlet Structure Design
- **UD-Detention, Version 3.07**
- **February 2017**
CULVERT STAGE-DISCHARGE SIZING (INLET vs. OUTLET CONTROL WITH TAILWATER EFFECTS)

Project: Hayden ADF EDOP
Basin ID: Detention Pond 2 Outlet

Design Information (Input):
Circular Culvert: Barrel Diameter in Inches
D = 24 inches
Square End with Headwall

OR:
Box Culvert: Barrel Height (Rise) in Feet
Height (Rise) = ft.
Width (Span) = ft.
Inlet Edge Type (choose from pull-down list)
1.5 : 1 Bevel w/ 90 Deg. Headwall

Number of Barrels
No = 1

Outlet Elevation at Culvert Invert OR Slope of Culvert (ft v./ft h.)
Outlet Elev = 6469.42 ft. elev.

Culvert Length in Feet
L = 66 ft.

Manning's Roughness
n = 0.024

Bend Loss Coefficient

Exit Loss Coefficient

Design Information (calculated):
Entrance Loss Coefficient
K_e = 0.50

Friction Loss Coefficient
K_f = 2.78

Sum of All Loss Coefficients
K_s = 4.28

Orifice Inlet Condition Coefficient
C_d = 0.86

Minimum Energy Condition Coefficient
K_{E,low} = -0.0342

Calculations of Culvert Capacity (output):

<table>
<thead>
<tr>
<th>Water Surface Elevation (ft., linked)</th>
<th>Tailwater Surface Elevation ft</th>
<th>Culvert Inlet-Control Flowrate cfs</th>
<th>Culvert Outlet-Control Flowrate cfs</th>
<th>Controlling Culvert Flowrate cfs (output)</th>
<th>Inlet Equation Used</th>
<th>Flow Control Used</th>
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<tbody>
<tr>
<td>6470.57</td>
<td></td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>No Flow (WS &lt; inlet)</td>
<td>N/A</td>
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<tr>
<td>6471.07</td>
<td>1.20</td>
<td>9.22</td>
<td>1.20</td>
<td>1.20</td>
<td>Min. Energy Eqn.</td>
<td>INLET</td>
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<tr>
<td>6471.57</td>
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<td>11.57</td>
<td>8.30</td>
<td>8.30</td>
<td>Regression Eqn.</td>
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<tr>
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<td>15.69</td>
<td>15.69</td>
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<td>OUTLET</td>
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<tr>
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<td>17.95</td>
<td>17.95</td>
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<td>OUTLET</td>
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<tr>
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<td>19.90</td>
<td>19.90</td>
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<td>23.29</td>
<td>23.29</td>
<td>23.29</td>
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<td>OUTLET</td>
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<tr>
<td>6475.57</td>
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<td>24.91</td>
<td>24.91</td>
<td>24.91</td>
<td>Regression Eqn.</td>
<td>OUTLET</td>
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<td>6476.57</td>
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<td>OUTLET</td>
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<tr>
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<td>29.03</td>
<td>29.03</td>
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<td>Orifice Eqn.</td>
<td>OUTLET</td>
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<td>31.46</td>
<td>31.46</td>
<td>Orifice Eqn.</td>
<td>OUTLET</td>
</tr>
</tbody>
</table>

Processing Time: 01.35 Seconds
Required Volume Calculation

Selected BMP Type = EDB

Water Quality Capture Volume (WQCV) = 0.026 acre-feet

Excess Urban Runoff Volume (EURV) = 0.030 acre-feet

2-yr Runoff Volume (P1 = 0.48 in.) = 0.010 acre-feet 0.48 inches

5-yr Runoff Volume (P1 = 0.66 in.) = 0.066 acre-feet 0.66 inches

10-yr Runoff Volume (P1 = 0.82 in.) = 0.229 acre-feet 0.82 inches

25-yr Runoff Volume (P1 = 1.24 in.) = 1.022 acre-feet 1.24 inches

50-yr Runoff Volume (P1 = 1.44 in.) = 4.428 acre-feet 1.44 inches

500-yr Runoff Volume (P1 = 1.97 in.) = 2.354 acre-feet 1.97 inches

Approximate 2-yr Detention Volume = 0.009 acre-feet

Approximate 5-yr Detention Volume = 0.062 acre-feet

Approximate 10-yr Detention Volume = 0.170 acre-feet

Approximate 25-yr Detention Volume = 0.132 acre-feet

Approximate 50-yr Detention Volume = 0.132 acre-feet

Approximate 100-yr Detention Volume = 0.219 acre-feet

Stage-Storage Calculation

Zone 1 Volume (WQCV) = 0.026 acre-feet

Zone 2 Volume (100-year - Zone 1) = 0.193 acre-feet

Select Zone 3 Storage Volume (Optional) = 0.219 acre-feet

Total Detention Basin Volume = 0.219 acre-feet

Initial Surcharge Volume (ISV) = user ft³

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 H:V

Basin Length-to-Width Ratio (R_{L/W}) = user

Initial Surcharge Area (A_{ISV}) = user ft²

Surcharge Volume Length (L_{ISV}) = user ft

Surcharge Volume Width (W_{ISV}) = user ft

Depth of Basin Floor (H_{BFS}) = user ft

Length of Basin Floor (L_{BFS}) = user ft

Width of Basin Floor (W_{BFS}) = user ft

Area of Basin Floor (A_{BFS}) = user ft²

Volume of Basin Floor (V_{BFS}) = user ft³

Depth of Main Basin (H_{MB}) = user ft

Length of Main Basin (L_{MB}) = user ft

Width of Main Basin (W_{MB}) = user ft

Area of Main Basin (A_{MB}) = user ft²

Volume of Main Basin (V_{MB}) = user ft³

Calculated Total Basin Volume (V_{total}) = user acre-feet
### Detention Basin Outlet Structure Design

#### UD-Detention, Version 3.07 (February 2017)

**Project:** Hayden ADF EDOP  
**Basin ID:** Detention Pond 3

#### Example Zone Configuration (Retention Pond)

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

  - Calculated Parameters for Underdrain
    - Underdrain Orifice Invert Depth = N/A ft (distance below the filtration media surface)
    - Underdrain Orifice Diameter = N/A inches
    - Underdrain Orifice Area = N/A sq. ft
    - 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)

  - Calculated Parameters for Plate
    - Invert of Lowest Orifice = 0.00 ft (relative to basin bottom at Stage = 0 ft)
    - Depth at top of Zone using Orifice Plate = 2.00 ft (relative to basin bottom at Stage = 0 ft)
    - Orifice Plate: Orifice Vertical Spacing = N/A ft
    - Orifice Plate: Orifice Area per Row = 0.78 sq. inches (diameter = 1 inch)
    - Orifice Plate: Orifice Area per Row = 0.78 sq. inches (diameter = 1 inch)

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

  - Calculated Parameters for Outlet Pipe
    - Invert of Vertical Orifice = N/A ft (relative to basin bottom at Stage = 0 ft)
    - Depth at top of Zone using Vertical Orifice = N/A ft (relative to basin bottom at Stage = 0 ft)
    - Vertical Orifice Area = N/A ft

- **User Input:** Overflow Weir (Dropbox) and Grate (Flat or Sloped)

  - Calculated Parameters for Overflow Weir
    - Overflow Weir Front Edge Height, Ho = 2.00 ft
    - Overflow Grate Open Area % = 80%
    - Overflow Grate Open Area w/ Debris = 0.00 ft

- **User Input:** Emergency Spillway (Rectangular or Trapezoidal)

  - Calculated Parameters for Spillway
    - Spillway Crest Length = 18.00 feet

- **User Input:** Vertical Orifice (Circular or Rectangular)

  - Calculated Parameters for Vertical Orifice
    - Vertical Orifice Diameter = inches
    - Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)

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

  - Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
    - Depth to Invert of Outlet Pipe = 0.00 ft
    - Circular Orifice Diameter = 17.00 inches

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

  - Calculated Parameters for Underdrain
    - Underdrain Orifice Invert Depth = N/A ft (distance below the filtration media surface)
    - Underdrain Orifice Diameter = N/A inches
    - Underdrain Orifice Area = N/A sq. ft
    - Underdrain Orifice Centroid = N/A feet

#### Calculated Runoff Volume (acre-ft)

<table>
<thead>
<tr>
<th>Stage (ft)</th>
<th>Zone Volume (ac-ft)</th>
<th>Outlet Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.19</td>
<td>0.036</td>
<td>Orifice Plate</td>
</tr>
<tr>
<td>1.39</td>
<td>0.193</td>
<td>weirPipe (Circular)</td>
</tr>
</tbody>
</table>

**Stage at Top of Freeboard:**
- **User Input:** Emergency Spillway (Rectangular or Trapezoidal)

  - Calculated Parameters for Spillway
    - Max Velocity through Grate 1 (fps) = 5.03

#### Routed Hydrograph Results

- **WQCV**
  - One-Hour Rainfall Depth (in) = 0.53
  - Calculated Runoff Volume (acre-ft) = 0.026
- **EUR**
  - Design Storm Return Period = 50 Year

#### Basins

<table>
<thead>
<tr>
<th>Zone 1 (WQCV)</th>
<th>Zone 2 (100-year)</th>
<th>Zone 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage (ft)</td>
<td>Zone Volume (ac-ft)</td>
<td>Outlet Type</td>
</tr>
<tr>
<td>Zone 1</td>
<td>0.98</td>
<td>0.34</td>
</tr>
<tr>
<td>Zone 2</td>
<td>0.08</td>
<td>0.12</td>
</tr>
<tr>
<td>Zone 3</td>
<td>0.02</td>
<td>0.03</td>
</tr>
</tbody>
</table>

- **Stage of Orifice Centroid (ft)**
  - 0.00

- **Max Velocity through Grate 1 (fps)**
  - 5.03

- **Freeboard above Max Water Surface:**
  - 1.00 feet

- **Inflow Hydrograph Volume (acre-ft)**
  - 0.026

- **Peak Inflow Q (cfs)**
  - 0.00

- **Stage of Orifice Centroid (ft)**
  - 0.00 0.30 0.60 0.90 1.20 1.50

- **Height of Grate Upper Edge, H**
  - 2.00 ft

<table>
<thead>
<tr>
<th>2 Year</th>
<th>5 Year</th>
<th>10 Year</th>
<th>25 Year</th>
<th>50 Year</th>
<th>100 Year</th>
<th>500 Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.19</td>
<td>0.10</td>
<td>0.08</td>
<td>0.06</td>
<td>0.05</td>
<td>0.04</td>
<td>0.03</td>
</tr>
<tr>
<td>0.53</td>
<td>0.48</td>
<td>0.46</td>
<td>0.44</td>
<td>0.42</td>
<td>0.40</td>
<td>0.39</td>
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</tbody>
</table>

- **Peak Outflow Q (cfs)**
  - 0.00

- **Stage at Top of Freeboard:**
  - 4.70 ft

- **Max Velocity through Grate (fps) (0.00)**
  - 0.00

- **Invert of Vertical Orifice**
  - N/A ft

- **Overflow Weir Slope = 0.00 H:V (enter zero for flat grate)**
  - 0.00

- **Peak Velocity through Grate 1 (fps)**
  - 0.00

- **Spatial Control:**
  - N/A

- **Max Velocity through Grate 2 (fps)**
  - 0.00

- **Max Velocity through Grate (fps) (0.00)**
  - 0.00

- **Max Velocity through Grate 3 (fps)**
  - 0.00

# Table 1: Calculated Runoff Volume (acre-ft)

<table>
<thead>
<tr>
<th>Stage (ft)</th>
<th>Zone Volume (ac-ft)</th>
<th>Outlet Type</th>
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<tr>
<td>0.09</td>
<td>0.051</td>
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</tr>
<tr>
<td>1.39</td>
<td>0.193</td>
<td>weirPipe (Circular)</td>
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**User Input:** Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)
**Project:** Hayden ADF EDOP  
**Basin ID:** Detention Pond 3 Outlet  
**Status:**

### Design Information (Input):

**Circular Culvert:**
- Barrel Diameter in Inches: 18 inches
- Inlet Edge Type (choose from pull-down list): Square End with Headwall

**Box Culvert:**
- Barrel Height (Rise) in Feet: Height (Rise)  
- Barrel Width (Span) in Feet: Width (Span)  
- Inlet Edge Type (choose from pull-down list): 1.5 : 1 Bevel w/ 90 Deg. Headwall
- Number of Barrels: No = 1
- Inlet Elevation at Culvert Invert: Inlet Elev = 6625 ft. elev.
- Outlet Elevation at Culvert Invert OR Slope of Culvert (ft v./ft h.): Outlet Elev = 6623.4 ft. elev.
- Culvert Length in Feet: L = 32 ft.
- Manning’s Roughness: n = 0.024
- Bend Loss Coefficient: $K_b = 0$
- Exit Loss Coefficient: $K_x = 1$

### Design Information (calculated):

- Entrance Loss Coefficient: $K_e = 0.50$
- Friction Loss Coefficient: $K_f = 1.98$
- Sum of All Loss Coefficients: $K_s = 3.48$
- Orifice Inlet Condition Coefficient: $C_d = 0.86$
- Minimum Energy Condition Coefficient: $KE_{low} = -0.1741$

### Calculations of Culvert Capacity (output):

<table>
<thead>
<tr>
<th>Water Surface Elevation (ft, linked)</th>
<th>Tailwater Surface Elevation ft</th>
<th>Culvert Inlet-Control Flowrate cfs</th>
<th>Culvert Outlet-Control Flowrate cfs</th>
<th>Controlling Culvert Flowrate cfs (output)</th>
<th>Inlet Equation Used</th>
<th>Flow Control Used</th>
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<td>6625.00</td>
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**Processing Time:** 11.56 Seconds
## Required Volume Calculation

