Technical Memorandum

To: Kathy Arnold
From: David Krizek
Company: Rosemont Copper Company
Date: November 30, 2010
Re: Rosemont Site Water Management Update Review Responses
Doc #: 271/10-320878-5.3
CC: Tom Furgason (SWCA)

1.0 Introduction

This Technical Memorandum provides responses to the review comments provided to Rosemont Copper Company (Rosemont) on the Site Water Management Update report prepared by Tetra Tech in March 2010 (Tetra Tech, 2010m). This five (5) volume update was a supplement to the report titled Site Water Management Plan (Tetra Tech, 2007b).

The review was performed by Golder Associates (Golder). Golder’s review was documented in a ten (10) page Technical Memorandum (Golder, 2010a). Golder summarized their review results in a table (Table 3 on page 10 of 10 of Golder, 2010a). This table was used to organize responses to the review comments. Table 1 below is a recreation of Golder’s summary table with added item numbers and page references to the review comments within the text of this Technical Memorandum. Golder’s Technical Memorandum titled Rosemont Copper Project, Technical Review of Site Water Management Update (Golder, 2010a) is provided in Attachment 1.

Prior to answering the questions/concerns listed in Table 1, clarifying statements within the text of Golder (2010a) are reproduced and italicized for some of the items.

<table>
<thead>
<tr>
<th>Flag</th>
<th>Percent of Material (by weight)</th>
<th>Item</th>
<th>Reference</th>
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<tbody>
<tr>
<td>Red Flags</td>
<td>Using smaller precipitation depth (18in) to calculate average annual runoff instead of NRCS recommended depth (24in)</td>
<td>1</td>
<td>Section 2.0</td>
</tr>
<tr>
<td></td>
<td>No volume check calculations using maximum saturation event conditions</td>
<td>2</td>
<td>Section 2.0</td>
</tr>
<tr>
<td></td>
<td>No calculations presented for pit diversion channel and pit stormwater pond</td>
<td>3</td>
<td>Section 3.0</td>
</tr>
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Memorandum

To: Bev Everson
Cc: Tom Furgason
From: Kathy Arnold
Doc #: 047/10-15.3.2
Subject: Transmittal of Technical Responses to Golder Review
Date: November 30, 2010

Rosemont is pleased to transmit the following documents:

- Rosemont Site Water Management Update Review Responses, Technical Memorandum, Tetra Tech, November 30, 2010

Rosemont is providing three hardcopies and two disk copies for the Forest and two hardcopies and one disk copy for SWCA of the technical memos. Copies of the reports are provided in a hardcopy format with the disk copies enclosed in the same number.
Table 1  Red Flags and Potential Fatal Flaws (Table 3 from Golder, 2010) (Continued)

<table>
<thead>
<tr>
<th>Flag</th>
<th>Percent of Material (by weight)</th>
<th>Item</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Flags</td>
<td>Methodology used for sediment calculations should be reviewed as it is believed to be inappropriate</td>
<td>4</td>
<td>Section 4.0</td>
</tr>
<tr>
<td></td>
<td>Lack of drainage from perimeter containment area</td>
<td>5</td>
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<tr>
<td></td>
<td>Demonstrate adherence to geometric recommendations on landform element suggestions previously proposed by Golder</td>
<td>6</td>
<td>Section 6.0</td>
</tr>
<tr>
<td></td>
<td>Lack of detail for sediment control designs during operations</td>
<td>7</td>
<td>Section 7.0</td>
</tr>
<tr>
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<td>Specific sediment yield is the same for pre- and post-mining conditions, which appears to be incorrect</td>
<td>8</td>
<td>Section 8.0</td>
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<tr>
<td>Potential Fatal Flaw</td>
<td>Storage on benches in unusual for long-term closure and could lead to massive failure</td>
<td>9</td>
<td>Section 9.0</td>
</tr>
<tr>
<td></td>
<td>Down chutes on both tailings facility and waste rock can lead to failure as riprap lining may be inappropriate</td>
<td>10</td>
<td>Section 10.0</td>
</tr>
<tr>
<td></td>
<td>Flow-through drains: potential long-term difficulties with maintenance and retaining discharge capacity</td>
<td>11</td>
<td>Section 11.0</td>
</tr>
<tr>
<td></td>
<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>
<td>12</td>
<td>Section 12.0</td>
</tr>
<tr>
<td></td>
<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>
<td>13</td>
<td>Section 13.0</td>
</tr>
</tbody>
</table>

2.0  Runoff Calculations

Item 1 refers to comments found in Section 2.0 of Golder (2010a).

Although National Resource Conservation System (NRCS) reports give the average-annual precipitation for the Rosemont area as ±24 inches, Tetra Tech used 18 inches in the developed regression equation to calculate average-annual runoff. This produced realistic results compared to observed data.

The use of 24 inches would have substantially over-predicted average-annual runoff, based upon measured runoff data published by the Arizona Department of Water Resources (ADWR). The ADWR Water Atlas contains up-to-date and accepted water data for the state of Arizona. It was used to check the calculation results, ensuring they resulted in real-world estimates of average-annual runoff at the Rosemont site.

Additional reasons for not using 24 inches of rainfall included the following:

a. There is no corroborative data in support the larger NRCS value of 24 inches; and
b. Application of 24 inches in regression equations yielded unacceptably high results for average-annual runoff at the Rosemont site when compared to measured data from nearby watercourses.

Item 2 also refers to comments found in Section 2.0 of Golder (2010a).

The worst-case average-annual runoff was calculated based upon observed extremes of measured average-annual runoff on similar watersheds throughout Arizona. Such extremes implicitly account for the occurrence of maximum saturation events.

It is accepted that for larger watersheds in Arizona, the major flood producing storms generally occur in the winter months due to frontal or convergence activity. A frontal or convergence storm, herein referred to as a general storm, produces large volumes of relatively low intensity rainfall over long durations. General storms are also typically large in areal extent.

For smaller watersheds, which are the types of watersheds that exist at the Rosemont site, the major flood producing storms generally occur in the summer months due to convective activity. A convective storm, herein referred to as a local storm, produces high intensity rainfall over a relatively short duration and a small areal extent. Occasionally, these storms can also be imbedded in general summer storms that are typically a result of tropical storms that move into the state from the Pacific Ocean.

Over the last 118 years, 89 flood and flash floods have impacted southeastern Arizona. Of these, 89 floods, 73 occurred during summer storm events and 16 occurred during winter storm events. This list was obtained from the National Oceanic and Atmospheric Administration (NOAA) website courtesy of the United States Geological Survey (USGS) (NOAA, 2010).

Also, Tetra Tech believes that the General Probable Maximum Precipitation (PMP) storm event would better represent the "maximum saturation event" for structures impounding water. This storm event represents the PMP event predicted to occur at the site. Per NOAA Atlas 14 (NOAA, 2009), the amount of rainfall (±10.1 inches) is approximately mid-way between an 8-day, 100-year and 200-year return-interval event. The 72-hour General PMP = 18.9 inches of rainfall (i.e., nearly double).

### 3.0 Pit Diversion and Pit Stormwater Pond Calculations

Attachment 2 provides calculations related to the Pit Diversion Channel in the Technical Memorandum titled *Rosemont Pit Diversion Channel Design* (Tetra Tech, 2010k).

Sizing of the Pit Stormwater Pond was described in Appendix K of the *Site Water Management Update* report (Tetra Tech, 2010m) in a Technical Memorandum titled *Rosemont Flow-Through Drain Sizing* (Tetra Tech, 2010i). The Pit Stormwater Pond is referred to as Pond S1C in this flow-through drain sizing memorandum (Tetra Tech, 2010i).

The Pit Stormwater Pond was designed to retain up to the 100-year-24 hour event and pass storm flows in excess of this event through culverts to a flow-through drain (South 2 Drain). The culverts were sized at maximum watershed conditions. The contributing watershed to the Pit Stormwater Pond (Pond S1C) is eventually eliminated at the ultimate pit extents (end of operations). As indicated in the *Site Water Management Update* report (Tetra Tech, 2010m), the
culverts may be reduced in size/number to allow peak flows to pass over the road and into the flow-through drain at the short-lived maximum watershed conditions.

### 4.0 Sediment Yield Calculations

Item 4 relates to comments found in Section 6.1 of Golder (2010a).

*The method used for the calculation of sediment yield for the site is the Pacific Southwest Inter-Agency Committee (PSIAC) method. This method was developed in 1968 in southern California and is recommended for basins that are larger than 10 mi$^2$ in size. The baseline and post-mining scenarios analyzed have basin areas of 8.20 mi$^2$ and 1.93 mi$^2$ respectively. Therefore Golder recommends that the sediment yield calculations be evaluated using a method that is more appropriate for this site.*

While it is true that the original developers of the PSIAC method recommended that the procedure be used for watershed delineations of ten (10) square miles or greater, subsequent studies in the intervening 40+ years have shown that the method can be used to reasonably characterize sediment yield from areas as small as 100 acres (e.g., Rasley, USDA NRCS, 1991). Over the past 30 years, Tetra Tech has used PSIAC on many small watersheds throughout southern Arizona, with highly satisfactory results. Attachment 3 provides a copy of the report titled *Proposed Revision (1991) of Sediment Yield Procedure - Pacific Southwest Inter-Agency Committee Report to the Water Management Subcommittee - October, 1968* (Rasley, USDA NRCS, 1991).

### 5.0 Drainage from Perimeter Containment Areas

Item 5 relates to comments found in Section 5.3 of Golder (2010a).

*There is no identifiable fatal flaw with the perimeter containment areas; however, there is a long-term concern with the lack of outlet from these locations. These may also potentially fill with sediment.*

In conjunction with the Detention Pools on the wide benches of the Waste Rock Storage Area, the Perimeter Containment Areas (PCAs) were designed to handle storm runoff from a General PMP event. The Detention Pools were typically designed to handle storm runoff from up to the 500-year, 24-hour event, with overflow reporting to the PCAs.

Section 4.0 of the Technical Memorandum titled *Rosemont Waste Rock Storage Area Stormwater Management* (Tetra Tech, 2010i) provides the sizing and functioning of the PCAs. Tetra Tech (2010i) was provided as Appendix G of the *Site Water Management Update* report (Tetra Tech, 2010m).

The PCAs are large containment areas located between the toe of the Rosemont Ridge Landform and a natural ridgeline. Only one (1) location (PCA3) required a fill section to provide containment of storm runoff to the adjacent drainage. This design achieves the goal of maintaining all Project activities within a single drainage which remains as one of Rosemont’s early initiatives.

The following summarizes the functioning of the PCAs. Included are storm runoff volumes reporting to the PCAs and their storage volumes:
• PCA1: This perimeter containment area is internal to the facilities with an outlet to a flow-through drain (South 2 Drain). Assuming PCA1 fills with sediment or its capacity becomes limited to convey/contain storm flows, stormwater would report to the Open Pit.

• PCA2: This perimeter containment area is located on the west side of the Waste Rock Storage Area and mainly receives storm runoff from the Pit Diversion Channel. This containment area was not designed to contain the General PMP event, but to pass storm volumes in excess of the 500-year, 24-hour event to PCA4 via a storm runoff channel.

• PCA3: This perimeter containment area is located on the south/southwest side of the Waste Rock Storage Area. A limited area reports to PCA3 since the storm runoff channel from PCA2 to PCA4 is located directly north of the PCA3. The capacity of PCA3 is estimated to be 127 acre-feet. Storm runoff reporting to PCA3 from a General PMP event is estimated to be about 90 acre-feet. Therefore, as currently sized, PCA3 is about 40-percent larger than required to contain runoff from a General PMP event.

• PCA4: This perimeter containment area is located on the south side of the Waste Rock Storage Area. As indicated above, storm runoff in excess of the 500-year, 24-hour event from PCA2 will be routed to PCA4. Storm runoff from PCA Basin 4 in the Waste Rock Storage Area reporting to PCA4 would also be in excess of the 500-year, 24-hour event. Therefore, storm runoff reporting to PCA4 is anticipated to be about 260 acre-feet from a General PMP event (this excludes the 500-year, 24-event volume from PCA2 and the Detention Ponds located on the wide benches of the Waste Rock Storage Area). The capacity of PCA4 is estimated to be about 360 acre-feet. Therefore, as currently designed, PCA4 is about 27-percent larger than required to contain runoff from a General PMP event.

• PCA5: This perimeter containment area is located on the southeast side of the Waste Rock Storage Area. A limited area reports to PCA5. The capacity of PCA5 is estimated to be 93 acre-feet. Storm runoff reporting to PCA3 from a General PMP event is estimated to be about 21 acre-feet. Therefore, as currently sized, PCA3 is over four (4) times larger than required to contain runoff from a General PMP event.

• PCA6: This perimeter containment area is located on the southeast side of the Waste Rock Storage Area. Storm runoff from PCA Basin 6 in the Waste Rock Storage Area in excess of the 500-year, 24-hour event will report to PCA6. Taking this volume into account, excess storm runoff reporting to PCA6 is anticipated to be about 104 acre-feet from a General PMP event (excludes 500-year, 24-year storm runoff from the Detention Ponds located on the wide benches of the Waste Rock Storage Area). The capacity of PCA6 is estimated to be about 104 acre-feet. Therefore, as currently designed, PCA6 has no extra capacity. However, PCA6 is designed to overflow to PCA7, which has an outlet, if needed, via PCA 8 to Lower Barrel Canyon located at the toe of the Rosemont Ridge Landform.

• PCA7: This perimeter containment area is located on the east side of the Waste Rock Storage Area. Storm runoff from PCA Basin 7 in the Waste Rock Storage Area in excess of the 500-year, 24-hour event will report to PCA7. Also, as indicated above, PCA6 has the ability to overflow to PCA7 if needed, since PCA6 has no extra storage capacity.
beyond the General PMP event. PCA7 can overflow to PCA8, which has an outlet to Lower Barrel Canyon at the base of the Rosemont Ridge Landform.

PCA7 has an estimated capacity of 328 acre-feet. The storm runoff volume reporting to PCA7 from a General PMP event is approximately 178.5 acre-feet (excludes 500-year, 24-hour storm runoff from Detention Ponds located on wide benches of the Waste Rock Storage Area). Therefore, PCA7 is about 46-percent larger than required to contain runoff from a General PMP event.

- **PCA8**: This perimeter containment area is located on the west side of the Waste Rock Storage Area. Storm runoff from a limited area reports to PCA8. PCA8 has a containment capacity of about 13.5 acre-feet. A General PMP event volume is about 9.4 acre-feet. Therefore, PCA8 is about 44-percent larger than required to contain runoff from a General PMP event. Additionally, PCA8 has an outlet to Lower Barrel Canyon. Temporary sediment control structures are also anticipated between PCA8 and the Compliance Point Dam in Lower Barrel Canyon during operations.

If required, potential concerns related to sizing of the PCAs could be remedied in the final design effort of the Rosemont Ridge Landform by slight adjustments to the toe setback from the natural ridgeline during the detailed design effort of the Project. Additionally, access to all of the PCA locations will be achieved via a Perimeter Access Road for maintenance as needed (i.e., sediment removal). Based on the planned construction of the screening berms and outer buttress areas, reclamation and revegetation of the outer slopes is anticipated early in operational life of the Project. Therefore, any sedimentation occurring in the PCAs would likely occur within the first five (5) to ten (10) years of the operation during which time cleanout could be accomplished by operational crews, etc.

### 6.0 Demonstrate Adherence to Geometric Landform Design by Golder

Item 6 relates to comments found in Section 6.2 of Golder (2010a).

> Golder was not requested to comment on the landforming arrangement, but feels compelled to do so as we have developed and estimated the hydraulic and erosion performance of the elements that were used to develop the landformed shape. We recommend that Tetra Tech develop a table showing adherence to the recommendations previously made by Golder in this regard.

Based on Golder’s report titled *Rosemont Mine Landforming - Evaluation of Mine Waste Slope Geometry* (Golder, 2010b), slope length recommendations were provided based on an assumed material gradation placed on the outer slopes in combination with various slope angles. Based on the material having a median particle size of $D_{50}=3$ inches, an intermediate lined drainage pool was recommended for a 500-foot high slope. This drainage pool was assumed mid-way of the slope at a transition from concave to convex slopes. An associated piping system was also illustrated as an option to remove storm runoff from the lined drainage pool. Detailed design would be required to confirm this arrangement.

In general, Tetra Tech placed drainage benches at a vertical spacing of 100 feet. This spacing was selected to reduce potential erosion on the slopes and to provide access to all areas of the Rosemont Ridge Landform. A mixed material with coarse fragments was assumed to be on the outer slopes by Tetra Tech.
7.0 Detailed Sediment Control Design During Operations

Item 7 relates to comments found in Section 6.2 of Golder (2010a).

_The report states that BMPs will be used during operations to manage sediment on the site; however, no specific definitions are described as to the locations and phasing of these sediment controls during operations. The report also calls for concurrent reclamation, which is very difficult in an arid climate. It is the recommended that BMPs be defined and that reliance on concurrent reclamation be minimized._

The following is summarized from Section 12.0 of the Site Water Management Update report (Tetra Tech, 2010m).

Except for access roads, development of the Rosemont Project will be up-gradient of the Compliance Point Dam and within the watersheds comprised of Barrel, Wasp, and McCleary Canyons. During early development of the Project, Best Management Practices (BMPs) will be employed to control sediment loading as needed to down-gradient receptors. Localized sediment control BMPs may also include the following:

- Temporary diversion channels and sediment traps;
- Containment berms;
- Riprap slope protection and culvert protection;
- Flow-through rock weirs; and
- Silt fencing, straw bales, and wattles.

Early revegetation of disturbed or reclaimed areas, or concurrent reclamation, is a significant part of the plan for this Project and will be employed as practicable. The site construction stormwater pollution prevention plan (SWPPP) will be updated prior to the start of construction activities. An appropriate Stormwater Permit application will be submitted to Arizona Department of Environmental Quality (ADEQ) and the SWPPP will be updated for the operational period.

Project facilities such as haul roads and stormwater ponding areas will also provide localized sediment control once constructed. Once the starter embankments for the South Dry Stack Tailings Facility and for the North Dry Stack Tailings Facility are constructed, they will provide sediment control for the entire area up-gradient of the structures. Stormwater runoff will filter through the flow-through drains and exit in Lower Barrel or McCleary Canyons, up-gradient of the Compliance Point Dam location.

Once the starter embankments are constructed, the outer surface will be reclaimed and revegetated as soon as practicable. Additionally, a rock facing is also planned for the lower east slope of the North Dry Stack Tailings Facility. As indicated, Project activities will generally occur behind the waste rock berms in the case of the Waste Rock Storage Area and Heap Leach Facility, or behind waste rock buttresses, in the case of the Dry Stack Tailings Facility. The design of the Rosemont Ridge Landform (Landform) as presented in the report titled Reclamation Concept Update (Tetra Tech, 2010d) minimizes direct runoff from the Landform to down-gradient receptors. Until the
reclaimed surfaces stabilize, sediment basins may be installed between the toe of the Landform and the Compliance Point Dam location for local sediment control. The Compliance Point Dam also functions as a sediment control structure, which is anticipated to be a six (6) foot high, porous rock structure where additional sediment controls will be applied as necessary to manage stormwater quality and where stormwater samples will be taken.

In summary, except for the outer slopes of the Rosemont Ridge Landform (screening berms and outer butters areas), all Project activity will be up-gradient of the starter buttresses for the South Tailings Dry Stack Tailings Facility or the North Dry Stack Tailings Facility. Therefore, operational sediment control will take place on limited areas on the North and South Dry Stack Tailings Facility outslopes.

Rosemont maintains as one (1) of their initiatives concurrent reclamation of the outslopes early in the operational life of the Project. Reclamation is anticipated to occur below completed working bench sections. In addition to temporary sediment containment structures, the following may be applied to the outslopes to control potential erosion from the side slopes prior to establishment of vegetation. These BMPs are mainly intended for areas reporting directly to the Compliance Point Dam location in Lower Barrel Canyon Wash. Slope areas reporting to perimeter containment areas (PCAs) or to flow-through drain inlet areas may receive less treatment.

