1.0 Introduction

This Technical Memorandum discusses stormwater management for the final reclaimed surface of the Dry Stack Tailings Facility associated with the proposed Rosemont Copper Project (Project) in Pima County, Arizona. The design of stormwater control structures, such as tailings bench stilling pools, riprap drop chutes, and detention pools on the top surface of the Dry Stack Tailings Facility are discussed herein. The Northwest and Northeast Decant Structures are also discussed along with the West Haul Road Channel. The analyses presented were performed on the base concept of the Rosemont Ridge Landform shown on Figure 1.

Detailed hydrologic and hydraulic analysis associated with the tailings bench drainage channels are explained in further detail in the Technical Memorandum titled Rosemont Dry Stack Tailings Facility Drainage Bench Analysis (Tetra Tech, 2010b).

2.0 Hydrologic Methodology Overview (NRCS Method)

The National Resources Conservation Services (NRCS) curve number method was used for the hydrologic analysis and design of stormwater control structures associated with the final reclaimed surface of the Dry Stack Tailings Facility. The NRCS method allows for many precipitation patterns to be applied to a watershed.

The following return periods were analyzed in relation to the stormwater control features analyzed herein:

- 500-year;
- 1,000-year; and
- The General Probable Maximum Precipitation (PMP).

The storms analyzed for estimating total runoff volume and peak runoff were the 24-hour NRCS Type II Distribution and the 72-hour General PMP Distribution.
Table 1 summarizes the storms that were considered for design purposes. For detailed information regarding the NRCS method, refer to the Technical Memorandum titled *Rosemont Copper Project Hydrology Method Justification* (Tetra Tech, 2010a).

<table>
<thead>
<tr>
<th>Parameters</th>
<th>NRCS Storms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Period (years)</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>Duration (hours)</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>72</td>
</tr>
<tr>
<td>Precipitation (inches)</td>
<td>6.57</td>
</tr>
<tr>
<td></td>
<td>6.00</td>
</tr>
<tr>
<td></td>
<td>18.90</td>
</tr>
<tr>
<td>Distribution</td>
<td>NRCS Type II</td>
</tr>
</tbody>
</table>

Stormwater management on the tailings benches required the calculation of hydrologic peak flows to size channels, stilling pools, and drop chutes. Hydrologic peak flows emanating from the tailings benches were previously determined utilizing the NRCS method along with Manning’s open-channel flow equation. Minimum design flows for the Tailings Bench Stilling Pools and Riprap Drop Chutes are a sum of the 500-year, 24-hour storm event peak flows from contributing basins. Additional information about the design of the drainage benches and V-channels are provided in the Technical Memorandum titled *Rosemont Dry Stack Tailings Facility Drainage Bench Analysis* (Tetra Tech, 2010b).

Stormwater management for the top of the North and South Dry Stack Tailings Facilities only required rainfall runoff volume calculations to design the appropriate structures since peak flows would be negligible due to the nearly flat surface and with the included pooling areas. Tetra Tech prepared a Technical Memorandum titled *Rosemont Waste Rock Storage Area Drainage Basin Analysis* (Tetra Tech, 2010b) that discusses the NRCS method for determining rainfall runoff volume.

### 3.0 Stormwater Management Overview

#### 3.1 Tailings Stormwater Drainage Benches

The outer surface slopes of the Dry Stack Tailings Facility consist of 50-foot wide drainage benches at an approximate vertical spacing of 100 feet. At a minimum, the drainage benches are capable of conveying the 500-year, 24-hour storm event with adequate freeboard.

These drainage benches are designed to slope longitudinally at 2% across the reclaimed face of the Rosemont Ridge Landform. In general, the V-channels will route stormwater from the reclaimed face the Dry Stack Tailings Facility either to stilling pools/drop structures designed to route stormwater off the face, to detention pools located on wide benches in the Waste Rock Storage Area, or to the perimeter toe of the Rosemont Ridge Landform into channels cut in natural ground.

As discussed in the Technical Memorandum titled *Rosemont Dry Stack Tailings Facility Drainage Bench Analysis* (Tetra Tech, 2010b), the 50-foot wide drainage benches were
designed to accommodate a drainage channel, a safety berm, and an access road. Two (2) stormwater channel designs were presented and analyzed:

- A V-channel; and
- A trapezoidal channel.

Both channel types were analyzed with and without volume losses due to sedimentation. The analysis indicated that the channels convey the peak flow from a 500-year, 24-hour event with adequate freeboard with 30 percent of the channel capacity lost due to sedimentation. As a note, and based on the analysis performed in the Technical Memorandum titled *Rosemont Copper Project Hydrology Method Justification* (Tetra Tech, 2010a), the peak flow generated by a 500-year, 24-hour storm event using the NRCS method was found to closely resemble the results using the Pima County Method (100-year event) as generated by the software package PC-Hydro.

### 3.2 Top of South Dry Stack Tailings Facility

Waste rock will be placed on top of the South Dry Stack Tailings Facility to form a ridge leading from the hillocks planned in the Waste Rock Storage Area to the top of the North Dry Stack Tailings Facility. Stormwater is intended to flow around the waste rock perimeter to the inlet of the West Haul Road Channel, located at the southwest corner of the South Dry Stack Tailings Facility as shown on Figure 1. Smaller storms, such as the 10-year, 24-hour event, will be retained in detention pools located on top of the reclaimed surface of the South Dry Stack Tailings Facility. A rock weir will be placed at the inlet of the West Haul Road Channel to control runoff from storms up to the 1,000-year, 24-hour event. These storm events will dissipate through the rock weir structure while larger events will pass over the structure.

A large containment berm, approximately ten (10) feet high, will be placed around the top perimeter of the South Dry Stack Tailings Facility. Containment of the runoff volume from a General PMP event is provided by this berm with stormwater routed to the West Haul Road Channel.

### 3.3 Top of North Dry Stack Tailings Facility

The top of the North Dry Stack Tailings Facility is designed to manage the General PMP storm with additional storage capacity to contain more extreme events. Similar to the South Dry Stack Tailings Facility, a ten (10) foot high containment berm is planned around the top perimeter of the North Dry Stack Tailings Facility.