Selected BMP Type = **EDB**

### Watershed Information
- **Watershed Area**: 14.10 acres
- **Watershed Length**: 1.850 ft
- **Watershed Slope**: 0.026 ft/ft
- **Watershed Imperviousness**: 2.00% percent

### BMP Descriptions
- **Percentage Hydrologic Soil Group A**: 0.0% percent
- **Percentage Hydrologic Soil Group B**: 0.0% percent
- **Percentage Hydrologic Soil Groups C/D**: 100.0% percent

### Water Quality Capture Volume (WQCV)
- **Water Quality Capture Volume (WQCV)**: 0.018 acre-feet

### Excess Urban Runoff Volume (EURV)
- **Excess Urban Runoff Volume (EURV)**: 0.021 acre-feet

### Runoff Volume Calculations
- **2-yr Runoff Volume**: 0.007 acre-feet (0.48 inches)
- **5-yr Runoff Volume**: 0.043 acre-feet (0.66 inches)
- **10-yr Runoff Volume**: 0.156 acre-feet (0.92 inches)
- **25-yr Runoff Volume**: 0.461 acre-feet (1.24 inches)
- **50-yr Runoff Volume**: 0.971 acre-feet (1.44 inches)
- **100-yr Runoff Volume**: 1.601 acre-feet (1.97 inches)

### Detention Volume Calculations
- **Approximate 2-yr Detention Volume**: 0.006 acre-feet
- **Approximate 5-yr Detention Volume**: 0.042 acre-feet
- **Approximate 10-yr Detention Volume**: 0.074 acre-feet
- **Approximate 25-yr Detention Volume**: 0.090 acre-feet
- **Approximate 50-yr Detention Volume**: 0.090 acre-feet
- **Approximate 100-yr Detention Volume**: 0.149 acre-feet

## Stage-Storage Calculation

### Example Zone Configuration (Retention Pond)

### Stage-Storage Table Builder

<table>
<thead>
<tr>
<th>Stage-Storage Description</th>
<th>Stage (ft)</th>
<th>Length (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Stage (ft)</th>
<th>Length (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Length (ft)</th>
<th>Area (ft²)</th>
<th>Volume (ft³)</th>
<th>Volume (ac-ft)</th>
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</thead>
<tbody>
<tr>
<td>Top of Micropool</td>
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<td>4.192</td>
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<td>4.517</td>
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<tr>
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<td>24.664</td>
<td>0.568</td>
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</tbody>
</table>

### Optional User Overides

- **1-hr Precipitation**
- **Area (ft²)**
- **Width (ft)**
- **Stage (ft)**
- **Stage - Storage Description**
- **Area (ft²)**
- **Volume (ft³)**
- **Volume (ac-ft)**

### Basin Details

- **Depth Increment**: 0.2 ft
- **Selected BMP Type**: EDB
- **Watershed Area**: 14.10 acres
- **Watershed Length**: 1.850 ft
- **Watershed Slope**: 0.026 ft/ft
- **Watershed Imperviousness**: 2.00% percent
- **Percentage Hydrologic Soil Group A**: 0.0% percent
- **Percentage Hydrologic Soil Group B**: 0.0% percent
- **Percentage Hydrologic Soil Groups C/D**: 100.0% percent
- **Desired WQCV Drain Time**: 40.0 hours
- **Location for 1-hr Rainfall Depths**: User Input
- **Water Quality Capture Volume (WQCV)**: 0.018 acre-feet
- **Excess Urban Runoff Volume (EURV)**: 0.021 acre-feet
- **2-yr Runoff Volume (P1 = 0.48 in.)**: 0.007 acre-feet (0.48 inches)
- **5-yr Runoff Volume (P1 = 0.66 in.)**: 0.043 acre-feet (0.66 inches)
- **10-yr Runoff Volume (P1 = 0.92 in.)**: 0.156 acre-feet (0.92 inches)
- **25-yr Runoff Volume (P1 = 1.24 in.)**: 0.461 acre-feet (1.24 inches)
- **50-yr Runoff Volume (P1 = 1.97 in.)**: 0.971 acre-feet (1.44 inches)
- **100-yr Runoff Volume (P1 = 1.97 in.)**: 1.601 acre-feet (1.97 inches)
- **Approximate 2-yr Detention Volume**: 0.006 acre-feet
- **Approximate 5-yr Detention Volume**: 0.042 acre-feet
- **Approximate 10-yr Detention Volume**: 0.074 acre-feet
- **Approximate 25-yr Detention Volume**: 0.090 acre-feet
- **Approximate 50-yr Detention Volume**: 0.090 acre-feet
- **Approximate 100-yr Detention Volume**: 0.149 acre-feet

## DETENTION BASIN STAGE-STORAGE TABLE BUILDER

<table>
<thead>
<tr>
<th>Stage-Storage Description</th>
<th>Stage (ft)</th>
<th>Length (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Stage (ft)</th>
<th>Length (ft)</th>
<th>Area (ft²)</th>
<th>Optional Override Length (ft)</th>
<th>Area (ft²)</th>
<th>Volume (ft³)</th>
<th>Volume (ac-ft)</th>
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</thead>
<tbody>
<tr>
<td>Zone 1 Volume (WQCV)</td>
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Detention Basin Outlet Structure Design

UD-Detention, Version 3.07 (February 2017)

Example Zone Configuration (Retention Pond)

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<th>Basin ID:</th>
<th>Project:</th>
<th>Detention Pond 4</th>
<th>Hayden ADF EDOP</th>
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<tr>
<td>Zone 1 (WQCV)</td>
<td>Zone 2 (WQCV)</td>
<td>Zone 3 (EURV)</td>
<td>Zone 4 (100-year)</td>
</tr>
<tr>
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<td>Volume (ac-ft)</td>
<td>Outlet Type</td>
<td>Stage (ft)</td>
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<td>0.19</td>
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<td>Orifice Plate</td>
<td>0.13</td>
</tr>
<tr>
<td>1.38</td>
<td>1.31</td>
<td>Pipe (Circular)</td>
<td>0.49</td>
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User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

<table>
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<tr>
<th>Calculated Parameters for Underdrain</th>
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<tbody>
<tr>
<td>Underdrain Orifice Invert Depth =</td>
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<tr>
<td>Underdrain Orifice Diameter =</td>
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<tr>
<td>Underdrain Orifice Area =</td>
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<tr>
<td>Underdrain Orifice Centroid =</td>
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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)

<table>
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<tr>
<th>Calculated Parameters for Plate</th>
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</table>

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

<table>
<thead>
<tr>
<th>Calculated Parameters for Orifice Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Orifice Diameter =</td>
</tr>
<tr>
<td>Vertical Orifice Area =</td>
</tr>
<tr>
<td>Vertical Orifice Invert Depth =</td>
</tr>
<tr>
<td>Vertical Orifice Slope Length =</td>
</tr>
</tbody>
</table>

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

<table>
<thead>
<tr>
<th>Calculated Parameters for Overflow Weir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overflow Weir Front Edge Height, Ho =</td>
</tr>
<tr>
<td>Overflow Weir Front Edge Slope =</td>
</tr>
<tr>
<td>Overflow Weir Slope Length =</td>
</tr>
<tr>
<td>Overflow Grate Open Area % =</td>
</tr>
<tr>
<td>Debris Clogging % =</td>
</tr>
</tbody>
</table>

User Input: Emergency Spillway (Rectangular or Trapezoidal)

<table>
<thead>
<tr>
<th>Calculated Parameters for Emergency Spillway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spillway Invert Stage=</td>
</tr>
<tr>
<td>Spillway End Slopes =</td>
</tr>
<tr>
<td>Basin Area at Top of Freeboard =</td>
</tr>
</tbody>
</table>

Route Hydrograph Results

<table>
<thead>
<tr>
<th>Design Storm Return Period</th>
<th>WQCV</th>
<th>EURV</th>
<th>2 Year</th>
<th>5 Year</th>
<th>10 Year</th>
<th>25 Year</th>
<th>50 Year</th>
<th>100 Year</th>
<th>500 Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>One-Hour Rainfall Depth (in)</td>
<td>0.53</td>
<td>1.07</td>
<td>0.48</td>
<td>0.66</td>
<td>0.82</td>
<td>1.05</td>
<td>1.24</td>
<td>1.44</td>
<td>1.97</td>
</tr>
<tr>
<td>Calculated Runoff Volume (acre-ft)</td>
<td>0.018</td>
<td>0.021</td>
<td>0.007</td>
<td>0.043</td>
<td>0.156</td>
<td>0.461</td>
<td>0.681</td>
<td>0.971</td>
<td>1.601</td>
</tr>
<tr>
<td>Inflow Hydrograph (acre-ft)</td>
<td>0.018</td>
<td>0.020</td>
<td>0.007</td>
<td>0.043</td>
<td>0.155</td>
<td>0.460</td>
<td>0.681</td>
<td>0.969</td>
<td>1.599</td>
</tr>
<tr>
<td>Precipitation Unit Peak Flow, Q (cfs)</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.04</td>
<td>0.11</td>
<td>0.30</td>
<td>0.41</td>
<td>0.56</td>
<td>0.90</td>
</tr>
<tr>
<td>Peak Inflow Q (cfs)</td>
<td>0.2</td>
<td>0.3</td>
<td>0.6</td>
<td>2.0</td>
<td>5.8</td>
<td>8.6</td>
<td>12.2</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td>Peak Outflow Q (cfs)</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.1</td>
<td>3.1</td>
<td>6.0</td>
<td>7.3</td>
<td>17.4</td>
</tr>
<tr>
<td>Structure Controlling Flow</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
<td>Plate</td>
</tr>
<tr>
<td>Max Velocity through Grate 1 (fps)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Max Velocity through Grate 2 (fps)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Time to Drain 99% of Inflow Volume (hours)</td>
<td>38</td>
<td>40</td>
<td>27</td>
<td>53</td>
<td>71</td>
<td>72</td>
<td>68</td>
<td>63</td>
<td>55</td>
</tr>
<tr>
<td>Maximum Ponding Depth (ft)</td>
<td>0.16</td>
<td>0.19</td>
<td>0.06</td>
<td>0.39</td>
<td>1.29</td>
<td>2.35</td>
<td>2.55</td>
<td>3.08</td>
<td>3.52</td>
</tr>
<tr>
<td>Maximum Volume Stored (acre-ft)</td>
<td>0.016</td>
<td>0.018</td>
<td>0.005</td>
<td>0.039</td>
<td>0.138</td>
<td>0.273</td>
<td>0.303</td>
<td>0.381</td>
<td>0.451</td>
</tr>
</tbody>
</table>
### Design Information (Input):

**Circular Culvert:**
- Barrel Diameter in Inches \(D = 18\) inches
- Inlet Edge Type (choose from pull-down list): Square End with Headwall

**OR:**

**Box Culvert:**
- Barrel Height (Rise) in Feet \(H = \) ft.
- Barrel Width (Span) in Feet \(B = \) ft.
- Inlet Edge Type (choose from pull-down list): 1.5 : 1 Bevel w/ 90 Deg. Headwall
- Number of Barrels \(N_b = 1\)
- Inlet Elevation at Culvert Invert \(E_{in} = 6640.5\) ft. elev.
- Outlet Elevation at Culvert Invert or Slope of Culvert \(E_{out} = 6639.3\) ft. elev.
- Culvert Length in Feet \(L = 24\) ft.
- Manning’s Roughness \(n = 0.024\)
- Bend Loss Coefficient \(K_b = 0\)
- Exit Loss Coefficient \(K_x = 1\)

### Design Information (calculated):

- Entrance Loss Coefficient \(K_e = 0.50\)
- Friction Loss Coefficient \(K_f = 1.48\)
- Sum of All Loss Coefficients \(K_s = 2.98\)
- Orifice Inlet Condition Coefficient \(C_d = 0.86\)
- Minimum Energy Condition Coefficient \(K_{E_{low}} = -0.1741\)

### Calculations of Culvert Capacity (output):

<table>
<thead>
<tr>
<th>Water Surface Elevation (ft., linked)</th>
<th>Tailwater Surface Elevation ft</th>
<th>Culvert Inlet-Control Flowrate cfs</th>
<th>Culvert Outlet-Control Flowrate cfs</th>
<th>Controlling Culvert Flowrate cfs (output)</th>
<th>Inlet Equation Used:</th>
<th>Flow Control Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>6641.00</td>
<td>1.10</td>
<td>6.93</td>
<td>1.10</td>
<td>Min. Energy. Eqn.</td>
<td>INLET</td>
<td></td>
</tr>
<tr>
<td>6641.20</td>
<td>2.10</td>
<td>7.38</td>
<td>2.10</td>
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<td>INLET</td>
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<td>INLET</td>
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<td>8.27</td>
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<tr>
<td>6641.80</td>
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<td>5.30</td>
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<td>INLET</td>
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<tr>
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<td>9.03</td>
<td>6.50</td>
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<td>INLET</td>
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<td>11.07</td>
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<td>INLET</td>
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<td>INLET</td>
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<td>12.21</td>
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<td>INLET</td>
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<td>12.72</td>
<td>11.80</td>
<td>Regression Eqn.</td>
<td>INLET</td>
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<tr>
<td>6643.40</td>
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<td>13.30</td>
<td>12.40</td>
<td>Regression Eqn.</td>
<td>INLET</td>
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<td>INLET</td>
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</tr>
<tr>
<td>6643.80</td>
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<td>14.25</td>
<td>13.60</td>
<td>Regression Eqn.</td>
<td>INLET</td>
<td></td>
</tr>
<tr>
<td>6644.00</td>
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<td>14.70</td>
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<td>INLET</td>
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</tr>
<tr>
<td>6644.20</td>
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<td>15.20</td>
<td>14.70</td>
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<td>INLET</td>
<td></td>
</tr>
<tr>
<td>6644.40</td>
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<td>15.59</td>
<td>15.20</td>
<td>Regression Eqn.</td>
<td>INLET</td>
<td></td>
</tr>
<tr>
<td>6644.60</td>
<td>15.70</td>
<td>16.03</td>
<td>15.70</td>
<td>Regression Eqn.</td>
<td>INLET</td>
<td></td>
</tr>
</tbody>
</table>

Processing Time: 14.26 Seconds
Purpose: The purpose of this calculation package is to show the calculated stage storage of the contact pond in the development conditions. The area of interest is shown on Figure 1. The contact pond as-built is shown on Figure 2.