- Incorporation of coarse rock on the lower slopes of the North Dry Stack Tailings Facility directly up-gradient of the Lower Barrel Canyon wash.
- Crimping of straw into the reclaimed surface to provide erosion stability.
- Surface treatments such as slope cross-cutting to provide erosion stability.

It is also assumed the growth media on the outslopes will be mixed with fairly coarse material. Ongoing test plots at the Rosemont Project site are underway on slopes with similar aspects (angle and length) as those proposed in the Reclamation Plan Update (Tetra Tech, 2010d). Lessons learned during the development and monitoring of these test plots will be used in the final design effort associated with the Rosemont Ridge Landform.

### 8.0 Pre- and post-Mining Sediment Yield Calculations

Item 8 refers to comments found in Section 6.1 of Golder (2010a).

Additionally, Golder has concerns with the results of the sediment yield calculations. Both baseline and post-mining conditions give the average-annual specific yield as 1.15 acre-feet/mi²/year. It is reasonable to expect that the baseline scenario will differ from the post-mining scenario because the addition of the landform will change the surface conditions. Currently no difference is indicated by the analysis results provided by TetraTech.

As a conservative approach, the same sediment factor of 1.15 acre-feet/mi²/year for the baseline condition watershed reporting to the Compliance Point Dam was applied to the post-mining condition. This assumption was reassessed and resulted in a factor of 0.78 acre-feet/mi²/year applicable to the post-mining condition, i.e., less sediment loss anticipated.
The baseline watershed area used in the analysis is approximately 8.20 square miles while the post-mining watershed reporting to the Compliance Point Dam location is approximately 1.32 square-miles. Of this 1.32 square mile area, about 50-percent is natural ground while the remaining ground is the face of the Dry Stack Tailings Facility. Drainage benches will be located on the face of the Dry Stack Tailings Facility every 100 feet of vertical rise. Inner bench slopes are planned at 3H:1V. Material on the outer slope is anticipated to be fairly coarse.

Attachment 4 provides the Rosemont Copper Project Sediment Yield Factor Rating table from the Technical Memorandum titled *Rosemont Baseline and Post-Mining Conditions—Sediment Delivery* (Tetra Tech, 2010e). As shown on the table, a factor of 1.15 acre-feet/mi²/year was estimated for the natural watershed area. A second factor was estimated for the bench areas of the Dry Stack Tailings Facility. This factor is shown on a second table provided in Attachment 4 titled Sediment Yield Factor Rating for Post-Mining Conditions – Rosemont Copper Company. For this area, a factor of 0.40 acre-feet/mi²/year was estimated. Based on the percentage of natural ground versus the area associated with the Rosemont Ridge Landform reporting to the Compliance Pont Dam, a factor of 0.40 was applied to the post-mining conditions.

In summary:

- The sediment yield factor rating for baseline conditions is 1.15 acre-feet/mi²/year resulting in 9.43 acre-feet per year of sediment delivery to the Compliance Point Dam location (based on 8.2 square-mile watershed area); and
- The sediment yield factor rating for post-mining conditions is 0.78 acre-feet/mi²/year resulting in 1.03 acre-feet per year of sediment delivery past the Compliance Point Dam location (based on 1.32 square-mile watershed area, about 50-percent of which is natural ground [1.15 acre-feet/mi²/year factor] and the other 50-percent is the east face of the North Dry Stack Tailings Facility [0.40 acre-feet/mi²/year factor]).

Accordingly, post-mining conditions result in about an 89-percent reduction in sediment delivery downstream of the Compliance Point Dam location.

### 9.0 Stormwater Storage on Benches

Item 9 refers to comments found in Section 5.4 of Golder (2010a).

*This issue, in our view, is such an unusual application that we wish to emphasize it here. It appears as if the consultant went to a lot of effort to size the facilities to minimize risk. Golder wishes to point out that it is unusual to store large amounts of water on top of waste rock and tailings facilities, and on benches, particularly after closure. It is recommended that appropriate stability calculations be executed to ensure that the geotechnical slope failures would not occur and that internal erosion might not lead to failure. Additionally, it is recommended that maintenance measures that will ensure that such containment volumes can be retained in the long term be outlined. Our concern is that a low spot that might develop on a perimeter berm could initiate a release, which can result in significant erosion. Such a low spot can be fairly small, but can lead to a massive release of all water in the containment area once erosion commences. This may lead to massive failure along the slopes of the waste rock and tailings facilities.*
As for storage on the benches, we recommend careful review of potential failure mechanisms. For example: Would it be possible for water to seep into the slope, eventually resulting in internal erosion and eventual failure of the slope? Such an erosion event can act in the same way as outlined in the previous paragraph, leading to a massive release of the water stored on the bench.

The detention of storm runoff on wide benches in the Waste Rock Storage Area is planned as part of the closure concept for the Rosemont Ridge Landform. These detention pools are designed to overflow to connecting pools as needed. Containment of the 500-year, 24-hour storm event is anticipated. Based on modeling performed as part of the *Infiltration, Seepage, and Fate and Transport Modeling Report – Revision 1* (Tetra Tech, 2010b), and the stability analysis performed as part of the *Reclamation Concept Update* report (see Appendix C1 of Tetra Tech, 2010d), global stability issues are not anticipated to occur. Additional stability analysis, including the development of a monitoring and maintenance program, will be performed as part of final design efforts for the Project.

10.0 Riprap Down Chutes

Item 10 refers to comments found in Section 5.1 of Golder (2010a).

This riprap protection on downchutes on the slopes of the tailings facility is designed to convey flow from bench channels to natural ground using the Robinson method. This method was originally developed using, to the best of Golder’s knowledge, a maximum \(d_{50}\) of 9 inches. The down chutes for the Rosemont project use rocks with median diameters (\(d_{50}\)) between 20-24 inches, which is outside the range of the Robinson method. Additionally, the ratio of normal flow depth to riprap thickness is much lower than 1. This leads to a situation where part of the water will likely flow through the rocks and out on top of them, as per the design intent. This can lead to unexpected failure.

Finally, the design calls for an 8 oz. min. geotextile fabric under the riprap. In Golder’s experience, geotextile fabric does not perform well as bedding material for riprap on steep slopes. Although, in some cases, riprap-lined chutes are still used on steep slopes, we recommend that its application for closure be reconsidered as such steep channels can be relatively unstable. This is not compatible with the closure demands of long-term stability.

The “Robinson” equation was actually developed using rock sizes up to 11 inches in diameter. The authors of the equation state: “Appropriate engineering judgment should be applied when extending this design information beyond the data base from which it was developed.” Tetra Tech applied appropriate engineering judgment by adding a factor of safety of 1.5 to the calculation of the \(D_{50}\) riprap size. Furthermore, resulting riprap sizes were found to be consistent with, or larger than, riprap sizes on steep slopes when using alternate procedures such as United States Geological Survey (USGS), Federal Highway Administration (FHWA), the Corps of Engineers (USACE), and local agencies (Pima County/City of Tucson).

The use of filter fabric underneath riprap placed on steep slopes is actually recommended by the USACE in their publication titled *EM 1110-2-1601 Hydraulic Design of Flood Control Channels* (USACE, 1994). Tetra Tech has elected to adopt the USACE guidance. Select pages of USACE (1994) are provided as Attachment 5 to this Technical Memorandum.
11.0 Flow-Through Drains

Golder’s review presented concerns about the potential clogging of rock drains, as a result of deposition of suspended sediments within the drain, or as a result of deposition of bedload at the inlet end of the drain.

Flow-through drains are designed to account for sediment accumulation at the inlet. Maintenance will be ongoing during mining activities, which includes reclamation at the site. The flow-through drains are significantly oversized to account for the possibility of some sediment clogging along the extent of the drain after termination of operations.

The fact that the free water surface in the rock drain must rise to accommodate the flow implies that a pool must form at the inlet end of the rock drain. This is true for all rock drains. As the water from the open channel enters the pool upstream of the inlet to a rock drain, the flow velocities decrease abruptly. As a result of this reduction in velocity, the bedload sediments that may be in transit along the bottom of the stream are deposited at the inlet to the pool. Since the bedload sediments are deposited at the upstream end of the pool, they are not transported to the inlet of the rock drain, and do not result in blockage of the inlet to the drain (Campbell-Golder Associates, Proceedings of the 13th Annual British Columbia Mine Reclamation Symposium on “Flow-Through Rock Drains, 1989). Campbell (1989) is provided as Attachment 6A.

In view of the above mentioned, it is foreseen that the size of particle to be accounted as the suspended solids to be transported to the inlet of the rock-drains will be fine grained sediment.

From Campbell’s study, within the range of fluctuation in the pool surface level, fine-grained sediment will be transported with the flow passing through the rock drains. Fine particles for the Rosemont area are mixtures of silt, loam, and clay (USDA Soil Survey) which classify as smaller than 0.074 mm in diameter or No. 200 US Standard Sieve size (ASTM D2487). The settling velocity of a 0.074 mm size particle would be approximately 3.4 millimeters per second or 0.011 Feet per second (ft/s) (Campbell, 1989).

Velocities for the Rosemont flow through drains were calculated using two equations: Leps (Leps, 1973) and Samani (Samani, 2007 and 2008). Samani (2007 and 2008) are provided as Attachment 6B and 6C, respectively. The velocities obtained were 0.47 ft/s and 0.067 ft/s respectively (see Attachment 6D). These results show that using the Samani equation produces more conservative values which may compensate for possible losses. In addition, the particle settling velocities of 0.011 ft/s would be 1/6th of the average velocity of flow through drain. In a turbulent flow field, particles having a settling velocity of about three (3) percent of flow velocity (0.002 ft/s), would not settle within the drain; these particles would be swept though the rock drain with the flow.

Conclusions from field observations on flow through drains stated at the Proceedings of the 13th Annual British Columbia Mine Reclamation Symposium (Campbell, 1989) are:

1. During periods of significant stream discharge, a pool will always be present at the inlet end of a rock drain. This pool serves to trap bedload at the location where the stream enters the pool. Consequently, deposition of bedload does not result in blockage at the inlet to the rock drain.
2. The pool at the inlet end of a rock drain also results in deposition of suspended sediment. The field study indicates that potential deposition of suspended sediment is not a factor that would result in reduction in the capacity of the through-flow rock drain.

In addition, the use of the PMP (Probable Maximum Precipitation) to determine the capacity of the incoming ponds was based on the analysis previously stated where a rainfall total is only 53.4 percent of the General PMP storm event. In Tetra Tech’s opinion, the General PMP better represents a maximum event at the Rosemont site.

An extensive sediment analysis associated with the flow-through drain inlets was prepared as part of the Technical Memorandum titled *Rosemont Flow-Through Drain Sedimentation Analysis* (Tetra Tech, 2010).

### 12.0 Water Storage on Top of Tailings Facility and Waste Rock Storage Area

Item 12 refers to comments found in Section 5.4 of Golder (2010a).

This issue, in our view, is such an unusual application that we wish to emphasize it here. It appears as if the consultant went to a lot of effort to size the facilities to minimize risk. Golder wishes to point out that it is unusual to store large amounts of water on top of waste rock and tailings facilities, and on benches, particularly after closure. It is recommended that appropriate stability calculations be executed to ensure that the geotechnical slope failures would not occur and that internal erosion might not lead to failure. Additionally, it is recommended that maintenance measures that will ensure that such containment volumes can be retained in the long term be outlined. Our concern is that a low spot that might develop on a perimeter berm could initiate a release, which can result in significant erosion. Such a low spot can be fairly small, but can lead to a massive release of all water in the containment area once erosion commences. This may lead to massive failure along the slopes of the waste rock and tailings facilities.

As for storage on the benches, we recommend careful review of potential failure mechanisms. For example: Would it be possible for water to seep into the slope, eventually resulting in internal erosion and eventual failure of the slope? Such an erosion event can act in the same way as outlined in the previous paragraph, leading to a massive release of the water stored on the bench.

In the report titled *Rosemont Copper Company Dry Stack Tailings Storage Facility Final Design Report* (AMEC, 2009) and in the *Rosemont Copper Company Dry Stack Tailings Storage Facility Stormwater Management Preliminary Design Report* (AMEC, 2010b), ponding of storm runoff incidental to the top surface of the reclaimed Dry Stack Tailings Facility was proposed as a closure concept. Containment of the Probable Maximum Flood (PMF) was anticipated on the top surface with an overflow spillway located on the west side of the Rosemont Ridge Landform. Overflow would report to a flow-through drain located in the area of the former Process Water Temporary Storage (PWTS) Pond.

Based on AMEC (2009) and on a report titled *Infiltration, Seepage, and Fate and Transport Modeling Report* (Tetra Tech, 2010b), infiltration of meteoric precipitation into the dry stack tailings is not anticipated to be a concern. Stability analysis performed by AMEC (2009) and in a technical memorandum titled *Dry Stack Facility Stability Analysis* (AMEC, 2010a) indicated satisfactory factors of safety per the Best Demonstrated Control Technology (BADCT) Guidance
Document (ADEQ, 2004). AMEC (2010a) was provided in Appendix C2 of the Reclamation Concept Update report (Tetra Tech, 2010d). A setback distance from the normal stormwater “pool” of about 1,000 feet was recommended by AMEC.

In the Reclamation Concept Update report (Tetra Tech, 2010d) and in the Site Water Management Update report (Tetra Tech, 2010m), this concept was modified to include two (2) decant structures in the northwest and northeast corners of the North Dry Stack Tailings Facility. Storm runoff in excess of a 1,000-year, 24-hour event will begin to decant off the top of the North Dry Stack Tailings Facility from small Detention Pools. A modified version of the Rosemont Ridge Landform design also has a northwest haul road left at closure. This northwest haul road could also be used to route excess storm flow to the former PWTS Pond area and the inlet of the South 1 Drain flow-through drain.

Section 9.0 of this Technical Memorandum discusses the retention of storms flows in Detention Pools on the wide benches of the Waste Rock Storage Area.

13.0 Allowance for Erosion in Containment Areas

Item 13 refers to comments found in Section 6.1 of Golder (2010a).

Golder produced a report Rosemont Mine Landforming – Evaluation of Mine Waste Slope Geometry dated February 17, 2010 wherein it was estimated that the expected erosion from the Rosemont landform surface prior to stabilization will be 14.4 inches. It is estimated that large amounts of this sediment will report to all areas where water will be ponded. This will therefore reduce the storage capacity of the bench storage areas and perimeter containment areas. Allowance for such storage loss should be made.

Section 5.0, “Drainage from Perimeter Containment Areas”, in this Technical Memorandum reviewed the sizing of the perimeter containment areas (PCAs). As illustrated in Section 5.0, many of the PCA locations have excess capacity in terms of containing storm runoff from a General PMP event. Those PCA locations without excess capacity typically overflow to other PCA locations having excess capacity or an outlet to Lower Barrel Canyon Wash, where additional sediment control structures will be located. Should sediment removal be required to maintain capacity of the PCAs, access to the PCAs will be available via the Perimeter Access Road for heavy equipment, etc.

Rosemont Copper has committed to beginning reclamation of the outer perimeter areas early in the life of the Project. This would include outer slope areas reporting to the PCAs. Section 7.0, “Detailed Sediment Control During Operations”, provides general details on possible BMP sediment controls on the outer slopes prior to the establishment of a vegetative cover.

As highlighted in the Technical Memorandum titled Rosemont Dry Stack Tailings Facility Drainage Bench Analysis (Tetra Tech, 2010f), which was previously provided as Appendix H of the Site Water Management Update report (Tetra Tech, 2010m), the bench drainage channels were designed to handle a minimum peak flow from a 500-year, 24-hour event. Additionally, assuming that 30-percent of the channel becomes filled with sediment, the capacity of these channels would still handle the peak flow from a 500-year, 24-hour event.

The drainage benches either report to natural ground, Detention Pools on the wide benches in the Waste Rock Storage Area, or to stilling pools which transition flows to drop chutes. The stilling pools are large “football field” sized structures typically 4.5 feet deep. As such, these
areas can accommodate large amounts of sediment loading. All of the drainage benches in the Dry Stack Tailings Facility areas will have access roads for inspections, maintenance, and sediment removal as needed.

Access roads will also be provided to all areas of the Waste Rock Storage Area. The Detention Pools located on the wide benches of the Waste Rock Storage Area are also large structures and are typically four (4) to eight (8) feet deep. All detention pool areas will also be accessible via an access road for inspections, maintenance or sediment removal as needed. As illustrated in the Technical Memorandum titled *Rosemont Waste Rock Storage Area Stormwater Management* (Tetra Tech, 2010), the detention pools are cumulatively designed to contain storm runoff from a 500-year, 24-hour event from a given watershed area. Overflow channels are located between the Detention Pools and drop chutes will be placed as needed to pass excess storm flows to the perimeter containment areas (PCAs). The design of the drop chutes locations in the Waste Rock Storage Area is anticipated to be constructed of thick layers of run-of-mine (ROM) rock placed directly on the waste rock slopes in the shape of channel chutes which mimic talus slopes as needed. Placement of these rock chutes will generally be determined during the detailed design phase of the Project.
References


ATTACHMENT 1
ROSEMONT COPPER PROJECT, TECHNICAL REVIEW
OF SITE WATER MANAGEMENT UPDATE
(GOLDER, 2010A)
TECHNICAL MEMORANDUM

Date: August 5, 2010

To: Dale Ortman

From: George Annandale, Jennifer Patterson, Craig Baxter

RE: ROSEMONT COPPER PROJECT, TECHNICAL REVIEW OF SITE WATER MANAGEMENT UPDATE

1.0 INTRODUCTION

Golder Associates (Golder) conducted a review of the Site Water Management Update for the Rosemont Copper Project (April 2010, Tetra Tech). The Site Water Management Update is presented in five volumes. The review consisted of reading the pertinent sections of the report and supporting documents and rendering a professional opinion regarding whether or not the data, assumptions, and methods used in the report conform to currently accepted industry practice. Review was limited to the goals specified by SWCA as listed in each section below as they relate only to water and erosion management. No review of geotechnical stability or other disciplines were addressed.

This memorandum summarizes the findings Golder’s review of the Site Water Management Update. The goal of the review is to identify any red flags and potential fatal flaws associated with the concepts used or the design of site stormwater management structures.

2.0 RUNOFF CALCULATIONS

Goal: Compare Tetra Tech’s selected method(s) of runoff calculation and the method(s) proposed by Pima County; comment on the applicability of all methods to the Rosemont Project.

Tetra Tech analyzed both the NRCS method and the Pima County method (PC-HYDRO) to determine the most suitable storm criteria for the Rosemont site. Table 1 ranks the design storms obtained by applying these methods in terms of severity.

Tetra Tech selected the NRCS method to determine peak flows and runoff volumes for the design of structures at the Rosemont site. Golder agrees this method is more appropriate because the Pima County method is more suitable for small urban watersheds and is not as conservative as the selected method.
TABLE 1
SUMMARY OF DESIGN STORM COMPARISON BY TETRATECH

<table>
<thead>
<tr>
<th>NRCS Method</th>
<th>Peak Flow Rate Ranking</th>
<th>Runoff Volume Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000-yr, 24-hr NRCS Type II Dist.</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>500-yr, 24-hr NRCS Type II Dist.</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>100-yr, 24-hr NRCS Type II Dist.</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>100-yr, 1-hr thunderstorm</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>100-yr, 1-hr compressed 6-hr event</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>100-yr, 1-hr NRCS Type II Dist.</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td>6-hr Local PMP</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>72-hr General PMP</td>
<td>9</td>
<td>1</td>
</tr>
<tr>
<td>Pima County Method (PC-HYDRO) 100-yr, 6-hr</td>
<td>4</td>
<td>6</td>
</tr>
</tbody>
</table>

Published reports give the average-annual precipitation as ±24 inches; however, Tetra Tech concludes that the average-annual precipitation is 18 inches. This was obtained by using both site-measured precipitation as well as back-calculating precipitation depth using average-annual runoff from the Arizona Water Atlas (106.7 ac-ft/sq-mi). This raises a few questions:

- How was the selected average rainfall of 18 inches used, and what was the sensitivity of that application compared to using the 24 inches average rainfall?
- Is the use of the Arizona Water Atlas appropriate? Golder understands that the water atlas back calculation was likely only used as a check of the site-calculated average rainfall. However, if one knows what the answer to a problem is, it is easy to select parameters for the back calculation to get to that answer. The question is whether those selected parameters are reasonable.
- How many years of site collected data were used to determine that the average-annual precipitation of 18 inches? Was the record long enough to justify not using the 24 inches average rainfall?