Runoff from a 1,000-year, 24-hour event will be contained in detention pools located on the top reclaimed surface of the North Dry Stack Tailings Facility. Larger rainfall events will accumulate within the basin beyond the storage capacity of the detention pools and stormwater will leave the facility via two (2) decant inlet structures. Stormwater in excess of the 1,000-year, 24-hour event will be piped to the Northeast and Northwest Tailings Bench Stilling Pools and routed off the face of the Dry Stack Tailings Facility via riprap drop chutes as illustrated on Figure 1. Discharge to these stilling pools will be controlled by the capacity of two (2) 36-inch diameter high-density, polyethylene (HDPE) pipes associated with the decant structures (see Section 5.4 for details).
4.0 Tailings Bench Stilling Pool and Riprap Drop Chute Design

The designs of the riprap drop chutes, which are sized to safely convey a minimum 500-year, 24-hour storm event, are primarily driven by hydrologic peak flows that will converge into the stilling pools at the inlet of each chute. For the design of the drop structures associated with the Dry Stack Tailings Facility, the sum of the peak flows at each of the stilling pools was used to size the drop structures. The peak flows used for this analysis are summarized in the following sections and were based on the Technical Memorandum titled *Rosemont Dry Stack Tailings Facility Drainage Bench Analysis* (Tetra Tech, 2010b).

The sum of the peak flows at the stilling pools is a conservative simplification in the design and analysis of the drop chute structures. In reality, stormwater flow from the bench V-channels would first fill the pooling area, then exit at the drop chute inlet by mimicking a reservoir routing effect. This reservoir routing effect was neglected due to the shallowness of the pools.

As shown on Figure 1, two (2) of the stilling pool locations, situated in the northeast and northwest areas of the North Dry Stack Tailings Facility, will have two (2) drop chute outlets. As such, quantifying the amount of runoff each chute would receive is uncertain. Therefore, the two (2) drop chutes were designed as if only one (1) drop chute were present.

4.1 Hydraulics Calculation Methodology

The method used to size the Riprap Drop Chutes was developed by the NRCS American Society of Agricultural Engineers (ASAE). An Excel spreadsheet released by the ASAE performs internal hydraulic calculations and is based on the *Design of Rock Chutes* (Robinson, et al, 1998). The spreadsheet requires specific chute, inlet, and outlet channel dimensions as well as the combined peak flows that are anticipated to occur at the particular structure. The spreadsheet also allows for assigning a desired factor of safety for each drop chute, and given the other necessary parameters, it verifies that the hydraulic controls will function properly.

In order to standardize the design of the drop structures, a $D_{50}$ rock size of 24 inches was selected for the riprap in all the structures. Also, a factor of safety of 1.5 for the riprap was achieved by varying the widths of the chute and the lengths of the inlet and outlet aprons at each structure. The ASAE spreadsheet *Rock Chute Design Data*, used for the analysis of the drop structures, is included in Attachment 1.

4.2 Southeast Riprap Drop Chutes

The stilling pools located at the southeast (SE) area of the Dry Stack Tailings Facility each have only one (1) contributing basin for the peak flow coming directly into the pools from the north. The upper drop chute, SE3 that exits the top pool, was sized for its reporting peak event, whereas the lower drop chutes, SE1 and SE2 that exit the downstream pools, were sized for their respective peak events as well as the peak flow generated from the upstream basin or basins. Table 2 below shows the SE Riprap Drop Chute’s cumulative design flows that were obtained from the Technical Memorandum titled *Rosemont Dry Stack Tailings Facility Drainage Bench Analysis* (Tetra Tech, 2010b).
Table 2  Design Flows for SE Riprap Drop Chutes

<table>
<thead>
<tr>
<th>Drop Structure ID</th>
<th>Contributing Basins ID</th>
<th>Drop Chute Design Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE1</td>
<td>SE Tailings Bench 1, 2, &amp; 3</td>
<td>591</td>
</tr>
<tr>
<td>SE2</td>
<td>SE Tailings Bench 1 &amp; 2</td>
<td>377</td>
</tr>
<tr>
<td>SE3</td>
<td>SE Tailings Bench 1</td>
<td>157</td>
</tr>
</tbody>
</table>

4.3  Northeast Riprap Drop Chutes

The stilling pools located at the northeast (NE) corner area of the Dry Stack Tailings Facility each have two (2) contributing basins for the peak flow coming directly into the pools. Also, these stilling pools each have two (2) drop chute structures. The upper drop chutes, NE3 and NE4 exiting the top pool, were sized to safely convey the peak flow event alone. The lower drop chutes, NE1 and NE2 leaving the bottom pool, were sized to safely convey the peak flow event and the peak flow from the upstream basins.

As previously stated, the sum of the peak flows at the stilling pools was simplified. Additionally, the design of the drop chutes associated with stilling pools having two (2) drop chutes assumed only one (1) was available to carry the entire peak flow. Table 3 shows the NE Riprap Drop Chute's cumulative design flows.

Table 3  Design Flows for NE Riprap Drop Chutes

<table>
<thead>
<tr>
<th>Drop Structure ID</th>
<th>Contributing Basins ID</th>
<th>Drop Chute Design Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NE1 &amp; NE2</td>
<td>NE Tailings Bench 1, 2, 6, &amp; 7</td>
<td>621</td>
</tr>
<tr>
<td>NE3 &amp; NE4</td>
<td>NE Tailings Bench 1 &amp; 6</td>
<td>247</td>
</tr>
</tbody>
</table>

4.4  Northwest Riprap Drop Chutes

The drop chutes located at the northwest (NW) corner area of the Dry Stack Tailings Facility were designed in the same manner as the NE structures. The NW stilling pools each have two (2) contributing basins for the peak flow coming directly into the pools. Both stilling pools also have two (2) drop chute structures. The upper drop chutes, NW3 and NW4 exiting the top pool, were sized to safely convey the peak flow event alone. The lower drop chutes, NW1 and NW2 leaving the bottom pool, were sized to safely convey the peak flow event and the peak flow from the upstream basins. Table 4 shows the NW Riprap Drop Chute’s cumulative design flows.
### 4.5 West Riprap Drop Chutes

The stilling pool located at the west (W) area of the Dry Stack Tailings Facility has two (2) contributing basins for the peak flow coming directly into the pool. The drop chute structure, W1, leaving the pool was designed to safely convey the peak flow event. Table 5 below shows the W Riprap Drop Chute’s design flow.