Background: The Hayden ash disposal facilities (ADF) design has been updated as apart of the EDOP renewal. A portion of the permit renewal is a revision to the contact pond design. The active disposal area of the landfill has a perimeter berm surrounding the ash placement to contain contact water and direct to the contact pond. A HDPE pipe will be placed in the perimeter berm to run contact water into the contact pond. The contact pond has been lined to contain all contaminated liquid and sized to store the minimum of the 100 year storm.

Methodology: The SCS Runoff Curve Number Method, in accordance with the NRCS TR-55 Manual has been used to estimate peak runoff flows for the drainage area shown on Figure 1. The placement area will be separated into 11 acre cells, with one cell active at a time. Calculations for the runoff generate a peak runoff depth for the entire watershed. The runoff amount is used to calculate the volume of water to be conveyed to the contact pond. Stage storage calculations were completed using these runoff volumes and the volume available in the contact pond at each corresponding elevation.

Notes: |
---|---
Data Input Cell | Calculated/Reference Cell

Attachments:
1. Phase I Grading Plan (Figure 1)
2. Contact Pond Asbuilt (Figure 2)

Conclusions: The contact pond was built to contain the 25 year storm with more than 2.5 feet of freeboard and is adequately sized to contain the contact water runoff generated by the ash disposal facility. If required, the contact pond can be used to contain groundwater or previous storm events until the elevation reaches 6608 and still contain the 25 year storm with more than 1 foot of freeboard. The 25 year storm is equal to 60,682 cubic feet which can be contained within the contact pond three times while maintaining the required freeboard. Additionally, the contact pond can contain the 100 year storm between the elevations of 6608 and 6610 and maintain more than 10 inches of freeboard.
Hydrologic Soil Group | Cover Type | Curve Number | Area (ac) | Curve Number x Area
--- | --- | --- | --- | ---
C | Newly Graded Areas | 91 | 11.00 | 1,001
Totals => | | 11.00 | 1,001

Weighted Curve Number, \( CN = \frac{\text{Curve Number} \times \text{Area}}{\text{Total Area}} \)

\[ CN = \frac{11.00 \times 1,001}{11.00} = 91 \]

**Time of Concentration**

Sheet Flow elevation range

- **Elevation difference, \( \Delta \)**
  \[ \Delta = \text{3 ft} \]
  (From AutoCAD)

- **Flow Length, \( L \)**
  \[ L = \text{300 ft} \]
  (From AutoCAD)

- **Slope, \( s = \frac{\Delta \text{elev}}{\text{length}} \)**
  \[ s = \text{1.000} \% \]

- **Flow Resistance Coefficient, \( k \)**
  \[ k = \text{0.150} \]
  See Attachment 3-2

- **Sheet Flow travel time, \( t_i = \frac{1.8 x (1.1 - K) x L^5}{S^{1/3}} \)**
  \[ t_i = \text{29.62 min} \]
  See Attachment 3-2

- **Shallow Concentrated Flow Time, \( t_t \)**
  \[ t_t = \text{0.49 hr} \]

- **Length of Shallow Concentrated Flow, \( L \)**
  \[ L = \text{575 feet} \]
  (From AutoCAD)

- **Shallow Concentrated Flow Time, \( t_t \)**
  \[ t_t = \text{16.25 min} \]

- **Time of Concentration, \( T_c = T_i + T_t \)**
  \[ T_c = \text{0.76 hr} \]

**Peak Discharge**

- **Drainage Area, \( A, \text{mi}^2 \)**
  \[ A = \text{0.0172 mi}^2 \]
  (See Above)

- **Rainfall Distribution Type**
  || || I I

- **Storm Frequency**
  \[ P = \text{2.40 2.80} \]
  See Attachment 3-5

- **Storm Rainfall Depth, \( P, \text{in} \)**
  \[ S = \text{0.99 0.99} \]
  See Attachment 3-6

- **Initial abstraction, \( I_a \)**
  \[ I_a = \text{0.20 0.20} \]
  See Attachment 3-6

- **Depth of runoff over entire watershed, \( Q = (P - I_a)^2 / [(P - I_a) + S] \)**
  \[ Q = \text{1.52 1.89} \]
  See Attachment 3-6

- **Initial abstraction / Rainfall Depth, \( I_a / P \)**
  \[ I_a / P = \text{0.08 0.07} \]
  See Attachment 3-6

- **Unit peak discharge, \( Q_u, \text{csm/in} \)**
  \[ Q_u = \text{420 420} \]
  See Attachment 3-7

- **Pond and swamp adjustment factor, \( F_p \)**
  \[ F_p = \text{1.0 1.0} \]
  See Attachment 3-8

- **Qp = Total Discharge from Watershed = Q_u A Q P F_p, cfs**
  \[ Q_p = \text{11.0 13.6} \]
Hayden Ash Disposal Facility
STAGE STORAGE CALCULATION

Calculation by: MDG  date: 7/25/2018
Checked by: JLL  date: 7/27/2018

Depth of Runoff (25-year), Q

\[
Q = 1.52 \text{ inch} = 0.13 \text{ feet}
\]

Maximum Area, A

\[
A = 11.00 \text{ acres}
\]

Total Runoff Volume, \( V = Q \times A \)

\[
V = 60,682 \text{ cubic feet}
\]

Depth of Runoff (100-year), Q

\[
Q = 1.89 \text{ inch} = 0.16 \text{ feet}
\]

Maximum Area, A

\[
A = 11.00 \text{ acres}
\]

Total Runoff Volume, \( V = Q \times A \)

\[
V = 75,290 \text{ cubic feet}
\]

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Depth (ft)</th>
<th>Area (ft²)</th>
<th>Average Area (ft²)</th>
<th>Incremental Volume (ft³)</th>
<th>Cumulative Volume (ft³)</th>
<th>Required Volume (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,603.95</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6,604</td>
<td>0.05</td>
<td>217</td>
<td>108</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>6,605</td>
<td>1.00</td>
<td>11,635</td>
<td>5,926</td>
<td>5,926</td>
<td>5,926</td>
<td>5,931</td>
</tr>
<tr>
<td>6,606</td>
<td>1.00</td>
<td>29,435</td>
<td>20,535</td>
<td>20,535</td>
<td>20,535</td>
<td>26,466</td>
</tr>
<tr>
<td>6,606.61</td>
<td></td>
<td></td>
<td></td>
<td>25-year Required Volume and Water Surface Elevation</td>
<td>60,682</td>
<td>60,682</td>
</tr>
<tr>
<td>6,607</td>
<td>1.00</td>
<td>41,977</td>
<td>35,706</td>
<td>35,706</td>
<td>35,706</td>
<td>62,172</td>
</tr>
<tr>
<td>6,607.37</td>
<td></td>
<td></td>
<td></td>
<td>100-year Required Volume and Water Surface Elevation</td>
<td>75,290</td>
<td>75,290</td>
</tr>
<tr>
<td>6,608</td>
<td>1.00</td>
<td>54,298</td>
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<td>48,138</td>
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<td>62,713</td>
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<td>78,537</td>
<td>74,832</td>
<td>74,832</td>
<td>74,832</td>
<td>247,854</td>
</tr>
</tbody>
</table>
### Table 2-2a Runoff curve numbers for urban areas

<table>
<thead>
<tr>
<th>Cover type and hydrologic condition</th>
<th>Average percent impervious area</th>
<th>Curve numbers for hydrologic soil group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td><strong>Fully developed urban areas (vegetation established)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Open space (lawns, parks, golf courses, cemeteries, etc.) (\frac{1}{1})</td>
<td>Poor condition (grass cover &lt; 50%)</td>
<td>68</td>
</tr>
<tr>
<td>Fair condition (grass cover 50% to 75%)</td>
<td>49</td>
<td>69</td>
</tr>
<tr>
<td>Good condition (grass cover &gt; 75%)</td>
<td>39</td>
<td>61</td>
</tr>
<tr>
<td><strong>Impervious areas:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paved parking lots, roofs, driveways, etc. (excluding right-of-way)</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Streets and roads:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paved; curbs and storm sewers (excluding right-of-way)</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Paved; open ditches (including right-of-way)</td>
<td>83</td>
<td>89</td>
</tr>
<tr>
<td>Gravel (including right-of-way)</td>
<td>76</td>
<td>85</td>
</tr>
<tr>
<td>Dirt (including right-of-way)</td>
<td>72</td>
<td>82</td>
</tr>
<tr>
<td><strong>Western desert urban areas:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural desert landscaping (pervious areas only) (\frac{1}{2})</td>
<td>63</td>
<td>77</td>
</tr>
<tr>
<td>Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)</td>
<td>96</td>
<td>96</td>
</tr>
<tr>
<td><strong>Urban districts:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Commercial and business</td>
<td>85</td>
<td>89</td>
</tr>
<tr>
<td>Industrial</td>
<td>72</td>
<td>81</td>
</tr>
<tr>
<td><strong>Residential districts by average lot size:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/8 acre or less (town houses)</td>
<td>65</td>
<td>77</td>
</tr>
<tr>
<td>1/4 acre</td>
<td>38</td>
<td>61</td>
</tr>
<tr>
<td>1/3 acre</td>
<td>30</td>
<td>57</td>
</tr>
<tr>
<td>1/2 acre</td>
<td>25</td>
<td>54</td>
</tr>
<tr>
<td>1 acre</td>
<td>20</td>
<td>51</td>
</tr>
<tr>
<td>2 acres</td>
<td>12</td>
<td>46</td>
</tr>
<tr>
<td><strong>Developing urban areas:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Newly graded areas (pervious areas only, no vegetation) (\frac{1}{2})</td>
<td>77</td>
<td>86</td>
</tr>
<tr>
<td><strong>Idle lands (CN’s are determined using cover types similar to those in table 2-2c):</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

1. Average runoff condition, and \(L_a = 0.2S\).
2. The average percent impervious area shown was used to develop the composite CN’s. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN’s for other combinations of conditions may be computed using figure 2-3 or 2-4.
3. CN’s shown are equivalent to those of pasture. Composite CN’s may be computed for other combinations of open space cover type.
4. Composite CN’s for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN’s are assumed equivalent to desert shrub in poor hydrologic condition.
5. Composite CN’s to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN’s for the newly graded pervious areas.
The time of concentration for both urban and non-urban areas is calculated as follows:

\[ t_c = t_i + t_t \]  
\[ (5.6.1) \]

Where:
- \( t_c \) = Time of concentration (min)
- \( t_i \) = Initial, Inlet, or overland flow time (min)
- \( t_t \) = Travel time in the ditch, channel, gutter, storm drain, etc. (min)

The initial or overland flow time, \( t_i \), may be calculated using the following equation:

\[ t_i = 1.8 \left(1.1 - K\right) \frac{L_o^{1/2}}{S^{1/3}} \]  
\[ (5.6.2) \]

Where:
- \( t_i \) = Initial or Overland Flow Time (min)
- \( K \) = Flow Resistance Coefficient
- \( L_o \) = Length of Overland Flow, (ft, 300-ft maximum)
- \( S \) = Average Watershed Slope (percent)

Equation 5.6.2 was originally developed for use with the Rational Formula method. The 5-year runoff coefficient, \( C_5 \), presented in Table 5.6.1 is recommended for the flow resistance coefficient, \( K \).