Also lacking in the runoff analyses is an assessment of the effects of the maximum saturation event. Arizona’s worst-case runoff volume conditions typically occur during consecutive precipitation days, as for example illustrated in Figure 1.

Experience in Arizona is that long duration, relatively low intensity rains often results in larger flow volumes than the 24-hr or shorter duration design storms. It is recommended that the maximum saturation event runoff be identified for the site and used to evaluate the capacity of the structures impounding water.
3.0 DESIGN CRITERIA FOR WATER CONTROL STRUCTURES

Goal: Concisely tabulate the design criteria selected by Tetra Tech for each water control structure and determine if the design calculations used the selected design criteria values. This information is summarized in Table 2.

As shown in Table 2, it is unknown if the Pit Stormwater Pond and Crusher Stormwater Pond meet the specified design criteria, because no detailed sizing calculations were included in the Site Water Management Update.

The client requested Golder to indicate concurrence with the application of the design criteria. Concurrence or not by Golder is indicated in the last column of Table 2.
### TABLE 2

**STORMWATER STRUCTURE DESIGN CRITERIA**

<table>
<thead>
<tr>
<th>Water Control Structure</th>
<th>Design Criteria Established in Volume 1</th>
<th>Criteria Followed?</th>
<th>Golder Concurrence?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pit Diversion Channel</td>
<td>Local PMP Event conveyance</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>Pit Stormwater Pond</td>
<td>General PMP Volume</td>
<td>Unknown</td>
<td>NO* + requires further clarification</td>
</tr>
<tr>
<td>Crusher Stormwater Pond</td>
<td>General PMP Volume</td>
<td>Unknown</td>
<td>NO* + requires further clarification</td>
</tr>
<tr>
<td>Permanent Diversion Channel No. 1</td>
<td>Local PMP Event conveyance, 200-yr, 24-hour erosion protection</td>
<td>YES</td>
<td>Why use different criteria? Clarify.</td>
</tr>
<tr>
<td>PWTS Pond and Settling Basin</td>
<td>100-yr, 24-hr event</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>Detention Basin No. 1</td>
<td>Manage General and Local PMP Volume, contain 200-yr, 24-hr</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>Permanent Diversion Channel No. 2</td>
<td>Local PMP Event conveyance, 200-yr, 24-hour erosion protection</td>
<td>YES</td>
<td>Why use different criteria? Clarify.</td>
</tr>
<tr>
<td>Detention Basin No. 2A</td>
<td>Manage General and Local PMP Volume, contain 200-yr, 24-hr</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>Detention Basin No. 2B</td>
<td>Manage General and Local PMP Volume, contain 200-yr, 24-hr</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>Detention Basin No. 3</td>
<td>Manage General and Local PMP Volume, contain 200-yr, 24-hr</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>Waste Rock Storage Area</td>
<td>Detention Pools on benches contain 500-yr, 24-hr event. PCAs capacity for General PMP event</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td>North Dry Stack Tailings Facility</td>
<td>Drainage channels and drop structures 500-yr, 24-hr.</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>South Dry Stack Tailings Facility</td>
<td>Depression areas on top of dry stack contain 1000-yr, 24-hr event, berms also on top control larger than general PMP event</td>
<td>YES</td>
<td>NO*</td>
</tr>
<tr>
<td></td>
<td>Drainage channels and drop structures 500-yr, 24-hr.</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>Depression areas on top of reclaimed surface. Storms up to 1,000-yr, 24-hr event controlled behind rock weir on top of dry stack.</td>
<td>YES</td>
<td>NO* Is rock weir watertight?</td>
</tr>
<tr>
<td></td>
<td>Larger flows discharged over weir to rock slope leading to flow-through drain</td>
<td>Unknown</td>
<td>Unclear what it meant by larger flows. How is stability ensured?</td>
</tr>
</tbody>
</table>

Note: NO* indicates that the storage volumes should be checked to also contain the maximum saturation event.
4.0 FLOW-THROUGH DRAINS

Goal: Review the design of the Flow-Through Drains and comment on their short- and long-term functional viability.

The purpose of Flow-Through Drains is to convey up-gradient water into the natural drainage downstream of the tailings and waste rock facilities. The Flow-Through Drains are constructed in addition to the typical under drains. The long-term viability of these structures is uncertain due to the potential effects of clogging by sediment. We recommend every effort be made to route water around the structures instead of using the flow-through drains. If this is not possible, then the Flow-Through Drains need to be constructed in a manner by which sediment can be trapped at the inlet and maintenance can be performed. Without an agreement to this maintenance, this structure poses, in our opinion, a fatal flaw.

Golder was requested to specifically comment on the entrance arrangement to the flow-through drains, shown in Figure 2. It is our opinion that sediment from upstream will likely clog the berm over the medium to long term. This is due to the fact that no upstream provision is made to prevent sediment from entering the berm.

![FIGURE 2
DETAIL OF THE FLOW-THROUGH INLET](image)

Both the long-term and short-term functionality of the Flow-Through drains are dependent upon the capacity of the upstream ponds. The capacity is based on the incoming runoff, which should be calculated using both PMP and maximum saturation event conditions to crosscheck results. The capacity is also based on the outflow rate, which is calculated using the following equation:
\[ Q = \left( \frac{1}{D} \right)^{\frac{1}{b+2}} \frac{aw}{(3 + b)^{b+2}} \left( H_{up}^{b+3} - H_{down}^{b+3} \right)^{\frac{1}{b+2}} \]

Where:

- \( a = \left( \frac{2gu}{a(d_{50} - \sigma)^{b-1}} \right)^{\frac{1}{b+2}} \)
- \( D = L - 0.7S_1 \)
- \( S_1 = H_{up} \cot \beta \)

- \( d_w \) is the particle diameter size where 50% of the total particles’ weight is smaller
- \( a \) and \( b \) are empirical coefficients of the equation related to the flow and particles
- \( u \) is the kinematic viscosity
- \( \sigma \) is the standard deviation of rock size distribution
- \( Q \) is the outflow rate through the rockfill dam structure
- \( H \) is the water depth inside the structure
- \( W \) is the width of the flow cross section
- \( \beta \) is the angle of the upstream and downstream dam face with horizontal
- \( L \) is the length of the dam


It appears this equation was developed to calculate flow through relatively short lengths of rockfill dams. It does not include allowances for losses due to long reaches or bends within the Flow-Through Drain. It is anticipated that the ponded water on the up-gradient portion of the tailings impoundment may not drain as quickly as calculated in the Management Plan.

### 5.0 REVIEW SITE STORMWATER CONTROLS

**Goal:** Review the design of the stormwater controls for the Rosemont Ridge Landform, including the Waste Rock Storage Area and Dry Stack Tailings Facility and comment on their short- and long-term functional viability.

#### 5.1 Dry Stack Tailings Facility

The Dry Stack Tailings Facility is broken into North and South facilities with very similar stormwater management designs for each facility. Depressions on top of the North tailings facility contain the 1,000-year, 24-hour storm event before allowing runoff to enter decanting structures and discharge off the tailings facility. Containment berms located on top of the North Dry Stack Tailings Facility have capacity to contain a volume from larger than the General PMP event. Similarly, the South Dry Stack Tailings Facility...
Facility has depressed areas to contain runoff from the 10-year, 24-hour event. Larger flows but smaller than the 1,000-year, 24-hour event will be retained behind a rock weir on the west side of the landform. Larger flows than the 1,000-year, 24-hour event will be discharged over the rock weir and will eventually be conveyed to a flow-through drain.

One concern with this type of design is the need for accuracy during construction. If one berm containing the water has a low-lying spot, the entire area of ponded water may escape causing massive erosion should water flow through that low-level spot. Another concern with this design is the estimated magnitude of the required capacity. Golder recommends that the volumes be checked using the maximum saturation event.

The riprap protection on downchutes on the slopes of the tailings facility is designed to convey flow from bench channels to natural ground using the Robinson method. This method was originally developed using, to the best of Golder’s knowledge, a maximum $d_{50}$ of 9 inches. The downchutes for the Rosemont project use rocks with median diameters ($d_{50}$) between 20-24 inches, which is outside the range of the Robinson method. Additionally, the ratio of normal flow depth to riprap thickness is much lower than 1. This leads to a situation where part of the water will likely flow through the rocks and not on top of them, as per the design intent. This can lead to unexpected failure.

Finally, the design specifies an 8 oz. min. geotextile fabric under the riprap. In Golder’s experience, geotextile fabric does not perform well as bedding for riprap on steep slopes. Although, in some cases, riprap-lined chutes are still used on steep slopes, we recommend that its application for closure be reconsidered as such steep channels can be relatively unstable. This is not compatible with the closure demands of long-term stability.

Drainage exiting the Dry Stack Tailings enter existing natural drainages at several points including the permanent diversion channel to the north side of the tailings facility, riprap lined downchutes, and channels flowing along benches. No erosion protection has been identified at these locations. These areas should be analyzed to ensure flow transitions from the engineered channels to the natural drainages without causing erosion to the natural channels.

5.2 Waste Rock Storage Area

Similar to the Dry Stack Tailings Facilities, the Waste Rock Storage Area has designed depression areas to contain a certain storm event. The Waste Rock Storage Area’s depression areas contain up to the 500-year, 24-hour storm event. Flows up to the General PMP event will be conveyed to the toe of the storage area and will be retained by perimeter containment areas (PCAs). Conveyance to the PCAs will be by rocked slopes on the 3:1 slopes of the Waste Rock Storage Area. No specifications for the gradation of the rock to be used on the 3:1 slopes were provided.
Concerns with this storage are similar to the Dry Stack Tailings Facility. The design will require tight controls on construction methods to ensure consistent elevations if the berms around all the benches. Additionally, the storage volumes should be checked using the maximum saturation event.

Golder was unable to locate designs for the downchutes on the waste rock storage area. The document indicated a need for riprap, but no structures were designed.

5.3 Perimeter Containment Areas
There is no identified fatal flaw with the perimeter containment areas; however, there is a long-term concern with the lack of outlet from these locations. These may also potentially fill with sediment.

5.4 Water Storage on Waste Rock and Tailings Facilities and Benches
This issue, in our view, is such an unusual application that we wish to emphasize it here. It appears as if the consultant went to a lot of effort to size these facilities to minimize risk. Golder wishes to point out that it is unusual to store large amounts of water on top of waste rock and tailings facilities, and on benches, particularly after closure. It is recommended that appropriate stability calculations be executed to ensure that geotechnical slope failures would not occur and that internal erosion might not lead to failure. Additionally, it is recommended that maintenance measures that will ensure that such containment volumes can be retained in the long term be outlined. Our concern is that a low spot that might develop on a perimeter berm could initiate a release, which can result in significant erosion. Such a low spot can be fairly small, but can lead to a massive release of all the water in the containment area once erosion commences. This may lead to massive failure along the slopes of the waste rock and tailings facilities.

As for storage on the benches, we recommend careful review of potential failure mechanisms. For example: Would it be possible for water to seep into the slope, eventually resulting in internal erosion and eventual failure of the slope? Such an erosion event can act in the same way as outlined in the previous paragraph, leading to a massive release of the water stored on the bench.

6.0 SEDIMENT CONTROLS AND YIELD
Goal: Review the sediment control design and sediment yield calculations and comment on the short- and long-term functional viability of the sediment control system and the applicability of the sediment yield calculations.

6.1 Sediment Yield Calculation Methodology
The method used for the calculation of sediment yield for the site is the Pacific Southwest Inter-Agency Committee (PSIAC) method. This method was developed in 1968 in Southern California and is recommended for basins that are larger than 10 mi$^2$ in size. The baseline and post-mining scenarios analyzed have basin areas of 8.20 mi$^2$ and 1.93 mi$^2$ respectively. Therefore, Golder recommends that the sediment yield calculations be evaluated using a method that is more appropriate for this site.
Additionally, Golder has concerns with the results of the sediment yield calculations. Both baseline and post-mining conditions give the average-annual specific sediment yield as 1.15 acre-feet/mi$^2$/year. It is reasonable to expect that the baseline scenario will differ from the post-mining scenario because the addition of the landform will change the surface conditions. Currently no difference is indicated by the analysis results provided by TetraTech.

Golder produced a report *Rosemont Mine Landforming – Evaluation of Mine Waste Slope Geometry* dated February 17, 2010 wherein it was estimated that the expected erosion from the Rosemont landform surface prior to stabilization will be 14.4 inches. It is anticipated that large amounts of this sediment will report to all areas where water will be ponded. This will therefore reduce the storage capacity of the bench storage areas and perimeter containment areas. Allowance for such storage loss should be made.

### 6.2 Sediment Control during Operations

The report states that BMPs will be used during operations to manage sediment on the site; however, no specific definitions are described as to the locations and phasing of these sediment controls during operations. The report also calls for concurrent reclamation, which is very difficult in an arid climate. It is recommended that BMPs be defined and that reliance on concurrent reclamation be minimized.

### 7.0 LANDFORMING

Golder was not requested to comment on the landforming arrangement, but feels compelled to do so as we have developed and estimated the hydraulic and erosion performance of the elements that were used to develop the landforming shape. We recommend that TetraTech develop a table showing adherence to the recommendations previously made by Golder in this regard.

### 8.0 CONCLUSION

Golder has classified concerns into two categories: red flags and potential fatal flaws associated with the Site Water Management Update. Those findings are summarized in 3.
### TABLE 3
**RED FLAGS AND POTENTIAL FATAL FLAWS**

<table>
<thead>
<tr>
<th>Red Flags</th>
<th>Potential Fatal Flaw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Using smaller precipitation depth (18in) to calculate average annual runoff</td>
<td>Storage on top of benches is unusual for long-term closure and could lead to massive</td>
</tr>
<tr>
<td>instead of NRCS recommended depth (24in)</td>
<td>failure</td>
</tr>
<tr>
<td>No volume check calculations using maximum saturation event conditions</td>
<td>Down chutes on both tailings facility and waste rock can lead to failure as riprap</td>
</tr>
<tr>
<td>No calculations presented for pit diversion channel and pit stormwater pond</td>
<td>lining may be inappropriate</td>
</tr>
<tr>
<td>Methodology used for sediment yield calculations should be reviewed as it</td>
<td>Flow-through drains: potential long-term difficulties with maintenance and retaining</td>
</tr>
<tr>
<td>is believed to be inappropriate</td>
<td>discharge capacity</td>
</tr>
<tr>
<td>Lack of drainage from perimeter containment areas</td>
<td>Water storage on top of tailings facility and waste rock dump is unusual for long-term</td>
</tr>
<tr>
<td>Demonstrate adherence to geometric recommendations on landform element</td>
<td>closure and could lead to massive failure</td>
</tr>
<tr>
<td>suggestions previously proposed by Golder</td>
<td>No allowance has been made for anticipated erosion from landforms into storage locations</td>
</tr>
<tr>
<td>Lack of detail for sediment control designs during operations</td>
<td>on benches and perimeter containment areas. 14 to 15 inches of erosion is anticipated</td>
</tr>
<tr>
<td>Specific sediment yield is the same for pre- and post-mining conditions,</td>
<td>from the landform areas.</td>
</tr>
<tr>
<td>which appears to be incorrect</td>
<td></td>
</tr>
</tbody>
</table>
ATTACHMENT 2
ROSEMONT PIT DIVERSION CHANNEL DESIGN
(TETRA TECH, 2010K)
1.0 Introduction

This Technical Memorandum discusses the hydrologic and hydraulic calculations performed for the Pit Diversion Channel (Channel) as part of the site water management for the Open Pit area at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. The Channel is located along the western extents of the Open Pit area as shown on Figure 1 and will be constructed early in the life of the Project. The Channel will also incorporate a Pit Electrical Loop Road over a portion of the Channel alignment. This Channel will be a permanent structure and was designed to pass the Local Probable Maximum Precipitation (PMP) storm event from its contributing basins.

Should modifications occur to this design, the general stormwater analysis presented herein would still be applicable assuming the configuration of basin areas and channel lengths were comparable to those analyzed.

In general, the Pit Diversion Channel will route stormwater from basins west and south of the Open Pit into the upper part of the Barrel Canyon drainage. At the end of the Channel, a large multi-plate culvert will be constructed to pass storm flows into the canyon. The Channel design also incorporates a drop structure and a large fill area.

During the early years of the Project, storm runoff from the Channel will enter Barrel Canyon and flow, unimpeded, down the drainage. Over time, placement of waste rock in the Waste Rock Storage Area will confine the flows. The design of the stormwater management features associated with the Waste Rock Storage Area takes storm flows from the Pit Diversion Channel into account.

Perimeter Containment Basins (PCAs) are located between the toe of the Waste Rock Storage Area and a natural ridgeline. The PCAs are designed to manage stormwater generated by a General PMP event from the Waste Rock Storage Area and from the Pit Diversion Channel. Stormwater management details for the Waste Rock Storage Area are provided in the Technical Memorandum titled *Rosemont Waste Rock Storage Area Stormwater Management* (Tetra Tech, 2010c).

The design method for the Pit Diversion Channel was based on the Natural Resources Conservation Service (NRCS) curve number procedure to estimate the peak stormwater runoff. This method, along with Manning’s open-channel flow equation, was used to calculate the
proper channel sizing and to design the drop structure and culvert. The methods used for the analysis are discussed in greater detail in the Technical Memorandum titled *Rosemont Hydrology Method Justification* (Tetra Tech, 2010a).

### 2.0 Hydrologic Method Overview (NRCS Method)

The NRCS method was developed for general hydrologic analysis and allows for various storm distributions and durations to be analyzed. The method is applicable to the analysis of large complex watershed systems, such as mining sites, where landscape conditions may change over time. Therefore, the NRCS method was selected for this analysis based on the information available and on the expectations for the analysis.

The analysis was performed using HEC-HMS, a hydrologic modeling software package developed by the U.S. Army Corps of Engineer’s (USACE). HEC-HMS incorporates the NRCS method and allows for the analysis of complex/integrated systems (i.e., multiple sub-basins, reservoir, and channel routing, etc.).

The primary input variables for determining the peak flow associated with stormwater runoff are:

- Precipitation;
- Storm distribution;
- Curve number;
- Basin delineation or area; and
- Time of concentration or lag time.

HEC-HMS uses these parameters and the NRCS unit hydrograph to estimate runoff after appropriate losses (i.e., infiltration). It is also used to develop a specific basin’s relationship between runoff versus time, presented as a hydrograph curve. The area under a hydrograph curve represents the basin’s expected total runoff volume and the apex of the hydrograph curve represents the basin’s estimated peak flow rate.

The estimated values for peak runoff were used for the hydraulic design of the Pit Diversion Channel and its corresponding elements. These input variables are presented in the following sections and are discussed in greater detail in the Technical Memorandum titled *Rosemont Hydrology Method Justification* (Tetra Tech, 2010a).

#### 2.1 Precipitation

Precipitation for the PMP storm event was estimated utilizing the procedures outlined in the Hydrometeorological Report No. 49 (HMR 49) published by the National Oceanic Atmospheric Administration (NOAA). 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), which is northeast of the Pit Diversion Channel.
2.2 Rainfall Storm Distributions

The NRCS method allows for many precipitation patterns to be applied to the watershed. The storm events considered for estimating peak runoff were the 6-hour Local PMP Distribution and the 72-hour General PMP Distribution. The Technical Memorandum titled *Rosemont Hydrology Method Justification* (Tetra Tech, 2010a), stated that a Local PMP event generates less volume than a General PMP, but larger peak flows because of its shorter duration and correspondingly higher rainfall intensities. For the Project site, a Local PMP storm is estimated to generate 15.0 inches of rainfall in six (6) hours. The General PMP event is estimated to generate 18.9 inches of rainfall over 72 hours.

The Local PMP storm event was selected as the design event for sizing the Pit Diversion Channel. Table 1 summarizes the Local PMP storm event and the General PMP storm event.

