In response to the April 5, 2011 Technical Memorandum prepared by Golder Associates (Golder) titled Review of “Rosemont Site Water Management Update Review Responses”, I have prepared this response for your records. It is noted that Golder had either not reviewed the documents referenced in earlier correspondence or had not had access to such documents.

The following sections mimic the sections outlined in Golder’s April 5th Technical Memorandum. A copy of the April 5th memorandum is provided as Attachment 1. However, the attachments to Golder’s memorandum are not included herein.

1.0 INTRODUCTION

There were no specific questions within this section. Specific concerns were addressed in Sections 2.0 through 13.0 within Golder’s April 5, 2011 Technical Memorandum.

2.0 RUNOFF CALCULATIONS

In Section 2.0 of the November 30, 2010 Technical Memorandum prepared by Tetra Tech titled Rosemont Site Water Management Update Review Responses, an explanation was provided for using an average annual rainfall of 18 inches versus 24 inches. It was implied in Golder’s April 5, 2011 Technical Memorandum that Tetra Tech may have arbitrarily selected 18 inches versus 24 inches. Additional explanation of the selection is provided below.

In the Technical Memorandum titled Rosemont Copper Design Storm Precipitation Data/Design Criteria (Tetra Tech, 2009), the average yearly rainfall of 17.37 inches, taken between the years 1952 and 2007 at the Nogales 6 N meteorological station, was selected for the Rosemont Project site based on the following:

- An on-site meteorological monitoring station was installed at the Rosemont Project site in 2006. For the recorded years between 2006 and 2008, the average annual rainfall was 17.12 inches. This closely matched the average annual rainfall recorded at the Nogales 6 N station.

- The Nogales 6 N station is the closest meteorological station to the Project site that had recorded both precipitation and evaporation and had a large dataset (approximately 50 years). The next closest station having both data sets was the University of Arizona in...
Memorandum

To: Beverly Everson
Cc: Tom Furgason
From: Kathy Arnold
Doc #: 052/11 – 15.3.2
Subject: Transmittal of Technical Memoranda
Date: May 6, 2011

Rosemont Copper is transmitting the following memoranda responding to an April 5, 2011 Golder Associates review that was submitted to Rosemont on April 25, 2011.

- Response to Golder Comments, Rosemont memorandum dated May 6, 2011

Rosemont was disappointed with the inflammatory language used in the tables associated with the Golder review and disagrees completely with the characterizations. We believe that this review did not incorporate all information available and necessary to completely evaluate the information. The responses herein address that missed data.

This memorandum is being transmitted in electronic form via email only and a hardcopy is being delivered. Please let me know if you require additional hardcopy versions of this document.
Tucson. Selecting a station that provided both long-term rainfall and evaporation was deemed important for data consistency and correlation.

- The Santa Rita Experimental Range meteorological station, although closer to the Project site than the Nogales 6 N station, did not have evaporation data in addition to precipitation. Precipitation was recorded at the Santa Rita station from 1950 to 2005 with a calculated average annual precipitation of 22.19 inches. This value was considered high for the Rosemont Project site since the average annual precipitation for the Rosemont area, estimated by Sellers (University of Arizona, 1977) for the period of 1931 to 1970, was approximately 16 inches. Also, based on available records from the Western Regional Climate Center (WRCC, 2009), the average annual precipitation for Helvetia, which was the closest recording station to Rosemont, was 19.72 inches for the period of 1916 to 1950.

In addition to the above, Tetra Tech prepared a Technical Memorandum titled *Baseline Regulatory (100-Yr) Hydrology and Average-Annual Runoff, Rosemont Copper Project* dated March 4, 2010. This Technical Memorandum also corroborated using an average annual rainfall value closer to 18 inches.

In terms of storage volumes on the wide benches of the waste rock storage area, Golder is referred to the following Technical Memoranda prepared by Tetra Tech:

- Tetra Tech titled *Rosemont Waste Rock Storage Area Stormwater Management* dated April 2, 2010

Golder has indicated that the volume calculations for the waste rock bench storage facilities could not be located. The Tetra Tech memorandum states that the detention pools located on the large waste rock benches are designed to handle the runoff volume from a 500-year, 24-hour event. Excess runoff volume will be routed to the Perimeter Containment Areas (PCAs). The detention pools, in combination with the PCAs, have the ability to handle runoff volumes generated by a General PMP event. As stated in previous correspondence, the PCA's generally have capacity in excess of the General PMP event volume. Cumulative volumes of the PCA/detention basins associated with each respective PCA watershed are provided (summarized in Table 3 of the Tetra Tech memo).

Golder has again in their April 5, 2011 review memo indicated a desire for Rosemont to perform volume checks using the maximum saturation event. Golder, however, does not provide any regulatory basis for this request. Furthermore, Rosemont has opted to design many of the on-site structures to handle the PMP event. Again not a regulatory driven criteria. Nonetheless, the General PMP event (18.9 inches of rainfall) used at the site is anticipated to produce a higher volume than that of a maximum saturation event (about 10.1 inches of rainfall).

### 3.0 PIT DIVERSION AND PIT STORMWATER POND CALCULATIONS

In Section 2.0 of the November 30, 2010 Technical Memorandum prepared by Tetra Tech titled *Rosemont Site Water Management Update Review Responses*, Golder was referred to Appendix K of the Site Water Management Update report in reference to runoff calculations. It appears that Golder may not have read through Appendix K but only the review comments in the November 30th Technical Memorandum.

The flow-through drains were designed to handle either the General or Local Probable Maximum Precipitation (PMP) events. This refers to the comment by Golder that a 100-year, 24-hour event to size the South 2 Drain.
Additionally, it is implied that the 100-year, 24-hour event was used to manage stormwater in the Pit Stormwater Pond watershed basin and the Crusher Stormwater Pond watershed basin. As stated in Appendix K, runoff is managed by retention of minor events (100-year, 24-hour storm) and passage of larger events (PMP storms) to the flow-through drain system. There was no intent to retain the PMP event in either the Pit Stormwater Pond or the Crusher Stormwater Pond. As a note, these are large watershed basins that will reduce in size as mining progresses. The basin reporting to the Pit Stormwater Pond will eventually be eliminated.

4.0 SEDIMENT YIELD CALCULATIONS

This item did not require further explanation. Tetra Tech’s response, in the November 30, 2010 Technical Memorandum prepared by Tetra Tech titled Rosemont Site Water Management Update Review Responses, was satisfactory to Golder.

5.0 DRAINAGE FROM PERIMETER CONTAINMENT AREAS

The Perimeter Containment Areas (PCAs) are generally located along the southern edge of the Rosemont Ridge Landform between the toe of the Landform and an adjacent natural ridge. The toe of the Landform was set-back from the ridgeline to ensure containment of storm runoff and sediments within Barrel drainage. These toe areas are generally segmented with smaller ridges that run parallel to the main ridgeline. These smaller ridges, in conjunction with the large containment volumes of the PCAs, generally prevent “cascading” flow from one PCA to the next. Stormwater that reaches these areas will be sources of infiltration.

PCAs on the west side of the Landform, including other inlets areas, report to flow-through drains. These flow-through drains will be a source of distributed infiltration along the drain and will also pass storm flows from the west side of the Rosemont Ridge Landform to the east side under larger storm events.

The reclamation design prepared by Tetra Tech provides maintenance access to all areas of the Rosemont Ridge Landform. During the 20+ year operational period, including the anticipated post-closure period, inspections and maintenance activities will be performed as needed to maintain adequate storm channel capacity and containment volume capacity. A long-term maintenance plan will be prepared based on the final construction plans and as-built conditions.

Tetra Tech has prepared several documents outlining the anticipated changes in surface water flow to downstream receptors (i.e., the Compliance Point Dam). The two documents listed below specifically address water flow. No further work is planned.

- Baseline Regulatory (100-Yr) Hydrology and Average-Annual Runoff, Rosemont Copper Project (Tetra Tech Technical Memorandum dated March 4, 2010); and
- Post-Mining Regulatory (100-Yr) Hydrology and Average-Annual Runoff, Rosemont Copper Project (Technical Memorandum dated March 5, 2010).

Refinements to down-gradient runoff via the flow-through drains was also prepared in another Tetra Tech Technical Memorandum titled Rosemont Infiltration Analysis – Revised, dated April 05, 2010.

6.0 DEMONSTRATE ADHERENCE TO GEOMETRIC LANDFORM DESIGN BY GOLDER
In relation to the slope recommendations contained within Golder’s report titled *Rosemont Mine Landforming — Evaluation of Mine Waste Slope Geometry*, Tetra Tech used a more conservative slope height/slope angle combination than that presented in the report. Available information of the materials to be placed on the outer slopes was available to both Golder and to Tetra Tech. Tetra Tech selected this conservative approach since the exact gradation of the outer cover materials will vary and is not completely known at this time. Rosemont currently has test plots on-site that mimic, as closely as possible, the general slope configuration selected by Tetra Tech. Due to the goal of concurrent reclamation of the outer surfaces, the opportunity to make modifications to the surface treatment will be available throughout the 20+ year operational period.

7.0  DETAILED SEDIMENT CONTROL DESIGN DURING OPERATIONS

This item did not require further explanation. Tetra Tech’s response, in the November 30, 2010 Technical Memorandum prepared by Tetra Tech titled *Rosemont Site Water Management Update Review Responses*, was satisfactory to Golder.

8.0  PRE- AND POST-MINING SEDIMENT YIELD CALCULATIONS

In this item Golder indicated that Tetra Tech estimated the sediment yield (at the Compliance Point Dam) based only on a reduction in catchment area. This is a correct observation. It should be noted that the lower slope area immediately above lower Barrel Wash will be covered with riprap. All other slopes ultimately reporting runoff directly to lower Barrel Wash will pass through large stilling pools where sediments are anticipated to drop out (see *Site Water Management Update* report). These areas will be maintained during operations and through-out the post-closure period. Therefore, additional sediment loading to the down-gradient watercourse is not expected from these areas. Additional sediment control/catchments are also anticipated during the early years of buttress construction to prevent sediment loading to the down-stream watercourse. The faces of Rosemont Ridge Landform will stabilize over the operational and post-closure period, thus in the long-term limited sediment loading is anticipated from these sources.

9.0  WATER STORAGE ON BENCHES

This section deals with concerns related to the Waste Rock Storage Area. Similar concerns associated with the Dry Stack Tailings Facility are provided in Section 12.0.

Golder states that the plan of placing detention pools on the surface of the Waste Rock Storage Area is not a common practice for closure. The intent of this design was to limit the free flow of stormwater off the Rosemont Ridge Landform (Landform), with the goal of preventing erosion. Stormwater is generally retained where generated.

It should be noted that these detention pools are placed on wide benches (between 100 to 300 feet wide). Leaving such wide post-closure bench areas is also not a common practice in the industry. However, because the Rosemont Ridge Landform will be constructed with closure in mind, designing these large benches into the Landform allowed for contouring as well as run-off retention and areas of enhanced vegetation growth.

The final design of these detention pool structures will likely include a sloped bottom with the deepest part of the pool located away from the slope face and with some compaction applied to the bottom of the pool. This would cause any seepage that may develop to be managed away from the slope face and
would not likely follow some predetermined path to the slope face, i.e., causing internal erosion. Fines are also expected to settle on the pool bottom, aiding in the retention of storm runoff due to decreased permeability. Although these detention pools are large, designed cumulatively hold the 500-year, 24-hour event volume, this would be an unusual event. Under average annual conditions, seepage is not anticipated due to the high evaporation rate of the area (see the August 2010 document prepared by Tetra Tech titled *Infiltration, Seepage, Fate and Transport Modeling Report – Revision 1*).

Regardless, additional stability and seepage analysis will be performed during the final design state and/or during the as-built stage when properties of the bench materials can be tested and better defined.