### Table 5.6.1 Design Runoff Coefficients

<table>
<thead>
<tr>
<th>Percentage Imperviousness</th>
<th>2-yr</th>
<th>5-yr</th>
<th>10-yr</th>
<th>25-yr</th>
<th>50-yr</th>
<th>100-yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.04</td>
<td>0.18</td>
<td>0.25</td>
<td>0.37</td>
<td>0.44</td>
<td>0.50</td>
</tr>
<tr>
<td>5%</td>
<td>0.08</td>
<td>0.18</td>
<td>0.28</td>
<td>0.39</td>
<td>0.46</td>
<td>0.52</td>
</tr>
<tr>
<td>10%</td>
<td>0.11</td>
<td>0.21</td>
<td>0.30</td>
<td>0.41</td>
<td>0.47</td>
<td>0.53</td>
</tr>
<tr>
<td>15%</td>
<td>0.14</td>
<td>0.24</td>
<td>0.32</td>
<td>0.43</td>
<td>0.49</td>
<td>0.54</td>
</tr>
<tr>
<td>20%</td>
<td>0.17</td>
<td>0.26</td>
<td>0.34</td>
<td>0.44</td>
<td>0.50</td>
<td>0.55</td>
</tr>
<tr>
<td>25%</td>
<td>0.20</td>
<td>0.28</td>
<td>0.36</td>
<td>0.46</td>
<td>0.51</td>
<td>0.56</td>
</tr>
<tr>
<td>30%</td>
<td>0.22</td>
<td>0.30</td>
<td>0.38</td>
<td>0.47</td>
<td>0.52</td>
<td>0.57</td>
</tr>
<tr>
<td>35%</td>
<td>0.25</td>
<td>0.33</td>
<td>0.40</td>
<td>0.48</td>
<td>0.53</td>
<td>0.57</td>
</tr>
<tr>
<td>40%</td>
<td>0.28</td>
<td>0.35</td>
<td>0.42</td>
<td>0.50</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>45%</td>
<td>0.31</td>
<td>0.37</td>
<td>0.44</td>
<td>0.51</td>
<td>0.55</td>
<td>0.59</td>
</tr>
<tr>
<td>50%</td>
<td>0.34</td>
<td>0.40</td>
<td>0.46</td>
<td>0.53</td>
<td>0.57</td>
<td>0.60</td>
</tr>
<tr>
<td>55%</td>
<td>0.37</td>
<td>0.43</td>
<td>0.48</td>
<td>0.55</td>
<td>0.58</td>
<td>0.62</td>
</tr>
<tr>
<td>60%</td>
<td>0.41</td>
<td>0.46</td>
<td>0.51</td>
<td>0.57</td>
<td>0.60</td>
<td>0.63</td>
</tr>
<tr>
<td>65%</td>
<td>0.45</td>
<td>0.49</td>
<td>0.54</td>
<td>0.59</td>
<td>0.62</td>
<td>0.65</td>
</tr>
<tr>
<td>70%</td>
<td>0.49</td>
<td>0.53</td>
<td>0.57</td>
<td>0.62</td>
<td>0.65</td>
<td>0.68</td>
</tr>
<tr>
<td>75%</td>
<td>0.54</td>
<td>0.58</td>
<td>0.62</td>
<td>0.66</td>
<td>0.68</td>
<td>0.71</td>
</tr>
<tr>
<td>80%</td>
<td>0.60</td>
<td>0.63</td>
<td>0.66</td>
<td>0.70</td>
<td>0.72</td>
<td>0.74</td>
</tr>
<tr>
<td>85%</td>
<td>0.66</td>
<td>0.68</td>
<td>0.71</td>
<td>0.75</td>
<td>0.77</td>
<td>0.79</td>
</tr>
<tr>
<td>90%</td>
<td>0.73</td>
<td>0.75</td>
<td>0.77</td>
<td>0.80</td>
<td>0.82</td>
<td>0.83</td>
</tr>
<tr>
<td>95%</td>
<td>0.80</td>
<td>0.82</td>
<td>0.84</td>
<td>0.87</td>
<td>0.88</td>
<td>0.89</td>
</tr>
<tr>
<td>100%</td>
<td>0.89</td>
<td>0.90</td>
<td>0.92</td>
<td>0.94</td>
<td>0.95</td>
<td>0.96</td>
</tr>
</tbody>
</table>
The overland flow length, \( L_o \), is generally defined as the length over which the flow characteristics appear as sheet flow or very shallow flow in broad, grassed swales. Changes in land slope, surface characteristics, and small drainage ditches or gullies will tend to force the overland flow into a combined flow condition, which results in higher flow velocities and shorter travel times. The initial flow time in both urban and non-urban areas shall be limited to the time to travel a distance of 300 feet.

For watersheds longer than 300 feet, the travel time, \( t_t \), must be added to the overland flow time. Travel time can be calculated using Manning's equation and the hydraulic properties of the storm drain, gutter, swale, ditch, or channel or can be approximated from Equation 5.6.3 and Table 5.6.2:

\[
V = C_v S_w^{0.5} \tag{5.6.3}
\]

Where:
- \( V \) = Velocity, fps
- \( S_w \) = watercourse slope, ft/ft
- \( C_v \) = Conveyance coefficient

The minimum conveyance coefficient, \( C_v \), that shall be used for a developed site shall be 7.0, corresponding to short pasture and lawns.

**Table 5.6.2 Travel Time Conveyance Coefficients**

<table>
<thead>
<tr>
<th>Land Surface</th>
<th>Conveyance Coefficient, ( C_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy meadow</td>
<td>2.5</td>
</tr>
<tr>
<td>Tillage/Field</td>
<td>5.0</td>
</tr>
<tr>
<td>Short pasture and lawns</td>
<td>7.0</td>
</tr>
<tr>
<td>Nearly bare ground</td>
<td>10</td>
</tr>
<tr>
<td>Grassed waterways</td>
<td>15</td>
</tr>
<tr>
<td>Paved areas and shallow swales</td>
<td>20</td>
</tr>
</tbody>
</table>

The time of concentration is then the sum of the initial flow time \( t_i \) and the travel time, \( t_t \). The minimum recommended \( t_c \) for non-urban watersheds is 10 minutes.

5.6.2.1.2 Urbanized Watersheds

Overland flow in urbanized watersheds can occur from the back of the lot to the street, in parking lots, in landscape areas, or within park areas and can be calculated using the procedure described for non-urbanized watersheds. Travel time, \( t_t \), to the first design point or inlet is often determined based on the conveyance coefficient for paved areas and shallow swales, but can be estimated using Manning's equation.

The time of concentration for the first design point in an urbanized watershed using this procedure should not exceed the time of concentration calculated using Equation 5.6.4, which was developed using rainfall/runoff data collected in urbanized regions (USDCM, 1969).

\[
t_c = \frac{L}{180} + 10 \tag{5.6.4}
\]
**Rainfall data sources**

This section lists the most current 24-hour rainfall data published by the National Weather Service (NWS) for various parts of the country. Because NWS Technical Paper 40 (TP-40) is out of print, the 24-hour rainfall maps for areas east of the 105th meridian are included here as figures B-3 through B-8. For the area generally west of the 105th meridian, TP-40 has been superseded by NOAA Atlas 2, the Precipitation-Frequency Atlas of the Western United States, published by the National Ocean and Atmospheric Administration.

**East of 105th meridian**


**West of 105th meridian**


**Alaska**


**Hawaii**


**Puerto Rico and Virgin Islands**

5.5.1 **INTRODUCTION**

Presented in this Section are design rainfall data for the minor and major storm events. These data are used to determine storm runoff peak flows and volumes in conjunction with the runoff models described in Section 5.6, Storm Runoff. All hydrologic analyses for Steamboat Springs shall utilize the rainfall data presented herein for calculating storm runoff.

5.5.2 **RAINFALL ANALYSIS**

For the City of Steamboat Springs, 24-hour point precipitation values are provided in [Table 5.5.1](attachment:2-5) for various recurrence interval storms. These data are from the NOAA Atlas 2, Volume III, 1973.

**Table 5.5.1 24-Hour Point Rainfall Values for Steamboat Springs**

<table>
<thead>
<tr>
<th>Recurrence Interval</th>
<th>24-Hour Precipitation Depths (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>1.3</td>
</tr>
<tr>
<td>5-year</td>
<td>1.7</td>
</tr>
<tr>
<td>10-year</td>
<td>1.9</td>
</tr>
<tr>
<td>25-year</td>
<td>2.4</td>
</tr>
<tr>
<td>50-year</td>
<td>2.6</td>
</tr>
<tr>
<td>100-year</td>
<td>2.8</td>
</tr>
</tbody>
</table>

5.5.3 **INTENSITY-DURATION-FREQUENCY CURVES FOR RATIONAL METHOD**

Rainfall intensities as a function of storm duration and recurrence interval are provided in [Table 5.5.2](attachment:2-5) and [Figure 5.5.1](attachment:2-5). These values were taken or derived from the values in the NOAA Atlas 2, Volume III, 1973.

**Table 5.5.2 Intensity-Duration-Frequency Values**

<table>
<thead>
<tr>
<th>Storm Duration (min)</th>
<th>Precipitation Intensity – Steamboat Springs (inches/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2-year Recurrence</td>
</tr>
<tr>
<td>5</td>
<td>2.04</td>
</tr>
<tr>
<td>10</td>
<td>1.56</td>
</tr>
<tr>
<td>15</td>
<td>1.32</td>
</tr>
<tr>
<td>30</td>
<td>0.92</td>
</tr>
<tr>
<td>60</td>
<td>0.58</td>
</tr>
</tbody>
</table>
5.6.2.3.1 CN Determination

If the SCS Method is specified for use in the basin model portion of the HEC-HMS model, a soil-cover curve number (CN) is used for computing excess precipitation. The curve number CN is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition.

The soil group is determined from published soil maps for the area, which correlate each soil name with the soil group. Land use and treatment class are determined during field visits or from aerial photographs. Procedures for determining land use and treatment class are found in Chapter 8 of the National Engineering Handbook, Section 4 (SCS, 1985). An antecedent moisture condition II (AMC-II) is recommended for the City of Steamboat Springs.

Having determined the soil group, land use and treatment class, and the antecedent moisture condition, CN values can be determined from Table 5.6.4 below. Additionally, for undeveloped areas, Table 5.6.5 may be used if the site meets any of the more specific cover types listed there. In watersheds with varying land use, a composite CN may also be calculated directly from imperviousness estimates using the following equation.

\[
CN = 98*\text{Imp} + X*(1-\text{Imp})
\]  

(5.6.6)

Where:

\( \text{Imp} \) = Imperviousness as a decimal
\( X \) = Adjustment factor based on NRCS Soil Type

<table>
<thead>
<tr>
<th>NRCS Soil Type</th>
<th>Adjustment Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>39</td>
</tr>
<tr>
<td>B</td>
<td>61</td>
</tr>
<tr>
<td>C</td>
<td>74</td>
</tr>
<tr>
<td>D</td>
<td>80</td>
</tr>
</tbody>
</table>

5.6.2.3.2 Losses

Once the curve number is determined, precipitation loss can be determined by first calculating the soil moisture storage deficit and then the initial abstraction using the equations below. The HEC-HMS model will also calculate loss and accumulated runoff when the initial abstraction is entered into the loss tab when using the SCS methodology.

\[
Q = \frac{(P - IA)^2}{((P - IA) + S)}
\]  

(5.6.7)

\[
S = \frac{1,000}{CN} - 10
\]  

(5.6.8)

\[
IA = 0.2 \times S
\]  

(5.6.9)

Where:

\( Q \) = Accumulated Excess (in)
\( P \) = Accumulated Rainfall Depth (in)
\( IA \) = Initial abstraction (in)
\( S \) = Currently Available Soil Moisture Storage Deficit (in)
\( CN \) = SCS Curve number
Exhibit 4-II  Unit peak discharge ($q_u$) for NRCS (SCS) type II rainfall distribution
Table 4-2
Adjustment factor ($F_p$) for pond and swamp areas that are spread throughout the watershed

<table>
<thead>
<tr>
<th>Percentage of pond and swamp areas</th>
<th>$F_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00</td>
</tr>
<tr>
<td>0.2</td>
<td>0.97</td>
</tr>
<tr>
<td>1.0</td>
<td>0.87</td>
</tr>
<tr>
<td>3.0</td>
<td>0.75</td>
</tr>
<tr>
<td>5.0</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Limitations
The Graphical method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular Hydrograph method (chapter 5). Use TR-20 if the watershed is very complex or a higher degree of accuracy is required.

- The watershed must be hydrologically homogeneous, that is, describable by one CN. Land use, soils, and cover are distributed uniformly throughout the watershed.
- The watershed may have only one main stream or, if more than one, the branches must have nearly equal $T_c$'s.
- The method cannot perform valley or reservoir routing.
- The $F_p$ factor can be applied only for ponds or swamps that are not in the $T_c$ flow path.
- Accuracy of peak discharge estimated by this method will be reduced if $I_a/P$ values are used that are outside the range given in exhibit 4. The limiting $I_a/P$ values are recommended for use.
- This method should be used only if the weighted CN is greater than 40.

- When this method is used to develop estimates of peak discharge for both present and developed conditions of a watershed, use the same procedure for estimating $T_c$.
- $T_c$ values with this method may range from 0.1 to 10 hours.

Example 4-1
Compute the 25-year peak discharge for the 250-acre watershed described in examples 2-2 and 3-1. Figure 4-2 shows how worksheet 4 is used to compute $q_p$ as 345 cfs.
The purpose of this calculation package is to evaluate the channel capacity and erosion protection for the peak flows from the drainage basins shown on Figure 1 at Hayden during the 100-year, 24 hour storm event.

Background: The Hayden ash disposal facilities (ADF) design being completed as part of the engineering, design and operations plan (EDOP). Stormwater on the ADF is currently conveyed through existing drainage channels. Through each phase of development additional channels will be constructed to accommodate peak flows from all ADF drainage basins.

Methodology: Capacity of the channels and benches were evaluated using the Manning's equation included in the Hydraflow extension for AutoCAD Civil 3D. The bench channel evaluated represents the bench with the largest tributary area. Perimeter channels are designed with shallow slopes (0.5 - 2.0%) and do not experience high velocities or shear stresses. The existing access road ditch will receive the peak flows from Channels A3 and C3 and was evaluated assuming the sum of peak flows from the two channels. The downslope channels that collect and convey flows from the benches to the perimeter channels are designed at steep (3:1) slopes and will experience high velocities and shear forces. These downslope channels will be stabilized using a turf reinforcement mat (TRM) and Flexamat lining. The velocity and shear force for each Flexamat-lined channel was calculated and compared against the maximum value provided by the manufacturer.