<table>
<thead>
<tr>
<th>Table 1 Design Storms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
</tr>
<tr>
<td>Distribution</td>
</tr>
<tr>
<td>Duration (hours)</td>
</tr>
<tr>
<td>Precipitation (inches)</td>
</tr>
</tbody>
</table>

2.3 Rainfall Losses – Curve Number

The NRCS developed a curve number (CN) procedure for estimating runoff from storm events. The curve number procedure is incorporated in this analysis.

Rainfall losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors (curve numbers) that represent the runoff potential of a soil type (i.e., the higher the curve number, the higher the runoff potential).

The NRCS classifies soils as “A”, “B”, “C”, or “D”, based on their hydrologic soil group to determine the runoff potential. Type “A” soils, such as sandy soils, have a very low runoff potential. Type “D” soils, such as heavy clay and/or shallow, rocky soils, have a very high runoff potential. Soil groups at the Project site were determined from the NRCS Soil Survey Geographic Database (SSURGO) data set. The main soils present near the Pit Diversion Channel and within the Project site are of type B, C, and D, which are further defined in Table 2.
Table 2  Hydrologic Soil Groups

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>Description*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type B (Moderately low run off potential)</td>
<td>These soils have a moderate infiltration rate when thoroughly wetted. They chiefly are moderately deep to deep, moderately well drained to well drained, soils that have moderately fine to moderately coarse textures. They have a moderate rate of water transmission (4 to 8 mm/hr), and are generally described as silty loam, and loam.</td>
</tr>
<tr>
<td>Type C (Moderately high run off potential)</td>
<td>These soils have a slow infiltration rate when thoroughly wetted. They chiefly have a layer that impedes downward movement of water or have moderately fine to fine texture. They have a slow rate of water transmission (1 to 4 mm/hr), and are generally described as sandy clay loam.</td>
</tr>
<tr>
<td>Type D (High run off potential)</td>
<td>These soils have a very low infiltration rate when thoroughly wetted. They chiefly consist of clay soils that have a high swelling potential, soils that have a permanent high water table, soils that have a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. They have a very slow rate of water transmission (0 to 1 mm/hr), and are generally described as clay loam and silty clay loam.</td>
</tr>
</tbody>
</table>

* Descriptions obtained from Pima County Regional Flood Control District’s PC-Hydro User Guide.

The curve number for the Pit Diversion Channel’s contributing basins was calculated using GIS data layers available for the hydrologic soil groups within the area and land use layers obtained from the Pima County Department of Transportation: Geographic Information Services Division. Based on the land use layers, the entire watershed for the Pit Diversion Channel falls under the arid range/desert shrub categorization classified in fair condition.

A curve number of 85 was calculated for the Pit Diversion Channel’s contributing basins. The curve number is based upon soil types, land use, an Antecedent Moisture Condition (AMC), and the related initial abstraction of 0.35 inches. The AMC accounts for the preexisting moisture conditions of the soils before the storm event. The initial abstraction is the total amount of precipitation in inches that is infiltrated and absorbed into the soil before runoff begins. The initial abstraction is developed in the following sections.

2.4  Drainage Basin Delineations

Three (3) separate drainage basins, each delineated with their unique contributing watershed, were analyzed as part of the design. These areas are shown on Figure 1.

2.5  Rainfall Runoff Excess

The NRCS method estimates rainfall runoff excess as a function of cumulative precipitation, soil cover, land use, and AMC using the following relationships:

\[
P_e = \frac{(P - I_a)^2}{P - I_a + S}
\]

\[
S = \frac{1000}{CN} - 10
\]
Where:

- $Pe =$ the accumulated precipitation excess, in inches;
- $P =$ the accumulated precipitation depth, in inches (Local PMP storm event);
- $S =$ the maximum soil water retention parameter, in inches;
- $Ia =$ the initial abstraction, in inches; and
- $CN =$ the curve number.

From an analysis of results for many small experimental watersheds, NRCS developed an empirical relationship between the initial abstraction and the maximum soil water retention parameter given by the following expression:

$$Ia = 0.2 \times S$$

Therefore, the determination of accumulated precipitation excess can be rewritten as:

$$Pe = \frac{(P - 0.2S)^2}{P + 0.8S}$$

### 2.6 Time of Concentration / Lag Time

The time of concentration ($Tc$) is the travel time for a flood-wave to travel from the hydraulically most distant point in the basin to the outlet during a period of the most intense rainfall excess. The time of concentration was determined by considering the most hydraulically distant flow path for each basin.

HEC-HMS requires a lag time that is equal to $0.6 \times Tc$. The lag time equation was developed from agricultural watershed data and has been adapted to small urban basins, less than 2000 acres in size, where the equation is defined as:

$$Lg = \frac{L^{0.8} (S + 1)^{0.7}}{1900 y^{0.5}}$$

Where:

- $Lg =$ the lag time, in hours;
- $L =$ the hydraulic length or the distance of the longest watercourse in the watershed, in feet;
- $y =$ the average watershed slope, in percent; and
- $S =$ the maximum soil water retention parameter in terms of the curve number noted before, in inches.

The unique characteristics required to determine each basin’s lag time were developed by utilizing Autodesk Land Desktop 2009. The lag times were then used in HEC-HMS to establish
the peak flows as the basis for the Channel design. Table 3 presents the parameters used to calculate the lag time.

<table>
<thead>
<tr>
<th>Basin ID</th>
<th>Basin Area (mi²)</th>
<th>Length (ft)</th>
<th>Slope (%)</th>
<th>CN</th>
<th>Lg (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PD-1</td>
<td>0.2029</td>
<td>5,009</td>
<td>51.55</td>
<td>85</td>
<td>8.171</td>
</tr>
<tr>
<td>PD-2</td>
<td>0.0474</td>
<td>3,436</td>
<td>58.00</td>
<td>85</td>
<td>5.697</td>
</tr>
<tr>
<td>PD-3</td>
<td>0.0907</td>
<td>4,806</td>
<td>61.00</td>
<td>85</td>
<td>7.267</td>
</tr>
</tbody>
</table>

* Parameters obtained from Autodesk Land Desktop 2009

2.7 Routing Method

In general, the Pit Diversion Channel is designed with an upper channel section sloped at 1% before entering the drop structure sloped at 25%. The lower channel section returns to the 1% slope after the drop structure and its outlet apron. The channel then widens out over the fill area before entering into the culvert.

Channel routing is the term applied to the NRCS method to account for the effects of channel storage on the runoff hydrograph as the storm event moves through a channel reach. Channel routing is used to translate and attenuate an upstream runoff hydrograph into a downstream hydrograph during the specific design storm event. The routing procedure has two (2) components – the routing method and the physical channel characteristics. The drop section of the Channel is sloped at 25% and the Kinematic Wave method was employed for routing of this section. The Muskingum-Cunge method was used for the routing of the remaining channel sections which are sloped at 1%.

The Muskingum-Cunge method was selected over the Kinematic Wave method for the flatter portions since it is intended for slopes less than 10%. The main advantage of these routing methods is that the relative coefficients are evaluated from physical channel characteristics and can be determined without existing flood hydrograph data. The physical channel characteristics used in the routing are presented in Table 4.

<table>
<thead>
<tr>
<th>Reach ID</th>
<th>Length (ft)</th>
<th>Slope (%)</th>
<th>Manning’s n</th>
<th>Shape</th>
<th>Bottom Width (ft)</th>
<th>Side Slope (H:V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reach-1</td>
<td>1,098</td>
<td>1.0</td>
<td>0.035</td>
<td>Trapezoidal</td>
<td>20</td>
<td>2:1</td>
</tr>
<tr>
<td>Reach-2</td>
<td>1,858</td>
<td>1.0</td>
<td>0.035</td>
<td>Trapezoidal</td>
<td>20</td>
<td>2:1</td>
</tr>
<tr>
<td>Reach-3</td>
<td>1,300</td>
<td>25.0</td>
<td>0.028</td>
<td>Trapezoidal</td>
<td>20</td>
<td>2:1</td>
</tr>
</tbody>
</table>

* Parameters obtained from Autodesk Land Desktop 2009

As shown on Figure 1, the channel section downstream of Junction-3 and upstream of Junction-2 was broken into two (2) reaches, where Reach-3 corresponds to the Pit Diversion Channel’s drop structure starting at Junction-3 and Reach-2 represents the typical lower channel section sloped at 1% after the drop structure. Although the Pit Diversion Channel widens downstream of Junction-2, the typical lower channel section was conservatively used to analyze Reach-1.
3.0 Hydraulic Method Overview (Manning’s Formula)

The hydraulic analysis used Manning’s open-channel flow equation, with cumulative peak flow rates, to size the sections of the Pit Diversion Channel. Manning’s formula for open-channel flow is as follows:

\[ Q = \frac{1.486A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}}{n} \]

Where:
- **Q** = The channel flow rate, in cubic feet per second (cfs);
- **A** = The cross sectional area of flow, in square feet (ft²);
- **R** = The hydraulic radius of flow, in feet;
- **S** = The longitudinal slope of the flow path for the channel, in feet/foot; and
- **n** = Manning’s roughness coefficient for the channel, unitless.

The following equations for the area and the hydraulic radius of the flow were obtained from the properties of triangles. The depth of flow was checked using these relationships to verify that the specific channel section was properly designed for safety:

\[ A = \frac{1}{2} \left( m_1 y^2 + m_2 y^2 \right) \]
\[ R = \frac{1}{2} \left( \sqrt{m_1^2 y^2 + y^2} + \sqrt{m_2^2 y^2 + y^2} \right) \]

Where:
- **A** = the cross sectional area of flow, in square feet (ft²);
- **R** = the hydraulic radius of flow, in feet;
- **m₁** = the side slope of the channel;
- **m₂** = the side slope of the channel; and
- **y** = the depth of flow, in feet.

4.0 Hydrologic and Hydraulic Analysis Results

Table 5 summarizes the cumulative peak runoff results from the hydrologic analysis using the 6-hour Local PMP storm event for the design of Pit Diversion Channel.
Table 5  Summary of Pit Diversion Channel Analysis

<table>
<thead>
<tr>
<th>Concentration ID</th>
<th>Peak Flow Rate (cfs)</th>
<th>Maximum Flow Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Junction-1</td>
<td>4,899</td>
<td>7.47</td>
</tr>
<tr>
<td>Junction-2</td>
<td>1,971</td>
<td>5.82</td>
</tr>
<tr>
<td>Junction-3</td>
<td>1,348</td>
<td>4.76</td>
</tr>
</tbody>
</table>

In general, the upper Channel section, upstream of Junction-3, is designed to convey the peak flow with:

- A bottom width of 20 feet;
- A depth of 7.5 feet;
- 2H:1V (Horizontal to Vertical) side-slopes; and
- Sloped at one (1) percent (%).

A cross section of the upper Channel section is shown on Figure 2, Section A.

The maximum depth of the peak flow is expected to be about five (5) feet in the Channel prior to entering the drop structure at Junction-3. This allows for about 2.5 feet of freeboard in the current preliminary design shown on Figures 1 and 2. Channel dimensions may be modified to achieve a minimum freeboard of one (1) foot.

Similarly, the Channel’s drop structure after Junction-3 is designed to convey the peak flow with:

- A bottom width of 20 feet;
- A depth of 7.5 feet;
- 2H:1V side-slopes; and
- Sloped at 25%.

A cross section of the drop structure is shown on Figure 2, Section B.

The maximum depth of the peak flow is expected to be about two (2) feet in the drop structure. This allows for about 5.5 feet of freeboard in the current preliminary design shown on Figures 1 and 2.

This increases the flow velocity and decreases the flow depth in the drop structure. The drop structure terminates with an outlet apron with:

- A bottom width of 20 feet;
- A bottom length of 85 feet;
- A depth of 11.5 feet;
- 2H:1V side-slopes; and
• Sloped at 0%.

The outlet apron is designed to dissipate the flow velocity after the drop structure.

After the outlet apron, the lower Channel section, upstream of Junction-2, is similar to the upper Channel section with:

• A bottom width of 20 feet;
• A depth of 8.5 feet;
• 2H:1V (Horizontal to Vertical) side-slopes; and
• Sloped at 1%.

A cross section of the lower Channel section is shown on Figure 2, Section C.

The maximum depth of the peak flow for this Channel section is about six (6) feet, with 2.5 feet for freeboard near Junction-2 based on the preliminary design.

Just prior to Junction-2 and leading to Junction-1, the lower Channel section widens to:

• A bottom width of 30 feet;
• 3H:1V side-slope toward the downstream side of the compacted fill embankment;
• A 1% side-slope swale on the upstream side of the fill;
• A depth of 9.5 feet; and
• Sloped at 1%.

A cross section of the lower Channel section between Junction-2 and Junction 1 is shown on Figure 2, Section D.

The maximum depth of the peak flow for this Channel section is about 3.5 feet, with six (6) feet for freeboard based on the preliminary design. The section is designed to convey the expected increase in cumulative peak flow.

Before entering into the culvert located near Junction-1, the Channel transitions back to a trapezoidal configuration with:

• A bottom width of 30 feet;
• A depth of 9.5 feet;
• 3H:1V side-slopes; and
• Sloped at 1%.

A cross section of the Channel near Junction-1 is shown on Figure 2, Section E.
The maximum depth of the peak flow at Junction-1 just prior to entering into the culvert is estimated to be about 7.5 feet, which allows for about two (2) feet of freeboard based on the preliminary design to convey the peak flow into the culvert inlet.

### 4.1 Pit Diversion Drop Structure

The Pit Diversion Channel’s drop structure located downstream of Junction-3 is about 1,300 feet in length with:

- A bottom width of 20 feet;
- A depth of 7.5 feet;
- 2H:1V side-slopes; and
- Sloped at 25%.

The Channel transitions to a flat outlet apron at the base of the drop structure that is about 100 feet in length with:

- A bottom width of 20 feet;
- A depth of 11.5 feet; and
- 2H:1V side-slopes.

The drop structure will be armored to protect the structure from the potential for erosion and to attenuate the energy generated from the peak storm event. The armor will consist of the following from bottom to top:

- A prepared subgrade;
- A geotextile;
- A minimum of six (6) inches of angular drainage rock;
- A geogrid for additional shear strength; and
- The entire drop structure, including the bottom and side-slopes, will be lined with Contech ArmorFlex® articulated concrete block that is connected in sheets by steel cable.

The outlet apron will be constructed in the same manner, with more robust Contech Ajacks® concrete block armoring over about three (3) inches of drainage rock for the energy dissipation. Attachment 1 provides recommendations to achieve proper erosion control for the drop structure design.
4.2 Pit Diversion Culvert

The final discharge point of the Pit Diversion Channel is at Junction-1 and is constrained by a natural ridge. A steel, low profile, arch-shaped, 180 feet long, multi-plate culvert will be installed at this location to pass storm flows into the upper portion of Barrel Canyon. The design of the steel culvert is a low profile arch shape that is:

- 16.5 feet tall;
- 29.5 feet wide; and
- sloped at 3%.

The culvert was hydraulically analyzed to determine its capacity to safely convey the peak flow and to verify that the inlet and outlet controls are properly designed. Attachment 2 contains the corresponding calculations and recommendations for the culvert design.

The existing natural ridge will be excavated along the culvert’s alignment with a bottom width of about 40 feet and side-slopes at angle-of-repose. The culvert will be installed on the following prepared foundation (from bottom to top):

- A prepared subgrade;
- A four (4) foot thick run-of-mine (ROM) riprap layer across the width of the excavation; and
- A loose soil cushion layer will be placed between the riprap and the culvert.

The culvert will then be installed along its alignment with recompacted fill material placed around and over the culvert to a maximum cover of five (5) feet according to the manufacturer’s guidelines. An access road will pass over this culvert location.

The inlet of the culvert will be armored with about a four (4) foot thick and 20 foot long layer of ROM riprap for erosion protection and for energy dissipation against the anticipated scouring velocities from the peak storm events. The inlet will be constructed with wingwalls designed to funnel the peak flow into the culvert.

The outlet of the culvert will be armored with about a four (4) foot thick and 100 foot long layer of ROM riprap and will be constructed with wingwalls designed for erosion protection and energy dissipation.

As an alternative, the multi-plate culvert may be replaced with a channel cut and a light vehicle bridge.

5.0 Conclusion

Based on the analysis and the preliminary design results discussed herein, the Pit Diversion Channel, drop structure, and culvert are capable of safely conveying the estimated stormwater runoff generated from the Local PMP design storm event.
6.0 References


ATTACHMENT 1
DROP STRUCTURE RECOMMENDATIONS
Rich Shelton  
Mine Market Manager  

March 31, 2010  

Gregory Hemmen, P.E.  
Tetra Tech  
1750 SW Harbor Way, Suite 400 | Portland, OR 97201  

Subject: Rosemont Channel – Armorflex Articulated Concrete Block  

Greg;  

Thank you for your continued interest in CONTECH Construction Products mine solutions. The following information and attached documents is in response to your inquiry regarding channel protection for Rosemont Mine in the Tucson, AZ area.  

The information provided is preliminary, additional information will be required for final sizing/selection of products as well as quantities. The quote at the end is preliminary only and does not constitute a price offering by Contech. The quote would be similar to what a contractor would see if it were to bid today and does not include any material escalators (price of concrete rising/falling in the next year nor does it account for fluctuation in steel prices).  

There will be some additional work and discussion that needs to happen between Tetra Tech and CONTECH but based on what you provided in our discussion on 3.30.2010, the following should be fairly close to accurate.  

Should you have additional comments or questions, do not hesitate to contact me.  

Sincerely,  

Richard Shelton  
Mine Market Manager  

Cc: Clayton Fawcett  
Keith Brooks  
Gene Zande
Drop 1

We worked off the following information to determine the proper block size:

- 1,500 cfs
- 25% bed slope
- 2:1 Side Slopes
- 26’ bed width (request was for 20’, however, 26’ is best we can do, currently)

Based on this information we are pleased to offer our Class 70 Tapered Articulated Concrete Block System (ACB) (please reference table below for pricing and approximate quantities).

Please note that the quantities are approximate based off preliminary information provided. We can also provide design calculations for the Armoflex should you require that information. Depth of channel needs to be discussed further. Channel alignment on Armorwedge bears further discussion.

When you review the table note that the acronyms are meant to read as follows 70 T = Armorflex Class 70 Tapered, and AW = ArmorWedge

*All quantities and material costs are estimates. Price reflects material costs only (no installation). Kindly pay particular attention to Note 4 beneath the table.

<table>
<thead>
<tr>
<th>Bottom width in feet</th>
<th>L Side Slope 2:1 w/ turndown – assume 7’ depth</th>
<th>R Side Slope @ 2:1 w/ turn down – assume 7’ depth</th>
<th>Total length of mat</th>
<th>Channel Length</th>
<th>Total sf</th>
<th>Block Class</th>
<th>~ SF Price Delivered</th>
<th>~ total price</th>
<th>Factor of Safety (FOS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>16</td>
<td>16</td>
<td>58</td>
<td>1,300</td>
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<td>70T</td>
<td>$x.xx</td>
<td>$xxx.xxx</td>
<td>1.5</td>
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<td>AW</td>
<td>$x.xx</td>
<td>$xxx.xxx</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Note 1: The Tapered and ArmorWedge block offered above is with the ACB, site specific fabric, delivery and Tensar BX 1100. We have not priced the cost of the 6” drainage medium underneath the block.

Note 2: Once channel subgrade is prepped, assume 5,000 – 12,000 square feet installed per day for the tapered Armorflex and 1,500-2,000 square feet per day for the ArmorWedge (which is handplaced only).

Note 3: Armorflex brochure follows this proposal with all dimensions (in email, as a separate attachment).

Note 4: Bottom width 22’ (line #2 of the table, is based upon the actual discharge cfs of 1,350 as stated in our conversation on 3.30.2010). Bottom width of actual discharge reduces bottom width of channel but did not change the sizing of the block.
STILLING BASINS

Previous data was supplied on 2.9.2010 to Ronson Chee of your Tucson office. We would be pleased to continue this conversation regarding various solutions for energy dissipation at the site.