<table>
<thead>
<tr>
<th>Drop Structure ID</th>
<th>Contributing Basins ID</th>
<th>Drop Chute Design Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>W Tailings Bench 1 &amp; 4</td>
<td>190</td>
</tr>
</tbody>
</table>

### 4.6 Southwest Riprap Drop Chutes

The stilling pools located at the southwest (SW) area of the Dry Stack Tailings Facility each have two (2) contributing basins for the peak flow coming directly into the pools. The upper drop chute, SW2 exiting the top pool, was sized for the peak flow event. The lower drop chute, SW1 leaving the downstream pool, was sized for the peak flow event and the peak flow from the upstream basins. Table 6 shows the SW Riprap Drop Chute’s cumulative design flows.

<table>
<thead>
<tr>
<th>Drop Structure ID</th>
<th>Contributing Basins/Peak Flow</th>
<th>Drop Chute Design Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW1</td>
<td>SW Tailings Bench 1, 2, 3, &amp; 4</td>
<td>404</td>
</tr>
<tr>
<td>SW2</td>
<td>SW Tailings Bench 1 &amp; 3</td>
<td>205</td>
</tr>
</tbody>
</table>

### 5.0 Top of Dry Stack Tailings Facility Design

As shown on Figure 1, stormwater from the top reclaimed surface of the South Dry Stack Tailings Facility will gradually flow around the waste rock ridge and be collected in a series of shallow detention pools that are hydraulically connected. Stormwater will eventually be routed to the inlet of the West Haul Road Channel located in the southwest corner of the South Dry Stack Tailings Facility.
Stack Tailings Facility. The basin for this southern top is assumed to be graded in a manner to evenly and effectively distribute stormwater into the detention pools.

Stormwater on the top reclaimed surface of the North Dry Stack Tailings Facility will accumulate in shallow detention pools that are hydraulically linked. Storm events generating excess runoff will be routed out of the facility via the Northwest and Northeast Decant Structures. These decant structures are located in the northwest and northeast detention pools, respectively.

The basin for this northern top is also assumed to be graded in a manner to evenly and effectively distribute stormwater among the detention pools. The Technical Memorandum titled *Rosemont Copper Project Hydrology Method Justification* (Tetra Tech, 2010a) discusses the minimum storm events used to calculate runoff volumes for permanent containment structures. Table 7 summarizes the NRCS parameters used to determine the total runoff volume generated for the basins located on top surface of the Dry Stack Tailings Facility.

<table>
<thead>
<tr>
<th>Table 7</th>
<th>NRCS Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
<td>500-year, 24-hour</td>
</tr>
<tr>
<td>P (inches)</td>
<td>6.00</td>
</tr>
<tr>
<td>CN</td>
<td>85</td>
</tr>
<tr>
<td>S (inches)</td>
<td>1.76</td>
</tr>
<tr>
<td>Q (inches)</td>
<td>4.30</td>
</tr>
<tr>
<td>I_A (inches)</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Tetra Tech also prepared a Technical Memorandum titled *Rosemont Waste Rock Storage Area Drainage Basin Analysis* (Tetra Tech, 2010c) that discusses the NRCS method for determining rainfall runoff volume. Table 8 summarizes the volumetric analysis for the Dry Stack Tailings Facility’s top basins examined herein together with the estimated storage capacities of their corresponding detention pools.

<table>
<thead>
<tr>
<th>Table 8</th>
<th>Summary of the Top of Dry Stack Tailings Facility Volumetric Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Stack Tailings Facility Basin Name</td>
<td>Area</td>
</tr>
<tr>
<td></td>
<td>(acres)</td>
</tr>
<tr>
<td>Top of South Tailings</td>
<td>211.4</td>
</tr>
<tr>
<td>Top of North Tailings</td>
<td>221.0</td>
</tr>
</tbody>
</table>

1. Detention pools are designed shallow at 2-ft deep with 4% side-slopes.
2. Excess runoff volume from the Top of South Dry Stack Tailings Facility for the storm events analyzed will be routed to the West Haul Road Channel.
3. Excess runoff volume from the Top of North Dry Stack Tailings Facility for the General PMP storm event will be routed to the Northwest and Northeast Decant Structures.
5.1 Top of South Dry Stack Tailings Detention Pools

The shallow, two (2) foot deep, detention pools located on the top of the South Dry Stack Tailings Facility are designed to retain smaller storm events such as the 10-year, 24-hour storm event with a combined storage capacity of about 31 ac-ft. Larger storm events will migrate out of these detention pools and will be routed to the inlet of the West Haul Road Channel as described in Section 3.2.

5.2 West Haul Road Channel

The West Haul Road Channel is located adjacent to a haul road having a 10% slope. The channel is configured as a ten (10) foot deep trapezoidal waterway with a 30-foot bottom width and 3H:1V (Horizontal to Vertical) side-slopes. This channel will receive attenuated stormwater runoff from the top of the South Dry Stack Tailings Facility. The West Haul Road Channel will be armored with a thick layer of run-of-mine (ROM) riprap for erosion protection and energy dissipation against the anticipated scouring velocities due to the channel grade.

5.3 Top of North Dry Stack Tailings Detention Pools

The shallow, two (2) foot deep, detention pools arranged on top of the reclaimed North Dry Stack Tailings Facility surface are designed to contain the 1,000-year, 24-hour storm event. The total storage capacity of the linked detention pools is about 175 ac-ft. It is assumed that the top of the reclaimed surface is graded flat.

The invert pipe elevations of the Northwest and Northeast Decant Structures allows for retention of the 1,000-year, 24-hour event in the pools. This corresponds to a volume of about 89 ac-ft and a depth of approximately one (1) foot in the detention pools. As a comparison, the General PMP event produces a runoff volume of about 312 ac-ft. This results in a depth of approximately 2.7 feet taken from the invert of the detention pools over the entire top of the North Dry Stack Tailings Facility (see the stage-storage curve shown in Illustration 1). As previously stated, a ten (10) foot high containment berm is located around the top of the North Dry Stack Tailing Facility.
5.4 Northwest and Northeast Decant Structures

The Northwest and Northeast Decant Structures, located at the corner areas of the northwest and northeast detention pools, both consist of a concrete inlet sump that is four (4) feet square by six (6) feet deep leading to a 36 inch HDPE pipe that outlets at the upper NE and NW Tailings Bench Stilling Pools. The concrete inlet sumps are anticipated to be covered with an overflow grate surrounded by graded filter rock.