Attachments: 1. Drainage Area Map (Figure 1)
2. AutoCAD Civil 3D Hydraflow (2018)
3. Flexamat Data Sheet (Attachment 1)
4. Flexamat Large-Scale Channel Erosion Testing Report (Attachment 2)

Conclusions:

<table>
<thead>
<tr>
<th>Channel ID</th>
<th>Peak Flows (cfs)</th>
<th>Channel Slope - Steep</th>
<th>Channel Slope - Shallow</th>
<th>Channel Lining</th>
<th>Channel Bottom Width (ft)</th>
<th>Flow Depth - Steep (ft)</th>
<th>Flow Depth - Shallow (ft)</th>
<th>Channel Design Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>2.7</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.27</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>A2</td>
<td>19.0</td>
<td>17.0%</td>
<td>3.5%</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.42</td>
<td>0.65</td>
<td>2.0</td>
</tr>
<tr>
<td>A3</td>
<td>11.9</td>
<td>1.8%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.43</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>B1</td>
<td>28.4</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.43</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C1</td>
<td>10.5</td>
<td>6.0%</td>
<td>0.5%</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.40</td>
<td>0.81</td>
<td>2.0</td>
</tr>
<tr>
<td>C2</td>
<td>12.2</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.63</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C3</td>
<td>7.3</td>
<td>1.0%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.39</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C4</td>
<td>2.4</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.11</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D1</td>
<td>22.0</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>12.0</td>
<td>0.66</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D2</td>
<td>14.2</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.29</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D3</td>
<td>12.5</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.27</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>Bench</td>
<td>8.8</td>
<td>1.0%</td>
<td>--</td>
<td>Grass</td>
<td>--</td>
<td>0.60</td>
<td>--</td>
<td>1.0</td>
</tr>
<tr>
<td>Access Road Ditch</td>
<td>19.2</td>
<td>5.5%</td>
<td>--</td>
<td>Grass</td>
<td>--</td>
<td>1.04</td>
<td>--</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Prepared By: Tyler Schmidt, EIT
Checked By: Joshua Lee, PE
Approved By: ___________________________ Date: 3/13/2018

Date: 3/19/2018

Date: 3/21/18
CHANNEL A1

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6648.00
- Slope (%) = 0.50
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 0.27
- Q (cfs) = 2.700
- Area (sqft) = 2.11
- Velocity (ft/s) = 1.28
- Wetted Perim (ft) = 8.71
- Crit Depth, Yc (ft) = 0.17
- Top Width (ft) = 8.62
- EGL (ft) = 0.30

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 2.70

---

**Elevation (ft)**

---

**Section**

---

**Depth (ft)**

---

**Reach (ft)**

---
CHANNEL A2 - SHALLOW

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6640.00
Slope (%) = 3.50
N-Value = 0.055

Highlighted
Depth (ft) = 0.65
Q (cfs) = 19.00
Area (sqft) = 5.82
Velocity (ft/s) = 3.27
Wetted Perim (ft) = 11.11
Crit Depth, Yc (ft) = 0.57
Top Width (ft) = 10.90
EGL (ft) = 0.82

Calculations
Compute by: Known Q
Known Q (cfs) = 19.00
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Calculation by: TJS       date: 3/13/2018
Checked by: JLL         date: 3/19/2018

Channel ID: A2

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = A/P \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

Specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF*t_d \]

\[ t_p > SF*t_d \]
CHANNEL A3

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6623.00
Slope (%) = 1.80
N-Value = 0.030

Highlighted
Depth (ft) = 0.43
Q (cfs) = 11.90
Area (sqft) = 3.56
Velocity (ft/s) = 3.34
Wetted Perim (ft) = 9.72
Crit Depth, Yc (ft) = 0.43
Top Width (ft) = 9.58
EGL (ft) = 0.60

Calculations
Compute by: Known Q
Known Q (cfs) = 11.90

Elev (ft) Section
Depth (ft)

Reach (ft)
Channel Report

CHANNEL B1

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6634.00
Slope (%) = 33.30
N-Value = 0.055

Highlighted
Depth (ft) = 0.43
Q (cfs) = 28.40
Area (sqft) = 3.56
Velocity (ft/s) = 7.97
Wetted Perim (ft) = 9.72
Crit Depth, Yc (ft) = 0.72
Top Width (ft) = 9.58
EGL (ft) = 1.42

Calculations
Compute by: Known Q
Known Q (cfs) = 28.40
STORM WATER MANAGEMENT CALCULATION

Channel ID: B1

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = A/P \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF \times t_d \]

\[ t_p > SF \times t_d \]
CHANNEL C1 - SHALLOW

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6669.00
Slope (%) = 0.50
N-Value = 0.055

Highlighted
Depth (ft) = 0.81
Q (cfs) = 10.50
Area (sqft) = 7.64
Velocity (ft/s) = 1.37
Wetted Perim (ft) = 12.12
Crit Depth, Yc (ft) = 0.39
Top Width (ft) = 11.86
EGL (ft) = 0.84

Calculations
Compute by: Known Q
Known Q (cfs) = 10.50
STORM WATER MANAGEMENT CALCULATION

Calculation by: TJS  date: 3/13/2018
Checked by:  JLL  date: 3/19/2018

Channel ID: C1

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = \frac{A}{P} \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF*t_d \]
\[ t_p > SF*t_d \]

Reference

- \( S_0 = 0.060 \text{ ft/ft} \) AutoCAD
- \( B = 7 \text{ ft} \)
- \( z = 3 \)
- \( n = 0.054 \) See Attachment 2-1
- \( Q = 10.5 \text{ ft}^3/\text{s} \) See Page 8
- \( d = 0.40 \text{ ft} \) See Page 8
- \( A = 3.28 \text{ ft}^2 \) See Page 8
- \( P = 12.12 \text{ ft} \) See Page 8
- \( R = 0.27 \text{ ft} \) See Page 8
- \( T = 9.40 \text{ ft} \) See Page 8
- \( V = 3.20 \text{ ft/s} \) See Page 8
- \( V_a = 30.0 \text{ ft/s} \) See Attachment 1
- \( y = 62.4 \text{ lb/ft}^3 \)
- \( t_d = 1.50 \text{ lb/ft}^2 \)
- \( t_p = 24.00 \text{ lb/ft}^2 \) See Attachment 1
- \( SF = 1.00 \)
- \( SF*t_d = 1.50 \text{ lb/ft}^2 \)
Channel Report

CHANNEL C2

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6648.00
Slope (%) = 0.50
N-Value = 0.030

Highlighted
Depth (ft) = 0.63
Q (cfs) = 12.20
Area (sqft) = 5.60
Velocity (ft/s) = 2.18
Wetted Perim (ft) = 10.98
Crit Depth, Yc (ft) = 0.43
Top Width (ft) = 10.78
EGL (ft) = 0.70

Calculations
Compute by: Known Q
Known Q (cfs) = 12.20

Elev (ft)

Section

Depth (ft)

Reach (ft)
**CHANNEL C3**

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6639.00
- Slope (%) = 1.00
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 0.39
- Q (cfs) = 7.300
- Area (sqft) = 3.19
- Velocity (ft/s) = 2.29
- Wetted Perim (ft) = 9.47
- Crit Depth, Yc (ft) = 0.31
- Top Width (ft) = 9.34
- EGL (ft) = 0.47

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 7.30
CHANNEL C4

Trapezoidal

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Width (ft)</td>
<td>7.00</td>
</tr>
<tr>
<td>Side Slopes (z:1)</td>
<td>3.00, 3.00</td>
</tr>
<tr>
<td>Total Depth (ft)</td>
<td>2.00</td>
</tr>
<tr>
<td>Invert Elev (ft)</td>
<td>6664.00</td>
</tr>
<tr>
<td>Slope (%)</td>
<td>33.33</td>
</tr>
<tr>
<td>N-Value</td>
<td>0.055</td>
</tr>
</tbody>
</table>

Highlighted

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (ft)</td>
<td>0.11</td>
</tr>
<tr>
<td>Q (cfs)</td>
<td>2.400</td>
</tr>
<tr>
<td>Area (sqft)</td>
<td>0.81</td>
</tr>
<tr>
<td>Velocity (ft/s)</td>
<td>2.98</td>
</tr>
<tr>
<td>Wetted Perim (ft)</td>
<td>7.70</td>
</tr>
<tr>
<td>Crit Depth, Yc (ft)</td>
<td>0.16</td>
</tr>
<tr>
<td>Top Width (ft)</td>
<td>7.66</td>
</tr>
<tr>
<td>EGL (ft)</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Calculations

Compute by:
- Known Q
- Known Q (cfs) = 2.40
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Channel ID: C4

Shear Stress

Average channel slope, \( S_0 = \frac{\Delta \text{elev}}{\text{length}} \)
Bottom Width, \( B \) = 7 ft
Side slope eg. 3:1, \( z \) = 3
Manning's Roughness coefficient, \( n \) = 0.054

Design Flow Rate, \( Q \) = 2.4 ft\(^3\)/s
Depth of Flow, \( d \) = 0.11 ft
Cross-sectional Area of Flow Prism, \( A \) = 0.81 ft\(^2\)
Wetted Perimeter of Flow Prism, \( P \) = 7.70 ft
Hydraulic Radius, \( R = \frac{A}{P} \)
Channel Top Width (water surface), \( T \) = 7.66 ft
Flow Velocity, \( V \) = 2.98 ft/s
Maximum Allowable Velocity, \( V_a \) = 30.0 ft/s

\[ V_a > V \]

specific weight of water, \( y \) = 62.4 lb/ft\(^3\)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \frac{t_p}{SF^*t_d} \)

\[ t_p > SF^*t_d \]
Channel Report

CHANNEL D1

Trapezoidal
Bottom Width (ft) = 12.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6610.00
Slope (%) = 0.50
N-Value = 0.030

Highlighted
Depth (ft) = 0.66
Q (cfs) = 22.00
Area (sqft) = 9.23
Velocity (ft/s) = 2.38
Wetted Perim (ft) = 16.17
Crit Depth, Yc (ft) = 0.46
Top Width (ft) = 15.96
EGL (ft) = 0.75

Calculations
Compute by: Known Q
Known Q (cfs) = 22.00

Elev (ft)  |  Section  |  Depth (ft)
--- | --- | ---
6613.00 |  | 3.00
6612.50 |  | 2.50
6612.00 |  | 2.00
6611.50 |  | 1.50
6611.00 |  | 1.00
6610.50 |  | 0.50
6610.00 |  | 0.00
6609.50 |  | -0.50
0 |  | 35
5 |  | 30
10 |  | 25
15 |  | 20
20 |  | 15
25 |  | 10
30 |  | 5
35 |  | 0

Reach (ft)
CHANNEL D2

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6610.00
Slope (%) = 33.33
N-Value = 0.055

Highlighted
Depth (ft) = 0.29
Q (cfs) = 14.20
Area (sqft) = 2.28
Velocity (ft/s) = 6.22
Wetted Perim (ft) = 8.83
Crit Depth, Yc (ft) = 0.47
Top Width (ft) = 8.74
EGL (ft) = 0.89

Calculations
Compute by: Known Q
Known Q (cfs) = 14.20

Elev (ft)  Section  Depth (ft)
6613.00
6612.50
6612.00
6611.50
6611.00
6610.50
6610.00
6609.50
0  2  4  6  8  10  12  14  16  18  20  22  24
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
Hayden Station Ash Disposal Facility  
STORM WATER MANAGEMENT CALCULATION  

Calculation by: TJS  
date: 3/13/2018  
Checked by: JLL  
date: 3/19/2018  

Channel ID: D2

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)

<table>
<thead>
<tr>
<th>( S_0 )</th>
<th>0.33 ft/ft</th>
</tr>
</thead>
</table>

Bottom Width, \( B \)

| \( B \) | 7 ft |

Side slope eg. 3:1, \( z \)

| \( z \) | 3 |

Manning's Roughness coefficient, \( n \)

| \( n \) | 0.054 |

Design Flow Rate, \( Q \)

| \( Q \) | 14.2 ft\(^3\)/s |

Depth of Flow, \( d \)

| \( d \) | 0.29 ft |

Cross-sectional Area of Flow Prism, \( A \)

| \( A \) | 2.28 ft\(^2\) |

Wetted Perimeter of Flow Prism, \( P \)

| \( P \) | 8.83 ft |

Hydraulic Radius, \( R = \frac{A}{P} \)

| \( R \) | 0.26 ft |

Channel Top Width (water surface), \( T \)

| \( T \) | 8.74 ft |

Flow Velocity, \( V \)

| \( V \) | 6.22 ft/s |

Maximum Allowable Velocity, \( V_a \)

| \( V_a \) | 30.0 ft/s |

Specific weight of water, \( y \)

| \( y \) | 62.4 lb/ft\(^3\) |

Shear Stress at max depth, \( t_d = ydS_0 \)

| \( t_d \) | 6.03 lb/ft\(^2\) |

Maximum Allowable Shear Stress, \( t_p \)

| \( t_p \) | 24.00 lb/ft\(^2\) |

Safety Factor, \( SF = \frac{SF*t_d}{SF*t_d} \)

| SF | 1.00 |

\( t_p > SF*t_d \)

OK
### CHANNEL D3

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6653.00
- Slope (%) = 33.33
- N-Value = 0.055