Sample photos and projects:

Class 50 Tapered Block in a copper mine in Arizona (confidentiality agreements prevent from naming the mine)

Armorwedge in a steep application – handles up to 43 fps
Armorflex to Ajax transition  
24" Ajax installed  
24" Ajax transition to rip rap  
* note the concrete wedge  
* Sits on 3" drainage medium  
* 4500 cfs at the transition point  
* Dissipates energy

Brochure Details (4 page) – separate brochure to follow via email
Sequencing sheet that a contractor will receive

<table>
<thead>
<tr>
<th>Mat #</th>
<th>Mat Installation Sequence</th>
<th>Unit Wt (lbs/sq.ft.)</th>
<th>Mat Length max (ft.)</th>
<th>Mat Width max (ft.)</th>
<th>Mat Width min (ft.)</th>
<th>Mat Weight (lbs)</th>
<th>Total Mat Coverage (sq.ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
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<td>20</td>
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Additional information for your bid documents and/or contractor.
Armortec Product Details

- **ArmorWedge**
- **ArmorRoad**
- **ArmorFlex - Open Cell**
- **A-Jacks**
- **ArmorStone**
- **ArmorLoc**
- **ArmorFlex - Close Cell**
- **ArmorFlex OS**

MANUFACTURING SPECIFICATION
ASTM D6684-04
**ArmorFlex®** (not to scale)

![Diagram of different block types]

**Top of Slope - Standard Detail**

**ArmorFlex Unit Specification**

<table>
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<th>Concrete Block Class</th>
<th>Open/Closed Cell</th>
<th>Nominal Dimensions (inches)</th>
<th>Gross Area/Unit (sq. ft.)</th>
<th>Block Weight (lbs)</th>
<th>lbs/sq. ft</th>
<th>Open Area %</th>
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**High Velocity Application Block Classes**

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### A-Jacks® Unit Specification

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### ArmorWedge® Unit Specification

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<th>UNIT WEIGHT (LBS)</th>
<th>SYSTEM WEIGHT (LBS)</th>
<th>UNIT COVERAGE (SF)</th>
<th>COMpressive STRENGTH (PSI)</th>
<th>MAXIMUM ABSORPTION (LBS/FT³)</th>
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### ArmorStone® Unit Specification

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<th>WEIGHT LBS PER UNIT</th>
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### Armortec Minimum Physical Requirements per ASTM 06684-04

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Hydraulic Analysis

Initial Project Parameters:

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Project Summary:

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Modified from Julien (1995)  
Analysis Date: 3/30/2010
Hydraulic Analysis

Determining n-Value:

Trapezoidal Channel:  
\[ Q_{unit} = \frac{Q}{(b + Z_L \times \text{depth} + Z_R \times \text{depth})} = 46.274 \]

Tapered Units:

If \( n < 0.02 \) then \( n = 0.02 \)

\[ n = 0.0202 \times Q_{unit}^{0.305} \times S^{0.488} = 0.033 \]

Non-Tapered Units:

If \( n < 0.032 \) then \( n = 0.032 \)

\[ n = 0.036 \times Q_{unit}^{0.305} \times S^{0.488} = 0.059 \]

Manning’s Equation:

Area = \( A_L + A_B + A_R \)

\[ A_L = \frac{1}{2} \times \text{Depth}^2 \times Z_L = 1.6 \text{ ft}^2 \]
\[ A_B = \text{Channel Bottom Width} \times \text{Depth} = 27.9 \text{ ft}^2 \]
\[ A_R = \frac{1}{2} \times \text{Depth}^2 \times Z_R = 1.6 \text{ ft}^2 \]

\[ \text{Area} = 31.2 \text{ ft}^2 \]

Wetted Perimeter = \( P_L + P_B + P_R \)

\[ P_L = \text{Depth} \times (Z_L^2 + 1)^{0.5} = 2.8 \text{ ft} \]
\[ P_B = \text{Channel Bottom Width} = 22.0 \text{ ft} \]
\[ P_R = \text{Depth} \times (Z_R^2 + 1)^{0.5} = 2.8 \text{ ft} \]

\[ \text{Wetted Perimeter} = 27.7 \text{ ft} \]

Hydraulic Radius = \( \frac{\text{Area}}{\text{Perimeter}} \)

\[ \text{Hydraulic Radius} = 1.1 \text{ ft} \]

Flow = \( 1.486 / n \times \text{Area} \times \text{Radius}^{2/3} \times \text{Slope}^{1/2} = 1253.1 \text{ cfs} \)

- Velocity = \( \frac{\text{Flow}}{\text{Area}} = 40.21 \text{ fps} \)

- Froude = \( \frac{\text{Velocity}}{\text{(Gravity} \times \text{Depth})^{1/2}} = 6.3 \)

Modified from Julien (1995)  
Analysis Date: 3/30/2010
Hydraulic Analysis

The following illustrates the use of the factor of safety method in the selection of block sizes for ACB's for revetment or bed armor. The following assumes that hydraulic testing has been performed for the block system to quantify a critical shear stress in the hydraulic situation. The flow depth, velocity, and site-generated shear stress were determined by the Manning's Equation.

Given: 70T

<table>
<thead>
<tr>
<th>Unit Parameters</th>
<th>Hydraulic Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_1 = 0.375$</td>
<td>$W = \text{Weight} = 128.3$ lbs</td>
</tr>
<tr>
<td>$\phi_2 = 0.971$</td>
<td>$b = \text{Block Width} = 1.292$ ft</td>
</tr>
<tr>
<td>$\phi_3 = 0.6$</td>
<td>$\tau_c = \text{Critical Shear} = 46.5$ psf</td>
</tr>
<tr>
<td>$\phi_4 = 0.971$</td>
<td>$S_c = \text{Sp. Gr. of Con.} = 2.1$</td>
</tr>
<tr>
<td>$\Delta Z = \text{Projection} = 0$ in</td>
<td>$S = \text{Bed Slope} = 0.25$</td>
</tr>
<tr>
<td>$\phi_1 = 0.375$</td>
<td>$W = \text{Weight} = 128.3$ lbs</td>
</tr>
<tr>
<td>$\phi_2 = 0.971$</td>
<td>$b = \text{Block Width} = 1.292$ ft</td>
</tr>
<tr>
<td>$\phi_3 = 0.6$</td>
<td>$\tau_c = \text{Critical Shear} = 46.5$ psf</td>
</tr>
<tr>
<td>$\phi_4 = 0.971$</td>
<td>$S_c = \text{Sp. Gr. of Con.} = 2.1$</td>
</tr>
</tbody>
</table>

Step 1: Compute Factor of Safety Parameters

$\theta_1 = 26.57^\circ$ (Angle for Side Slope) $\theta_0 = 14.0^\circ$ (Angle for Bed Slope)

$W_s = W * ((S_c - 1) / S_c) = 67.2$ lbs (Submerged Weight)

$\tau_o = K_b \gamma y \sin (\tan^{-1} S_o) = 19.22$ lbs/ft$^2$ (Design Shear Stress)

$\eta_o = \tau_o / \tau_c = 0.41$ (Stability Number for Horizontal Surface)

$a_0 = (\cos^2 \theta_1 - \sin^2 \theta_0)^{1/2} = 0.86^\circ$

$\theta = \arctan ((\sin \theta_0 \cos \theta_1) / (\sin \theta_1 \cos \theta_0)) = 26.6^\circ$

$\beta = \arctan ((\cos (\theta_0 + \theta)) / ((\theta_4 / \theta_3 + 1)) * (1 - a_0^2)^{1/2} / (\eta_o * \phi_2 / \phi_1)) + \sin (\theta_0 + \theta)) = 21.83^\circ$

$\eta_1 = ((\phi_4 / \phi_3 + \sin (\theta_0 + \theta + \beta)) / (\theta_4 / \theta_3 + 1)) * \eta_o = 0.40$ (Stability Number for Slope Surface)

$\delta = 90^\circ - \beta - \theta = 41.60^\circ$

Step 2: Consider Effects for Specified Projection

$F_L = F_D = 0.5 \Delta Z b \rho V_{des}^2 = 0.00$ lbs (Lift & Drag Forces)

Step 3: Compute Factor of Safety

$SF = (\phi_2 / \phi_1 * a_0) / ((1 - a_0^2)^{1/2} * \cos \beta + \eta_1 * (\phi_2 / \phi_4) + (\phi_3 * F_D * \cos \delta + \phi_4 * F_L) / (\phi_1 * W_s)) = 1.5$

Modified from Julien (1995)  
Analysis Date: 3/30/2010
Hydraulic Analysis

Detailed Calculations

If $H = \text{horizontal component of side slope}$, then $\theta_1 = \tan^{-1}(1/H)$

If $S = \text{bed slope}$, then $\theta_0 = \tan^{-1}(S)$

**For $\tau_o$:**

\[
\tan^{-1} S_o = 14.04 \quad \text{sin (tan}^{-1} S_o) = 0.243
\]

**For $a_0$:**

\[
\begin{align*}
\cos \theta_1 &= 0.894 \quad \cos^2 \theta_1 = 0.800 \\
\sin \theta_0 &= 0.243 \quad \sin^2 \theta_0 = 0.059
\end{align*}
\]

**For $\theta$:**

\[
\begin{align*}
\sin \theta_0 \cos \theta_1 &= 0.217 \quad (\sin \theta_0 \cos \theta_1) / (\sin \theta_1 \cos \theta_0) = 0.500 \\
\sin \theta_1 &= 0.447 \\
\cos \theta_0 &= 0.970 \\
\sin \theta_1 \cos \theta_0 &= 0.434
\end{align*}
\]

**For $\beta$:**

\[
\begin{align*}
\cos (\theta_0 + \theta) &= 0.759 \quad (\theta_4 / \theta_3 + 1) * (1 - a_0^2)^{1/2} / (\eta_1 * \theta_2 / \theta_1) = 1.245 \\
\sin (\theta_0 + \theta + \beta) &= 0.886 \quad (\theta_4 / \theta_3 + 1) * (1 - a_0^2)^{1/2} / (\eta_1 * \theta_2 / \theta_1) + \sin (\theta_0 + \theta) = 1.895 \\
\sin (\theta_0 + \theta) &= 0.651
\end{align*}
\]

**For $\eta_1$:**

\[
\begin{align*}
\theta_4 / \theta_3 &= 1.618 \quad (\theta_4 / \theta_3) = 2.505 \\
\sin (\theta_0 + \theta + \beta) &= 0.886 \quad (\theta_4 / \theta_3) + (\sin (\theta_0 + \theta + \beta)) / (\theta_4 / \theta_3 + 1) = 0.957 \\
\theta_4 / \theta_3 + 1 &= 2.618
\end{align*}
\]

**For $F_L = F_D$:**

\[
\rho = 1.940 \quad \text{slugs/ft}^3
\]

**For SF:**

\[
\begin{align*}
\theta_3 / \theta_1 * a_0 &= 2.229 \quad (\theta_3 * F_D * \cos \delta + \theta_4 * F_L) / (\theta_1 * W_s) = 0.000 \\
(1 - a_0^2)^{1/2} * \cos \beta &= 0.472 \quad (1 - a_0^2)^{1/2} * \cos \beta + \eta_1 * (\theta_4 / \theta_3) + (\theta_4 * F_D * \cos \delta + \theta_2 * F_L) / (\theta_1 * W_s) = 1.496 \\
\eta_1 * (\theta_2 / \theta_3) &= 1.024 \\
\cos \delta &= 0.748 \\
\theta_3 * F_D * \cos \delta + \theta_1 * F_L &= 0.000 \\
\theta_1 * W_s &= 25.200
\end{align*}
\]

Modified from Julien (1995)  
Analysis Date: 3/30/2010
### Parameters for Factor of Safety Calculations

<table>
<thead>
<tr>
<th>Block Class</th>
<th>Submerged Weight (lbs)</th>
<th>$\theta_1$ (ft)</th>
<th>$\theta_2$ (ft)</th>
<th>$\theta_3$ (ft)</th>
<th>$\theta_4$ (ft)</th>
<th>$\tau_c$ (psf) 0 Degrees</th>
<th>Width (ft)</th>
<th>Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40T</td>
<td>35.500</td>
<td>0.198</td>
<td>0.971</td>
<td>0.317</td>
<td>0.971</td>
<td>31.800</td>
<td>1.292</td>
<td>67.773</td>
</tr>
<tr>
<td>50T</td>
<td>44.800</td>
<td>0.250</td>
<td>0.971</td>
<td>0.400</td>
<td>0.971</td>
<td>36.900</td>
<td>1.292</td>
<td>85.527</td>
</tr>
<tr>
<td>60T</td>
<td>56.000</td>
<td>0.313</td>
<td>0.971</td>
<td>0.500</td>
<td>0.971</td>
<td>42.100</td>
<td>1.292</td>
<td>106.909</td>
</tr>
<tr>
<td>70T</td>
<td>67.200</td>
<td>0.375</td>
<td>0.971</td>
<td>0.600</td>
<td>0.971</td>
<td>46.500</td>
<td>1.292</td>
<td>128.291</td>
</tr>
</tbody>
</table>

Modified from Julien (1995)  
Analysis Date: 3/30/2010
LOW PROFILE ARC SHAPE: 99A24-24

\[ Q_{\text{out channel}} = 4899 \text{ cfs} \quad A_{\text{channel}} = 377.6 \text{ ft}^2 \quad \frac{V^2}{2g} = 168.3259 = 2.62 \quad y_c = 7.28 \]

\[ L_{\text{culvert}} = 130 \text{ ft} \quad \text{Slope} = 0.03 \text{ ft/ft} \quad E_c = 9.90 \quad \text{minimum specific energy} \]

\[ A_{\text{culvert}} = 16 \text{ ft} \quad \text{(allowable headwater)} \]

\[ \text{Culvert span} = 27.08 \text{ ft} \quad \text{Aluminum Box Culvert type 81 A6 with square edge in a headwall} \]

\[ \text{Culvert rise} = 16.42 \text{ ft} \quad \text{(d)} \]

\[ \text{Culvert Area} = 412 \text{ ft}^2 \quad \text{(A)} \]

Assuming inlet control submerged and orifice flow, the governing hydraulic equation is the orifice-flow equation given as:

\[ Q = C_d A_o \sqrt{2g (HW)} \]

Where:

\( Q = \text{flow discharge} \)
\( C_d = \text{coefficient of discharge} \)
\( A_o = \text{Cross-sectional area of inlet} \)
\( HW = \text{head on the inlet invert of the culvert} \)

Checking for inlet submerge: 

\[ \frac{Q}{A_d^{0.5}} = 2.93 < 4 \quad \text{Thus unsubmerged culvert equation should be used:} \]

\[ \frac{HW}{d} = E_c + K \left[ \frac{Q}{A_d^{0.5}} \right]^M - 0.5S \]

where: \( K, M = \text{Constants for different types of inlets.} \)

For a arc culvert mitered to slope: \( K = 0.03 \quad M = 1 \)

\[ \text{HW} = 11.09 \text{ ft} \quad < 16 \text{ o.k.} \]

Checking outlet: \( n = 0.024 \quad \text{corrugated storm drain} \)

Downstream trapezoidal channel:

\[ \text{bottom width} = 30 \text{ ft} \quad \text{side slopes 3H:1V} \]

\[ \text{Slope} = \]
TAILWATER CALCULATION:

Q 4899 cfs
n 0.035 Excavated channel-smooth and uniform rock cuts (normal)

\[ A = \frac{Q \cdot n}{1.49 \cdot R^{\frac{3}{2}} \cdot S^2} = y(b + my) \]

\[ R = \frac{A}{P} = \frac{y(b + my)}{b + 2y(1 + m^2)^{\frac{1}{2}}} \]

Critical depth \( y_c \) 7.28 ft
Flow velocity: \( V = 12.84 \text{ ft/s} \)
\( S = 0.03 \text{ ft} \)
\( L = 180 \text{ ft} \)
\( A = 412 \text{ ft}^2 \)

TW = \( \frac{(y_c + d)}{2} = 11.85 \text{ ft} \)

Substituting the previous value in the tailwater equation:

\[ HW = TW - S\phi L + (1 + K_e + \frac{L}{4R}) \frac{Q^2}{2gA^2} \]

where:
\[ f = \frac{L}{4R} = \frac{2gA^2 L}{1.49 \cdot R^{\frac{5}{2}}} \]

Where:
\( K_e = \) entrance loss coefficient = 0.7 for an arch, corrugated metal mitered t conform fill slope
\( F = \) Darcy-Weisbach friction factor
\( R = \) full flow hydraulic radius
\( A = \) culvert cross section

HW = 10.80 ft < 11.09 ft (inlet control head)
SUPER-SPAN™ and SUPER-PLATE®

**Over 4000 SUPER-SPANS in Place**

Since 1967, more than 4,000 structures have been built on five continents. That makes SUPER-SPAN the most widely accepted, long-span, corrugated steel design in the world.

SUPER-SPAN structures with individual spans up to 50 feet are serving as bridges, railroad overpasses, stream enclosures, vehicular tunnels, culverts, and conveyor conduits. Installations have involved almost every job condition possible, including severe weather and unusual construction time constraints.

**National specification**

SUPER-SPAN’s popularity has resulted in a national specification written for long-span, corrugated metal structures by the American Association of State Highway and Transportation Officials. A.A.S.H.T.O. Standard Specifications (Section 12.7) for Highway Bridges provide for the selection of acceptable combinations of plate thickness, minimum cover requirements, plate radius and other design factors. Material is covered by A.A.S.H.T.O. M 167 AND ASTM A 761. Installation is covered by A.A.S.H.T.O. standard specification for highway bridges (Sec. 12) and ASTM A 761.

**Acceptance**

Many state and federal agencies recognize the excellent performance and economy of SUPER-SPAN corrugated structures. In a 1979 memorandum, the chief of FHWA’s Bridge Division noted that in the previous 15 years, several hundred CONTECH SUPER-SPAN Culverts had been erected in the United States and Canada and their performance had been excellent.

In a 1983 report to the Secretary of Transportation, the General Accounting Office stated, “Some innovations, such as using certain long-span culverts rather than building conventional bridges, have substantially lowered bridge costs.”

**Aluminum Long-Span structures (SUPER-PLATE)**

SUPER-PLATE structures add both longitudinal stiffeners (thrust beams) and circumferential stiffeners (reinforcing ribs) to conventional Aluminum Structural Plate to achieve larger sizes. Clear spans in excess of 30 feet and clear areas over 435 square feet are achievable with SUPER-PLATE. Available shapes include low-profile and high-profile arch and horizontal ellipse. Consult a CONTECH representative for additional information.

High-profile arch SUPER-SPAN (43'-3" span, 27' rise) in Hamilton, Ohio to span a wetland and to provide a wildlife crossing.
General design and installation characteristics

As conventional round structures increase in diameter beyond 16-18 feet, they become more difficult to install. It becomes increasingly difficult to both control the shape and to achieve good backfill support. CONTech’s SUPER-SPAN and SUPER-PLATE help overcome these problems through the use of both special shapes and concrete thrust beams.

SUPER-SPAN/SUPER-PLATE solves the problem

The horizontal ellipse, low-profile and high-profile arch shapes are wide-span, reduced-rise structures. They provide large open areas with less rise than comparable circular shapes. Sidewalls are compact with a modest radius to provide a more rigid pipe wall to compact against. At the same time, the large radius top arc of these structures is flatter and, therefore, has less tendency to peak as it supports the sides (see Figure 9).

![Figure 9](image)

By contrast, pear and pear-arch shapes provide relatively high-rise structures. These shapes orient their sides at the derivable angle to the soil pressures (see Figure 10). Their smaller radius crowns are typically heavy gauge to provide the necessary restraint at the top.

![Figure 10](image)

The thrust beam is the key element to SUPER-SPAN and SUPER-PLATE success. Besides providing perfect backfill in the important area above the spring line, it acts as a floating footing for the critical large radius top arch of the structure. It fixes the end of the arch, stiffening it and reducing deflection as backfill goes over the top.

The thrust beam also provides a solid vertical surface that is easy to backfill against to obtain excellent compaction*. After installation, the beam effectively controls possible horizontal spreading of the top arch.