It is stated in Golder’s April 5th Technical Memorandum that Golder was not provided volume calculations associated with these detention pools. This information is readily available in the following:

- Tetra Tech titled *Rosemont Waste Rock Storage Area Stormwater Management* dated April 2, 2010

### 10.0 RIPRAP DOWNCHUTES


One of the primary empirical procedures recommended in TS14C for sizing rock riprap on steep slopes is the ARS Rock Chutes method (Robinson, Rice, and Kadavy 1998). It is highlighted in a subset of Technical Supplement 14C provided as Attachment 3. This procedure is the one that Tetra Tech employed to calculate rock sizes for the rock-chute spillways at the Rosemont.

With regard to slope ranges, note that the TS14C clearly states that the ARS Rock Chutes method can be used for slopes up to 40 percent. The slopes of the rock-chute spillways at Rosemont are designed at 33 percent.

With regard to particle sizes, Golder states...“the use of the Robinson method is limited to a maximum rock size of 11 inches, as confirmed by TetraTech in their response.” It is noted that Tetra Tech did not say that the Robinson method was limited to rocks of a maximum of 11 inches in size. What was actually said was “The ‘Robinson’ equation was actually developed using rock sizes up to 11 inches in diameter,” and that the authors went on the state “Appropriate engineering judgment should be applied when extending this design information beyond the data base from which it was developed.”

In this regard, note that TS14C contains a full-page design example for a rock chute spillway under the title “Rock Chute Design Data.” Again, see the document in Attachments 2 and 3.

In the referenced design example provided by NRCS TS14C, a unit discharge of 13.65 cfs/ft is on a 20 percent chute slope. A unit discharge of 13.65 cfs/ft is in the range of the unit discharges proposed for the rock-chute spillways at Rosemont. Note that the size of the D₅₀ rock for this design example is 16.2 inches (factor of safety = 1.20). This D₅₀ size is nearly 50 percent greater than the 11-inch maximum size to which Golder claims that Tetra Tech says the Robinson method (i.e., the ARS Rock Chute method) is limited. So, obviously, the NRCS does not believe that the ARS Rock Chute method is limited to maximum rock sizes of 11 inches. The NRCS obviously believes that appropriate engineering judgment can be applied, as exhibited in the design example contained in TS14C.
In this regard, TS14C states that:

Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. A factor of safety is often advisable to account for unknowns and uncertainty. In some cases, the factor of safety is part of the sizing formulas provided. Where a factor of safety is not built into the procedure, the designer should multiply the resulting size by an appropriate value. Appropriate engineering judgment should be applied when assigning a factor of safety... Typically, a factor of safety will range from 1.1 to 1.5. The risk and uncertainty associated with a project should be reflected in the factor of safety.

Tetra Tech’s designers are cognizant of the unknowns and uncertainties associated with the placement of rock riprap on such steep slopes, as described in the preceding text excerpted from TS14C. This is why a high factor of safety of 1.5 was applied by Tetra Tech.

In addition, the TS14C indicates, and the design example corroborates, that the blanket thickness of the rock-chute spillway should be 2D_{50}. In fact, this is what the ARS Rock Chute method recommends. In the referenced design example, the flow depth is only about 40 percent of the thickness of the rock chute.

Furthermore, the design example clearly indicates the use of an 8-oz, minimum, geotextile everywhere underneath the rock-chute spillway. Again, the NRCS does not seem to be worried about placing geotextile under rock on steep slopes... in fact, they recommend it.

Finally, the NEH is a nationally recognized manual for engineering design, and is used throughout the country by numerous public and private entities. Tetra Tech’s designers have indicated no reason, nor supportive data, to suggest that the application of procedures contained in the NEH are “indefensible,” as could be construed from the latest statements made by Golder.

11.0 FLOW-THROUGH DRAINS

Tetra Tech titled Rosemont Flow-Through Drain Sedimentation Analysis dated August 31, 2010 and is provided as Attachment 4 prepared a Technical Memorandum. This Technical Memorandum is referred to as a response to Golder’s review comments requesting quantitative sedimentation analysis associated with the flow-through drain inlets.

12.0 WATER ON TOP OF TAILINGS FACILITY AND WASTE ROCK STORAGE AREA

This section deals with concerns related to the Dry Stack Tailings Facility. Similar concerns associated with the Waste Rock Storage Area are provided in Section 9.0.

It is stated in Golder’s April 5th Technical Memorandum that Golder was unable to locate the following reports that were used to answer stability and seepage concerns in Golder’s previous set of review comments dated August 5, 2010.

- Rosemont Copper Company Dry Stack Tailings Storage Facility Design report (AMEC, 2009); and
- Infiltration, Seepage, and Fate and Transport Modeling Report (Tetra Tech, 2010).

These documents are and have been readily available via inquiry to SWCA as well on Rosemont’s website. Additionally, Tetra Tech has also prepared an updated document titled infiltration, Seepage, Fate and Transport Modeling Report – Revision 1, dated August 2010.
In addition to the documents listed above, Tetra Tech also listed several other documents that were pertinent to the original Golder review comment such as AMEC’s report titled *Rosemont Copper Company Dry Stack Tailings Storage Facility Stormwater Management Preliminary Design Report* (April, 2010).

Golder’s concerns related to “internal erosion” have been addressed by AMEC’s documents. The dry stack tailings are compacted directly underneath the waste rock buttress areas and runoff detention pools will be located away from the buttress slope. The dry stack tailings surface will be sloped toward these pools away from the outer perimeter. Additionally, seepage is not expected to develop in the tailings (see the updated document titled *Infiltration, Seepage, Fate and Transport Modeling Report – Revision 1.*) Outlets are also designed on the top surfaces as part of the stormwater control plan to control large storm events. Also, a large containment berm will be left around the perimeter of the dry stack tailings as part of the last buttress lift.

There is no basis for Golder’s statement that ponding water on the top of the dry stack tailings structure is not a good or standard practice. Based on the stability, seepage, and water quality aspects of the Rosemont Dry Stack Tailing Facility, there are no technical disqualifications to doing do.

### 13.0 ALLOWANCE FOR EROSION IN CONTAINMENT AREAS

Golder has indicated that the volume calculations for the waste rock bench storage facilities could not be located. Golder is referred to the Technical Memorandum prepared by Tetra Tech titled *Rosemont Waste Rock Storage Area Stormwater Management* dated April 2, 2010 (specifically in Table 3 of the memo). The memorandum states that the detention pools located on the large waste rock benches are designed to handle the runoff volume from a 500-year, 24-hour event. Excess runoff volume will be routed to the Perimeter Containment Areas (PCAs). The detention pools, in combination with the PCAs, have the ability to handle the General PMP event. As stated in previous correspondence, the PCA’s generally have capacity in excess of the General PMP event volume. Cumulative volumes of the detention basins associated with each respective PCA watershed are provided.

In reference to the drainage bench channels associated with the dry stack tailings facility, Golder is referred to Tetra Tech’s Technical Memorandum titled *Rosemont Dry Stack Tailings Facility Drainage Bench Analysis* dated January 28, 2010. Tetra Tech selected a 30% channel sedimentation allowance for the express purpose of demonstrating that the selected channel size was large and could still accommodate the peak design flow with this sedimentation factor. There does not seem to be any rational to compare Golder’s estimated average annual erosion loss of 14.4 inches to the channel capacity. Golder’s value is a hypothetical average annual number. On-going maintenance would preclude this accumulation. The surface is also expected to stabilize over time as vegetation is established and as fines are washed out and the surface becomes more course.

As a note, the 14.4 inches of average annual erosion was based on slope configurations different than that selected by Tetra Tech. Tetra Tech selected a more conservative slope configuration (slope height/slope length).

The reclamation design prepared by Tetra Tech provides maintenance access to all areas of the Rosemont Ridge Landform. During the 20+ year operational period, including the anticipated post-closure period, inspections and maintenance activities will be performed as needed to maintain adequate storm channel capacity and containment volume capacity. A long-term maintenance plan will be prepared based on the final construction plans and as-built conditions.
14.0 CONCLUSION

There were no specific questions/concerns summarized within this section. Specific concerns were addressed in Sections 2.0 through 13.0 within Golder's April 5, 2011 Technical Memorandum.
Attachment 1
Golder Technical Memorandum
Rosemont Copper Project
Review of “Rosemont Site Water Management Update Review Responses”
April 5, 2011
1.0 INTRODUCTION

At the request of SWCA Environmental Consultants (SWCA), Golder Associates Inc. (Golder) conducted on August 5, 2010, a review of the TetraTech design document Site Water Management Update for the Rosemont Copper Project (Site Water Management Update) and submitted comments to SWCA on a document titled Rosemont Copper Project, Technical Review of Site Water Management (Golder 2010a). That memorandum is provided herein as Attachment 1. The review consisted of reading the pertinent sections of the report and supporting documents and rendering a professional opinion regarding whether the data, assumptions, and methods used in the report conform to currently accepted industry practice in the disciplines of hydrology and sediment transport only. In addition, Golder was requested to render a professional opinion whether the conclusions reached in the report appear reasonable.

TetraTech provided responses to Golder’s review on November 30, 2010 (TetraTech 2010a). The TetraTech document is provided herein as Attachment 2.

This memorandum provides additional comments to TetraTech’s responses. For consistency, this technical memorandum provides direct response to TetraTech’s comments and is organized following TetraTech’s response section headings and numbers. Table 1 provides a summary of Golder’s original concerns after the initial review of the Site Water Management Update as well as columns stating whether TetraTech has, in Golder’s opinion, sufficiently addressed or partially addressed the concern, or if the concern still remains outstanding. Justification for the current status of each concern is given in the following sections.
### Table 1 Red Flags and Potential Fatal Flaws

<table>
<thead>
<tr>
<th>Concern</th>
<th>Section</th>
<th>Addressed</th>
<th>Partially Addressed</th>
<th>Outstanding</th>
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<tbody>
<tr>
<td>Using smaller precipitation depth (18in) to calculate average annual runoff instead of NRCS recommended depth (24in)</td>
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<td>No volume check calculations using maximum saturation event conditions</td>
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<tr>
<td>No calculations presented for pit diversion channel and pit stormwater pond</td>
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<td>Methodology used for sediment yield calculations should be reviewed as it is considered to be inappropriate</td>
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<td>Lack of drainage from perimeter containment areas</td>
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<tr>
<td>Demonstrate adherence to geometric recommendations on landform element suggestions previously proposed by Golder</td>
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<tr>
<td>Lack of detail for sediment control designs during operations</td>
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<tr>
<td>Specific sediment yield is the same for pre- and post-mining conditions, which appears to be incorrect</td>
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<tr>
<td>Storage on top of benches is unusual for long-term closure and could lead to massive failure</td>
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<tr>
<td>Down chutes on both tailings facility and waste rock can lead to failure as riprap lining may be inappropriate protection type</td>
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<tr>
<td>Flow-through drains: Potential long-term difficulties with maintenance and retaining discharge capacity</td>
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<tr>
<td>Water storage on top of tailings facility and waste rock dump is unusual for long-term closure and could lead to massive failure</td>
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<td>x</td>
</tr>
<tr>
<td>No allowance has been made for anticipated erosion from landforms into storage locations on benches and perimeter containment areas; 14 to 15 inches of erosion is anticipated from the landform areas</td>
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<td>x</td>
</tr>
</tbody>
</table>
2.0 RUNOFF CALCULATIONS

TetraTech's response to Golder's comment was that there is no corroborative data supporting 24-inch average annual rainfall and that using 24 inches gives unacceptably high results when compared to measured data from nearby watercourses. Golder's interpretation of this comment is that the higher average annual rainfall reported by NRCS of 24 inches was not used because TetraTech believes it leads to unrealistic results. Therefore, TetraTech chose the lower average annual rainfall amount of 18 inches as reported by the Arizona Department of Water Resources Water Atlas. This appears to be qualitative and Golder is uncertain how to assess the selection. The question that comes to mind is: How is it determined what is realistic or unrealistic?