**Highlighted**
- Depth (ft) = 0.27
- Q (cfs) = 12.50
- Area (sqft) = 2.11
- Velocity (ft/s) = 5.93
- Wetted Perim (ft) = 8.71
- Crit Depth, Yc (ft) = 0.44
- Top Width (ft) = 8.62
- EGL (ft) = 0.82

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 12.50

---

<table>
<thead>
<tr>
<th>Elev (ft)</th>
<th>Section</th>
<th>Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6653.00</td>
<td></td>
<td>-0.50</td>
</tr>
<tr>
<td>6653.50</td>
<td></td>
<td>-0.50</td>
</tr>
<tr>
<td>6654.00</td>
<td></td>
<td>0.00</td>
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<tr>
<td>6654.50</td>
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<td>0.00</td>
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<tr>
<td>6655.00</td>
<td></td>
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<tr>
<td>6655.50</td>
<td></td>
<td>0.50</td>
</tr>
<tr>
<td>6656.00</td>
<td></td>
<td>1.00</td>
</tr>
</tbody>
</table>

---

Reach (ft)
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Calculation by: TJS    date: 3/13/2018
Checked by: JLL    date: 3/19/2018

Channel ID: D3

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = \frac{A}{P} \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF \times t_d \]

\[ t_p > SF \times t_d \]
## Bench

### Triangular
- Side Slopes (z:1) = 20.00, 3.00
- Total Depth (ft) = 1.00
- Invert Elev (ft) = 6650.00
- Slope (%) = 1.00
- N-Value = 0.030

### Calculations
- Compute by: Known Q
- Known Q (cfs) = 8.80

### Highlighted
- Depth (ft) = 0.60
- Q (cfs) = 8.800
- Area (sqft) = 4.14
- Velocity (ft/s) = 2.13
- Wetted Perim (ft) = 13.91
- Crit Depth, Yc (ft) = 0.52
- Top Width (ft) = 13.80
- EGL (ft) = 0.67

### Elev (ft) vs Section

![Graph of Elev (ft) vs Section with specific data points and calculations displayed.](image-url)
Access Road Ditch

**Triangular**
- Side Slopes (z:1) = 2.50, 2.50
- Total Depth (ft) = 2.50
- Invert Elev (ft) = 6600.00
- Slope (%) = 5.50
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 1.04
- Q (cfs) = 19.20
- Area (sqft) = 2.70
- Velocity (ft/s) = 7.10
- Wetted Perim (ft) = 5.60
- Crit Depth, Yc (ft) = 1.30
- Top Width (ft) = 5.20
- EGL (ft) = 1.82

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 19.20
**DRAINAGE BASIN SUMMARY TABLE**

<table>
<thead>
<tr>
<th>BASIN ID</th>
<th>AREA (acres)</th>
<th>100 YEAR PEAK FLOW (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20.7</td>
<td>19.0</td>
</tr>
<tr>
<td>A1</td>
<td>2.9</td>
<td>2.7</td>
</tr>
<tr>
<td>BENCH</td>
<td>9.0</td>
<td>6.8</td>
</tr>
<tr>
<td>B</td>
<td>30.2</td>
<td>33.5</td>
</tr>
<tr>
<td>B1</td>
<td>21.7</td>
<td>28.4</td>
</tr>
<tr>
<td>C</td>
<td>14.1</td>
<td>12.2</td>
</tr>
<tr>
<td>C1</td>
<td>11.4</td>
<td>10.5</td>
</tr>
<tr>
<td>C2</td>
<td>2.1</td>
<td>2.4</td>
</tr>
<tr>
<td>D</td>
<td>83.9</td>
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<td>D1</td>
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<tr>
<td>D2</td>
<td>16.0</td>
<td>14.2</td>
</tr>
<tr>
<td>D3</td>
<td>11.5</td>
<td>12.5</td>
</tr>
</tbody>
</table>

**LEGEND**
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- DESIGN MAJOR CONTOUR
- DESIGN MINOR CONTOUR
- BASIN BOUNDARY
- SUB BASIN BOUNDARY
- TIME OF CONCENTRATION
- FLOW PATH
- ACCESS ROAD
- BASIN POINT OF INTEREST

**FIGURE 1**
POST DEVELOPMENT DRAINAGE BASIN MAP
XCEL ENERGY HAYDEN, CO
Data Sheet

Flexamat® is a tied concrete block mat used to control erosion in swales, slopes, ditches, channels, shorelines and any area where soil sediment may be lost due to water runoff.

The matting consists of pyramidal concrete blocks that are interconnected utilizing a high tensile strength polypropylene geogrid. The completed mat yields a high strength, ultra-flexible hard armor system of Erosion Control. Flexamat’s superior Percentage of Open Area (POA) affords an ideal zone for vegetation growth while remaining a permanent armor against long-term erosional forces.

General Composition of Materials

<table>
<thead>
<tr>
<th>Blocks</th>
<th>5000 PSI, Wet-cast Portland Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interlocking Biaxial Geogrid</td>
<td>Fornit 30/30 – Polypropylene Geogrid with 2,055 lb/ft biaxial strength</td>
</tr>
<tr>
<td>Underlayment Options</td>
<td><strong>Standard</strong> Polypropylene netting Curlex® II ECB <strong>Plus</strong> Polypropylene netting Recyclex® TRM-V Culrex® II ECB <strong>Fabric</strong> 10 oz NW fabric *More options available upon request</td>
</tr>
</tbody>
</table>

Manufacturing Values

<table>
<thead>
<tr>
<th>Flexamat Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roll Width</td>
<td>4’, 5.5’, 8’, 10’, 12’ 16’</td>
</tr>
<tr>
<td>Roll Length</td>
<td>30’, 40’, 50’ / custom</td>
</tr>
<tr>
<td>Material Weight</td>
<td>10 lbs./sf</td>
</tr>
<tr>
<td>Block Size</td>
<td>6.5” x 6.5” x 2.25”</td>
</tr>
<tr>
<td>Percentage Open Area (POA)</td>
<td>30% min.</td>
</tr>
</tbody>
</table>

Performance

<table>
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<tr>
<th>Test</th>
<th>Tested Value</th>
<th>Bed Slope</th>
<th>Soil Classification</th>
<th>Limiting Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM 6460</td>
<td>Shear Stress</td>
<td>30%</td>
<td>Sandy Loam (USDA)</td>
<td>24 PSF</td>
</tr>
<tr>
<td>ASTM 6460</td>
<td>Velocity</td>
<td>20%</td>
<td>Loam (USDA)</td>
<td>30 ft./sec</td>
</tr>
</tbody>
</table>
Large-Scale Channel Erosion Testing
(ASTM D 6460 modified)

of

Flexamat
Channel Lining
over
Sandy Loam

February 2009

Submitted to:
Motz Enterprises, Inc.
9415 Montgomery Rd, Ste H
Cincinnati, Ohio 45242

Attn: Mr. Jim Motz

Submitted by:
TRI/Environmental, Inc.
9063 Bee Caves Road
Austin, TX 78733

C. Joel Sprague
Project Manager
TEST RESULTS

Average soil loss and the associated hydraulic shear calculated from flow and depth measurements made during the testing are the principle data used to determine the performance of the product tested. This data is entered into a spreadsheet that transforms the flow depth and velocity into an hydraulic shear stress and the soil loss measurements into average Clopper Soil Loss Index (CSLI). A graph of shear versus soil loss for the protected condition is shown in Figure 9. The associated velocities are plotted in Figure 10. The graphs include a polynomial regression line fit to the test data to facilitate a projection of the limiting shear stress, $\tau_{\text{limit}}$, and limiting velocity, $V_{\text{limit}}$, since $\frac{1}{2}$-inch of soil loss was not achieved during testing.

<table>
<thead>
<tr>
<th>Test # (run # - target shear)</th>
<th>Flow depth (in)</th>
<th>Flow velocity (fps)</th>
<th>Flow (cfs)</th>
<th>Manning’s roughness, $n$</th>
<th>Max Bed Shear Stress (psf)</th>
<th>CSLI (in)</th>
<th>Cumm. CSLI (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1-4</td>
<td>3.79</td>
<td>6.56</td>
<td>4.13</td>
<td>0.058</td>
<td>5.82</td>
<td>-0.06</td>
<td>-0.06</td>
</tr>
<tr>
<td>R1-8</td>
<td>5.07</td>
<td>8.88</td>
<td>7.48</td>
<td>0.052</td>
<td>7.79</td>
<td>-0.05</td>
<td>-0.11</td>
</tr>
<tr>
<td>R1-12</td>
<td>6.99</td>
<td>11.06</td>
<td>12.87</td>
<td>0.051</td>
<td>10.74</td>
<td>-0.07</td>
<td>-0.18</td>
</tr>
<tr>
<td>R1-16</td>
<td>11.03</td>
<td>14.88</td>
<td>27.30</td>
<td>0.052</td>
<td>16.95</td>
<td>-0.11</td>
<td>-0.29</td>
</tr>
<tr>
<td>R2-4</td>
<td>3.61</td>
<td>6.38</td>
<td>3.82</td>
<td>0.058</td>
<td>5.55</td>
<td>-0.04</td>
<td>-0.04</td>
</tr>
<tr>
<td>R2-8</td>
<td>5.21</td>
<td>8.69</td>
<td>7.53</td>
<td>0.054</td>
<td>8.00</td>
<td>-0.05</td>
<td>-0.09</td>
</tr>
<tr>
<td>R2-12</td>
<td>7.10</td>
<td>10.81</td>
<td>12.77</td>
<td>0.053</td>
<td>10.92</td>
<td>-0.05</td>
<td>-0.14</td>
</tr>
<tr>
<td>R2-16</td>
<td>10.80</td>
<td>14.56</td>
<td>26.19</td>
<td>0.052</td>
<td>16.60</td>
<td>-0.11</td>
<td>-0.25</td>
</tr>
<tr>
<td>R3-4</td>
<td>3.53</td>
<td>6.31</td>
<td>3.70</td>
<td>0.057</td>
<td>5.42</td>
<td>-0.04</td>
<td>-0.04</td>
</tr>
<tr>
<td>R3-8</td>
<td>5.31</td>
<td>8.56</td>
<td>7.58</td>
<td>0.055</td>
<td>8.17</td>
<td>-0.07</td>
<td>-0.11</td>
</tr>
<tr>
<td>R3-12</td>
<td>6.88</td>
<td>10.63</td>
<td>12.17</td>
<td>0.053</td>
<td>10.57</td>
<td>-0.07</td>
<td>-0.17</td>
</tr>
<tr>
<td>R3-16</td>
<td>10.88</td>
<td>14.88</td>
<td>26.95</td>
<td>0.051</td>
<td>16.71</td>
<td>-0.13</td>
<td>-0.30</td>
</tr>
</tbody>
</table>
The purpose of this calculation package is to evaluate the channel capacity and erosion protection for the peak flows from the drainage basins shown on Figure 1 at Hayden during the 100-year, 24 hour storm event.

Background: The Hayden ash disposal facilities (ADF) design being completed as part of the engineering, design and operations plan (EDOP). Stormwater on the ADF is currently conveyed through existing drainage channels. Through each phase of development additional channels will be constructed to accommodate peak flows from all ADF drainage basins.

Methodology: Capacity of the channels and benches were evaluated using the Manning's equation included in the Hydraflow extension for AutoCAD Civil 3D. The bench channel evaluated represents the bench with the largest tributary area. Perimeter channels are designed with shallow slopes (0.5 - 2.0%) and do not experience high velocities or shear stresses. The existing access road ditch will recieve the peak flows from Channels A3 and C3 and was evaluated assuming the sum of peak flows from the two channels. The downslope channels that collect and convey flows from the benches to the perimeter channels are designed at steep (3:1) slopes and will experience high velocities and shear forces. These downslope channels will be stabilized using a turf reinforcement mat (TRM) and Flexamat lining. The velocity and shear force for each Flexamat-lined channel was calculated and compared against the maximum value provided by the manufacturer.

Attachments: 1. Drainage Area Map (Figure 1)
   2. AutoCAD Civil 3D Hydraflow (2018)
   3. Flexamat Data Sheet (Attachment 1)
   3. Flexamat Large-Scale Channel Erosion Testing Report (Attachment 2)

Conclusions:

<table>
<thead>
<tr>
<th>Channel ID</th>
<th>Peak Flows (cfs)</th>
<th>Channel Slope - Steep</th>
<th>Channel Slope - Shallow</th>
<th>Channel Lining</th>
<th>Channel Bottom Width (ft)</th>
<th>Flow Depth - Steep (ft)</th>
<th>Flow Depth - Shallow (ft)</th>
<th>Channel Design Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>2.7</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.27</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>A2</td>
<td>19.0</td>
<td>17.0%</td>
<td>3.5%</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.42</td>
<td>0.65</td>
<td>2.0</td>
</tr>
<tr>
<td>A3</td>
<td>11.9</td>
<td>1.8%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.43</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>B1</td>
<td>28.4</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.43</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C1</td>
<td>10.5</td>
<td>6.0%</td>
<td>0.5%</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.40</td>
<td>0.81</td>
<td>2.0</td>
</tr>
<tr>
<td>C2</td>
<td>12.2</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.63</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C3</td>
<td>7.3</td>
<td>1.0%</td>
<td>--</td>
<td>Grass</td>
<td>7.0</td>
<td>0.39</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>C4</td>
<td>2.4</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.11</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D1</td>
<td>22.0</td>
<td>0.5%</td>
<td>--</td>
<td>Grass</td>
<td>12.0</td>
<td>0.66</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D2</td>
<td>14.2</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.29</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>D3</td>
<td>12.5</td>
<td>33.3%</td>
<td>--</td>
<td>Flexamat w/ TRM</td>
<td>7.0</td>
<td>0.27</td>
<td>--</td>
<td>2.0</td>
</tr>
<tr>
<td>Bench</td>
<td>8.8</td>
<td>1.0%</td>
<td>--</td>
<td>Grass</td>
<td>--</td>
<td>0.60</td>
<td>--</td>
<td>1.0</td>
</tr>
<tr>
<td>Access Road Ditch</td>
<td>19.2</td>
<td>5.5%</td>
<td>--</td>
<td>Grass</td>
<td>--</td>
<td>1.04</td>
<td>--</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Channel Report

Hydraflo Express Extension for Autodesk AutoCAD® Civil 3D® by Autodesk, Inc.