The decant structures were designed to control the rate of flow to the downstream stilling pools and drop chutes. The inverts of the structures will be set one (1) foot above the bottom of the detention pools in order for the 1,000-year, 24-hour event to be retained. When the depth of the water in the detention pools is greater than one (1) foot, stormwater will begin to enter the decant structures.

As the depth of the water within the basin increases due to storms beyond the 1,000-year, 24-hour event, the pressure head above a decant structure’s invert will become greater and thus the flow into the structure will increase. When the depth of the water is between one (1) foot and about 2.5 feet deep, the flow into the structure is controlled by the weir configuration of the inlet. As the depth of the water increases beyond 2.5 feet, the structure’s discharge is driven...
by orifice type flow that is expected as stormwater enters the inlet of the HDPE pipe from the concrete sump. The anticipated rate of flow through the HDPE pipe was also considered. However, the flow regime for a given decant structure is limited by weir flow and orifice flow as depicted in Illustration 2. Considering that the General PMP storm event corresponds to a stage depth of approximately 2.7 feet, the discharge into the receiving stilling pools and drop chutes from the HDPE pipe outlet is dictated by the pipe inlet's orifice type flow at a minimal rate of approximately 85 cfs.

Illustration 2     Northwest and Northeast Decant Structures Typical Stage-Discharge

6.0      Conclusion

Based on the analysis performed herein for the base concept of the Rosemont Ridge Landform, the stormwater control measures are sufficient to properly manage the anticipated runoff generated from storms ranging from the 500-year, 24-hour event up to the PMP event. Should modifications to the Rosemont Ridge Landform shape occur, this stormwater management analysis and the design of corresponding stormwater control structures would still be applicable assuming the configuration of basin areas and channel lengths were comparable to those analyzed herein.
7.0 REFERENCES


ATTACHMENT 1
NRCS ROCK CHUTE DESIGN DATA
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - SE 1
Designer: Ronson Chee
Date: 3/10/2010

**Input Channel Geometry**

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 25.0 ft.</td>
<td>Bw = 40.0 ft.</td>
<td>Bw = 30.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 (F_s)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>Side slopes = 3.0 (m:1)</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.020 ft./ft.</td>
<td>Bed slope = 0.330 ft./ft.</td>
<td>Bed slope = 0.0200 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

**Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage area</td>
<td>na acres</td>
</tr>
<tr>
<td>Rainfall</td>
<td>O = 0-3 in.</td>
</tr>
<tr>
<td>Apron elev. --- Inlet</td>
<td>86.0 ft.</td>
</tr>
<tr>
<td>--- Outlet</td>
<td>3.0 ft.</td>
</tr>
<tr>
<td>Chute capacity = Q25-year</td>
<td>591.0 cfs</td>
</tr>
<tr>
<td>Total capacity = Q100-year</td>
<td>20.0 cfs</td>
</tr>
<tr>
<td>Note: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.</td>
<td></td>
</tr>
</tbody>
</table>

**Input tailwater (Tw):**

- Qhigh = 591.0 cfs
- Qlow = 20.0 cfs

**Profile and Cross Section (Output)**

- H_plv = 0.42 ft. (0.26 ft.)
- H_plv = 2.76 ft.
- H_plv = 0.8 ft. (0.09 ft.)
- H_plv = 2.61 ft.
- H_plv = 2.34 ft.
- (0.18 ft.)
- Yc = 1.81 ft. (0.2 ft.)
- 0.715yc = 1.29 ft. (0.14 ft.)
- z1 = 0.98 ft. (0.11 ft.)
- H_drop = 80 ft.
- H_drop = 2.11 ft. (0.29 ft.)
- Tw+d = 5.11 ft. - Tw o.k.
- (3.29 ft.) - Tw o.k.
- 23.7 in. (971 lbs. - 50% round / 50% angular)
- 8 oz. Min. Geotextile
- 15(D50)(F_s)
- Note: When the normal depth in the inlet channel is less than the weir head (H_p), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

**Notes:**

1) Output given as High Flow (Low Flow) values.

2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.

3) Critical depth occurs 2yc - 4yc upstream of crest.

4) Use min. 8 oz. non-woven geotextile under rock.

**Profile Along Centerline of Chute**

- q_c = 13.77 cfs/ft.
- Equivalent unit discharge
- F_s = 1.50
- Factor of safety (multiplier)
- z_c = 0.98 ft.
- Normal depth in chute
- n-value = 0.06
- Manning's roughness coefficient
- D50(F_s) = 23.7 in. (971 lbs. - 50% round / 50% angular)
- 2(D50)(F_s) = 47.4 in.
- Rock chute thickness
- Tw + d = 5.11 ft.
- Tailwater above outlet apron
- z_2 = 2.98 ft.
- Hydraulic jump height

**Typical Cross Section**

- **High Flow Storm Information**

**Note:**

*** The outlet will function adequately
**Rock Chute Design Data**

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

**Input Channel Geometry**

<table>
<thead>
<tr>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 20.0 ft.</td>
<td>Bw = 25.0 ft.</td>
</tr>
<tr>
<td>Bw = 25.0 ft.</td>
<td>Bw = 25.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>Factor of safety = 1.50 ((F_s))</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.0200 ft./ft.</td>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
</tr>
<tr>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Freeboard = 0.5 ft.</td>
</tr>
<tr>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

**Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)**

- **Drainage area**: na acres
- **Rainfall**: 0 - 3 in., 3 - 5 in., 5+ in.
- **Apron elev. --- Inlet**: 106.0 ft. --- **Outlet**: 3.0 ft. --- \((H_{drop} = 100 \text{ ft.})\)
- **Chute capacity**: Q25-year
- **Total capacity**: Q100-year