Golder's principal concern relates not to peak discharge but to water volume. The maximum saturation event is focused on volume rather than peak discharge and usually occurs over several consecutive days of precipitation. TetraTech does not provide a quantitative response to the issue as to whether the maximum storage requirements were adequately determined; particularly as it affects the storage in various facilities including the storage on the waste dump benches (see section 9.0 for specific concerns on bench storage). Golder was unable to locate and verify the calculations determining the size of the storage on the waste dump benches in the documentation provided. The only calculations for the benches that were identified were discharge calculations to size conveyance channels.

Golder recommends verification of the selected storm event and quantitative assessment of maximum storage requirements.

3.0 PIT DIVERSION AND PIT STORMWATER POND CALCULATIONS

TetraTech indicates that they used the 100-year/24-hour storm to size the pond and South 2 Drain. The design criteria established in the Site Water Management Update require that the Pit Stormwater Pond and the Crusher Stormwater Pond have the capacity to contain the PMP volume of rainfall. Using the 100-year event does not satisfy that design criterion.

Golder recommends that TetraTech evaluate the potential effect of the maximum saturation event.

4.0 SEDIMENT YIELD CALCULATIONS

TetraTech's response is acceptable.

5.0 DRAINAGE FROM PERIMETER CONTAINMENT AREAS

TetraTech does not address the question as to why only one of the PCAs has an outlet to a natural stream. Golder's interpretation is that some of the PCAs will discharge water into the flow-through drains, but that the water collected in others will be lost to the natural system. Please indicate whether that is the intent and whether it is acceptable from an environmental point of view.
9.0 WATER STORAGE ON BENCHES

It is Golder’s opinion that placing unlined ponds on granular material benches without drainage provision is not good or standard practice. Water storage on benches may lead to either or both seepage on the waste rock dump slopes and concentrated flow on the slope face, which could result in slope instability and undue erosion. Additionally, Golder is concerned about the cover materials for the dry stack and waste rock. The dry stack is likely to be compacted very fine materials with low permeability; the waste rock is likely to be Run of Mine, large to medium rock with high permeability. This difference may exacerbate the concerns listed above.

Volume calculations for the storage ponds on the benches were not included in the materials provided to Golder and, therefore, could not be reviewed. Therefore, we cannot comment on the appropriateness of the size of these ponds.

Golder has not implemented a design where ponded water is stored on waste rock dumps. Therefore, it is requested that TetraTech provide case studies and/or regulatory acceptance of this approach. Additionally, it is recommended that TetraTech provide more detail as it relates to pond design (volume and lining details) and to present stability and seepage analyses indicating that the behavior of unlined ponds without drainage provision will not result in undue slope failure and will not result in erosion initiated by seepage on the waste dump slope surfaces.

10.0 RIPRAP DOWNCHUTES

Golder remains concerned about the fact that TetraTech uses the empirical equations for sizing the riprap outside of their intended ranges, both as far as slope ranges and particle sizes are concerned. The USGS and FHWA methods are not intended for steep slope design. The USACE method can be used to a maximum slope of 20%, while the use of the Robinson method is limited to a maximum rock size of 11 inches, as confirmed by TetraTech in their response.

The slope angles requiring protection is on the order of 33%, which is larger than those assumed by the USGS, FHWA, and the USACE. Therefore, those methods cannot justifiably be used to size riprap for the steep slopes at Rosemont Mine. TetraTech’s specification of riprap sizes on the order of 2 to 2.5 times the maximum tested sizes is a large extrapolation. Placing such large rock sizes on steep channels subjects the rock to forces that are different to those tested. As indicated before, it is Golder’s experience that riprap channels with such large rock elements often fail at much lower discharges than assumed.

As a frame of reference, one can relate the calculated flow depth to the thickness of the proposed riprap. The calculated flow depth is just below 1 foot, while the thickness of the proposed riprap protection is up to 48 inches thick (two times the median particle size of 24 inches). The design is proportionally not defensible.
TetraTech justifies the use of the geo-fabric by referring to the USACE. As previously noted, the USACE maximum channel slope is 20%, which is considerably smaller than the 33% at Rosemont mine. Golder remains concerned about the use of geotextile under riprap on steep slopes.

Golder recommends that TetraTech consider using other means of protecting the steep channels against erosion.

11.0 FLOW-THROUGH DRAINS

TetraTech refers to an extensive flow-through drain sedimentation analysis that was performed. Golder found the TetraTech referenced hydraulic analysis documentation (Campbell 1989) but could not find any detailed analysis related to sedimentation of the drains or inlets to the drains. Golder's sedimentation concerns relate principally to the inflow conditions at the rock berm. The only analysis that we are aware of is that which is attached to TetraTech's response, referred to as the "LEPS CALCULATION WORKSHEET." This calculation is a simple comparison between flow velocity and settling velocity, which can hardly be viewed as a detailed sedimentation analysis. Sedimentation against and in front of the rock berm inlet has not been quantitatively considered. The reference to Campbell (1989) relates to qualitative assessments of conditions. Quantification of these issues is in order, due to the fact that it is now possible to do so with present technology.

TetraTech is requested to provide quantitative sedimentation calculations.

12.0 WATER ON TOP OF TAILINGS FACILITY AND WASTE ROCK STORAGE AREA

TetraTech indicates that it relies on the AMEC (2009) report entitled "Rosemont Copper Company Dry Stack Tailings Storage Facility Final Design Report" and on the TetraTech (2010b) report entitled "Infiltration, Seepage and Fate and Transport Modeling Report" as proof that the dry stack facility will be stable and that infiltration will not be a problem. Golder was unable to locate these reports and cannot comment on this reliance by TetraTech. Golder could not locate any calculations addressing the issue of internal erosion.

The concerns relating to storage on the benches of the waste rock dump have been expounded upon in section 9.0 of this memorandum.

TetraTech is requested to address these concerns, particularly the issues related to potential internal erosion.

13.0 ALLOWANCE FOR EROSION IN CONTAINMENT AREAS

TetraTech indicates that there is excess capacity: If the facility is evaluated as a whole and if sedimentation reduces upstream capacity, then it can be captured in downstream facilities. Golder could
not locate volume calculations for determining the size of the bench storage facilities. The 500-year/24-hour calculations relate to discharge and not volume assessment.

It is recommended that TetraTech determine whether the 30\% arbitrary allowance for sedimentation in the bench channels is comparable to the amount of anticipated erosion from the slopes. Golder estimated erosion of 14.4 inches on average over all the slopes.

Golder is also uncertain whether the closure plan includes long-term maintenance, which may be required to keep the bench storage facilities operational. TetraTech is requested to address the long-term maintenance of these structures.

### 14.0 CONCLUSION

Golder summarized the potential Red Flags and Fatal Flaws found after the initial review of the Site Water Management Update on August 5, 2010, and has provided a summary of the extent to which TetraTech responded to these issues in Table 1. Golder has provided explanations for the current status of each concern and TetraTech is requested to respond to these unresolved concerns.

### 15.0 REFERENCES


Attachment 2
Technical Supplement 14C of U. S. Department of Agriculture
Soil Conservation Service
Engineering Handbook
Chute Spillways, Section 14
chute spillways

section 14
The aim of this handbook is to present in brief and usable form information on the application of engineering principles to the problems of soil and water conservation. While this information will be sufficient for the solution of most problems, other sources of reference material should be used when applicable.

The scope of the handbook necessarily is limited to phases of engineering which pertain directly to the program of the Soil Conservation Service. Therefore, emphasis is given to problems involving the use, conservation, and disposal of water, and the design and use of structures most commonly used for water control. Typical problems in soil and water conservation are described, basic considerations are set forth, and step-by-step procedures are outlined to enable the engineer to understand a recommended solution. These solutions will help in training engineers and will promote nation-wide uniformity in procedures. Since some phases of the field of conservation engineering are relatively new, further experience may result in improved methods which will require revision of the handbook from time to time.

This section of the Engineering Handbook has been written by Paul D. Doubt, civil engineer. Richard M. Matthews and other members of the Design Section staff have helped materially with the calculations and in the preparation of charts and examples. This work was done under the general direction of M. M. Culp, Head, Design Section. A preliminary draft was submitted to field engineers and others for review. Their suggestions led to improvements in the text and are sincerely appreciated.

Many sources of information have been utilized in developing the material. Original contributions are acknowledged in the text.

This revision removes the references to concrete volumes which appeared in the original handbook. Concrete volumes for the elements of chute spillways may be obtained by using currently approved computer programs.
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3. Dimensions of Chute Spillway

First Situation—Introduction

Relationship of Hydraulic Design, Structural Design, and Economics

Relationship of Spillway Storage, Design Discharge $Q_r$, Vertical Drop $Z$, and Site Location

Outline of the Method for Determining the Dimensions of a Chute

I. The Associated Spillway is Insignificant

A. Bottom Slope of Spillway is Known

1. Chute spillways having associated appurtenance costs which are not significantly dependent on the dimensions of the chute

2. Chute spillways having associated appurtenance costs which are dependent on the dimensions of the chute

B. Bottom Slope of Spillway is Unknown

1. Chute spillways having associated appurtenance costs which are dependent on the dimensions of the chute

II. The Associated Spillway Storage is Significant

A. Bottom Slope of the Spillway is Known

1. Chute spillways having associated appurtenance costs which are not dependent on the dimensions of the chute

2. Chute spillways having associated appurtenance costs which are dependent on the dimensions of the chute

B. Bottom Slope of Spillway is Unknown

1. Chute spillways having associated appurtenance costs which are dependent on the dimensions of the chute

Types of Inlets for other Functions or when the Wave Freeboard is Sufficiently Large
# ENGINEERING STANDARD DRAWINGS

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Cover photo: Stone may be needed as a foundation on which to implement other restoration features such as soil bioengineering practices. Stone may also be needed to form an erosion resistant layer. How large, how thick, and how deeply keyed-in are questions that are addressed in the design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

(210-VI-NEH, August 2007)
# Stone Sizing Criteria

## Technical Supplement 14C

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Stone Sizing Criteria

Purpose

Many channel protection techniques involve rock or stone as a stand-alone treatment or as a component of an integrated system. Stone used as riprap can also be a component of many streambank soil bioengineering projects. Many Federal and state agencies have developed methods and approaches for sizing riprap, and several of those techniques are briefly described in this document. Stone sizing methods are normally developed for a specific application, so care should be exercised in matching the selected method with the intended use. While many of these were developed for application with stone riprap revetments, they are also applicable for other designs involving rock, as well.

Introduction

When the attacking forces of flowing water exceed the resisting forces of the existing channel material, channel protection is needed as part of a restoration design. Channel protection typically ranges from soil bioengineering treatments to more traditional armor ing methods. Numerous methods have been developed for the design and sizing of riprap. Several common techniques for estimating the required stone size are briefly outlined in this document. The designer is encouraged to review the complete development of a selected method and assess the relevance of the assumptions behind that selected method to their application. In this document, the words rock and stone are used interchangeably.

Size is one of many considerations when designing riprap for use in protecting channel bed and banks. The designer must also address issues such as material strength, density, angularity, durability, length-to-width ratio, gradation, bedding, piping potential, and channel curvature. These important design and construction considerations are addressed in NEH054 TS14K.

Basic concepts

Description of forces on a stone

A rock will be stable until the lift and drag forces of moving water exceed a critical value or threshold. Therefore, for a given rock size subjected to a given force of moving water, there is some unit discharge where the rock will move and become unstable. Forces on a submerged stone, as indicated in figure TS14C-1, typically consist of the force exerted by the flowing water \( F_w \), drag force \( F_d \) associated with flow around the object (skin friction and form drag), lift force \( F_l \) associated with flow around the particle (pressure differences caused by streamline curvature and increased velocity around a particle), submerged weight of the stone \( F_s \), and resisting force due to the particle interlock and/or contact between stones \( F_e \).

While some methods are based on a particle force balance, all rock sizing methods are essentially empirical techniques. Field performance data, physical models, and theoretical developments have all contributed to the diverse set of approaches used to determine stable stone sizes for restoration designs.