Friday, Mar 9 2018

CHANNEL A1

Trapezoidal

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Width (ft)</td>
<td>7.00</td>
</tr>
<tr>
<td>Side Slopes (z:1)</td>
<td>3.00, 3.00</td>
</tr>
<tr>
<td>Total Depth (ft)</td>
<td>2.00</td>
</tr>
<tr>
<td>Invert Elev (ft)</td>
<td>6648.00</td>
</tr>
<tr>
<td>Slope (%)</td>
<td>0.50</td>
</tr>
<tr>
<td>N-Value</td>
<td>0.030</td>
</tr>
</tbody>
</table>

Highlighted

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (ft)</td>
<td>0.27</td>
</tr>
<tr>
<td>Q (cfs)</td>
<td>2.700</td>
</tr>
<tr>
<td>Area (sqft)</td>
<td>2.11</td>
</tr>
<tr>
<td>Velocity (ft/s)</td>
<td>1.28</td>
</tr>
<tr>
<td>Wetted Perim (ft)</td>
<td>8.71</td>
</tr>
<tr>
<td>Crit Depth, Yc (ft)</td>
<td>0.17</td>
</tr>
<tr>
<td>Top Width (ft)</td>
<td>8.62</td>
</tr>
<tr>
<td>EGL (ft)</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Calculations

Compute by: Known Q
Known Q (cfs) = 2.70

Diagram:

Elev (ft)       | Section            | Depth (ft) |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6651.00</td>
<td></td>
<td>3.00</td>
</tr>
<tr>
<td>6650.50</td>
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<td>2.50</td>
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<td>6650.00</td>
<td></td>
<td>2.00</td>
</tr>
<tr>
<td>6649.50</td>
<td></td>
<td>1.50</td>
</tr>
<tr>
<td>6649.00</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>6648.50</td>
<td></td>
<td>0.50</td>
</tr>
<tr>
<td>6648.00</td>
<td></td>
<td>0.00</td>
</tr>
<tr>
<td>6647.50</td>
<td></td>
<td>-0.50</td>
</tr>
</tbody>
</table>

Reach (ft)
CHANNEL A2 - SHALLOW

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6640.00
Slope (%) = 3.50
N-Value = 0.055

Highlighted
Depth (ft) = 0.65
Q (cfs) = 19.00
Area (sqft) = 5.82
Velocity (ft/s) = 3.27
Wetted Perim (ft) = 11.11
Crit Depth, Yc (ft) = 0.57
Top Width (ft) = 10.90
EGL (ft) = 0.82

Calculations
Compute by: Known Q
Known Q (cfs) = 19.00
## Storm Water Management Calculation

**Channel ID:** A2

### Shear Stress

- **Average channel slope,** $S_0 = \Delta \text{elev} / \text{length}$
- **Bottom Width,** $B = 7$ ft
- **Side slope eg. 3:1,** $z = 3$
- **Manning's Roughness coefficient,** $n = 0.054$

### Design Flow Rate, $Q$

<table>
<thead>
<tr>
<th>$Q$</th>
<th>19.0 $\text{ft}^3/\text{s}$</th>
</tr>
</thead>
</table>

### Depth of Flow, $d$

<table>
<thead>
<tr>
<th>$d$</th>
<th>0.42 ft</th>
</tr>
</thead>
</table>

### Cross-sectional Area of Flow Prism, $A$

<table>
<thead>
<tr>
<th>$A$</th>
<th>3.47 $\text{ft}^2$</th>
</tr>
</thead>
</table>

### Wetted Perimeter of Flow Prism, $P$

<table>
<thead>
<tr>
<th>$P$</th>
<th>11.11 ft</th>
</tr>
</thead>
</table>

### Hydraulic Radius, $R = A/P$

<table>
<thead>
<tr>
<th>$R$</th>
<th>0.31 ft</th>
</tr>
</thead>
</table>

### Channel Top Width (water surface), $T$

<table>
<thead>
<tr>
<th>$T$</th>
<th>9.52 ft</th>
</tr>
</thead>
</table>

### Flow Velocity, $V$

<table>
<thead>
<tr>
<th>$V$</th>
<th>5.48 $\text{ft/s}$</th>
</tr>
</thead>
</table>

### Maximum Allowable Velocity, $V_a$

<table>
<thead>
<tr>
<th>$V_a$</th>
<th>30.0 $\text{ft/s}$</th>
</tr>
</thead>
</table>

### Specific weight of water, $y$

<table>
<thead>
<tr>
<th>$y$</th>
<th>62.4 $\text{lb/ft}^3$</th>
</tr>
</thead>
</table>

### Shear Stress at max depth, $t_d = ydS_0$

<table>
<thead>
<tr>
<th>$t_d$</th>
<th>4.46 lb/ft²</th>
</tr>
</thead>
</table>

### Maximum Allowable Shear Stress, $t_p$

<table>
<thead>
<tr>
<th>$t_p$</th>
<th>24.00 lb/ft²</th>
</tr>
</thead>
</table>

### Safety Factor, $SF = SF^*t_d$

<table>
<thead>
<tr>
<th>$SF^*t_d$</th>
<th>4.46 lb/ft²</th>
</tr>
</thead>
</table>

### Validation

- $V_a > V$
- $t_p > SF^*t_d$
### CHANNEL A3

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6623.00
- Slope (%) = 1.80
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 0.43
- Q (cfs) = 11.90
- Area (sqft) = 3.56
- Velocity (ft/s) = 3.34
- Wetted Perim (ft) = 9.72
- Crit Depth, Yc (ft) = 0.43
- Top Width (ft) = 9.58
- EGL (ft) = 0.60

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 11.90

---

**Elev (ft)**

<table>
<thead>
<tr>
<th>Elev (ft)</th>
<th>Section</th>
<th>Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6623.00</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>6623.50</td>
<td>4</td>
<td>0.50</td>
</tr>
<tr>
<td>6624.00</td>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td>6624.50</td>
<td>12</td>
<td>1.50</td>
</tr>
<tr>
<td>6625.00</td>
<td>16</td>
<td>2.00</td>
</tr>
<tr>
<td>6625.50</td>
<td>20</td>
<td>2.50</td>
</tr>
<tr>
<td>6626.00</td>
<td>24</td>
<td>3.00</td>
</tr>
</tbody>
</table>

---

**Reach (ft)**
CHANNEL B1

**Trapezoidal**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Width (ft)</td>
<td>7.00</td>
</tr>
<tr>
<td>Side Slopes (z:1)</td>
<td>3.00, 3.00</td>
</tr>
<tr>
<td>Total Depth (ft)</td>
<td>2.00</td>
</tr>
<tr>
<td>Invert Elev (ft)</td>
<td>6634.00</td>
</tr>
<tr>
<td>Slope (%)</td>
<td>33.30</td>
</tr>
<tr>
<td>N-Value</td>
<td>0.055</td>
</tr>
</tbody>
</table>

**Highlighted**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (ft)</td>
<td>0.43</td>
</tr>
<tr>
<td>Q (cfs)</td>
<td>28.40</td>
</tr>
<tr>
<td>Area (sqft)</td>
<td>3.56</td>
</tr>
<tr>
<td>Velocity (ft/s)</td>
<td>7.97</td>
</tr>
<tr>
<td>Wetted Perim (ft)</td>
<td>9.72</td>
</tr>
<tr>
<td>Crit Depth, Yc (ft)</td>
<td>0.72</td>
</tr>
<tr>
<td>Top Width (ft)</td>
<td>9.58</td>
</tr>
<tr>
<td>EGL (ft)</td>
<td>1.42</td>
</tr>
</tbody>
</table>

**Calculations**

- Compute by: Known Q
- Known Q (cfs) = 28.40
Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = A/P \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF*t_d \]

\[ t_p > SF^*t_d \]
**CHANNEL C1 - SHALLOW**

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6669.00
- Slope (%) = 0.50
- N-Value = 0.055

**Highlighted**
- Depth (ft) = 0.81
- Q (cfs) = 10.50
- Area (sqft) = 7.64
- Velocity (ft/s) = 1.37
- Wetted Perim (ft) = 12.12
- Crit Depth, Yc (ft) = 0.39
- Top Width (ft) = 11.86
- EGL (ft) = 0.84

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 10.50

---

**Elev (ft)**

**Section**

**Depth (ft)**

---

Reach (ft)
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Calculation by: TJS
date: 3/13/2018

Checked by: JLL
date: 3/19/2018

Channel ID: C1

Shear Stress

Average channel slope, \( S_0 = \frac{\Delta \text{elev}}{\text{length}} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

| \( S_0 \) | 0.060 ft/ft |
| \( B \) | 7 ft |
| \( z \) | 3 |
| \( n \) | 0.054 |

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = \frac{A}{P} \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

| \( Q \) | 10.5 ft³/s |
| \( d \) | 0.40 ft |
| \( A \) | 3.28 ft² |
| \( P \) | 12.12 ft |
| \( R \) | 0.27 ft |
| \( T \) | 9.40 ft |
| \( V \) | 3.20 ft/s |
| \( V_a \) | 30.0 ft/s |

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\( SF \times t_d \)

\[ t_p > SF \times t_d \]

| \( y \) | 62.4 lb/ft³ |
| \( t_d \) | 1.50 lb/ft² |
| \( t_p \) | 24.00 lb/ft² |
| \( SF \) | 1.00 |
| \( SF \times t_d \) | 1.50 lb/ft² |

Reference

AutoCAD
See Page 8
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See Page 8
See Attachment 2-1
See Attachment 1

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CHANNEL C2

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6648.00
Slope (%) = 0.50
N-Value = 0.030

Highlighted
Depth (ft) = 0.63
Q (cfs) = 12.20
Area (sqft) = 5.60
Velocity (ft/s) = 2.18
Wetted Perim (ft) = 10.98
Crit Depth, Yc (ft) = 0.43
Top Width (ft) = 10.78
EGL (ft) = 0.70

Calculations
Compute by: Known Q
Known Q (cfs) = 12.20
CHANNEL C3

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6639.00
Slope (%) = 1.00
N-Value = 0.030

Highlighted
Depth (ft) = 0.39
Q (cfs) = 7.300
Area (sqft) = 3.19
Velocity (ft/s) = 2.29
Wetted Perim (ft) = 9.47
Crit Depth, Yc (ft) = 0.31
Top Width (ft) = 9.34
EGL (ft) = 0.47

Calculations
Compute by: Known Q
Known Q (cfs) = 7.30

Elev (ft)  |  Section  | Depth (ft)
---|---|---
6642.00  |  | 3.00
6641.50  |  | 2.50
6641.00  |  | 2.00
6640.50  |  | 1.50
6640.00  |  | 1.00
6639.50  |  | 0.50
6639.00  |  | 0.00
6638.50  |  | -0.50
0  | 12 |
2  | 14 |
4  | 16 |
6  | 18 |
8  | 20 |
10 | 22 |
12 | 24 |
Reach (ft)
Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc.