**Input tailwater \((Tw)\):**

- **Q_{high}**: 377.0 cfs
- **Q_{low}**: 20.0 cfs

**Profile and Cross Section (Output)**

1. **Note**: When the normal depth \((y_n)\) in the inlet channel is less than the weir head \((H_p)\), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

**Profile Along Centerline of Chute**

- **q_e**: 13.48 cfs/ft. \(\text{Equivalent unit discharge}\)
- **F_S**: 1.50 \(\text{Factor of safety (multiplier)}\)
- **z_t**: 0.97 ft. \(\text{Normal depth in chute} \)
- **n-value**: 0.06 \(\text{Manning's roughness coefficient} \)
- **D_{50}(F_s) = 23.4 \text{ in. (938 lbs. - 50% round / 50% angular)} \)
- **2(D_{50})(F_s) = 46.9 in. \(\text{Rock chute thickness} \)
- **Tw + d = 4.8 ft. \(\text{Tailwater above outlet apron} \)
- **z_o = 2.92 ft. \(\text{Hydraulic jump height} \)

**Typical Cross Section**

- **Use \(H_p\) along chute but not less than \(z_2\).**

**High Flow Storm Information**

- **Outlet**: will function adequately
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - SE 3  County: Pima
Designer: Ronson Chee  Checked by:
Date: 3/10/2010  Date:

Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

Drainage area = na acres  Rainfall = O 0 - 3 in.  ● 3 - 5 in.  ○ 5 in.
Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- (Hdrop = 100 ft.)
Chute capacity = Q25-year  Minimum capacity (based on a 5-year, 24-hour storm with a 3 - 5 inch rainfall)
Total capacity = Q100-year
Qhigh = 157.0 cfs  High flow storm through chute  through the chute (principal spillway) or
Qlow = 20.0 cfs  Low flow storm through chute  in combination with an auxiliary spillway.

Profile and Cross Section (Output)

1) Output given as High Flow (Low Flow) values.
2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.
3) Critical depth occurs 2yc - 4yc upstream of crest.
4) Use min. 8 oz. non-woven geotextile under rock.

Profile Along Centerline of Chute

Equivalent unit discharge
q_e = 12.1 cfs/ft.
Factor of safety (multiplier)
F_s = 1.50
Normal depth in chute
z_f = 0.9 ft.
Manning’s roughness coefficient
n-value = 0.099
Rock chute thickness
D50(Fs) = 22.1 in. (791 lbs. - 50% round / 50% angular)

Notes:

**The outlet will function adequately**

Typical Cross Section

High Flow Storm Information
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - NE 1 & 2
Designer: Ronson Chee
County: Pima
Checked by: 
Date: 3/10/2010

Input Channel Geometry

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 25.0 ft.</td>
<td>Bw = 40.0 ft.</td>
<td>Bw = 30.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 (F_s)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>Side slopes = 3.0 (m:1)</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.0200 ft./ft.</td>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
<td>Bed slope = 0.0200 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

- **Drainage area = na acres**
- **Rainfall = 0 - 3 in. 3 - 5 in. 5+ in.**
- **Apron elev. --- Inlet = 86.0 ft. --- Outlet = 3.0 ft. --- (H_{drop} = 80 ft.)**
- **Chute capacity = Q25-year 24-hour storm with a 3 - 5 inch rainfall**
- **Total capacity = Q100-year**
- **Q_{high} = 621.0 cfs** High flow storm through chute
- **Q_{low} = 20.0 cfs** Low flow storm through chute

<table>
<thead>
<tr>
<th><strong>Input tailwater (Tw):</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tw+d = 5.17 ft. - Tw o.k.</td>
</tr>
<tr>
<td>(3.29 ft.) - Tw o.k.</td>
</tr>
</tbody>
</table>

**Profile and Cross Section (Output)**

- **Slope = 0.02 ft./ft.**
- **h_{pw} = 0.42 ft. (0.26 ft.)**
- **h_{pw} = 2.84 ft.**
- **h_{ch} = 0.83 ft. (0.09 ft.)**
- **H_{ch} = 2.69 ft.**
- **H_{p} = 2.43 ft. (0.18 ft.)**
- **H_{p} = 1.86 ft. (0.2 ft.)**
- **H_{slo} = 0.715y_c = 1.33 ft. (0.14 ft.)**
- **z_{1} = 1.01 ft. (0.11 ft.)**
- **H_{drop} = 80 ft.**
- **2.17 ft. (0.29 ft.)**
- **Outlet Channel**
- **Slope = 0.02 ft./ft.**
- **d = 3 ft. (1 ft. minimum suggested)**
- **Velocity_{outlet} = 7.83 fps at normal depth**

1. **Note:** When the normal depth (y_c) in the inlet channel is less than the weir head (H_p), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

**Profile Along Centerline of Chute**

- **q_t = 14.43 cfs/ft.** Equivalent unit discharge
- **F_s = 1.50** Factor of safety (multiplier)
- **z_{1} = 1.01 ft.** Normal depth in chute
- **n-value = 0.06** Manning’s roughness coefficient
- **D_{50} (F_s) = 24.3 in. (1046 lbs. - 50% round / 50% angular)**
- **2(D_{50})(F_s) = 48.6 in.** Rock chute thickness
- **Tw + d = 5.17 ft.** Tailwater above outlet apron
- **z_{2} = 3.07 ft.** Hydraulic jump height

**Typical Cross Section**

- **Use H_p along chute but not less than z_{2}**

**High Flow Storm Information**

- **hpv = 0.42 ft. (0.26 ft.)**
- **Hpe = 2.84 ft. 0.83 ft. (0.09 ft.)**
- **Tailwater depth plus d must be at or above the Energy Grade Line**
- **Hce = 2.69 ft.**
- **Energy Grade Line**
- **hydraulic jump height for the chute to function.**
- **3) Critical depth occurs 2y_c - 4y_c upstream of crest.**
- **0.715y_c = 1.33 ft.**
- **4) Use min. 8 oz. non-woven geotextile under rock.**
- **Hp = 2.43 ft. (0.14 ft.)**
- **z_{1} = 1.01 ft.**
- **z_{2} = 3.07 ft.**
- **z_{2} = 0.14 ft.**
- **Freeboard = 0.5 ft.**
- **FS = 1.50** Factor of safety (multiplier)
- **z_{1} = 1.01 ft.** Normal depth in chute
- **n-value = 0.06** Manning’s roughness coefficient
- **D_{50} (F_s) = 24.3 in. (1046 lbs. - 50% round / 50% angular)**
- **2(D_{50})(F_s) = 48.6 in.** Rock chute thickness
- **Tw + d = 5.17 ft.** Tailwater above outlet apron
- **z_{2} = 3.07 ft.** Hydraulic jump height