Velocity-based approaches and boundary shear or stress-based approaches are the two prominent classes of methods that have been used to evaluate the erosion resistance of materials. While shear or stress-based approaches are considered more academically correct, velocity-based methods are still widely used. The design stress and the design discharge do not necessarily represent the same conditions.

Figure TS14C-1 Forces on a submerged stone

Flow direction

(210-VI-NEH, August 2007)
Flow conditions

The flow conditions associated with a particular application will have a major influence on selecting the right rock sizing method. While it is difficult to select a single criterion that separates rock sizing methods, high energy and low energy are used in this development. For example, a technique developed for the design of a riprap blanket revetment in a low-energy environment would not necessarily be suitable for estimating the minimum stone size in a high-energy environment, where the stone projects into the flow. Such applications, including instream habitat boulders, grade stabilization, and stream barbs, should be addressed with impinging flow design techniques. Table TS14C-1 lists some of the flow descriptors that can be associated with high- and low-energy flow conditions. Photographs of the different energy conditions where stone is applied as part of the solution are shown in figures TS14C-2 through 4. In figure TS14C-2, riprap is used to control a headcut. Riprap chutes can be used to control erosion from a headcut in a channel or in a side inlet to a channel. Riprap for this type of structure would fall in the steep-slope, high-energy design. Figure TS14C-3 shows riprap used to prevent erosion from flow from a side inlet to a channel. This structure also prevents a headcut from moving into the field. As illustrated in figure TS14C-4, if the toe of the slope is eroding, and it cannot be controlled with bioengineering alone, lining the toe of the slope with stone may be a solution. Riprap for this type of structure would fall in the mild slope, low-energy design.

The appropriate rock sizing method must consider the flow energy associated with the particular application. While there are exceptions, most rock sizing methods were developed for either a high- or low-energy flow condition.

Table TS14C-1 High-energy vs. low-energy conditions

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<td>Steep slope</td>
<td>Mild slope</td>
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<td>High turbulence</td>
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<td>Impinging flow</td>
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<td>Rapidly varied flow</td>
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TS14C-2 (210-VI-NEH, August 2007)
Sizing techniques

There are many techniques for sizing stone, and each method has advantages and disadvantages. Many techniques were derived under specific conditions and developed for particular applications. While this list is not complete and the description is not exhaustive, several commonly used methods are presented. The designer should review the applicability of a technique before choosing it to size stone for a particular project. Following is a brief description of several rock sizing techniques.

Isbash method
The Isbash formula (Isbash 1936) was developed for the construction of dams by depositing rocks into moving water. The Isbash curve should only be used for quick estimates or for comparisons. A coefficient is provided to target high- and low-turbulence flow conditions, so this method can be a high- or low-energy application. The equation is:

\[ V_c = \sqrt{\frac{g}{C}} \left( \frac{Y_s - Y_w}{Y_s} \right) \left( D_{50} \right)^{0.50} \]  

(eq. TS14C-1)

where:

- \( V_c \) = critical velocity (ft/s)
- \( C \) = 0.86 for high turbulence
- \( C \) = 1.20 for low turbulence
- \( g \) = 32.2 ft/s²
- \( Y_s \) = stone density (lb/ft³)
- \( Y_w \) = water density (lb/ft³)
- \( D_{50} \) = median stone diameter (ft)

A graphical solution is provided in figure TS14C-5 (ch. 16 of the Engineering Field Manual) This graph should be used only for quick estimates at a conceptual design level.

The U.S. Army Corps of Engineers (USACE) provides additional guidance for the use of the Isbash technique in EM 1110-2-1601. The required inputs are channel velocity, specific gravity of the stone, and a turbulence coefficient. The turbulence coefficient has two values that represent either high turbulence or low turbulence. The graphical solution for this is shown in figure TS14C-6(a) and (b).
Figure TS14C-6  Graphical solution for Isbash technique

Basic equations:

\[ V = C \left[ 2g \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right) \right]^{1/2} D_{50} \]

\[ D_{50} = \left( \frac{8W_{50}}{\gamma_s} \right)^{1/3} \]

where:

- \( V \) = Velocity, ft/s
- \( \gamma_s \) = Specific stone weight, lb/ft\(^3\)
- \( \gamma_w \) = Specific weight of water, 62.5 lb/ft\(^3\)
- \( W_{50} \) = Weight of stone, subscript denotes Percent of total weight of material containing stone of less weight
- \( D_{50} \) = Spherical diameter of stone having the same weight as \( W_{50} \)
- \( C = \) Isbash constant (0.86 for high turbulence level flow and 1.20 for low turbulence level flow)
- \( g = \) Acceleration of gravity, ft/s\(^2\)

Stone stability velocity vs. stone diameter

Hydraulic design chart 712-1 (Sheet 1 of 2)
### Basic equations:

\[
V_c = C \left[ \frac{\gamma_s - \gamma_w}{\gamma_w} \right] \left( \frac{D_{so}}{2} \right)^{\frac{1}{2}}
\]

\[
D_{so} = \left( \frac{8W_{so}}{\pi \gamma_s} \right)^{\frac{1}{3}}
\]

**Where:**
- \( V_c \) = Velocity, ft/s
- \( \gamma_s \) = Specific stone weight, lb/ft\(^3\)
- \( \gamma_w \) = Specific weight of water, 62.5 lb/ft\(^3\)
- \( W_{so} \) = Weight of stone, subscript denotes percent of total weight of material containing stone of less weight
- \( D_{so} \) = Spherical diameter of stone having the same weight as \( W_{so} \)
- \( C \) = Isbash constant (0.86 for high turbulence level flow and 1.20 for low turbulence level flow)
- \( g \) = Acceleration of gravity, ft/s\(^2\)

### Stone stability

**Velocity vs. stone diameter**

Hydraulic design chart 712-1

(Sheet 2 of 2)
National Cooperative Highway Research Program Report 108

This method (Anderson, Paintal, and Davenport 1970) is suggested for design of roadside drainage channels handling less than 1,000 cubic foot per second and a maximum slope of 0.10 foot per foot. Therefore, this application can be used for high- or low-energy applications. Photo documentation shows that most of the research was done on rounded stones. This method will give more conservative results if angular rock is used.

\[ \tau_c = \frac{RS_s}{4} \]  
(eq. TS14C-2)

\[ \tau_c = 4D_{50} \]  
(eq. TS14C-3)

\[ D_{50} = \frac{RS_s}{4} \]  
(eq. TS14C-4)

- \( \tau_c \) = critical tractive stress
- \( \gamma \) = 62.4 lb/ft³
- \( R \) = hydraulic radius (ft)
- \( S_s \) = energy slope (ft/ft)
- \( D_{50} \) = median stone diameter (ft)

A similar approach has been proposed by Newbury and Gaboury (1993) for sizing stones in grade control structures. This relationship is:

tractive force (kg/m²) = incipient diameter (cm)

USACE—Maynord method

This low-energy technique for the design of riprap is used for channel bank protection (revetments). This method is outlined in USACE guidance as provided in EM 1110-2-1601, and is based on a modification to the Maynord equation:

\[ D_{so} = FS \times \frac{C_v}{C_s} \times \frac{V}{\sqrt{K_s \times g \times d}} \]  
(eq. TS14C-5)

where:
- \( D_{so} \) = stone size in ft; in percent finer by weight
- \( d \) = water depth (ft)
- \( FS \) = factor of safety (usually 1.1 to 1.5), suggest 1.2
- \( C_s \) = stability coefficient Z=2 or flatter C=0.30, (0.3 for angular rock, 0.375 for rounded rock)
- \( C_v \) = velocity distribution coefficient (1.0 for straight channels or inside of bends, calculate for outside of bends)
- \( C_T \) = thickness coefficient (use 1.0 for 1 \( D_{10} \) or 1.5 \( D_{so} \) whichever is greater)
- \( \gamma_w \) = specific weight of water (lb/ft³)
- \( \gamma_s \) = specific weight of stone (lb/ft³)
- \( V \) = local velocity; if unknown use 1.5 \( V_{average} \)
- \( g \) = 32.2 ft/s²
- \( K_s \) = side slope correction as computed below

\[ K_s = \left[ 1 + \sin^2 \frac{\theta}{2} \right] \]  
(eq. TS14C-6)

where:
- \( \theta \) = angle of rock from the horizontal
- \( \phi \) = angle of repose (typically 40°)

Note that the local velocity can be 120 to 150 percent of the average channel velocity or higher. The outside bend velocity coefficient and the side slope correction can be calculated:

\[ C_v = 1.28 - 0.2 \log \left( \frac{R}{W} \right) \]  
(eq. TS14C-7)

where:
- \( R \) = centerline bend radius
- \( W \) = water surface width

In the analysis used to develop this formula, failure was assumed to occur when the underlying material became exposed. It should be noted that while many of the other techniques specify a \( D_{so} \), Maynord (1992) specifies a \( D_{30} \) which will typically be 15 percent smaller than the \( D_{50} \). This assumes a specific gradation of:

\[ 1.8D_{15} < D_{50} < 4.6D_{50} \]  
(eq. TS14C-8)

The USACE developed this method for the design of riprap used in either constructed or natural channels which have a slope of 2 percent or less and Froude numbers less than 1.2. As a result, this technique is not appropriate for high-turbulence areas.

Maynord's side-slope and invert equation is for cases where the protective blanket is constructed with a relatively smooth surface and has no significant projections. It is appropriate for use to size stone-toe protection. However, it has been suggested that with some adjustment to the coefficients (typically using a velocity coefficient of 1.25 and a local velocity equal to 160% of the channel velocity), Maynord's method can
be used for exposed boulders or stones exposed to impinging flow.

**U.S. Bureau of Reclamation method**
This high-energy technique is outlined in U.S. Bureau of Reclamation (USBR) EM-25 (Peterka 1958) and was developed for sizing riprap below a stilling basin. It was empirically developed using 11 prototype installations with velocities ranging from 1 foot per second to 20 foot per second. The formula is:

\[ D_{50} = 0.0122V^{0.6} \]  
(eq. TS14C-9)

where:
- \( D_{50} \) = median stone diameter (ft)
- \( V \) = average channel velocity (ft/s)

**U.S. Geological Survey method** (Blodgett 1981)
This technique is based on analysis of field data of 39 large events from sites in Arizona, Washington, Oregon, Nevada, and California. Riprap protection failed in 14 of the 39 cases. An envelope curve was empirically developed to represent the difference between sites that performed without damage and those that were damaged by particle erosion. The formula is:

\[ D_{50} = 0.01V^{2.44} \]  
(eq. TS14C-10)

where:
- \( D_{50} \) = median stone diameter (ft)
- \( V \) = average channel velocity (ft/s)

This method typically provides overly conservative results.

**Tillatoba model study**
This study (Blaisdell 1973) provides an equation for sizing stone to remain stable in the turbulent flow found below stilling basins. This high-energy technique results in an estimate for \( D_{50} \)

\[ D_{50} = 0.00116 \frac{V^3}{\sqrt{d}} \]  
(eq. TS14C-11)

where:
- \( V \) = velocity (ft/s)
- \( d \) = flow depth (ft)
- \( D_{50} \) = stone diameter (ft)

**USACE steep slope riprap design**
This high-energy technique is outlined in standard USACE guidance as provided in EM 1110-2-1601. It is designed for use on slopes from 2 to 20 percent. However, the side slopes should be 1V:2.511 or flatter. A typical application would be a rock-lined chute. The formula is:

\[ D_{50} = \frac{1.95S_0^{0.55}(C_q)^{2}}{g} \]  
(eq. TS14C-12)

where:
- \( D_{50} \) = stone size; m percent finer by weight
- \( S_0 \) = channel slope
- \( q \) = unit discharge (q = Q/b, where b = bottom width of chute and Q is total flow)
- \( C = \) flow concentration factor (usually 1.25, but can be higher if the approach is skewed)
- \( g \) = gravitational constant

This equation is applicable to thickness = 1.5 \( D_{100} \) angular rock, unit weight of 167 pounds per cubic foot, \( D_{85}/D_{15} \) from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope with no tailwater. This equation typically predicts conservative sizes.