SUPER-SPAN and SUPER-PLATE structures, by means of their shape and thrust beams (which reduce the central angle of the effective top arch to 80 degrees) have added stability against deflection and snap-through buckling. They can be economically designed and installed within recognized AASHTO/AISI critical stresses and seam strength limits.

Horizontal reinforcement bars are tied to CONTech bent and threaded rods to provide reinforcement for the concrete thrust beam.
Structural design

<table>
<thead>
<tr>
<th>Top Radius R, Ft.</th>
<th>0.111&quot; (12)</th>
<th>0.138&quot; (10)</th>
<th>0.168&quot; or 0.188&quot; (8 or 7)</th>
<th>0.218&quot; (5)</th>
<th>0.249&quot; (3)</th>
<th>0.280&quot; (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15'</td>
<td>2.5'</td>
<td>2.5'</td>
<td>2.5'</td>
<td>2.0'</td>
<td>2.0'</td>
<td>2.0'</td>
</tr>
<tr>
<td>15'-17'</td>
<td>3.0'</td>
<td>3.0'</td>
<td>2.5'</td>
<td>2.0'</td>
<td>2.0'</td>
<td>2.0'</td>
</tr>
<tr>
<td>17'-20'</td>
<td>3.0'</td>
<td>2.5'</td>
<td>2.5'</td>
<td>2.0'</td>
<td>2.0'</td>
<td>2.0'</td>
</tr>
<tr>
<td>20'-23'</td>
<td>3.0'</td>
<td>3.0'</td>
<td>3.0'</td>
<td>3.0'</td>
<td>3.0'</td>
<td>3.0'</td>
</tr>
<tr>
<td>23'-25'</td>
<td>4.0'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. Designs listed are for steel 6" x 2" corrugation only. For aluminum 9" x 2 1/2" corrugation design, please contact your local CONTECH representative.
2. Heights of cover for highway live loads given are to top of concrete pavement or bottom of flexible pavement.
3. Minimum covers for E 80 live loads are approximately twice those for HS 20. However, E 80 minimums must be established for individual applications.
4. Minimum covers for construction loads and similar heavy wheel loads must be established for individual applications.
5. The table assumes a granular backfill over the crown of the structure to the full minimum cover depth (height) compacted to not less than 90 percent AASHTO T180 density.
6. Call a CONTECH representative for Peer shape gauges.

A SUPER-SPAN or SUPER-PLATE structure is essentially an engineering combination of steel and soil. Maximum fill heights are calculated on the basis of A.A.S.H.T.O./AISI design methods using top radius to calculate ring compression (thrust=pressure x R,) with allowable wall stress of 16,500 psi. In the design method, AISI requires a seam strength safety factor of two, while A.A.S.H.T.O. requires a seam strength safety factor of three.

In accordance with A.A.S.H.T.O., buckling and flexibility factors are not calculated. These factors are covered by the minimum thickness/minimum cover table on this page and special geometry limitations spelled out by A.A.S.H.T.O.

Shallow fill

Minimum designs are shown in Table 53. Ordinarily, shallow cover structures will be at the minimum (shown in the tables) thickness required for installation and to prevent against buckling. Wall stresses can be checked in deep cover applications by adding the soil load to the appropriate live load.

When adding the total live load over the structure, it is necessary to distribute it over an appropriate area of the structure which varies with the fill height.

Special designs

Structure sizes shown in Tables 55 through 61 are standard shapes. Intermediate or larger sizes are available. These special sizes also are designed in accordance with the A.A.S.H.T.O. design method.

Minimum covers shown in Table 53 are based on standard construction. Somewhat lower covers are possible with special measures such as using concrete relieving slabs. Special designs are also available for fill heights exceeding the normal limitations of standard structures. Your CONTECH representative can provide information on special requirements.

Foundation

The foundation under the structure and sidefill zones must be evaluated by the design engineer to ensure adequate bearing capacity. Differential settlement between the structure and sidefill must be minimal.

Hydraulic design

The most commonly used SUPER-SPAN and SUPER-PLATE hydraulic shapes are the horizontal ellipse, the low-profile arch, and the high-profile arch. Hydraulic data for these shapes are presented in tabular and graphical form in the current edition of the Handbook of Steel Drainage and Highway Construction Products. Standard procedures are presented in the Hydraulics chapter of the handbook to determine the headwater depth required for a given flow through these structures under both inlet and outlet control conditions.

In addition, the hydraulic design series of publications from FHWA offers guidance regarding hydraulic capacity of these structures.

Installation precautions

During the installation and prior to the construction of permanent erosion control and end-treatment protection, special precautions may be necessary. The structure must be protected from unbalanced loads and from any structural loads or hydraulic forces that might bend or distort the unsupported ends of the structure. Erosion wash out of previously placed soil support must be prevented to ensure that the structure maintains its load capacity.
CONTECH SUPER-SPAN structures have proven both practical and economical to construct in a wide range of applications and conditions. Nevertheless, there are basic rules of installation that must be obeyed to ensure acceptable performance.

Comprehensive installation and inspection standards are furnished with every SUPER-SPAN purchase. These documents should be studied thoroughly by the contractor and engineer. The following material highlights the key elements involved in the proper construction of a CONTECH SUPER-SPAN.

Excavation, foundation and bedding

There must be adequate distance between the SUPER-SPAN and questionable native soils. Bedding must be pre-shaped for structures with inverts. A loose soil cushion should be provided for the bottom plates. Base channels for arches must be square to the centerline on arch structures.

Erection

Plates can be placed either one at a time or in preassembled units of two or more plates in a ring.

All bolts in a newly hung plate or assembly should be tightened before adding the next unit above it. This should be done only with the plates in proper relation to each other for correct curvature and alignment in the structure. It may be necessary to use cables, props, or jigs to keep the plates in position during tightening.

The structure cross-section must be checked regularly during assembly. Its shape must be symmetrical, with the plates forming smooth, continuous curves. Longitudinal seams should be tight and plate ends should be parallel to each other.

Backfilling

SUPER-SPANs are flexible structures, therefore care is required during the placement and compaction of backfill. An effective system to monitor the structure during the backfilling process must be established.

Select an approved structure backfill material for the zone around the SUPER-SPAN. Establish soil density curves and determine proper frequencies and procedures for testing. The equipment used to place and compact fill around and over the structure should be selected based on the quality of the backfill and the shape of the SUPER-SPAN. Such plans should be verified in the initial backfilling stages.

Use only backfilling methods and equipment that obtain specified density without excessive movement or deformation of the structure.

Backfill material

CONTECH’s specification for backfill material contains the following as listed in the A.A.S.H.T.O. Bridge Specification:

1. Granular type soils shall be used as structure backfill (the envelope next to the metal structure). Well-graded sand and gravel that is sharp, rough, and angular is preferred.

Approved stabilized soil shall be used only under direct supervision of a competent, experienced soils engineer. Plastic or cohesive soils should not be used.

2. The structure backfill material shall conform to one of the following soil classifications from A.A.S.H.T.O. Specification M145, Table 2: for height of fill less than 12 feet, A-1, A-2-4 and A-2-5; for height of fill of 12 feet and more and all pear or pear-arch structures, A-1. Structure backfill shall be placed and compacted to not less than 90 percent density, per A.A.S.H.T.O. T 180.

3. The extent of the select structural backfill outside the maximum span is dependent on the quality of the adjacent embankment, loading and shape of the structure. It may be necessary to excavate native soil at the sides to provide an adequate width needed for compaction. For ordinary installations with a good quality, well-compacted embankment or in situ soil adjacent to the structure backfill, a minimum width of structural backfill six feet beyond the structure is usually required. The engineer must evaluate the in situ conditions to ensure adequate bearing capacity. The structure backfill shall extend to the minimum cover elevation (Table 53—page 80) above the structure.

Monitoring Backfill

Regular monitoring is required during backfilling to assure a structure with a proper shape and that compaction levels are achieved. A CONTECH technician will confirm the structure’s shape before backfilling, then monitor the shape and verify compaction readings until the backfill reaches the minimum cover level.

Special requirements

Very large or high structures sometimes call for additional special provisions for shape control during backfilling.

The minimum stiffness requirements for some structures shown in Table 53 on Page 80 may need to be augmented by increased design stiffness or mandatory top loading. Top loading requires the placement of a modest blanket of soil on the crown when backfill is approximately at the springline height.
### Galvanized Steel 6" x 2" Corrugation

#### Table 57. TYPICAL HIGH PROFILE ARCH SHAPES

(All Dimensions to Inside Crests)

<table>
<thead>
<tr>
<th>Structure Number</th>
<th>Maximum Span</th>
<th>Bottom Span</th>
<th>Total Rise</th>
<th>Top Rise</th>
<th>Top Radius</th>
<th>Upper Side Radius ( R_s )</th>
<th>Lower Side Radius ( R_l )</th>
<th>Angle Below Horizontal ( \Delta )</th>
<th>Approx. Area (Sq. Ft.)</th>
<th>Shape Factor ( R_s/R_l )</th>
</tr>
</thead>
<tbody>
<tr>
<td>69A15-9</td>
<td>20'-1&quot;</td>
<td>19'-7&quot;</td>
<td>9'-1&quot;</td>
<td>6'-6&quot;</td>
<td>13'-1&quot;</td>
<td>4'-6&quot;</td>
<td>5'-5&quot;</td>
<td>11'-18&quot;</td>
<td>152</td>
<td>2.91</td>
</tr>
<tr>
<td>69A18-18</td>
<td>20'-8&quot;</td>
<td>18'-10&quot;</td>
<td>12'-1&quot;</td>
<td>7'-3&quot;</td>
<td>13'-1&quot;</td>
<td>4'-6&quot;</td>
<td>5'-5&quot;</td>
<td>21'-44&quot;</td>
<td>214</td>
<td>2.40</td>
</tr>
<tr>
<td>75A15-18</td>
<td>21'-6&quot;</td>
<td>19'-10&quot;</td>
<td>11'-8&quot;</td>
<td>6'-9&quot;</td>
<td>14'-3&quot;</td>
<td>4'-6&quot;</td>
<td>5'-5&quot;</td>
<td>20'-0&quot;</td>
<td>215</td>
<td>3.13</td>
</tr>
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<td>12'-43&quot;</td>
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Note: Other sizes are available for special designs.

![Diagram of High Profile Arch](image-url)

End View – High Profile Arch

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ATTACHMENT 3

PROPOSED REVISION (1991) OF SEDIMENT YIELD PROCEDURE - PACIFIC SOUTHWEST INTER-AGENCY COMMITTEE REPORT OF THE WATER MANAGEMENT SUBCOMMITTEE

(RASLEY, USDA NRCS, 1991)
Upper Colorado River Basin
Rangeland Salinity Control Project

PROPOSED REVISION (1991)
OF
PACIFIC SOUTHWEST INTERAGENCY COMMITTEE
SEDIMENT YIELD PROCEDURE

Utah

SALT YIELD
SEDIMENTATION
RANGELAND CONDITION
HYDROLOGY
TREATMENT POTENTIAL
EROSION

SEPTEMBER 1991
PROPOSED REVISION (1991) OF SEDIMENT YIELD PROCEDURE
PACIFIC SOUTHWEST INTER-AGENCY COMMITTEE
REPORT OF THE WATER MANAGEMENT SUBCOMMITTEE
OCTOBER, 1968

Robert C. Rasely, Geologist, Soil Conservation Service
Salt Lake City, Utah, 1991
ABSTRACT

The Pacific Southwest Interagency Committee, 1968, Sediment Yield Procedure is a resource evaluation tool that can be used to characterize sediment and salt yield from various sized hydrologic units, watersheds and geomorphic units. This sediment yield model is a documented reliable procedure that will result in quantification of sediment and salt yield. A sediment delivery ratio can be applied to derive sediment and salt delivery quantification from the modelled hydrologic unit or watershed to a downstream delivery point. These proposed revisions incorporate recent research into the procedure and improve the utility of the procedure. The revisions include applying the procedure to three planning frameworks: Present Condition, Future Without Project Condition and Future With Project Condition, and to a burned watershed (wildfire) condition. All of these planning models procedures can be used in a timely manner for planning purposes or for emergency watershed protection evaluations. A new evaluation sheet is presented for efficient field use. Emphasis is placed on the necessity of maintaining the field oriented interdisciplinary method of applying the sediment yield model. An example of the use of the revised sediment yield model is cited for the Colorado River Basin Rangeland Salinity Project, State of Utah, 1990-1991, as conducted by an interagency, interdisciplinary team.
Background and Experience: The Pacific Southwest Inter-Agency Committee, October, 1968, Report of the Water Management Subcommittee, Factors Affecting Sediment Yield and Measures for the Reduction of Erosion and Sediment Yield is generally known by its acronym the PSIAC (pronounced si-ak) procedure. This procedure has been tested against other sediment yield procedures in measured field conditions and found to be a reliable model of sediment yield (Renard, 1980) and (Shown, 1970). The Department of Interior, Bureau of Land Management (BLM) and the Department of Agriculture, Soil Conservation Service (SCS) have authorized the use of the PSIAC procedure since 1968 for modelling the sediment yield rates on rangeland in the intermountain west. The SCS in Utah has been using the PSIAC procedure extensively in watershed planning and emergency fire response work since 1983.

USGS, Circular 256 (King and Mace, 1953), entitled SEDIMENTATION IN SMALL RESERVOIRS ON THE SAN RAFAEL SWELL, gives detailed hydrologic studies, geologic formations, rock weathering, climate, sediment trapping efficiency and measured sediment yield for 15 reservoirs and debris basins over a period of 8-12 years beginning in 1936. The data presented in Table 5 (King and Mace, 1953) relates the average annual sedimentation rates in acre-feet per square mile to rock weathering categories. These data form an excellent basis for evaluating PSIAC procedure sediment yield rates in the same or comparable rock units throughout the upper Colorado Plateau. None of the SCS modelled rangeland sediment yield rates using the PSIAC procedure exceeded the ratings given in King and Mace (1953). It should be noted that some of the SCS fire related ratings did exceed the King and Mace (1953) rates. These high ratings occurred on areas where fire had been extremely hot, burned almost all vegetation to the ground, burned the root mass to a depth of a few inches and located on the very steep Wasatch Mountain Front, and therefore, warranted the high sediment yield ratings.

PROPOSED REVISION

A revised PSIAC (1968) procedure would serve as a documentation tool for quantification of present and future impacts of conservation plans on rangeland. This revised procedure could also be used to document, quantify, characterize and predict environmental impacts for rangeland planning in the preparation of National Environmental Protection Act documents.

Modifications 1 through 7 relate to the overall character of the PSIAC sediment yield procedure. Modifications 8 through 16 are proposed updates to the actual rating of the individual PSIAC sediment yield factors (a through i). Modification 17 is a proposed revision of the PSIAC Sediment Yield Factor Rating Sheet.

Modification 1: It is proposed that the title of the PSIAC procedure be restructured for consistent reference. The original document has a title page that presents either a misleading title
at the top of the page or at best a cumbersome phrasing on the whole title page that leaves the user with a reference to a committee rather than to a subject. It is proposed that the title be changed to: PACIFIC SOUTHWEST INTER-AGENCY COMMITTEE SEDIMENT YIELD PROCEDURE - REVISED 1991. The acronym PSIAC would still be valid for easy reference and clear purposeful title with a date would be available for formal reference.

Modification 2: It is proposed that the PSIAC procedure be considered applicable for land units as small as 100 acres as opposed to the current lower applicability limit of 10 square miles. Renard (1980) favorably compared PSIAC results with measured field data in watersheds 100 acres or greater. SCS experience in PSIAC ratings on wildfire impacted watersheds that drain into urbanized areas along the Wasatch Front has found the PSIAC procedure to be applicable to units of 100 acres or greater (Nelson and Rasely, 1989; Nelson and Rasely, 1990; Rasely and Robison, 1987; Robison and Rasely, 1988).

Modification 3: It is proposed that the PSIAC procedure does not necessarily have to be applied to complete hydrologic units. The PSIAC procedure may be applied to geomorphic units of 100 acres or greater. The geomorphic units can be defined on any combination of the following criteria: land use, climate, precipitation, elevation, geology, soils, erosion condition, hydrology or other criteria that are used to determined that a landform is unique. This modification should be used with discretion to avoid excessive lumping or splitting of a watershed into too gross a division or too many divisions. In other words, PSIAC can be used for land use planning on major hydrologic units by dividing the hydrologic unit into appropriate geomorphologies.

Modification 4: Modification 3 presents the problem that a PSIAC rating in a geomorphic unit will be a rating on a land form and not on a discrete hydrologic unit with a defined outlet or yield point. It is proposed that such a geomorphic unit PSIAC rating be used in conjunction with a sediment delivery ratio procedure. A sediment delivery ratio (SDR) is generally defined as a percentage of the total gross estimated sediment yield that is transported to a watershed outlet or major stream channel. The SCS National Engineering Handbook (USDA, SCS, NEH) defines sediment delivery ratio theory and describes techniques for estimating a sediment delivery ratio. The PSIAC sediment yield should be routed to a perennial flowing stream channel or appropriate delivery point.

Modification 5: It is proposed that the PSIAC sediment yield procedure may be used to model salt yield from rangeland caused by accelerated erosion. The salt is a natural constituent of some soils and is associated with rock fragments as a cementation agent within sedimentary rock. Salt is also derived directly from the weathering of limestone, halite, evaporite and gypsum rock formations. The salt content is defined as the percentage by weight of salt contained within a soil and associated rock...
fragments. Some judgment must be exercised in establishing the salt content of a soil that is actively becoming sediment because large limestone or lime rich sedimentary rock fragments may not completely break down in transport to yield the full percentage of salt content.

The salt content of a soil can be obtained by direct sampling and chemical analysis in a laboratory or by estimating the salt content from previous published analyses of comparable soils. It should be noted that SCS soil survey reports contain some soil salt data. This data often refers only to the solution prone soil salt content and does not take into account the salt content of rock fragments that can be yielded by a combination of mechanical and chemical weathering processes that occur in the stream transport of sediment.

The salt delivery percentage is not always a direct relationship with the sediment delivery percentage. In the Colorado Plateau, as well as in the intermountain basin region, streams will trap sediment in channel bar and bank deposits. In some very active streams with dominant mechanical weathering characteristics the salt content of stream sediments is yielded long before the sediment is transported out of the watershed and may even be yielded before the sediment is trapped in deposition within the watershed. The decision of what percentage of salt is contained in a sediment yield load and how much of that percentage is yielded is a matter of a combination of empirical analyses and judgement based on experience and observed stream transport characteristics.

Modification 6: It is proposed that the numerical sediment yield classification system be reversed to reflect that the highest sediment yield be given the highest category rating. The present rating of low numerical rating for high sediment yield class has led to considerable confusion. It is also proposed that the five categories of sediment yield be given written characterizations that will allow for consistent written accounts of the severity of sediment yield categories. The proposed written classifications are as follows:

<table>
<thead>
<tr>
<th>Old Class</th>
<th>New Class</th>
<th>Range (ac-ft/sq mi)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>&gt; 3.0</td>
<td>Very High</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>1.0-3.0</td>
<td>High</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>0.5-1.0</td>
<td>Moderately High</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>0.2-0.5</td>
<td>Moderate</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>&lt; 0.2</td>
<td>Low</td>
</tr>
</tbody>
</table>

It is recommended that the point rating be eliminated from the above classification scheme because in the original PSIAC there is a discrepancy between the 75 point class limit and the assigned sediment yield value of 1.0 acre-feet/square mile (75 points is equal to 1.225 acre-feet/square mile). The use of acre-feet for the development of classification is adequate for rating purposes.
Modification 7: It is recommended that the section of the PSIAC, 1968 report referred to on the title page as "Selection and Evaluation of Measures for Reduction of Erosion and Sediment Yield in the Pacific Southwest" be eliminated. The elimination of this major section of the report is necessary because the rapidity with which the field of conservation land treatment is changing. The 1968 section is out of date and if a new section were proposed, it too would become out of date in a very short time.