**The outlet will function adequately**
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - NE 3 & 4
County: Pima
Designer: Ronson Chee
Checked by: __________________________
Date: 3/10/2010

Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

<table>
<thead>
<tr>
<th>Drainage area = na acres</th>
<th>Rainfall = O 0-3 in.  O 3-5 in.  O 5+ in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- (hdrop = 100 ft.)</td>
<td></td>
</tr>
<tr>
<td>Chute capacity = Q25-year 24-hour storm with a 3 - 5 inch rainfall)</td>
<td></td>
</tr>
<tr>
<td>Total capacity = Q100-year Program</td>
<td></td>
</tr>
<tr>
<td>Qhigh = 247.0 cfs High flow storm through chute</td>
<td></td>
</tr>
<tr>
<td>Qlow = 20.0 cfs Low flow storm through chute</td>
<td></td>
</tr>
</tbody>
</table>

Note: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.

Input tailwater (Tw):

Tw+d = 4.58 ft. - Tw o.k. (3.37 ft.) - Tw o.k.

Profile and Cross Section (Output)

Notes:
1) Output given as High Flow (Low Flow) values.
2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.
3) Critical depth occurs 2yc - 4yc upstream of crest.
4) Use min. 8 oz. non-woven geotextile under rock.

Profile Along Centerline of Chute

Notes:
q = 13.63 cfs/ft. Equivalent unit discharge
Fp = 1.50 Factor of safety (multiplier)
zr = 0.97 ft. Normal depth in chute
n-value = 0.06 Manning's roughness coefficient
D50(Fs) = 23.6 in. (955 lbs. - 50% round / 50% angular)

Profile Cross Section

Typical Cross Section

High Flow Storm Information
**Rock Chute Design Data**

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

**Project:** Rosemont Reclamation - NW 1 & 2  
**County:** Pima

**Designer:** Ronson Chee  
**Checked by:**

**Date:** 3/10/2010

---

### Input Channel Geometry

- **Inlet Channel**
  - Bw = 20.0 ft.
  - Side slopes = 10.0 (m:1)
  - Bed slope = 0.0200 ft./ft.
  - Freeboard = 0.5 ft.

- **Chute**
  - Bw = 25.0 ft.
  - Factor of safety = 1.50 ($F_s$)
  - Side slopes = 3.0 (m:1)
  - Bed slope = 0.330 ft./ft.

- **Outlet Channel**
  - Bw = 20.0 ft.
  - Side slopes = 3.0 (m:1)
  - Bed slope = 0.0200 ft./ft.
  - Base flow = 0.0 cfs

---

### Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

- **Drainage area:** na acres  
- **Rainfall:** na in.
- **Apron elev. --- Inlet:** 106.0 ft.  
- **Outlet:** 3.0 ft.  
- **Chute capacity:** Q25-year = 368.0 cfs
- **Total capacity:** Q100-year = 24-hour storm with a 3 - 5 inch rainfall

**Note:** The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.

### Profile and Cross Section (Output)

- **Energy Grade Line**
  - $h_{pv} = 0.2$ ft. (0.1 ft.)
  - $h_{se} = 2.56$ ft.
  - $h_{ou} = 0.75$ ft. (0.13 ft.)
  - $H_p = 2.5$ ft.
  - $0.40(D_{50}) = 51$ ft. radius
  - $40(D_{50}) = 51$ ft. radius
  - $10yc = 18$ ft.
  - $yc = 1.75$ ft. (0.27 $ft.$)
  - $z_1 = 0.95$ ft. (0.15 ft.)
  - $H_{drop} = 100$ ft.
  - $H_{apron} = 1.98$ ft. (0.37 ft.)
  - $Outlet$ Channel
  - $Slope = 0.02$ ft./ft.
  - $Velocity_{outlet} = 7.15$ fps

**Note:** When the normal depth ($y_n$) in the inlet channel is less than the weir head ($H_p$), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

---

### Profile Along Centerline of Chute

- **$q_i = 13.18$ cfs/ft.**  
  - Equivalent unit discharge
- **$F_s = 1.50$**  
  - Factor of safety (multiplier)
- **$z_1 = 0.95$ ft.**  
  - Normal depth in chute
- **$n-value = 0.06$**  
  - Manning’s roughness coefficient
- **$D_{50}(F_s) = 23.2$ in. (905 lbs. - 50% round / 50% angular)**
- **$2(D_{50})(F_s) = 46.3$ in.**  
  - Rock chute thickness
- **$Tw+d = 4.98$ ft.**  
  - Tailwater above outlet apron
- **$z_2 = 2.88$ ft.**  
  - Hydraulic jump height

**Note:** The outlet will function adequately when $z_2$ is not less than $z_2$.
**Rock Chute Design Data**

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

**Project:** Rosemont Reclamation - NW 3 & 4  
**County:** Pima  
**Designer:** Ronson Chee  
**Date:** 3/10/2010

---

### Input Channel Geometry

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 10.0 ft.</td>
<td>Bw = 10.0 ft.</td>
<td>Bw = 10.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 ($F_s$)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>Side slopes = 3.0 (m:1)</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.0200 ft./ft.</td>
<td>Bed slope = 0.3300 ft./ft.</td>
<td>Bed slope = 0.0200 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

---

### Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

<table>
<thead>
<tr>
<th>Drainage area = na acres</th>
<th>Rainfall = 0 - 3 in.</th>
<th>3 - 5 in.</th>
<th>5+ in.</th>
<th>Note: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- ($H_{inp} = 100$ ft.)</td>
<td>Chute capacity = Q25-year [24-hour storm with a 3 - 5 inch rainfall]</td>
<td>Total capacity = Q100-year [24-hour storm with a 3 - 5 inch rainfall]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q$_{high}$ = 155.0 cfs</td>
<td>High flow storm through chute --- Tw (ft.) = Program 0.33</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q$_{low}$ = 20.0 cfs</td>
<td>Low flow storm through chute --- Tw (ft.) = Program</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

### Profile and Cross Section (Output)

**Notes:**
1. Output given as High Flow (Low Flow) values.  
2. Tailwater depth plus $d$ must be at or above the hydraulic jump height for the chute to function.  
3. Critical depth occurs $2y_c - 4y_c$ upstream of crest.  
4. Use min. 8 oz. non-woven geotextile under rock.