**USACE habitat boulder design**
This technique is outlined in USACE guidance provided in EMRRP-SR-11. It is developed for sizing boulder clusters in a channel for habitat enhancement. This high-energy relationship is an incipient motion relation for fully immersed boulders in turbulent flow on a flat bed. This method is for impinging flow. The formula is:

\[ D = \frac{18(\text{depth})S_r}{(SG - 1)} \]  
(eq. TS14C-13)

where:
- \( D \) = minimum stone size
- \( \text{depth} \) = channel depth
- \( S_r \) = channel friction slope
- \( SG \) = specific gravity of the stone

This equation has also been used to size stones for use in low instream weirs. However, estimating the friction slope across a drop can be difficult.

**Abt and Johnson (1991)**
Abt and Johnson (1991) conducted near-prototype flume studies to determine riprap stability when subjected to overtopping flows such as in spillway flow or in sloping loose-rock grade control structures. Slopes varied from 2 to 20 percent. Riprap design criteria for overtopping flows were developed for two conditions: stone movement and riprap layer failure. Criteria were
developed as a function of median stone size, unit discharge, and embankment slope. The equation is:

\[ D_{50} = (q_{aveg})^{0.54} \times S^{0.45} \times 5.23 \]  
(eq. TS14C-14)

where:

- \( D_{50} \) = stone size in inches; m percent finer by weight
- \( q_{aveg} \) = unit discharge (ft³/s/ft)
- \( S \) = channel slope (ft/ft) and S between 0.02 and 0.20 ft/ft
- \( q_{aveg} = (q_{aveg})^{0.74} \times 1.35q_{aveg} \)  
(eq. TS14C-15)

Stone movement occurred at approximately 74 percent of the flow, causing layer failure. It was determined from testing that rounded stone should be oversized by approximately 40 percent to provide the same protection as angular stone.

**ARS rock chutes**

This design technique (Robinson, Rice, and Kadavy 1998) is primarily targeted at high-energy applications. Loose riprap with a 2 \( D_{50} \) blanket thickness composed of relatively uniform, angular riprap was tested to overtopping failure in models and field scale structures. This method applies to bed slopes of 40 percent and less. This technique can be used for low slope, and thus, low-energy applications, but it is particularly useful for slopes greater than 2 percent. A factor of safety appropriate for the project should be applied to the predicted rock size. The equations are:

for \( S < 0.1 \)

\[ D_{50} = 12(1.923qS^{2})^{0.529} \]  
(eq. TS14C-16)

0.10<\( S < 0.40 \)

\[ D_{50} = 12(0.233qS^{0.34})^{0.529} \]  
(eq. TS14C-17)

where:

- \( D_{50} \) = median stone size (in)
- \( q \) = highest stable unit discharge (ft³/s/ft)
- \( S \) = channel slope (ft/ft)

A spreadsheet program (Lorenz, Lobrecht, and Robinson 2000) is available to assist in sizing riprap on steep slopes. A screen capture of this spreadsheet program is shown in figure TS14C-7.

This method is best used in steep slopes for grade control, embankment overtopping, or on side inlets from fields to a major drainage outlet. The spreadsheet provides much additional information related to rock chutes such as guidance on inlet and outlet conditions, quantity estimates, and hydrology.

**California Department of Transportation RSP**

This technique was developed by the California Department of Transportation (CALTRANS) for designing rock slope protection (RSP) for streams and riverbanks. Unlike most of the other available techniques, it results in a recommended minimum weight of the stone. The equation is:

\[ W = 0.00002 \times \frac{VM \times V_{s}^{2}}{(G - 1)^{2}} \times \sin^{3}(r - \alpha) \]  
(eq. TS14C-18)

where:

- \( W \) = minimum rock weight (lb)
- \( V \) = velocity (ft/s)
- \( VM = 0.67 \) if parallel flow
- \( VM = 1.33 \) if impinging flow
- \( G = \) specific gravity of rock (typically 2.65)
- \( r = \) angle of repose (70° for randomly placed rock)
- \( \alpha = \) outside slope face angle to the horizontal (typically a maximum of 33°)

The weight indicated by this method should be used in conjunction with standard CALTRANS specifications and gradations.

**Far West states (FWS)—Lane’s Method**

Vito A. Vanoni worked with the Northwest E&WP Unit to develop the procedure from the ASCE paper entitled "Design of Stable Alluvial Channels" (Lane 1955a). The equation is:

\[ D_{50} = \frac{3.5}{C \times K} \times Y \times X \times D \times S_{f} \]  
(eq. TS14C-19)

where:

- \( D_{50} \) = stone size, (in)
- \( C = \) correction for channel curvature
- \( K = \) correction for side slope
- \( S_{f} = \) channel friction slope (ft/ft)
- \( d = \) depth of flow (ft)
- \( \gamma_{w} = \) density of water

This is generally considered to be a conservative technique. It assumed that the stress on the sides of the channel were 1.4 times that of the bottom. This
Rock Chute Design Data

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

Project: Spillway protection
County: Woodbury

Input Channel Geometry

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 20.0 ft.</td>
<td>Bw = 20.0 ft.</td>
<td>Bw = 40.0 ft.</td>
</tr>
<tr>
<td>Side slopes = 4.0 (m:1)</td>
<td>Factor of safety = 1.20 ( ( J ))</td>
<td>Side slopes = 4.0 (m:1)</td>
</tr>
<tr>
<td>n-value = 0.035</td>
<td>( J ) = 2.5:1 max.</td>
<td>n-value = 0.045</td>
</tr>
<tr>
<td>Bed slope = 0.0060 ft.</td>
<td>Bed slope (5:1) = 0.200 ft.</td>
<td>Bed slope = 0.0050 ft.</td>
</tr>
<tr>
<td>Freeboard = 0.5 ft.</td>
<td>Outlet apron depth, ( d = 1.0 ) ft.</td>
<td>Base flow = 0.0 ft.</td>
</tr>
</tbody>
</table>

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

Drainage area = 450.0 acres
Rainfall = 0.0 3.3 in. 0.0 in.

Chute capacity = Q5-year
Minimum capacity (based on a 5-year, 24-hour storm with a 3-5 inch rainfall)
Input tailwater (Tw):

Outlet Channel

Profile and Cross Section (Output)

Notes:
1) Output given as High Flow (Low Flow) values.
2) Tailwater depth must be at least 4 ft. downstream of the chute to function.
3) Critical depth occurs \( 2x - 4x \) upstream of crest.
4) Use 8 oz. non-woven geotextile under rock.

Profile Long Centerline of Chute

<table>
<thead>
<tr>
<th>Typical Cross Section</th>
<th>High Low Storm Information</th>
</tr>
</thead>
</table>

(210-VI-NEH, August 2007) TS14C-9
is about 1.8 times the actual stress on the sides of a straight channel. It is very close to the stresses on the sides in a curved channel reach. The curved corrections included in the procedure only make the conservative answer even more conservative. In addition, it was developed for stones with a specific gravity of 2.56. However, it has been successfully applied on many projects. This procedure may be used with figure TS14C-8 and is:

\[ \frac{D_{75}}{S_{cr}} = \frac{3.5}{C_{gr}} \times \gamma_w \times d \times S \]

**Notes**

1. Ratio of channel bottom width to depth (d) greater than 4
2. Specific gravity of rock not less than 2.56
3. Additional requirements for stable riprap include fairly well-graded rock, stable foundation, and minimum section thickness (normal to slope) not less than \(D_{75}\) at maximum water surface elevation and 3 \(D_{75}\) at the base.
4. Where a filter blanket is used, design filter material grading in accordance with criteria in NRCS Soil Mechanics Note I.

<table>
<thead>
<tr>
<th>(\text{Ro}/W_w)</th>
<th>(C)</th>
<th>Side slope</th>
<th>(K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-6</td>
<td>0.6</td>
<td>1-1/2H:1V</td>
<td>0.52</td>
</tr>
<tr>
<td>6-9</td>
<td>0.75</td>
<td>1-3/4H:1V</td>
<td>0.63</td>
</tr>
<tr>
<td>9-12</td>
<td>0.90</td>
<td>2H:1V</td>
<td>0.72</td>
</tr>
<tr>
<td>straight channel</td>
<td>1.0</td>
<td>2-1/2H:1V</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3H:1V</td>
<td>0.87</td>
</tr>
</tbody>
</table>

\(R_c = \text{Curve radius}\)
\(W_w = \text{Water surface width}\)
\(S = \text{Energy slope or channel grade}\)
\(w = 62.4\)

**Figure TS14C-8** Lane's method

**Step 1** Enter figure TS14C-8 with energy slope (channel grade) and flow depth.

**Step 2** Track right to side slope.

**Step 3** Track up to ratio of curve radius to water surface width.

**Step 4** Track right to estimate required riprap size.
U.S. Department of Transportation Federal Highway Administration techniques

Several additional computational techniques for designing riprap are available from the U.S. Department of Transportation Federal Highway Administration (FHWA). While these are not described in detail, a brief description of each is provided in table TS14C-2.

Review the references (FHWA HEC 1987, 1988, 2001a, 2001b) to obtain the design relationships and application manuals for these methods.

Summary guide of selected techniques

Attributes of selected methods are summarized in table TS14C-3 to allow the user to quickly select a method.

The designer should not be surprised if the different techniques produce different answers. The user needs to recognize the limits and applicability of each technique and match it to the site and project conditions.

Table TS14C-2 Federal Highway Administration techniques

<table>
<thead>
<tr>
<th>Technique</th>
<th>High or low energy</th>
<th>Slopes</th>
<th>Typical application(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HEC-11</td>
<td>Both</td>
<td>Not specified</td>
<td>Rock revetment, stilling basins, river closures</td>
</tr>
<tr>
<td>HEC-15</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>HEC-18</td>
<td>High</td>
<td>2% to 40%</td>
<td>Overtopping, rock chutes, grade protection</td>
</tr>
</tbody>
</table>

Table TS14C-3 Summary of techniques

<table>
<thead>
<tr>
<th>Technique</th>
<th>High or low energy</th>
<th>Slopes</th>
<th>Typical application(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>Both</td>
<td>Not specified</td>
<td>Rock revetment, stilling basins, river closures</td>
</tr>
<tr>
<td>108 Report</td>
<td>Both</td>
<td>&lt;10%</td>
<td>Quick assessments for stable stone requirements</td>
</tr>
<tr>
<td>Maynord</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>High</td>
<td>2% to 20%</td>
<td>Overtopping, grade protection</td>
</tr>
<tr>
<td>ARS - rock chute</td>
<td>High</td>
<td>2% to 40%</td>
<td>Overtopping, rock chutes, grade protection</td>
</tr>
<tr>
<td>USBR</td>
<td>High</td>
<td>Not specified</td>
<td>Riprap below a stilling basin</td>
</tr>
<tr>
<td>USGS Blodgett</td>
<td>Both</td>
<td>Not specified</td>
<td>Riprap stability</td>
</tr>
<tr>
<td>USACE Steep Slope Riprap</td>
<td>High</td>
<td>2% to 20%</td>
<td>Rock chutes, grade protection</td>
</tr>
<tr>
<td>USACE Habitat Boulder</td>
<td>High</td>
<td>Not specified</td>
<td>Instream boulders for habitat enhancement</td>
</tr>
<tr>
<td>CALTRANS RSP</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Lane’s (FWS)</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Stone bank protection, stream barbs with adjustments</td>
</tr>
</tbody>
</table>

(210-VI-NEH, August 2007)
Factor of safety

Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. A factor of safety is often advisable to account for unknowns and uncertainty. In some cases, the factor of safety is part of the sizing formulas provided. Where a factor of safety is not built into the procedure, the designer should multiply the resulting size by an appropriate value. Appropriate engineering judgment should be applied when assigning a factor of safety. Maynard (1992) suggests a minimum factor of safety of 1.1. Typically, a factor of safety will range from 1.1 to 1.5. The risk and uncertainty associated with a project should be reflected in the factor of safety.