Friday, Mar 9 2018

CHANNEL C4

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6664.00
Slope (%) = 33.33
N-Value = 0.055

Calculate
Compute by: Known Q
Known Q (cfs) = 2.40

Highlighted
Depth (ft) = 0.11
Q (cfs) = 2.400
Area (sqft) = 0.81
Velocity (ft/s) = 2.98
Wetted Perim (ft) = 7.70
Crit Depth, Yc (ft) = 0.16
Top Width (ft) = 7.66
EGL (ft) = 0.25

Elev (ft)                      Section                          Depth (ft)
6667.00                      0.00                          3.00
6666.50                      0.50                          2.50
6666.00                      1.00                          2.00
6665.50                      1.50                          1.50
6665.00                      2.00                          1.00
6664.50                      2.50                          0.50
6664.00                      3.00                          0.00
6663.50                      3.50                          -0.50

Reach (ft)
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Channel ID: C4

Shear Stress

Average channel slope, \( S_0 = \Delta \text{elev} / \text{length} \)
Bottom Width, \( B \)
Side slope eg. 3:1, \( z \)
Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)
Depth of Flow, \( d \)
Cross-sectional Area of Flow Prism, \( A \)
Wetted Perimeter of Flow Prism, \( P \)
Hydraulic Radius, \( R = A/P \)
Channel Top Width (water surface), \( T \)
Flow Velocity, \( V \)
Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

specific weight of water, \( y \)
Shear Stress at max depth, \( t_d = ydS_0 \)
Maximum Allowable Shear Stress, \( t_p \)
Safety Factor, \( SF = \)
\[ SF*t_d \]

\[ t_p > SF*t_d \]
 CHANNEL D1

**Trapezoidal**
- Bottom Width (ft) = 12.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6610.00
- Slope (%) = 0.50
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 0.66
- Q (cfs) = 22.00
- Area (sqft) = 9.23
- Velocity (ft/s) = 2.38
- Wetted Perim (ft) = 16.17
- Crit Depth, Yc (ft) = 0.46
- Top Width (ft) = 15.96
- EGL (ft) = 0.75

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 22.00

---

**Elev (ft)**

**Section**

**Depth (ft)**

Reach (ft)
CHANNEL D2

Trapezoidal
Bottom Width (ft) = 7.00
Side Slopes (z:1) = 3.00, 3.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6610.00
Slope (%) = 33.33
N-Value = 0.055

Highlighted
Depth (ft) = 0.29
Q (cfs) = 14.20
Area (sqft) = 2.28
Velocity (ft/s) = 6.22
Wetted Perim (ft) = 8.83
Crit Depth, Yc (ft) = 0.47
Top Width (ft) = 8.74
EGL (ft) = 0.89

Calculations
Compute by: Known Q
Known Q (cfs) = 14.20
Hayden Station Ash Disposal Facility

STORM WATER MANAGEMENT CALCULATION

Calculation by: TJS  date: 3/13/2018
Checked by: JLL      date: 3/19/2018

Channel ID: D2

Shear Stress

Average channel slope, \( S_0 = \frac{\Delta \text{elev}}{\text{length}} \)

Bottom Width, \( B \)

Side slope eg. 3:1, \( z \)

Manning's Roughness coefficient, \( n \)

Design Flow Rate, \( Q \)

Depth of Flow, \( d \)

Cross-sectional Area of Flow Prism, \( A \)

Wetted Perimeter of Flow Prism, \( P \)

Hydraulic Radius, \( R = \frac{A}{P} \)

Channel Top Width (water surface), \( T \)

Flow Velocity, \( V \)

Maximum Allowable Velocity, \( V_a \)

\[ V_a > V \]

Specific weight of water, \( y \)

Shear Stress at max depth, \( t_d = ydS_0 \)

Maximum Allowable Shear Stress, \( t_p \)

Safety Factor, \( SF = \frac{SF^*t_d}{t_p} \)

\[ t_p > SF^*t_d \]

Reference

AutoCAD

See Attachment 2-1

See Page 15

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See Page 15

See Attachment 1

OK

OK
**CHANNEL D3**

**Trapezoidal**
- Bottom Width (ft) = 7.00
- Side Slopes (z:1) = 3.00, 3.00
- Total Depth (ft) = 2.00
- Invert Elev (ft) = 6653.00
- Slope (%) = 33.33
- N-Value = 0.055

**Highlighted**
- Depth (ft) = 0.27
- Q (cfs) = 12.50
- Area (sqft) = 2.11
- Velocity (ft/s) = 5.93
- Wetted Perim (ft) = 8.71
- Crit Depth, Yc (ft) = 0.44
- Top Width (ft) = 8.62
- EGL (ft) = 0.82

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 12.50

---

**Graph**

The graph shows the elevation and depth changes along the channel. The x-axis represents the reach in feet, ranging from 0 to 24 feet, while the y-axis represents the elevation from 662.50 to 666.00 feet. The depth is indicated along the right side of the graph, ranging from 0.00 to 3.00 feet. The channel's profile is represented by a series of points, indicating the depth at various reaches. The highlighted section is marked with a blue triangle, indicating specific points or conditions within that section.
### Shear Stress

Average channel slope, $S_0 = \Delta \text{elev} / \text{length}$

Bottom Width, $B$

Side slope eg. 3:1, $z$

Manning's Roughness coefficient, $n$

Design Flow Rate, $Q$

Depth of Flow, $d$

Cross-sectional Area of Flow Prism, $A$

Wetted Perimeter of Flow Prism, $P$

Hydraulic Radius, $R = A/P$

Channel Top Width (water surface), $T$

Flow Velocity, $V$

Maximum Allowable Velocity, $V_a$

\[
V_a > V
\]

Specific weight of water, $y$

Shear Stress at max depth, $t_d = ydS_0$

Maximum Allowable Shear Stress, $t_p$

Safety Factor, $SF = SFS_d$

\[
t_p > SF^*t_d
\]
Bench

**Triangular**
- Side Slopes (z:1) = 20.00, 3.00
- Total Depth (ft) = 1.00
- Invert Elev (ft) = 6650.00
- Slope (%) = 1.00
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 0.60
- Q (cfs) = 8.800
- Area (sqft) = 4.14
- Velocity (ft/s) = 2.13
- Wetted Perim (ft) = 13.91
- Crit Depth, Yc (ft) = 0.52
- Top Width (ft) = 13.80
- EGL (ft) = 0.67

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 8.80

---

![Diagram](image-url)
Access Road Ditch

**Triangular**
- Side Slopes (z:1) = 2.50, 2.50
- Total Depth (ft) = 2.50

- Invert Elev (ft) = 6600.00
- Slope (%) = 5.50
- N-Value = 0.030

**Highlighted**
- Depth (ft) = 1.04
- Q (cfs) = 19.20
- Area (sqft) = 2.70
- Velocity (ft/s) = 7.10
- Wetted Perim (ft) = 5.60
- Crit Depth, Yc (ft) = 1.30
- Top Width (ft) = 5.20
- EGL (ft) = 1.82

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 19.20

---

**Graph**
- Elev (ft) vs. Reach (ft)
- Section
- Depth (ft)
**DRAINAGE BASIN SUMMARY TABLE**

<table>
<thead>
<tr>
<th>BASIN ID</th>
<th>AREA (acres)</th>
<th>100 YEAR PEAK FLOW (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20.7</td>
<td>19.0</td>
</tr>
<tr>
<td>A1</td>
<td>2.9</td>
<td>2.7</td>
</tr>
<tr>
<td>BENCH</td>
<td>9.0</td>
<td>6.8</td>
</tr>
<tr>
<td>B</td>
<td>30.2</td>
<td>33.5</td>
</tr>
<tr>
<td>B1</td>
<td>11.4</td>
<td>28.4</td>
</tr>
<tr>
<td>C</td>
<td>14.1</td>
<td>12.2</td>
</tr>
<tr>
<td>C1</td>
<td>11.4</td>
<td>10.5</td>
</tr>
<tr>
<td>C2</td>
<td>2.1</td>
<td>2.4</td>
</tr>
<tr>
<td>D</td>
<td>83.9</td>
<td>54.8</td>
</tr>
<tr>
<td>D1</td>
<td>41.3</td>
<td>24.0</td>
</tr>
<tr>
<td>D2</td>
<td>16.0</td>
<td>14.2</td>
</tr>
<tr>
<td>D3</td>
<td>11.5</td>
<td>12.5</td>
</tr>
</tbody>
</table>

**Legend**
- **EXISTING MAJOR CONTOUR**
- **EXISTING MINOR CONTOUR**
- **DESIGN MAJOR CONTOUR**
- **DESIGN MINOR CONTOUR**
- **BASIN BOUNDARY**
- **SUB BASIN BOUNDARY**
- **TIME OF CONCENTRATION**
- **FLOW PATH**
- **ACCESS ROAD**
- **BASIN POINT OF INTEREST**

**FIGURE 1**
POST DEVELOPMENT DRAINAGE BASIN MAP
XCEL ENERGY HAYDEN, CO

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Data Sheet

Flexamat® is a tied concrete block mat used to control erosion in swales, slopes, ditches, channels, shorelines and any area where soil sediment may be lost due to water runoff.

The matting consists of pyramidal concrete blocks that are interconnected utilizing a high tensile strength polypropylene geogrid. The completed mat yields a high strength, ultra-flexible hard armor system of Erosion Control. Flexamat’s superior Percentage of Open Area (POA) affords an ideal zone for vegetation growth while remaining a permanent armor against long-term erosional forces.

General Composition of Materials

<table>
<thead>
<tr>
<th>Blocks</th>
<th>5000 PSI, Wet-cast Portland Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interlocking Biaxial Geogrid</td>
<td>Fornit 30/30 – Polypropylene Geogrid with 2,055 lb/ft biaxial strength</td>
</tr>
<tr>
<td>Underlayment Options</td>
<td>Standard Polypropylene netting Curlex® II ECB Plus Polypropylene netting Recyclex® TRM-V Culrex® II ECB Fabric 10 oz NW fabric *More options available upon request</td>
</tr>
</tbody>
</table>

Manufacturing Values

<table>
<thead>
<tr>
<th>Flexamat Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roll Width</td>
<td>4’, 5.5’, 8’, 10’, 12’ 16’</td>
</tr>
<tr>
<td>Roll Length</td>
<td>30’, 40’, 50’ / custom</td>
</tr>
<tr>
<td>Material Weight</td>
<td>10 lbs./sf</td>
</tr>
<tr>
<td>Block Size</td>
<td>6.5” x 6.5” x 2.25”</td>
</tr>
<tr>
<td>Percentage Open Area (POA)</td>
<td>30% min.</td>
</tr>
</tbody>
</table>

Performance

<table>
<thead>
<tr>
<th>Test</th>
<th>Tested Value</th>
<th>Bed Slope</th>
<th>Soil Classification</th>
<th>Limiting Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM 6460</td>
<td>Shear Stress</td>
<td>30%</td>
<td>Sandy Loam (USDA)</td>
<td>24 PSF</td>
</tr>
<tr>
<td>ASTM 6460</td>
<td>Velocity</td>
<td>20%</td>
<td>Loam (USDA)</td>
<td>30 ft./sec</td>
</tr>
</tbody>
</table>
Large-Scale Channel Erosion Testing  
(ASTM D 6460 modified)

of

Flexamat
Channel Lining
over
Sandy Loam

February 2009

Submitted to:  
Motz Enterprises, Inc.  
9415 Montgomery Rd, Ste H  
Cincinnati, Ohio 45242  
Attn: Mr. Jim Motz

Submitted by:  
TRI/Environmental, Inc.  
9063 Bee Caves Road  
Austin, TX 78733

C. Joel Sprague  
Project Manager
TEST RESULTS

Average soil loss and the associated hydraulic shear calculated from flow and depth measurements made during the testing are the principle data used to determine the performance of the product tested. This data is entered into a spreadsheet that transforms the flow depth and velocity into an hydraulic shear stress and the soil loss measurements into and average Clopper Soil Loss Index (CSLI). A graph of shear versus soil loss for the protected condition is shown in Figure 9. The associated velocities are plotted in Figure 10. The graphs include a polynomial regression line fit to the test data to facilitate a projection of the limiting shear stress, \( \tau_{\text{limit}} \), and limiting velocity, \( V_{\text{limit}} \), since \( \frac{1}{2} \)-inch of soil loss was not achieved during testing.

Table 3. Summary Data Table – Protected Test Reach

<table>
<thead>
<tr>
<th>Test # (run # - target shear)</th>
<th>Flow depth (in)</th>
<th>Flow velocity (fps)</th>
<th>Flow (cfs)</th>
<th>Manning’s roughness, n</th>
<th>Max Bed Shear Stress (psf)</th>
<th>CSLI (in)</th>
<th>Cumm. CSLI (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1-4</td>
<td>3.79</td>
<td>6.56</td>
<td>4.13</td>
<td>0.058</td>
<td>5.82</td>
<td>-0.06</td>
<td>-0.06</td>
</tr>
<tr>
<td>R1-8</td>
<td>5.07</td>
<td>8.88</td>
<td>7.48</td>
<td>0.052</td>
<td>7.79</td>
<td>-0.05</td>
<td>-0.11</td>
</tr>
<tr>
<td>R1-12</td>
<td>6.99</td>
<td>11.06</td>
<td>12.87</td>
<td>0.051</td>
<td>10.74</td>
<td>-0.07</td>
<td>-0.18</td>
</tr>
<tr>
<td>R1-16</td>
<td>11.03</td>
<td>14.88</td>
<td>27.30</td>
<td>0.052</td>
<td>16.95</td>
<td>-0.11</td>
<td>-0.29</td>
</tr>
<tr>
<td>R2-4</td>
<td>3.61</td>
<td>6.38</td>
<td>3.82</td>
<td>0.058</td>
<td>5.55</td>
<td>-0.04</td>
<td>-0.04</td>
</tr>
<tr>
<td>R2-8</td>
<td>5.21</td>
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<td>7.53</td>
<td>0.054</td>
<td>8.00</td>
<td>-0.05</td>
<td>-0.09</td>
</tr>
<tr>
<td>R2-12</td>
<td>7.10</td>
<td>10.81</td>
<td>12.77</td>
<td>0.053</td>
<td>10.92</td>
<td>-0.05</td>
<td>-0.14</td>
</tr>
<tr>
<td>R2-16</td>
<td>10.80</td>
<td>14.56</td>
<td>26.19</td>
<td>0.052</td>
<td>16.60</td>
<td>-0.11</td>
<td>-0.25</td>
</tr>
<tr>
<td>R3-4</td>
<td>3.53</td>
<td>6.31</td>
<td>3.70</td>
<td>0.057</td>
<td>5.42</td>
<td>-0.04</td>
<td>-0.04</td>
</tr>
<tr>
<td>R3-8</td>
<td>5.31</td>
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<td>0.055</td>
<td>8.17</td>
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</tr>
<tr>
<td>R3-12</td>
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<td>0.053</td>
<td>10.57</td>
<td>-0.07</td>
<td>-0.17</td>
</tr>
<tr>
<td>R3-16</td>
<td>10.88</td>
<td>14.88</td>
<td>26.95</td>
<td>0.051</td>
<td>16.71</td>
<td>-0.13</td>
<td>-0.30</td>
</tr>
</tbody>
</table>