It is recommended that a conservation land treatment reference section be added to the report at the end of the sediment yield procedure section. This new section would refer the user to the annually updated planning manuals and technical guides of the following agencies: (1) US Department of Agriculture, Soil Conservation Service, various conservation planning handbooks and manuals; (2) US Department of Agriculture, Forest Service, various conservation planning procedures and documents; (3) US Department of Interior, Bureau of Land Management, various conservation planning handbooks and manuals; (4) Cooperative Extension Service, an organization in every state funded by state and federal money, various conservation planning handbooks and manuals. The BLUE PAGES in telephone directories will have addresses and telephone numbers of state and field offices of the above mentioned agencies. The handbooks, manuals and other references or guides can be obtained through the Freedom of Information Act for a reproduction fee (or free for a small number of pages) by contacting state or national offices of the above agencies.

Modification 8: It is proposed that the PSIAC rating factor Geology (a) is given too much weight in the rating system (ten points) and that this rating factor be given a rating maximum of five points. This modification proposal is based on research that has occurred since 1968. Hereford (1976) documents the sediment yield from the shale and sandstone units in the Chinle Formation as having a 1.8 year recurrence interval. This is an almost two year time gap between sediment yield within drainages of the Colorado Plateau that have deeply weathered sandstone and shale bedrock on the surface. These badlands areas are subject to intense and infrequent thunderstorm events. After a major sediment yielding storm event, the next storms would result in minimal to no sediment yield due to the upland having been stripped of weathered soil and sediment. It would take an average of almost two years for the bedrock to weather by freeze and thaw, temperature differentials and chemical weathering before the area would yield sediment again.

The PSIAC sediment yield procedure results in an annual figure of sediment yield from an area. In light of the above work by Hereford (1976) it is proposed that the Geology (a) rating factor should not be given equal weight as the Soil (b) rating factor.
The maximum rating of five is considered appropriate in view of recent research.

Modification 9: It is proposed that the Soil (b) rating description in the text be modified to emphasize that the entire 60 inch soil profile is not involved in the sediment yield and erosion process. The top two to three inches is generally all of the soil profile that is involved in concentrated overland flow sediment yield. The sediment yield character of these few inches of soil is critical to rating this factor.

Should gully erosion be dominant within an area, then the entire soil and sediment profile would be involved in the sediment yield and erosion process but the PSIAC sediment yield procedure would not be the appropriate procedure to characterize an area of dominant gully erosion. The PSIAC procedure should be used to characterize upland and channel erosion. A direct volume sediment yield procedure (USDA, SCS, 1983) should be used to characterize an area dominated by gully sediment yield. Together the two procedures would serve to characterize the sediment yield of the area.

Modification 10: It is proposed that the Topography (e) be rated by a weighted average for slope-area in conjunction with the characterization in PSIAC (1968, pages 5-6). The weighted slope-area average can be based on field observation, planimeter work or GIS data. The geomorphic unit should be divided into general slope-related subunits. Then the area of the subunit and the characteristic slope should be determined. The geomorphic unit average slope may then be calculated by the following example: A butte, mesa, bench and escarpment topography characteristic of the Colorado Plateau may have 15% of the area with a characteristic slope of 70%, 35% of the area with a characteristic slope of 5% and 50% of the area with a characteristic slope of 10%. The overall average slope for a geomorphic unit is obtained by multiplying the percent of the area by its average slope percent; this should be done for all the individual slope-areas in the geomorphic unit. These values are then added together to give the weighted average slope percent for the geomorphic unit; which in the case of this example is 17.25% or 17%. A weighted slope average calculation table and a slope percent rating chart is included on the revised PSIAC Rating Chart.

The rating of this factor should be based on the watershed considerations as documented in the PSIAC (1968, pages 5-6) as well as the weighted slope average. Experienced judgement in all PSIAC ratings is critical to the obtaining the best results.

Modification 11: It is proposed that the factor (f) topic name be revised to read EFFECTIVE GROUND COVER and it is proposed that the Effective Ground Cover (f) rating format be modified to specifically state that vegetation, litter and rock fragments each be rated by determining the percentage of effective ground
Ground cover should represent those factors which impede surface flow, affect hydrologic time of concentration and aid infiltration. Canopy should be a peripheral to negligible factor. The last sentence in the Ground Cover section (PSIAC, 1968, page 7) states "For instance, in areas of pinyon-juniper forest having the same percentages of ground cover as an area of grass, the absence of understory in some of the pinyon-juniper stands would allow a higher erosion rate than in the area of grass.". Field experience in rating concentrated overland flow erosion in conjunction with the last statement in the Ground Cover section leads to the conclusion that canopy is incidental in the concentrated overland flow erosion process. Effective ground cover should be defined as any material that slows concentration of overland flow, spreads water flows, permits greater infiltration time and would lower peak flows.

It should be emphasized that the PSIAC sediment yield procedure is quite different from the Universal Soil Loss Equation, USLE, (Wischmeier and Smith, 1978) because the USLE evaluates on-site soil disturbance in relationship to agricultural cropland which is the gross soil erosion in an individual soil and farm field setting and the PSIAC sediment yield procedure rates sediment delivery from rangeland and mountainland which is net soil loss in a watershed hydrologic unit setting. Also, the USLE was developed in a gentle to moderate slope, deep soil regime in the midwest where precipitation occurs dominantly as frontal storms of long duration and the annual precipitation rates are around 30 inches. The PSIAC procedure was developed for a low precipitation area of steep slopes, relatively thin erosive soils, alluvial fan topography and a predominantly infrequent high intensity thunderstorm precipitation regime. Each procedure has its area of applicability. The concepts of gross soil erosion (USLE) and net sediment yield are separate processes and should not be equated. Also, the concept of ground cover canopy (USLE) and ground cover in a concentrated overland flow sense (PSIAC) should not be equated.

In terms of litter, it should be noted that not all litter is effective in controlling erosion or overland flow. An area with 15% litter on the surface may have most of the litter oriented parallel to flow paths. Also, sticks may have only the ends in contact with the ground, thereby forming an arc with flow possible under the middle of the stick. This is ineffective litter. Litter should be considered in an "effective" sense. Litter oriented perpendicular to flow paths and in contact with the ground is effective litter. Abundant small pieces of litter may actually be floated by overland flows and not act as a sediment yield control agent. The rating of litter should be carefully considered in terms of effectiveness.

The ground cover rating section on field sheets should include the items VEGETATION, LITTER and ROCK with some space behind each
word for the percentage rating of each item. This allows the
tater to document and consider the relative value of each item in
terms of sediment yield. These items could be important in
determining FWOP conditions and FWP proposed land treatment
measures. The vegetation, litter and rock ratings should be made
in the field by direct observation and paced transect. Each item
should be rated to the nearest five percentage point value. This
eliminates developing arguable digit ratings and relates to the
sensitivity level of the -10 to +10 rating system (see below).

The percentage rating of effective ground cover is made by
totaling the percentages of vegetation, litter and rock. An
Effective Ground Cover rating chart is included in the Revised
PSIAC Rating Sheet.

In developing the FWP and FWOP conditions, the modelling of the
Ground Cover (f) factor is critical. The increase of litter and
vegetation should be estimated from experience in previous type
of treatments or rated by example in the field if a previously
treated area is available.

Modification 12: It is proposed that the Land Use (g) rating
factor be re-evaluated. The cultivated land factor should be
removed from rating consideration. If cropland is present within
a geomorphic unit; for instance, it cannot be assumed that
because 50% of the area is cultivated that a necessarily high
sediment yield rating is deserved. The sediment yield rating of
cultivated cropland should be characterized by the USLE procedure
(or other appropriate model) and a sediment delivery ratio should
be applied for complete characterization.

The term "intensively grazed" should be removed as a rating
criterion. Good management and a properly implemented grazing
plan can allow an area to be intensively grazed without
appreciable increases in sediment yield. There may be on-site
gross soil disturbance but this problem affects productivity, not
sediment yield. The factor that should be considered in this
context is the quality of grazing management. The following
terminology is proposed for the high, moderate and low rating
categories. The 10 point category line (b) should read "Almost
all the area overgrazed or historical overgrazing impacts are
still wide spread.". Overgrazing implies management problems and
historical overgrazing implies former management problems that
have not been accounted for in recent management. In the 0 point
category line (c) should read "Less than 50% of area overgrazed
or historically overgrazed.". The -10 point category line (c)
should read "Good grazing practices in effect or historic
overgrazing damage controlled by present land use practices."

The 10 point category should be augmented to reflect road
construction quality in regard to potential sediment yield and
road drainage. Gravel (dirt) or unimproved roads are common in
the intermountain region. Roads can significantly contribute to
sediment delivery or significantly contribute to sediment yield
control depending on construction, design and maintenance. The rating line should read "Roads in need of maintenance or need drainage control design.". Excessive concentrated road drainage can lead to significant gullying and damaging concentrated flow erosion. If road bank erosion is significant, it can be characterized by the direct volume method (USDA, SCS, 1983).

In the 10 point and 0 point categories there should be items involving the badlands land type. The 10 point rating line should read "Almost all of the area is badlands with minimal armoring.". The 0 point rating line should read "almost all of the area is badlands with 50% armor on slopes.". Badlands is a land type that is not accounted for in the PSIAC (1968) procedure. Not all badlands are in a high yielding condition. For instance the Book Cliffs region of Utah is a badlands escarpment composed of a sandstone caprock underlain by hundreds of feet of highly erosive Mancos Shale on steep slopes. The sediment yield from the Book Cliffs varies greatly with the amount of natural armoring that has occurred from the weathering of the overlying sandstone caprock.

Based on the above discussion, it is proposed that this category be renamed LAND TYPE AND MANAGEMENT QUALITY. This change is necessary because badlands is a land type (not a use), wildfire burned areas are a management problem with high sediment yield rates (not a land use) and grazing, overgrazing, historic overgrazing, logging and roads relate to sediment yield in terms of land management (not as a land use).

Modification 13: It is proposed that in the Upland Erosion (h) the reference to rill erosion be eliminated and replaced with the term "concentrated flow erosion". Rill erosion in the last 20 years has become predominantly associated with cultivated cropland erosion in the phrase "sheet and rill erosion" and the USLE. Concentrated flow erosion is defined as storm related natural flow paths where sediment is eroded and transported during storm events. These paths are not gullies but can eventually become gullies in some cases. The flow paths can laterally develop as well as developing more tributary paths. These flow paths enter into major channels or gullies or may distribute into lower sloping depositional zones. The result can be significant sediment loss over a wide area of land. This is the type of sediment yield and erosion process that the PSIAC sediment yield procedure was developed to characterize.

The reference to landslide erosion should be eliminated. This type of erosion can be characterized by a direct volume measurement estimate method and should not be included in the PSIAC procedure.

In the 10 point category the (b) wind erosion portion should be eliminated. If wind erosion is a significant problem, then a wind erosion equation (USDA, SCS, 1984) should be used to characterize the problem. The wind erosion values can be used in
conjunction with the PSIAC sediment yield and other data to fully characterize the erosion and management problems of a geomorphic unit.

The percentage value of the surface area of a geomorphic unit used to rate this factor should be developed in relationship to the percentage value of effective ground cover \((f)\). The sum of upland erosion area \((h)\) and ground cover \((f)\) cannot exceed 100% and in reality should not equal 100%. The potential area open to concentrated overland flow erosion is not the total remaining area after effective ground cover is estimated. The percentage of surface area assigned to this factor should be developed by direct observation of the density of concentrated flow paths, the opportunity for sheet flow sediment yield between developed flow paths and the opportunity for development of new concentrated flow paths. This percentage figure should be assigned to the nearest five percentage points. An Upland Erosion rating chart is included on the Revised PSIAC Rating Sheet.

Experienced judgement and field observation are the main tools for developing the FWP and FWOP conditions for this factor. Should an area that is in good management condition be locally analogous to the one under study, it can be used as a guide for rating the future conditions.

Modification 14: A comparison of the rating of factors \((a,b,c,d,e,f,g)\) that affect sediment yield to the rating of active sediment yield factors \((h,i)\) as stated in the PSIAC procedures (1968, page 2) is a recommended step in the rating process that should occur in the field. It is proposed that a value of 10 points be used as a maximum difference between the two factor values. Any differential value greater than 10 points should either be justified by observed field conditions (and noted on the field rating sheet) or the ratings should undergo a re-evaluation for consistency. It is also recommended that the raters confer in the field and determine if the total rating reflects their personal view of the sediment yield and erosion status of the geomorphic unit. This determination may be characterized as a "gut reaction" test - does the rating reflect what you see there? - a pertinent question to consider before leaving the field.

Modification 15: It is proposed that the title page following page 10 in the original PSIAC text be added to the page 10 text as a major section title. This title should be centered and underlined. The title should be revised to read: "An Example of the Use of the PSIAC Rating Sheet for Evaluating Sediment Yield". The sediment yield classification chart of modification 6 should be repeated.

The following example should be substituted for the 15 square miles watershed: A geomorphic unit of approximately 1760 acres (1.75 square miles) in central Utah was evaluated for sediment yield. The area is characterized by sagebrush vegetation with
little understory, rolling-hhummocky badlands topography, in a thin silt-loam soil with abundant calcium carbonate salt, dominated by weathered Jurassic age Morrison formation shale with occasional sandstone fragments on the surface. The following rating for the Present condition was performed:

<table>
<thead>
<tr>
<th>Factors</th>
<th>Sediment Yield Characteristics</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>a Geology</td>
<td>Lacustrine shales, Jur-Morrison</td>
<td>5</td>
</tr>
<tr>
<td>b Soils</td>
<td>Hi shrink-swell, dispersive, hi CaCO3, minor rock frags</td>
<td>8</td>
</tr>
<tr>
<td>c Climate</td>
<td>Infreq convective storms, freeze-thaw dominant, approx 12&quot; ppt</td>
<td>7</td>
</tr>
<tr>
<td>d Runoff</td>
<td>Hi peak flows, low volumes</td>
<td>5</td>
</tr>
<tr>
<td>e Topography</td>
<td>60% area 15% slope = 9.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>40% area 8% slope = 3.2%</td>
<td>5</td>
</tr>
<tr>
<td>f Eff. Grnd.</td>
<td>Veg 10%, Litter 5%, Rock 10%</td>
<td>8</td>
</tr>
<tr>
<td>Cover</td>
<td>Total Eff. Grnd. Cover = 25%</td>
<td></td>
</tr>
<tr>
<td>g Land Type</td>
<td>Hist. overgrazed, impacts evident, pedestal erosion, area mostly badlands</td>
<td>10</td>
</tr>
<tr>
<td>Mgmt. Qual.</td>
<td>&gt;50% of area involved in conc. flow eros. with minor gullying</td>
<td>25</td>
</tr>
<tr>
<td>h Upland Erosion</td>
<td>Moderate flow depths, minor eroding banks and bed, 4th order ephemeral gully develop.</td>
<td>8</td>
</tr>
</tbody>
</table>

Subtotal a-g = 39; Subtotal h-i = 33 Total Rating = 72

A total rating of 72 points is an annual sediment yield of 1.1 acre-feet per square mile (approximately 3.3 ton per acre) with a sediment yield classification of 4 - High. Laboratory tests on salt content of sediment are 10%, 8.5% and 9%. It is determined that 9% is a representative figure. The annual salt yield from the geomorphic unit is 9% of 3.3 tons per acre or approximately 0.3 tons per acre. The geomorphic unit has a channel outlet to an intermittent channel with 32 miles of canyon-type channel delivery to the Green River. There are minor occurrences of sandbars and flood plain deposits along the channels within the geomorphic unit. It is estimated that approximately 90% of the sediment from the geomorphic unit actually are transported to the Green River. This percentage is known as the sediment delivery ratio (SDR). In this example it will be assumed that the trapped sediment contains its percentage of salt. It some cases the sediment deposited in bars and flood plains will have already yielded its salt to solution through mechanical and chemical erosion and therefore, the SDR will not apply to the salt value (then the salt percent should be applied to the sediment tonnage before the SDR is applied).
The Present Condition annual salt yield to the Colorado River is estimated by the following calculation:
0.3 ton/acre x 1760 acres x .9 (SDR) = 475 tons of salt (Present).

The above example was for the Present condition. The Future Without Project condition is determined to be a continuation of the Present conditions with a slight increase in the Upland Erosion factor by 2 points and a 2 point increase in the Channel Erosion factor. Thereby giving a total rating of 76 points for an annual sediment yield of 1.275 acre-feet per square mile (approximately 3.8 tons per acre). The FWOP annual estimated salt yield is calculated as follows:
3.8 ton/acre x .09 salt % x 1760 acres x .9 SDR = 542 tons of salt (FWOP).

The next rating is for the Future With Project condition. It is determined in the field that this geomorphic unit would respond to a change in management and land treatment planning. The proposed changes are some livestock exclusion of major channel areas, fencing, chaining or harrowing of strips of sagebrush to disturb the potential for rapid collection of overland flows and to disrupt concentrated flow paths. Gully areas up to 2 feet deep will be filled with litter and sediment. An appropriate vegetative mix is proposed for the reseeding. Cattle will be limited to a late spring-early summer use. The following is the FWP rating for the geomorphic unit:

<table>
<thead>
<tr>
<th>Factors</th>
<th>Sediment Yield Characteristics</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>a Geology</td>
<td>Lacustrine shales, Jur-Morrison</td>
<td>5</td>
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<td>b Soils</td>
<td>Hi shrink-swell, dispersive, hi CaCO3, minor rock frags</td>
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<td>c Climate</td>
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<tr>
<td>e Topography</td>
<td>60% area 15% slope = 9.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>40% area 8% slope = 3.2%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Weighted slope Ave.= 12%</td>
<td>6</td>
</tr>
<tr>
<td>f Eff. Grnd. Cover</td>
<td>Veg 20, Litter 15%, Rock 10%</td>
<td></td>
</tr>
<tr>
<td>g Land Type Mgmt. Qual.</td>
<td>Hist. overgrazed, impacts</td>
<td>-1</td>
</tr>
<tr>
<td></td>
<td>under control, land treatment practices implemented</td>
<td></td>
</tr>
<tr>
<td>h Upland Erosion</td>
<td>&gt;25% of area involved in conc. flow eros. with gullying controlled</td>
<td>10</td>
</tr>
<tr>
<td>i Channel Eros. and Sed. Trans.</td>
<td>Moderate flow depths, eroding banks and bed controlled, 4th order ephemeral gully develop. controlled</td>
<td>6</td>
</tr>
</tbody>
</table>

Subtotal a-g = 24; Subtotal h-i = 16 Total Rating = 40
The rating of 40 points is a sediment yield of 0.35 ac-ft/sq. mi. or approximately 1.1 tons per acre. The average annual salt yield rate is .09 x 1.1 tons per acre = 0.1 tons per acre salt. The annual salt yield from the geomorphic unit is as follows: 0.1 tons/acre x 1760 acres x .9 (SDR) = 158 tons of salt (FWP).

For planning purposes, the annual salt benefit value for the geomorphic unit is the FWOP value of 542 tons minus the FWP value of 158 tons for a net benefit value of 384 tons of salt.

The annual sediment load values may be calculated in a similar manner as outlined above and would be as follows for the example:

Present - 3.3 tons/acre x 1760 acres x .9 SDR = 5227 tons
1.1 ac-ft/sq mi x 2.75 sq mi x .9 SDR = 2.7 ac-ft

FWOP - 3.8 tons/acre x 1760 acres x .9 SDR = 6019 tons
1.275 ac-ft/sq mi x 2.75 sq mi x .9 SDR = 3.2 ac-ft

FWP - 1.1 tons/acre x 1760 acres x .9 SDR = 1742 tons
.35 ac-ft/sq mi x 2.75 sq mi x .9 SDR = .9 ac-ft.

For planning purposes, the annual sediment benefit values are calculated as follows:

6019 tons (FWOP) - 1742 tons (FWP) = 4277 tons
3.2 ac-ft (FWOP) - .9 ac-ft (FWP) = 2.3 ac-ft.

Modification 16: It is proposed that the factor rating sheet on an unnumbered page in the original PSIAC text be replaced with the two page "PSIAC Sediment Yield Factor Rating Sheet" attached to the end of this appendix. The new rating sheet is a modification of the PSIAC sediment yield rating sheets prepared by the Department of Interior, Bureau of Land Management. The modifications would reflect the changes proposed in the above 15 modifications. These sheets would be used in the field for ratings and documentation of the rating factor data. It is proposed that these two sheets be published or issued to field personnel in a front-to-back printing mode. In practical use, completed rating sheets could be used in reports as data documentation of field sediment yield evaluations. It is urged that a maximum amount of notation be made on these sheets in the field to record the development of the rating process.