Example calculations

Example calculations are presented for selected methods to illustrate the variability associated with rock sizing methods. The examples may also provide a new user with confirmation that they are correctly applying a method.

Example problem: Mild slope

Problem: For the following flow conditions, determine the required rock size for stone toe protection.

\[ G_s = 2.65 \text{ or } \gamma_s = 165.36 \text{ lb/ft}^3 \]

Width = 40 ft

\[ n = 0.045 \]

Slope = 0.01 ft/ft

Depth = 6 ft

Solution: Solve relevant hydraulic parameters

\[ V_1 = 9.1 \text{ ft/s} \]

\[ Q = 2,200 \text{ ft}^3/\text{s} \]

\[ Y_{er} = 4.54 \text{ ft} \]

The riprap size determined from several methods is:

<table>
<thead>
<tr>
<th>Method</th>
<th>( D_{50} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>6.5 in</td>
</tr>
<tr>
<td>Maynard</td>
<td>4.6 in, 5.5 in</td>
</tr>
<tr>
<td>Lane’s (FWS)</td>
<td>15 in, 12.7 in</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>8.1 in</td>
</tr>
<tr>
<td>ARS rock chute</td>
<td>3.6 in</td>
</tr>
</tbody>
</table>

Discussion: The computed critical depth indicates that this is a subcritical flow. The design calls for a revetment-type protection, so the stones are not projecting into the flow. Therefore, this is a low-energy flow condition. The Isbash (1936) and the Maynard (1992) methods both indicate a \( D_{50} \) of about 5.5 to 6.5 inches. These methods were developed for conditions that are similar to those in the problem statement. Therefore, a stone size of 6 inches with an appropriate factor of safety should be acceptable.

Lane’s (1955a) FWS method provides a conservative estimate of 12.7 inches. While this technique is used in similar situations, a conservative answer is expected. The Abt and Johnson (1991) method and the ARS method (Robinson, Rice, and Kadavy 1998) were developed for steeper high-energy flow conditions (>2); therefore, use of these methods would not be advisable for this application.

Example problem: Steep slope

Problem: For the following flow conditions, determine the required rock size for a rock chute.

\[ G_s = 2.65 \text{ or } \gamma_s = 165.36 \text{ lb/ft}^3 \]

Width = 40 ft

\[ n = 0.045 \]

Slope = 0.06 ft/ft

Depth = 3.5 ft

Solution: Solve relevant hydraulic parameters

\[ V_1 = 16.7 \text{ ft/s} \]

\[ Q = 2,340 \text{ ft}^3/\text{s} \]

\[ Y_{er} = 4.7 \text{ ft} \]

The riprap size determined from several methods is:

<table>
<thead>
<tr>
<th>Method</th>
<th>( D_{50} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>1.6 ft</td>
</tr>
<tr>
<td>Maynard</td>
<td>1.6 ft, 1.9 ft</td>
</tr>
<tr>
<td>Lane’s (FWS)</td>
<td>3.7 ft, 3.2 ft</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>1.3 ft</td>
</tr>
<tr>
<td>ARS rock chute</td>
<td>1.1 ft</td>
</tr>
</tbody>
</table>

Discussion: The computed critical depth indicates that this is a supercritical flow. While similar in prediction, the Isbash and the Maynard (1992) methods were not developed for conditions that are described in the problem statement. The Abt and Johnson (1991), as well as the ARS rock chute methods (Robinson, Rice, and Kadavy 1998), were derived for similar conditions to the problem statement. Therefore, the 1.1 to 1.3 foot \( D_{50} \) riprap with an appropriate factor of safety should be acceptable.
Conclusion

Rock is often used where long-term durability is needed, velocities are high, periods of inundation are long, and there is a significant threat to life and property. Whether a streambank project involves the use of rock as part of a stand-alone treatment or as a component of an integrated system, the determination of the required stone size requires engineering analysis. Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. Since stone sizing methods are normally developed for a specific application, care should be exercised matching the selected method with the project purpose and site condition. Therefore, the intended application should dictate which rock sizing technique is used. By using several methods, the designer will often see a convergence of rock sizes for a given application.
Attachment 3
Subset of Technical Supplement 14C
Cover photo: Stone may be needed as a foundation on which to implement other restoration features such as soil bioengineering practices. Stone may also be needed to form an erosion resistant layer. How large, how thick, and how deeply keyed-in are questions that are addressed in the design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
developed as a function of median stone size, unit discharge, and embankment slope. The equation is:

\[ D_{50} = (q_{\text{design}})^{0.57} \times S^{0.41} \times 5.23 \]  

(eq. TS14C-14)

where:
- \( D_{50} \) = stone size in inches; \( m \) percent finer by weight
- \( q_{\text{design}} \) = unit discharge (ft\(^3\)/s/ft)
- \( S \) = channel slope (ft/ft) and \( S \) between 0.02 and 0.20 ft/ft

\[ \left( q_{\text{design}} \right) = \left( \frac{q_{\text{inflow}}}{0.74} \right) \]  

(eq. TS14C-15)

Stone movement occurred at approximately 74 percent of the flow, causing layer failure. It was determined from testing that rounded stone should be oversized by approximately 40 percent to provide the same protection as angular stone.

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For \( S < 0.1 \)

\[ D_{50} = 12 \left( 1.923q^{0.1} S^{0.72} \right) \]  

(eq. TS14C-16)

For \( 0.10 < S < 0.40 \)

\[ D_{50} = 12 \left( 0.233q^{0.58} S^{0.70} \right) \]  

(eq. TS14C-17)

where:
- \( D_{50} \) = median stone size (in)
- \( q \) = highest stable unit discharge (ft\(^3\)/s/ft)
- \( S \) = channel slope (ft/ft)

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\[ W = \frac{0.00002 \times \frac{V}{S}}{\left( \frac{S_o}{1.3} \right)^3} \times \frac{V^e \times G_s}{\sin^{1/2}(r - a)} \]  

(eq. TS14C-18)

where:
- \( W \) = minimum rock weight (lb)
- \( V \) = velocity (ft/s)
- \( V^e \) = specific gravity of rock (typically 2.65)
- \( r \) = angle of repose (70° for randomly placed rock)
- \( a \) = outside slope face angle to the horizontal (typically a maximum of 33°)

The weight indicated by this method should be used in conjunction with standard CALTRANS specifications and gradations.

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Vito A. Vanoni worked with the Northwest E&WP Unit to develop the procedure from the ASCE paper entitled “Design of Stable Alluvial Channels” (Lane 1955a). The equation is:

\[ D_{75} = \frac{3.5 \times C \times K \times \gamma \times D \times S_t}{\gamma_h} \]  

(eq. TS14C-19)

where:
- \( D_{75} \) = stone size, (in)
- \( C \) = correction for channel curvature
- \( K \) = correction for side slope
- \( S_t \) = channel friction slope (ft/ft)
- \( \gamma \) = depth of flow (ft)
- \( \gamma_h \) = density of water

This is generally considered to be a conservative technique. It assumed that the stress on the sides of the channel were 1.4 times that of the bottom. This
Rock Chute Design Data

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

Project: Spillway protection
Designer: Jim Villa
Checked by: 
Date: 3/30/2006

Input Channel Geometry

<table>
<thead>
<tr>
<th>Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side slopes</td>
<td>Side slopes</td>
<td>Side slopes</td>
</tr>
<tr>
<td>Factor of safety</td>
<td>n-value</td>
<td>n-value</td>
</tr>
<tr>
<td>Bed slope</td>
<td>Bed slope</td>
<td>Bed slope</td>
</tr>
<tr>
<td>Freeboard</td>
<td>Outlet apron depth</td>
<td>Base flow</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw = 20.0 ft.</td>
<td>Bw = 40.0 ft.</td>
<td></td>
</tr>
<tr>
<td>n-value</td>
<td>n-value</td>
<td></td>
</tr>
<tr>
<td>Bed slope</td>
<td>Bed slope</td>
<td></td>
</tr>
<tr>
<td>Freeboard</td>
<td>Outlet apron depth</td>
<td></td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Drainage area</th>
<th>Rainfall</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>450.0 acres</td>
<td>0.0-3 in.</td>
<td>The total required capacity is routed through the chutes (principal spillway) or in combination with an auxiliary spillway.</td>
</tr>
</tbody>
</table>

Profile and Cross Section (Output)

**Profile Along Centerline of Chute**

1. **Use** $H_2$ **along chute but not less than** $z_2$.

2. **Note:** When the normal depth ($y_e$) in the inlet channel is less than the weir head ($H_d$), 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.

**Typical Cross Section**

**High Flow Storm Information**

(210-VI-NEH, August 2007)
Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. A factor of safety is often advisable to account for unknowns and uncertainty. In some cases, the factor of safety is part of the sizing formulas provided. Where a factor of safety is not built into the procedure, the designer should multiply the resulting size by an appropriate value. Appropriate engineering judgment should be applied when assigning a factor of safety. Maynard (1992) suggests a minimum factor of safety of 1.1. Typically, a factor of safety will range from 1.1 to 1.5. The risk and uncertainty associated with a project should be reflected in the factor of safety.
Attachment 4
Tetra Tech Technical Memorandum
Rosemont Flow-Through Drain Sedimentation Analysis
August 31, 2010
1.0 Introduction

The design and sizing of the flow-through drains planned for the Rosemont Copper Project (Project) in Pima County, Arizona were highlighted in two (2) technical memoranda presented in the Site Water Management Update report (Tetra Tech, 2010a).

- Rosemont Flow-Through Drain Design (Tetra Tech, 2010b); and

These technical memoranda and others in the Site Water Management Update report provide the basis for the hydraulic calculations performed to size the drains. The flow-through drains are large structures, extending laterally 100 to 200 feet across depending on the location within the flow-through drain network. The drains were designed to handle either the General or Local Probable Maximum Precipitation (PMP) events.

This Technical Memorandum provides estimates of sediment loading to the drain inlet areas and also expands on the control of these sediments at the inlet areas from that provided in the Site Water Management Update report (Tetra Tech, 2010a). Sediment loading to the drain inlets was estimated using the current version of the software package SEDCAD (Sediment, Erosion, Discharge by Computer Aided Design), which is a comprehensive sedimentology program that incorporates standard hydrology and hydraulic principles. This program is used to analyze the design and efficiency of alternative surface water, erosion, and sediment control systems.

2.0 Flow-Through Drain Overview

Flow-through drains are large rock structures designed to preserve the stability of the proposed Rosemont Ridge Landform and to provide the following functions:

- A hydraulic connection between the up-gradient side of the Rosemont Ridge Landform and the down-gradient side;
- Protection of facilities during operational periods; and
Separation between the wash areas and the dry stack tailings with waste rock.

As indicated above, the design of the flow-through drains were detailed in the Technical Memorandum titled *Rosemont Flow-Through Drain Design* (Tetra Tech, 2010b) as part of the *Site Water Management Update* report (Tetra Tech, 2010a). Sizing of the flow-through drains was dependent on the contributing area reporting to the drain inlet. The flow-through drains were generally sized to accommodate the maximum contributing watershed reporting to the drain inlet, thus reflecting the worst case scenario that would be encountered during operations and/or post-closure conditions. The drain systems were also generally designed to safely convey the Local or General PMP events.

The analysis and sizing of the flow-through drains assumed that ponding would occur at the upstream end of the drains. This allowed modeling of the system as a detention reservoir routing system with the ponding area acting as the reservoir, the basin hydrograph controlling inflow to the reservoir, and the drain acting as the outlet.

An inflow hydrograph for each of the reservoir/pond locations was developed using the largest possible contributing basin determined from preliminary waste rock placement/sequencing plans (prepared by Moose Mountain) and the base concept of the Rosemont Ridge Landform. Elevation-Area storage functions (stage-storage curves), which were based on existing ground topography, the proposed waste rock and tailings surfaces, and the facility embankments, were determined for each of the ponding areas located at the flow-through drain inlets.