---

### Profile Along Centerline of Chute

**Notes:**
- Use $H_p$ along chute but not less than $z_2$.  
- The outlet will function adequately

---

**Typical Cross Section**

- **Freeboard:** 0.5 ft.  
- **Berm:** 10 ft.  
- **Rock Chute Bedding:** 44 in.  
- **8 oz. Min. Geotextile:**  
- **Rock Chute Thickness:** 44 in.  
- **Wall function:**  

---

**High Flow Storm Information**

- **$q_e$:** 11.97 cfs/ft. Equivalent unit discharge  
- **$F_s$:** 1.50 Factor of safety (multiplier)  
- **$z_t$:** 0.89 ft. Normal depth in chute  
- **$n$:** 0.059 Manning’s roughness coefficient  
- **$D_{50}(F_s)$:** 22 in. (777 lbs. - 50% round / 50% angular)  
- **Rock chute thickness:** 44 in.  
- **Tailwater above outlet apron:** 4.7 ft.  
- **Hydraulic jump height:** 2.68 ft. (0.76 ft.)  
- **Velocity$_{inlet}$:** 4.78 fps at normal depth  
- **Velocity$_{outlet}$:** 6.06 fps at normal depth  

---

**Profile Along Centerline of Chute**

- **$q_e$:** 11.97 cfs/ft. Equivalent unit discharge  
- **$F_s$:** 1.50 Factor of safety (multiplier)  
- **$z_t$:** 0.89 ft. Normal depth in chute  
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- **$D_{50}(F_s)$:** 22 in. (777 lbs. - 50% round / 50% angular)  
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- **Velocity$_{inlet}$:** 4.78 fps at normal depth  
- **Velocity$_{outlet}$:** 6.06 fps at normal depth  

---

**Notes:**
1. Output given as High Flow (Low Flow) values.  
2. Tailwater depth plus $d$ must be at or above the hydraulic jump height for the chute to function.  
3. Critical depth occurs $2y_c - 4y_c$ upstream of crest.  
4. Use min. 8 oz. non-woven geotextile under rock.
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - W 1  County: Pima
Designer: Ronson Chee  Checked by:
Date: 3/10/2010

Input Channel Geometry

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 10.0 ft.</td>
<td>Bw = 15.0 ft.</td>
<td>Bw = 10.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 (F_s)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>Side slopes = 3.0 (m:1)</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.0200 ft./ft.</td>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
<td>Bed slope = 0.0200 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

<table>
<thead>
<tr>
<th>Drainage area = na acres</th>
<th>Rainfall = O 0-3 in.</th>
<th>3-5 in.</th>
<th>5+ in.</th>
<th>Note: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- (Hdrop = 100 ft.)</td>
<td>Chute capacity = Q25-year</td>
<td>24-hour storm with a 3-5 inch rainfall)</td>
<td>Input tailwater (Tw):</td>
<td></td>
</tr>
<tr>
<td>Total capacity = Q100-year</td>
<td>Q_{high} = 190.0 cfs</td>
<td>High flow storm through chute</td>
<td>Tw (ft.) = Program</td>
<td></td>
</tr>
<tr>
<td>Q_{low} = 20.0 cfs</td>
<td>Low flow storm through chute</td>
<td>Tw (ft.) = Program</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Profile and Cross Section (Output)

<table>
<thead>
<tr>
<th>Energy Grade Line</th>
<th>Inlet Channel</th>
<th>Hydraulic Jump Height, z_2 = 2.5 ft. (0.59 ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_{py} = 0.14 ft. (0.14 ft.)</td>
<td>h_{py} = 0.62 ft. (0.17 ft.)</td>
<td>Tw + d = 4.89 ft. - Tw o.k.</td>
</tr>
<tr>
<td>H_{se} = 2.2 ft.</td>
<td>H_{se} = 2.15 ft.</td>
<td>(3.54 ft.) - Tw o.k.</td>
</tr>
<tr>
<td>Inlet Channel</td>
<td>Slope = 0.02 ft./ft.</td>
<td>(0.46 ft.)</td>
</tr>
<tr>
<td>1/2yn = 1.5 ft. (0.49 ft.)</td>
<td>10yc = 15 ft.</td>
<td>40(D_{50}) = 46 ft.</td>
</tr>
<tr>
<td>Velocity_{inlet} = 5.04 fps at normal depth</td>
<td>0.715yc = 1.1 ft. (0.27 ft.)</td>
<td></td>
</tr>
<tr>
<td>n = 0.060 (0.05)</td>
<td>z_1 = 0.84 ft. (0.21 ft.)</td>
<td></td>
</tr>
<tr>
<td>Inlet Apron</td>
<td>H_{drop} = 100 ft.</td>
<td></td>
</tr>
<tr>
<td>H_p = 2.06 ft. (0.46 ft.)</td>
<td>1.89 ft. (0.54 ft.)</td>
<td></td>
</tr>
<tr>
<td>15(D_{50})(F_s)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 oz. Min. Geotextile</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** When the normal depth (yn) in the inlet channel is less than the weir head (H_p), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

Profile Along Centerline of Chute

<table>
<thead>
<tr>
<th>q_t = 10.77 cfs/ft.</th>
<th>Equivalent unit discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td>F_s = 1.50</td>
<td>Factor of safety (multiplier)</td>
</tr>
<tr>
<td>z_t = 0.84 ft.</td>
<td>Normal depth in chute</td>
</tr>
<tr>
<td>n-value = 0.059</td>
<td>Manning’s roughness coefficient</td>
</tr>
<tr>
<td>D_{50}(F_s) = 20.8 in. (658 lbs. - 50% round / 50% angular)</td>
<td></td>
</tr>
<tr>
<td>2(D_{50})(F_s) = 41.6 in.</td>
<td>Rock chute thickness</td>
</tr>
<tr>
<td>Tw + d = 4.89 ft.</td>
<td>Tailwater above outlet apron</td>
</tr>
<tr>
<td>z_2 = 2.5 ft.</td>
<td>Hydraulic jump height</td>
</tr>
</tbody>
</table>