With the flow-through drain serving as an outlet, stormwater flow passing through the drain increases as the stage or the water surface elevation (WSE) increases. Discharge rating curves (Elevation-Discharge functions) between the WSE in the reservoir and outflow were also developed. The sizing and configurations of the flow-through drains are detailed in the Technical Memorandum titled *Rosemont Flow-Through Drain Sizing* (Tetra Tech, 2010c) referenced above as part of the *Site Water Management Update* report (Tetra Tech, 2010a).

Prior to the placement of waste rock or tailings over the flow-through drains, these structures will be covered with a non-woven, 10 ounce (oz) geotextile, which will act as a separation barrier and provide a filtering mechanism to prevent sediments from entering into the drains from above and from the sides. The rock drain inlets will typically be extended approximately 30 feet beyond the toe of the Rosemont Ridge Landform. A filter geotextile will be placed at these drain inlet areas and covered with about a ten (10) foot thick layer of rock to prevent sedimentation of the flow-through drain inlets. This typical configuration (Illustration 1) will allow for maintenance of the drain inlets, as needed. The filter geotextile at the drain inlets is only anticipated during the operational period and for a limited post-closure period. As appropriate, the same geotextile protection would be employed at the Detention Basin embankments designed by AMEC (see report titled *Dry Stack Tailings Storage Facility Stormwater Management Design Report* (AMEC, 2010) in Appendix A of the *Site Water Management Update* report (Tetra Tech, 2010a).
For post-closure conditions, this drain inlet configuration could be modified by removal of the filter geotextile and possible replacement of the rock layer by a graded rock filter. Modifications to the inlets would be dependent upon field conditions. A graded rock filter design would allow the flow-through drains to be self-cleaning by allowing sediments to settle out and deposit at this interface as surface water transitions from turbulent flow at the inlet to laminar flow through the graded rock filter and into the drain.

The watershed surfaces reporting to the drain inlets are expected to stabilize over time with a corresponding reduction in sediment loading. Sediment loading would be also mitigated with the addition of Best Management Practices (BMPs) throughout the Project area and especially upgradient of the drain inlets.

3.0 Sedimentology Methodology Overview

SEDCAD is a comprehensive sedimentology program that incorporates standard hydrology and hydraulic principles and is used to analyze the design and efficiency of alternative surface water, erosion and sediment control systems utilized for projects involving earth-disturbing activities such as surface mining. For the purposes of this Technical Memorandum, only the contributing watershed basins and the corresponding ponds utilized for the sizing of the flow-through drains were investigated for this sedimentology analysis.

SEDCAD was employed in this analysis to model the contributing watersheds and their ponds as sediment basins in order to conservatively predict and quantify the potential sedimentation at the inlet area of the various drains. The following paragraphs describe the hydrology and sedimentology inputs required to create a complete model run for each contributing watershed basin and inlet pond. Attachment 1 contains all of the input data for the watershed basins considered in the analysis.

3.1 Design Storm and Distribution Information

Precipitation data for the design storms utilized herein was acquired from the National Oceanic and Atmospheric Association (NOAA) Atlas 14 Point Precipitation website (NOAA, 2009). The
coordinates used to obtain precipitation data for the Project site were 31.862 N 110.692 W, at an elevation of 4,429 feet above mean sea level (amsl).

The Natural Resource Conservation Service (NRCS) has developed synthetic hyetographs for the entire U.S., for 6-hour and 24-hour storm events, called "type curves". The U.S. is divided into four (4) regions where specific "type curves" can be applied. The widely used 24-Hour NRCS Type II Distribution was analyzed herein for SEDCAD’s hydrologic model.

Based on the NRCS Type Curve Map, Arizona falls within the Type II region, which is characterized by high-intensity storms such as those experienced in Pima County. A Type II storm represents one of the more intense storm patterns defined by NRCS. Table 1 summarizes the design storms that were considered for the analysis.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>NRCS Storms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Period (yrs)</td>
<td>2 5 10 25 50 100</td>
</tr>
<tr>
<td>Duration (hrs)</td>
<td>24 24 24 24 24 24</td>
</tr>
<tr>
<td>Precipitation (in)</td>
<td>2.21 2.75 3.18 3.77 4.23 4.75</td>
</tr>
<tr>
<td>Distribution</td>
<td>NRCS Type II</td>
</tr>
</tbody>
</table>

3.2 Structure Networking

The structure networking within the program was simplified for this analysis to include each pond acting as a sediment basin control structure with its corresponding contributing basin. The Elevation-Area storage functions, Elevation-Discharge functions, and worst-case scenario watershed basin models were entered into SEDCAD to simulate the ponding effect for each pond studied in this analysis.

3.3 Hydrograph

Similar to the hydrologic analysis performed in the Technical Memorandum titled Rosemont Flow-Through Drain Sizing (Tetra Tech, 2010c), an inflow hydrograph was developed for each design storm for the associated watershed basin and with the following input parameters:

- The basin area, in acres;
- The time of concentration, in hours;
- The NRCS curve number (CN); and
- The unit hydrograph response shape.
With the exception of the unit hydrograph response shape, the other inputs of basin area, time of concentration, and curve number were derived for each of the basins from the HEC-HMS hydrologic analysis previously defined in the Technical Memorandum titled Rosemont Flow-Through Drain Sizing (Tetra Tech, 2010c). Attachment 1 provides the input values to the SEDCAD program. An important note is that the time of concentration, Tc, is related to lag time, Lg, by the expression \( Tc = 1.67 \times Lg \).

The TR-55 emulator within the SEDCAD program was conservatively selected as the unit hydrograph response shape. The United States Department of Agriculture (USDA) developed Technical Release 55 (TR-55) for urban hydrology of small watersheds. TR-55 is based upon the same NRCS principles and methodologies utilized in the HEC-HMS hydrologic analysis for sizing the flow-through drains, and therefore produces similar and comparable results for peak flows.

### 3.4 Sedimentgraph

Similar to the hydrograph, a sedimentgraph can be developed for a watershed basin with the following input parameters:

- The eroded particle size distribution (EPSD);
- The soil erodibility factor, K;
- The soil cover type factor, C;
- The soil control practices factor, P;
- The representative overland flow slope length, in feet; and
- The representative overland flow slope, in %.

The eroded particle size distribution (EPSD) was estimated from the particle size distribution of a common soil type within the Project area referred to as the Gila Conglomerate. In ephemeral channels of the Southwestern United States, sediment often moves in a step-wise manner because of transmission losses (Renard 1975). Water from storms originating in the upper reaches of a watershed is often completely absorbed in the channel before reaching the outlet. Therefore, it was a conservative assumption to consider that a typical extreme monsoon storm would transport particles finer than the #4 sieve number (4.75 mm). The Gila Conglomerate particle size distribution data was modified to mimic an expected EPSD for its universal use in the SEDCAD model. This estimated EPSD assumes that 100 percent of the Gila Conglomerate soil material finer than the #4 sieve (measured from the laboratory sieve analysis) will be effectively eroded. It was also necessary to classify sediment material into a group suitable for application in the SEDCAD calculations. Therefore, the particle weight percentage assigned to the sieve #4 sieve was assumed to be 100% (see Attachment 1).

The erodibility factor, K, was conservatively selected for the common soil type in the Project area, the Gila Conglomerate (K=0.24 for fine sandy loam, loamy very fine sand, sandy loam). The soil cover type factor, C, which accounts for canopy, surface cover, and surface roughness, was also conservatively selected as having no appreciable canopy for grass with 40 percent ground cover anticipated during post-closure conditions (C = 0.10) and for grass with zero
percent ground cover anticipated during operations \((C = 0.45)\). Similarly, the soil control practices factor, \(P\), which substantiates the utilization of sediment control practices, was set equal to 1.0 to conservatively account for no use of sediment control practices (see Attachment 1).

The representative slope length, and slope for the probable overland flow of surface water into a more concentrated flow regime, was approximated for each basin based upon the existing ground topography, the proposed waste rock and tailings surfaces, and the facility embankments (see Attachment 1).

4.0 Sedimentology Results

The results of the sediment transport analysis to all of the flow-through drain inlets are provided in Attachment 1. Table 2 below presents a summary of the typical results for Pond S1A that will be referenced for purposes of discussion.

<table>
<thead>
<tr>
<th>POND ID</th>
<th>SEDCAD Results: NRCS Type II, 241 pt. Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24-hr Design Storm (yrs) 2 5 8 10 25 50 100</td>
</tr>
<tr>
<td>Annual Event Probability</td>
<td>0.50 0.20 0.13 0.10 0.04 0.02 0.01</td>
</tr>
<tr>
<td>Peak Flow (cfs)</td>
<td>279 404 466 507 650 764 893</td>
</tr>
<tr>
<td>Post-Closure Sediment (C = 0.10)</td>
<td>535 805 944 1,037 1,374 1,648 1,967</td>
</tr>
<tr>
<td>(tons)</td>
<td>0.26 0.40 0.47 0.51 0.68 0.81 0.97</td>
</tr>
<tr>
<td>(ac-ft)</td>
<td>0.13 0.08 0.06 0.05 0.03 0.02 0.01</td>
</tr>
<tr>
<td>Operations Sediment (C = 0.45)</td>
<td>2,406 3,624 4,250 4,668 6,183 7,416 8,853</td>
</tr>
<tr>
<td>(tons)</td>
<td>1.19 1.79 2.10 2.30 3.05 3.66 4.37</td>
</tr>
<tr>
<td>(ac-ft)</td>
<td>0.59 0.36 0.28 0.23 0.12 0.07 0.04</td>
</tr>
</tbody>
</table>

SEDCAD produces sediment output in tons of material and volume in acre-feet (ac-ft). The volume of this sediment was conservatively calculated utilizing the dry specific weight of sand sediment deposits (with no expected consolidation over time) of 93 pounds per cubic foot (pcf) or 2025.5 tons/ac-ft. The related annual sediment yield contribution, which is presented as ac-ft/year above in Table 2, was calculated for post-closure and operations conditions by multiplying the respective 24-hr design storm event's estimated sediment volume by its annual probability of occurrence.

It is important to note that the basin peak flow into Pond S1A, from a 100-year, 24-hour event, results in a peak flow of 893 cubic feet per second (cfs) from the SEDCAD analysis and 946 cfs using the HEC-HMS analysis. This is also true for all the basins studied herein when comparing the SEDCAD results in Attachment 1 to the HEC-HMS results highlighted in the Technical Memorandum titled *Rosemont Flow-Through Drain Sizing* (Tetra Tech, 2010c).
The peak flows and the post-closure and operations sedimentation values for the 8-year, 24-hour design storm event was conservatively estimated by linear interpolation between the 5-year and 10-year events in order to ensure that the sum of the respective annual event probabilities of occurrence was equal to 1.0, which was utilized for the sedimentology analysis described in the next section.

5.0 Sedimentology Analysis

The sedimentology results presented in Section 4.0, and as part of Attachment 1, were utilized along with Elevation-Area storage functions (stage-storage curves) for each flow-through inlet ponding area to analyze their specific responses to sediment loading during operations and post-closure conditions. Table 3 below presents a summary of the sedimentology analysis for all of the inlet ponding areas.

Stage-storage curves were developed for each of the flow-through drain inlet ponds based upon existing ground topography, the proposed waste rock and tailings surfaces, and the facility embankments, etc. These curves were used to determine the partial pond storage capacity in order to determine the design height of the drain inlet as well as the total pond storage capacity to the crest of each pond (see Table 3). The annual sediment yield or rate, measured in ac-ft/yr for each pond, was estimated for post-closure conditions and for the operational period by adding all of the respective pond's annual sediment yield contributions for each 24-hour design storm event (see Table 2 for Pond S1A and Attachment 1 for all ponds).

Also considered in this analysis was the average-annual sediment yield for the Project area using the Pacific Southwest Inter-Agency Committee (PSIAC) Method as estimated at 1.15 ac-ft per square mile per year (ac-ft/m²/yr) or 1.80E-3 ac-ft/ac/yr in the Technical Memorandum titled Rosemont Baseline and Post-Closure Conditions - Alternatives Sediment Delivery (Tetra Tech, 2010d). This average-annual sediment yield was multiplied by the various basin areas to determine the estimated annual sediment yield into the respective ponds based upon the PSIAC Method. The annual sediment yields estimated for post-closure and operations conditions using SEDCAD compared with the annual sediment yield referenced from the PSIAC Method.

The time for each flow-through drain inlet area to reach its respective partial pond sediment capacity was then estimated (see Table 3). For example, it is estimated to take about six (6) years for pond S1B to fill the partial storage capacity with sediments during operational conditions. This illustrates the effective use of catchments to control sediments prior to reaching the drain inlets.
Table 3  Summary of Pond Sedimentology Analysis

<table>
<thead>
<tr>
<th>POND ID</th>
<th>Flow-Through Drain Height</th>
<th>Partial Pond Storage Capacity to Drain Height</th>
<th>Total Pond Storage Capacity to Crest</th>
<th>Post-Closure Sediment Yield</th>
<th>Operations Sediment Yield</th>
<th>PSIAC Ave. Sediment Yield</th>
<th>Partial Pond Sediment Capacity (Post-Closure)</th>
<th>Partial Pond Sediment Capacity (Operations)</th>
<th>Partial Pond Sediment Capacity (PSIAC Ave.)</th>
<th>Total Pond Sediment Capacity (PSIAC Ave.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>(ac-ft)</td>
<td>(ac-ft)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
<td>(ac-ft/yr)</td>
</tr>
<tr>
<td>S1A</td>
<td>20</td>
<td>31.9</td>
<td>556</td>
<td>0.38</td>
<td>1.70</td>
<td>0.44</td>
<td>84</td>
<td>19</td>
<td>73</td>
<td>1,264</td>
</tr>
<tr>
<td>S1B</td>
<td>20</td>
<td>29.6</td>
<td>223</td>
<td>1.09</td>
<td>4.91</td>
<td>0.56</td>
<td>27</td>
<td>6</td>
<td>53</td>
<td>400</td>
</tr>
<tr>
<td>S1C</td>
<td>20</td>
<td>20.5</td>
<td>247</td>
<td>0.08</td>
<td>0.38</td>
<td>0.10</td>
<td>245</td>
<td>54</td>
<td>205</td>
<td>2,467</td>
</tr>
<tr>
<td>S1D</td>
<td>20</td>
<td>103.9</td>
<td>397</td>
<td>1.70</td>
<td>7.64</td>
<td>1.05</td>
<td>61</td>
<td>14</td>
<td>99</td>
<td>379</td>
</tr>
<tr>
<td>S2A</td>
<td>30</td>
<td>62.8</td>
<td>492</td>
<td>0.08</td>
<td>0.37</td>
<td>0.38</td>
<td>767</td>
<td>171</td>
<td>167</td>
<td>1,307</td>
</tr>
<tr>
<td>S3A</td>
<td>20</td>
<td>89.0</td>
<td>394</td>
<td>0.05</td>
<td>0.23</td>
<td>0.20</td>
<td>1,711</td>
<td>381</td>
<td>451</td>
<td>1,999</td>
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<tr>
<td>S3B</td>
<td>20</td>
<td>116.3</td>
<td>795</td>
<td>0.23</td>
<td>1.01</td>
<td>0.60</td>
<td>516</td>
<td>115</td>
<td>192</td>
<td>1,315</td>
</tr>
<tr>
<td>S3C</td>
<td>20</td>
<td>91.0</td>
<td>1,636</td>
<td>0.51</td>
<td>2.29</td>
<td>1.47</td>
<td>179</td>
<td>40</td>
<td>62</td>
<td>1,114</td>
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<tr>
<td>SM1</td>
<td>30</td>
<td>274.9</td>
<td>4,965</td>
<td>0.49</td>
<td>2.19</td>
<td>1.37</td>
<td>565</td>
<td>126</td>
<td>201</td>
<td>3,621</td>
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<tr>
<td>N1A</td>
<td>30</td>
<td>105.5</td>
<td>188</td>
<td>0.17</td>
<td>0.78</td>
<td>0.35</td>
<td>612</td>
<td>136</td>
<td>302</td>
<td>540</td>
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<tr>
<td>N1B</td>
<td>30</td>
<td>62.7</td>
<td>311</td>
<td>0.70</td>
<td>3.15</td>
<td>0.99</td>
<td>90</td>
<td>20</td>
<td>64</td>
<td>315</td>
</tr>
<tr>
<td>N2</td>
<td>23</td>
<td>8.9</td>
<td>331</td>
<td>0.25</td>
<td>1.13</td>
<td>0.35</td>
<td>35</td>
<td>8</td>
<td>25</td>
<td>948</td>
</tr>
<tr>
<td>N3</td>
<td>23</td>
<td>10.0</td>
<td>301</td>
<td>0.17</td>
<td>0.76</td>
<td>0.09</td>
<td>59</td>
<td>13</td>
<td>117</td>
<td>3,513</td>
</tr>
<tr>
<td>NM1</td>
<td>38</td>
<td>198.2</td>
<td>2,566</td>
<td>0.28</td>
<td>1.26</td>
<td>0.96</td>
<td>710</td>
<td>158</td>
<td>206</td>
<td>2,668</td>
</tr>
</tbody>
</table>
6.0 Conclusion

The results of the sediment loading calculations show that the partial pond storage capacities ahead of the flow-through drain inlets provide sufficient sediment storage capacity to allow for periodic cleaning and maintenance of these areas during operational conditions. Placement of a filter geotextile at the drain inlet will help prevent sediments filtering into the drains. The flow-through drains will be extended beyond the toe of the Rosemont Ridge Landform, thus facilitating maintenance of the filter geotextile, as needed.

For most post-closure conditions, the drain inlet configuration may be modified by the removal of the filter geotextile and protective rock cover. Depending on the post-closure conditions, a graded rock filter may be constructed at the drain inlets. A graded rock filter design would allow the flow-through drains to be self-cleaning by allowing sediments to settle out and deposit at this interface as surface water transitions from turbulent flow at the inlet to laminar flow through the graded rock filter and into the drain.

The watershed surfaces reporting to the drain inlets are expected to stabilize over time with a corresponding reduction in sediment loading. BMPs could also be employed up-gradient of the drain inlets to control stormwater flow velocities, etc. BMP options include, but are not limited to, features such as additional sediment basins/sediment traps and check dams.
7.0 REFERENCES


ATTACHMENT 1
SED CAD ANALYSIS AND RESULTS
## SEDCAD - Watershed Basin and Pond Sedimentology Analysis for the Rosemont Project Area

**Gila Conglomerate Soil Data**

<table>
<thead>
<tr>
<th>Sieve Number</th>
<th>Sieve Size (mm)</th>
<th>ASTM D 6913</th>
<th>100% Effective Erosion past 3/8&quot;</th>
<th>100% Effective Erosion past #4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>% Finer by Wt.</td>
<td>% Finer by Wt.</td>
<td>% Finer by Wt.</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>19.05</td>
<td>97.1</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.525</td>
<td>88.2</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>#4</td>
<td>4.750</td>
<td>77.5</td>
<td>87.9</td>
<td>100</td>
</tr>
<tr>
<td>#10</td>
<td>2.000</td>
<td>55.4</td>
<td>62.8</td>
<td>71.5</td>
</tr>
<tr>
<td>#20</td>
<td>1.000</td>
<td>43.8</td>
<td>49.7</td>
<td>56.5</td>
</tr>
<tr>
<td>#40</td>
<td>0.445</td>
<td>34.2</td>
<td>38.8</td>
<td>44.1</td>
</tr>
<tr>
<td>#60</td>
<td>0.250</td>
<td>29.8</td>
<td>33.8</td>
<td>38.5</td>
</tr>
<tr>
<td>#100</td>
<td>0.150</td>
<td>25.1</td>
<td>28.6</td>
<td>32.4</td>
</tr>
<tr>
<td>#140</td>
<td>0.106</td>
<td>22.7</td>
<td>25.7</td>
<td>29.3</td>
</tr>
<tr>
<td>#200</td>
<td>0.075</td>
<td>19.7</td>
<td>22.3</td>
<td>25.4</td>
</tr>
</tbody>
</table>

**Precipitation Design Storm Events**

- 2-yr, 24-hr = 2.21 in
- 5-yr, 24-hr = 2.75 in
- 10-yr, 24-hr = 3.18 in
- 25-yr, 24-hr = 3.77 in
- 50-yr, 24-hr = 4.23 in
- 100-yr, 24-hr = 4.75 in

**Hydrology Calculations**

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<th>BASIN ID (1)</th>
<th>BASIN AREA (acres)</th>
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<th>Longest Course (ft)</th>
<th>Ave. Slope</th>
<th>Tc</th>
<th>Sedimentology Factors</th>
<th>Representative Slope Length (ft)</th>
<th>Representative Slope Size (%)</th>
<th>Eroded Particle Size Distribution (EPSD)</th>
<th>100% Effective Erosion past #4</th>
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**AVE:**
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<th>Drain Height (ft)</th>
<th>Partial Pond Storage Capacity (ac-ft (pr))</th>
<th>Total Pond Storage Capacity (ac-ft (pr))</th>
<th>Post-Closure Storage Yield (ac-ft/yr)</th>
<th>Operations</th>
<th>PSDC Average Sediment Yield (ac-ft/yr)</th>
<th>Partial Pond Sediment Capacity (ac (Ha))</th>
<th>Partial Pond Sediment Capacity (ac (Ha))</th>
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4. The C factor, accounting for 3. The erodibility factor, K, was conservatively selected for fine sandy loam, loamy very fine sand, sandy loam as appropriate for the dominant soil type in the Rosemont Project Area, the Gila Conglomerate, 9. The 6. The representative slope length and slope for anticipated overland flow are approximate and were estimated utilizing AutoCAD.

1. The basins and their associated... sediment yield for the Project... post-closure and operations... are only those... estimated from the Rosemont Flow-Through Drain Sizing Technical Memorandum.

2. The watershed hydrology input data for the SEDCAD Analysis was referenced from the Rosemont Flow-Through Drain Sizing Technical Memorandum.

3. The rainfall factor, K, was conservatively selected for fine sandy loam, loamy very fine sand, sandy loam as appropriate for the dominant soil type in the Rosemont Project Area, the Gila Conglomerate.

4. The C factor, accounting for canopy, surface cover, and surface roughness, was conservatively selected as having no appreciable canopy for grass with 45% ground cover anticipated during Post-Closure (C = 0.10) and for grass with 5% ground cover anticipated during Operations (C = 0.45).

5. The P factor, accounting for no anticipated utilization of sediment control practices, was conservatively set equal to 1.

6. The representative slope length and slope for anticipated overland flow are approximately accurate utilizing AutoCAD.

7. The volume of sediment was conservatively calculated utilizing the dry specific weight of sand sediment deposits with no expected consolidation over time of 93 lb/ft³ or 2055 kg/m³ (Ref. Erosion and Sedimentation by Pierre Y. Julien, 1958 pg 241).

8. The average annual sediment yield was calculated for post-closure and operations conditions by multiplying the respective 24-hr design storm event's estimated sediment volume by its annual probability of occurrence.

9. The peak flows and post-closure and operations sedimentation values were conservatively estimated by linear interpolation between the 5-yr and 10-yr events in order that the sum of the respective annual prediction probabilities of occurrence is equal to 1.

10. The average annual sediment yield estimated for post-closure and operations conditions by the sum of the average-annual sediment yields for each 24-hr design storm event since the sum of the associated annual probabilities of occurrence for each respective event is equal to 1.

11. The average annual sediment yield for the Project area using the Pacific Southwest Inter-Agency Committee (PSIAC) Method was estimated at 1.15 ac-ft/ct or 1.80E-3 ac-ft/ac-yr (Ref. Rosemont Baseline and Post-Closure Conditions - Alternatives Sediment Delivery Technical Memorandum by Zeiler).