**The outlet will function adequately**

Typical Cross Section

**High Flow Storm Information**
**Rock Chute Design Data**

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

**Project:** Rock Chute Design Data  
**County:** Pima  
**Designer:** Ronson Chee  
**Checked by:**  
**Date:** 3/10/2010

### Input Channel Geometry

<table>
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<tr>
<th>Inlet Channel</th>
<th>Chute</th>
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<tr>
<td>Bw = 20.0 ft.</td>
<td>Bw = 30.0 ft.</td>
<td>Bw = 20.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 ($F_s$)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
<td>Side slopes = 3.0 (m:1)</td>
<td>n-value = 0.040</td>
</tr>
<tr>
<td>Bed slope = 0.0200 ft./ft.</td>
<td>Bed slope (3:1) = 0.330 ft./ft.</td>
<td>Bed slope = 0.0200 ft./ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.00 ft.</td>
</tr>
</tbody>
</table>

### Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

- **Drainage area = na** acres
- **Rainfall:** 0 - 3 in., 3 - 5 in., 5+ in.
- **Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- $H_{drop} = 100$ ft.**
- **Chute capacity = Q25-year 24-hour storm with a 3 - 5 inch rainfall**
- **Total capacity = Q100-year**
- **$Q_{high} = 404.0$ cfs High flow storm through chute**
- **$Q_{low} = 20.0$ cfs Low flow storm through chute**

**Note:** The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.

**Input tailwater (Tw):**
- **$Q_{high} = 404.0$ cfs**
- **$Q_{low} = 20.0$ cfs**

### Profile and Cross Section (Output)

1. **Note:** When the normal depth ($y_n$) in the inlet channel is less than the weir head ($H_p$), i.e., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

### Profile Along Centerline of Chute

- **$H_p = 2.22$ ft.**
- **$y_n = 1.79$ ft.**
- **$H_{drop} = 100$ ft.**
- **$D_{50} = 50$ ft. radius**
- **$n = 0.069$ (0.041)**

**Notes:**
1. Output given as **High Flow (Low Flow)** values.
2. Tailwater depth plus $d$ must be at or above the hydraulic jump height for the chute to function.
3. Critical depth occurs $2yc - 4yc$ upstream of crest.
4. Use min. 8 oz. non-woven geotextile under rock.

**Typical Cross Section**  
**High Flow Storm Information**

- **$q_t = 12.33$ cfs/ft.**  
- **$F_s = 1.50$**  
- **$z_f = 0.91$ ft.**  
- **$D_{50} = 22.4$ in. (815 lbs. - 50% round / 50% angular)**  
- **$2(D_{50})F_s = 44.7$ in.**  
- **$Tw + d = 5.09$ ft. - Tw o.k.**  
- **$z_2 = 2.76$ ft.**

**Equivalent unit discharge**  
**Factor of safety (multiplier)**  
**Normal depth in chute**  
**Manning’s roughness coefficient**  
**Rock chute thickness**  
**Tailwater above outlet apron**  
**Hydraulic jump height**  
**The outlet will function adequately**
Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: Rosemont Reclamation - SW 2
Design: Ronson Chee
Date: 3/10/2010

Input Channel Geometry

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 10.0 ft.</td>
<td>Bw = 15.0 ft.</td>
<td>Bw = 10.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 10.0 (m:1)</td>
<td>Factor of safety = 1.50 (Fs)</td>
<td>Side slopes = 3.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.040</td>
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</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, d = 3.0 ft.</td>
<td>Base flow = 0.0 cfs</td>
</tr>
</tbody>
</table>

Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

<table>
<thead>
<tr>
<th>Drainage area = na acres</th>
<th>Rainfall = 0 - 3 in.</th>
<th>3 - 5 in.</th>
<th>5+ in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apron elev. --- Inlet = 106.0 ft. --- Outlet = 3.0 ft. --- (Hdrop = 100 ft.)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chute capacity = Q25-year</td>
<td>Minimum capacity (based on a 5-year, 24-hour storm with a 3 - 5 inch rainfall)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total capacity = Q100-year</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qhigh = 205.0 cfs High flow storm through chute</td>
<td>Tw (ft.) = Program 0.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qlow = 20.0 cfs Low flow storm through chute</td>
<td>Tw (ft.) = Program</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Profile and Cross Section (Output)

<table>
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<tr>
<th>Notes:</th>
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<tbody>
<tr>
<td>1) Output given as High Flow (Low Flow) values.</td>
</tr>
<tr>
<td>2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.</td>
</tr>
<tr>
<td>3) Critical depth occurs 2yc - 4yc upstream of crest.</td>
</tr>
<tr>
<td>4) Use min. 8 oz. non-woven geotextile under rock.</td>
</tr>
</tbody>
</table>

Profile Along Centerline of Chute

<table>
<thead>
<tr>
<th>Notes:</th>
</tr>
</thead>
<tbody>
<tr>
<td>q = 11.53 cfs/ft. Equivalent unit discharge</td>
</tr>
<tr>
<td>F = 1.50 Factor of safety (multiplier)</td>
</tr>
<tr>
<td>z = 0.87 ft. Normal depth in chute</td>
</tr>
<tr>
<td>n-value = 0.059 Manning’s roughness coefficient</td>
</tr>
<tr>
<td>D50(Fs) = 21.6 in. (733 lbs. - 50% round / 50% angular)</td>
</tr>
<tr>
<td>2(D50)(F) = 43.2 in. Rock chute thickness</td>
</tr>
<tr>
<td>Tw + d = 4.96 ft. Tailwater above outlet apron</td>
</tr>
<tr>
<td>z2 = 2.62 ft. Hydraulic jump height</td>
</tr>
</tbody>
</table>

Typical Cross Section

High Flow Storm Information

* Use Hp along chute but not less than z2.