Modification 17: It is proposed that the PSIAC sediment yield graph on the back of the BLM PSIAC rating sheet be replaced by a PSIAC sediment yield chart that eliminates the field plotting of sediment yield from a 2-log graph. A chart of a direct sediment yield values for PSIAC ratings ranging from 1 to 135 is included in the Revised PSIAC Rating Sheet. The chart was developed from Renard (1980, p. 167) and BLM graphic plot of PSIAC. The equation used to developed the chart is:
X ac-ft/mi sq = antilog \([0.015656 \times \text{PSIAC rating} - 1.085666]\)

It is further recommended that the sediment yield conversion graph (BLM) be included in the text portion of the PSIAC procedure.

THE REVISED PSIAC PROCEDURE AS USED IN THE COLORADO RANGELAND SALINITY CONTROL PROJECT

The above proposed modifications to the PSIAC procedure were implemented in the Colorado River Basin Rangeland Salinity Control Project, State of Utah, 1990-1991.

Watersheds were divided into geomorphic units. A PSIAC sediment yield rating was modelled for each of the geomorphic units that reflected the present condition of the unit. The PSIAC procedure was then used to model a Future Without Project (FWOP) condition and a Future With Project (FWP) condition.

The present condition was modelled through direct field observation by an interdisciplinary team. Each PSIAC factor was rated by a specialist for that factor or by use of technical data.

The rating factors (a) Surface Geology, (b) Soils, (c) Climate and (e) Topography are constant within a geomorphic unit for the future condition models (FWOP and FWP).

The FWOP condition is the extension of present condition trends into the future for a number of years relating to the life of land treatment planning measures. Factor ratings can be increased, decreased or remain the same by continuation of current land treatment practices and current management planning.

The rating factor of Land Use (g) was rated first for the FWOP because this factor influences the other factor ratings. The factor rating would increase if present trends are interpreted to be causing the resource condition to degrade and therefore, increase sediment yield potential. The factor rating would decrease if the land will respond to current land treatment practices that would improve the resource condition and therefore decrease sediment yield potential. The rating would remain the same if the present trends have stabilized the land resources. The rating is arrived at by consultation between land use planners and technical specialists.

The Ground Cover (f) rating factor is modelled for the FWOP condition based on the anticipated impact of the current Land Use (g) and the present trends continuing into the future. Greater vegetative ground cover created by increased litter and/or increased vegetation would lower the rating. A degrading present trend would raise the rating.
The Runoff (d) factor is modelled for the FWOP condition based on the increased or decreased ground cover. An increase of Ground Cover (f) would lower the rating and a decrease in Ground Cover (f) would raise the rating.

The Upland Erosion (h) factor rating is modelled for the FWOP condition based on the Runoff (d) and Ground Cover (f) ratings. There would be less potential for concentrated overland flow erosion if ground cover is increased and runoff is slowed. Therefore, the rating would decrease. There would be greater potential for concentrated overland flow if ground cover is decreased and runoff is accelerated. Therefore, the rating would increase.

The Channel Erosion and Sediment Transport (i) rating factor is modelled for the FWOP condition based on the Upland Erosion rating. There would be an increase in channel erosion and sediment transport rating if concentrated overland flow is increased. There would be a decrease in the rating if concentrated overland flow was decreased. Therefore, gullies and channels would re-vegetate and stabilize.

Future With Project Condition (FWP): The FWP condition is the model of proposed land treatment practices that will control the accelerated sediment and salt yield problems. The FWP condition is modelled for a 5 to 50 year planning period. Factor ratings can be increased, decreased or remain the same by implementation of conservation oriented land treatment practices and management planning.

The factor of Land Use (g) is rated first for the FWP because this rating influences the other factor ratings. Future conditions with improved land use practices should not increase the rating. The rating would be decreased if the land will respond to proposed land treatment practices that would improve the resource condition and therefore decrease sediment yield potential. The rating would remain the same if the present land use practices have stabilized the land resources. The rating is arrived at by consultation between land use planners and technical specialists.

The Ground Cover (f) factor rating is modelled for the FWP condition based on the anticipated impact of proposed Land Use (g) practices. Greater vegetative ground cover created by increased litter and/or increased vegetation would lower the rating. A degrading trend should not occur in the FWP condition.

The Runoff (d) factor rating is modelled for the FWP condition based on the increased ground cover and implementation of mechanical practices that increase surface roughness, retard flow, and increase filtration. An increase in Ground Cover (f) and surface roughness would lower the rating. The rating could remain the same as in the Present condition if the proposed practices would not significantly impact on runoff.
The Upland Erosion (h) factor rating is modelled for the FWP condition based on the Runoff (d) and Ground Cover (f) ratings. There would be less potential for concentrated overland flow erosion if ground cover is increased and runoff is slowed. Therefore, the rating would decrease. The factor rating could remain the same as the Present condition if there is no significant impact on upland erosion due to the proposed land treatment practices.

The Channel Erosion and Sediment Transport (i) rating factor is modelled for the FWP condition based on the Upland Erosion rating. There would be a decrease in the rating if concentrated overland flow and runoff was decreased. Therefore, gullies and channels would support vegetation and stabilize.

SUMMARY AND CONCLUSIONS

The proposed revisions of PSIAC would accomplish the following goals: (1) update the procedure with recent research, (2) increase the utility of the procedure in the field, (3) broaden the scope of the procedure to include its use on geomorphic areas as well as watersheds, (4) define the use of the procedure to characterize future sediment yield rates for the present conditions continued into the future and (5) define the use of the procedure to model rates for various alternative future conditions involving proposed land treatment planning measures.
REFERENCES


U.S. Department of Agriculture, Soil Conservation Service, Sediment Sources, Yields and Delivery Ratios, National Engineering Handbook Section 3, Chapter 6, updated periodically.


Watershed: __________________________  State: ________________  Condition: Present, FWOP, FWI, Fire

Geomorphic Unit __________________________

Map ____________ Location: T __ R __ S __ I/4 1/4 1/4

<table>
<thead>
<tr>
<th>Factor Value</th>
<th>Subtotal (a)</th>
<th>Subtotal (h)</th>
<th>Total Rating</th>
</tr>
</thead>
</table>

**Factor Value**

(a) Surface Geology
- Geologist
  - Marine shales and related mudstones and siltstones
  - Rocks of medium hardness
  - Massive, hard formations

(b) Soils
- Soil Scientist
  - Fine textured; easily dispersed; saline-alkaline; high shrink-swell characteristics
  - Medium textured soil
  - High percentage of rock fragments

(c) Climate
- Local Knowledge
  - Storms of several days' duration with short periods of intense rainfall
  - Humid climate with rainfall of low intensity
  - Almost all of area overgrazed or historic overgrazing impacts still active

(d) Runoff
- Hydrologist
  - High peak flows per unit area
  - Low peak flows per unit area
  - Almost all of area is badlands with 50% of area covered with armor

(e) Topography
- Map & Field
  - Steep upland slopes (in excess of 30%)
  - Gentle upland slopes (less than 5%)
  - Wide shallow channels with flat gradients and short flow duration

(f) Effective Ground Cover
- Land Use Planner
  - Ground cover does not exceed 20%
  - Noticeable litter
  - Area completely protected by vegetation, rock fragments, litter

- Range Conservationist
  - Vegetation sparse; little or no litter
  - No rock in surface soil cover
  - Little opportunity for rainfall to reach erodible material

(g) Land Type and Management Quality
- Land Planner
  - Almost all of area overgrazed or historic overgrazing impacts still active
  - No recent logging

- Management Quality
  - Good grazing management or historic overgrazing impact under control
  - Almost all of area is badlands with minimal armor

(h) Upland Erosion
- Geologist
  - Almost all of area overgrazed or historic overgrazing impacts still active
  - No apparent signs of erosion

(i) Channel Erosion and Sediment Transport
- Geologist
  - No recent logging
  - Wide shallow channels with flat gradients and short flow duration

Weighted Slope %

Rating Chart (g) on back

Rating Chart (h) on back

**Factor Value**

Subtotal (a) - (g) Subtotal (h) - (i) Total Rating

**(AcFumi²) X (3) Conversion Factor = ___ Tons/acre**

Sheet ___ of ___
Instructions: Interpolation between sediment yield levels in each factor may be made. High values for columns (a) through (g) should correspond to high values for (h) and (i). If the difference between the total (a) through (g) and the total of (h) and (i) is greater than 10 points, then either a field related justification is necessary or the factor ratings should be reevaluated. The total rating should be reviewed from a field perspective with this question: "Does this rating reflect field observations of erosion and sediment yield for the geomorphic unit?"

<table>
<thead>
<tr>
<th>Factor (e) Chart</th>
<th>Topography</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>Pts</td>
</tr>
<tr>
<td>&gt;30 - 20</td>
<td>18 - 20 - 10</td>
</tr>
<tr>
<td>28 - 18</td>
<td>15 - 17 - 8</td>
</tr>
<tr>
<td>26 - 16</td>
<td>12 - 14 - 6</td>
</tr>
<tr>
<td>24 - 14</td>
<td>9 - 11 - 4</td>
</tr>
<tr>
<td>22 - 12</td>
<td>6 - 8 - 2</td>
</tr>
<tr>
<td>20 - 10</td>
<td>&lt;5 - 0</td>
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</table>

<table>
<thead>
<tr>
<th>Factor (f) Chart</th>
<th>Effective Ground Cover</th>
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</thead>
<tbody>
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<td>%</td>
<td>Pts</td>
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<tr>
<td>&lt;20</td>
<td>10</td>
</tr>
<tr>
<td>25 - 30</td>
<td>8</td>
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<tr>
<td>35 - 40</td>
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</tr>
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<td>45 - 50</td>
<td>3</td>
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<td>55 - 60</td>
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<td>95 - 100</td>
<td>-5</td>
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<tr>
<td>100</td>
<td>-10</td>
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<table>
<thead>
<tr>
<th>Factor (h) Chart</th>
<th>Upland Erosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>Pts</td>
</tr>
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<td>10 - 5</td>
<td>4</td>
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<tr>
<td>5 - 0</td>
<td>2</td>
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<table>
<thead>
<tr>
<th>Total Rating vs Annual Sediment Yield Chart</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pts ac-ft/sq mi</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>1 &lt;0.10</td>
</tr>
<tr>
<td>2 &lt;0.10</td>
</tr>
<tr>
<td>3 &lt;0.10</td>
</tr>
<tr>
<td>4 &lt;0.10</td>
</tr>
<tr>
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<td>40 0.34</td>
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Notes:
ATTACHMENT 4
BASELINE AND POST-MINING SEDIMENT YIELD
(PSIAC METHOD)
## ROSEMONT COPPER PROJECT SEDIMENT YIELD FACTOR RATING

<table>
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<tr>
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<th>Climate</th>
<th>Runoff</th>
<th>Topography</th>
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<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
<td>(c)</td>
<td>(d)</td>
<td>(e)</td>
</tr>
<tr>
<td>a. Marine shales and related mudstones and siltstones</td>
<td>Fine textured; easily dispersed; saline-alkaline; high shrink-swell characteristics</td>
<td>Storms of several days duration with short period of intense rainfall</td>
<td>High peak flows per unit area</td>
<td>Steep upland slopes (in excess of 30%)</td>
</tr>
<tr>
<td>b. Single grain silts and fine sands</td>
<td>Frequent intense convective storms</td>
<td>Large volume of flow per unit area</td>
<td>High relief; little or no floodplain development</td>
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<tr>
<td>c. Freeze-thaw occurrence</td>
<td></td>
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<tr>
<td>(10)</td>
<td>(10)</td>
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<tr>
<td>a. Rocks of Medium hardness</td>
<td>Medium textured soil</td>
<td>Storms of moderate duration and intensity</td>
<td>Moderate peak flows per unit area</td>
<td>Moderate upland slopes (less than 20%)</td>
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<tr>
<td>b. Moderately weathered</td>
<td>Occasional rock fragments</td>
<td>Infrequent convective storms</td>
<td>Moderate volume of flow per unit area</td>
<td>Moderate fan or floodplain development</td>
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<tr>
<td>c. Moderately fractured</td>
<td>Caliche layers</td>
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<tr>
<td>a. Massive, hard formations</td>
<td>High percentage of rock fragments</td>
<td>Humid climate with rainfall of low intensity</td>
<td>Low peak flows per unit area</td>
<td>Gentle upland slope (less than 5%)</td>
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<td>b. Aggregated clays</td>
<td>Precipitation in form of snow</td>
<td>Low volume of runoff per unit area</td>
<td>Extensive alluvial plains</td>
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<tr>
<td>c. High in organic matter</td>
<td>And climate, low intensity storms</td>
<td>Late runoff event</td>
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<tr>
<td>d. And climate; rare convective storms</td>
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### Subtotal (a) - (e): 42.5

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<th>Upland Erosion</th>
<th>Channel Erosion and Sediment Transport</th>
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<tr>
<td>(f)</td>
<td>(g)</td>
<td>(h)</td>
<td>(i)</td>
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<tr>
<td>Ground cover does not exceed 20%</td>
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<tr>
<td>a. Vegetation sparse; little or no litter</td>
<td>More than 50% cultivated</td>
<td>More than 50% of the area characterized by rill and gully or landslide erosion</td>
<td>Eroding banks continuously or at frequent intervals with large depths and long flow duration</td>
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<td>b. No rock in surface soil</td>
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### Subtotal (f) - (i): 32.5

Total Rating: 75

SEDIMENT YIELD (ac-ft/sq mi/yr): 1.15

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<th>Climate</th>
<th>Runoff</th>
<th>Topography</th>
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<td>Storms of several days' duration with short period of intense rainfall; Frequent intense convective storms</td>
<td>High peak flows per unit area; Large volume of flow per unit area</td>
<td>Steep upland slopes (in excess of 30%)</td>
</tr>
<tr>
<td>b)</td>
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<tr>
<td>c) Freeze-thaw occurrence</td>
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<td>(5)</td>
<td>(5)</td>
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<td>(10)</td>
<td>(20)</td>
</tr>
<tr>
<td>a) Rocks of Medium hardness</td>
<td>Medium textured soil</td>
<td>Storms of moderate duration and intensity</td>
<td>Moderate peak flows per unit area</td>
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</tr>
<tr>
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<td>Infrquent convective storms</td>
<td>Moderate volume of flow per unit area</td>
<td>Moderate fan or floodplain development</td>
</tr>
<tr>
<td>c) Moderately fractured</td>
<td>Caliche layers</td>
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<tr>
<td>a) Massive, hard formations</td>
<td>High percentage of rock fragments</td>
<td>Humid climate with rainfall of low intensity</td>
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<td>b) Aggregated days</td>
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<td>And climate: low intensity storms</td>
<td>Low volume of runoff per unit area</td>
<td>Extensive alluvial plains</td>
</tr>
<tr>
<td>c) High in organic matter</td>
<td>And climate: low intensity storms</td>
<td>Rare runoff event</td>
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<td></td>
</tr>
<tr>
<td>d) Caliche layers</td>
<td>And climate: low intensity storms</td>
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<td></td>
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<table>
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<tr>
<th>Ground Cover</th>
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<tbody>
<tr>
<td>(f)</td>
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<td>(h)</td>
<td>(i)</td>
</tr>
<tr>
<td>a) Vegetation sparse; little or no litter</td>
<td>More than 50% cultivated</td>
<td>More than 50% of the area characterized by all and gully or landslide erosion</td>
<td>Eroding banks continuously or at frequent intervals with large depths and long flow duration</td>
</tr>
<tr>
<td>b) No rock in surface soil</td>
<td>Almost all of area intensively grazed</td>
<td>All of area recently burned</td>
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</tr>
<tr>
<td>c) All of area recently burned</td>
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</tr>
<tr>
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<td>(0)</td>
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<tr>
<td>a) Noticeable litter</td>
<td>Less than 25% cultivated</td>
<td>About 25% of the area characterized by all and gully or landslide erosion</td>
<td>Moderate flow depths, medium flow duration with occasionally eroding banks or bed</td>
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<tr>
<td>b) if trees present understory not well developed</td>
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<tr>
<td>c) Less than 50% intensively grazed</td>
<td>Ordinary road and other construction</td>
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<td>d)</td>
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</tr>
<tr>
<td>(10)</td>
<td>(10)</td>
<td>(10)</td>
<td>(10)</td>
</tr>
<tr>
<td>a) Area completely protected by vegetation, rock fragments, litter</td>
<td>No cultivation</td>
<td>No apparent signs of erosion</td>
<td>Erosion of channels with flat gradients and short flow duration</td>
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<tr>
<td>b) Little opportunity for rainfall to reach erodible material</td>
<td>No recent logging</td>
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<td>Channel in massive rock, large boulders, or well vegetated</td>
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<tr>
<td>c) Low intensity grazing</td>
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<td>Artificially controlled channels</td>
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| Factor Value | 5 | 0 | 10 | 10 | 2 | 0 |

| Comments | Used 5 | Used 0 | Used 10 | Used 10 | Used 20 |

| Subtotal (a) - (g) | 35 | Subtotal (h) - (i) | 10 | Total Rating | 45 |

Sediment Yield Factor Rating for Post-Mining Conditions – Rosemont Copper Company

U.S. Department of the Interior
Bureau of Land Management

Tetra Tech, Inc.
ATTACHMENT 5
ENGINEERING AND DESIGN – HYDRAULIC DESIGN OF FLOOD CONTROL CHANNELS
(USACE, 1994)
| CECW-EH-D Engineer Manual 1110-2-1601 | Department of the Army  
U.S. Army Corps of Engineers  
Washington, DC 20314-1000 | EM 1110-2-1601  
1 July 1991/  
30 June 1994 |
|--------------------------------------|---------------------------------|-----------------|
| Engineering and Design               | HYDRAULIC DESIGN OF FLOOD CONTROL CHANNELS | Distribution Restriction Statement  
Approved for public release; distribution is unlimited. |
1. This Change 1 to EM 1110-2-1601, 1 Jul 91:
   a. Updates Chapter 2.
   b. Updates Chapter 3.
   c. Adds Chapter 5, which describes methods for predicting n values for the Manning equation.
   d. Updates the Table of Contents to reflect the changes in Chapters 2 and 3 and the addition of Chapter 5.
   e. Updates the preceding chapters to reflect the addition of Chapter 5.
   f. Adds references in Chapters 3 and 5 to Appendix A.
   g. Adds updated plates in Chapter 3 to Appendix B.
   h. Inserts page F-18, which was inadvertently omitted.
   i. Updates Appendix H.
   j. Adds symbols in Chapter 5 to Appendix I.

2. Substitute the attached pages as shown below:

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<th>Remove page</th>
<th>Insert page</th>
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</thead>
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<td>2-1 and 2-2</td>
<td>2-1 and 2-2</td>
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<td>3</td>
<td>3-1 thru 3-10</td>
<td>3-1 thru 3-12</td>
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<td>5</td>
<td>—</td>
<td>5-1 thru 5-16</td>
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<td>F-17 and F-18</td>
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<td>I-1 thru I-4</td>
<td>I-1 thru I-4</td>
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</tbody>
</table>
3. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:

[Signature]

WILLIAM D. BROWN
Colonel, Corps of Engineers
Chief of Staff
Hydraulic Design of Flood Control Channels
Engineering and Design
HYDRAULIC DESIGN OF FLOOD CONTROL CHANNELS

1. **Purpose.** This manual presents procedures for the design analysis and criteria of design for improved channels that carry rapid and/or tranquil flows.

2. **Applicability.** This manual applies to major subordinate commands, districts, and laboratories having responsibility for the design of civil works projects.

3. **General.** Procedures recommended herein are considered appropriate for design of features which are usable under most field conditions encountered in Corps of Engineers projects. Basic theory is presented as required to clarify presentation and where the state of the art, as found in standard textbooks, is limited. In the design guidance, where possible, both laboratory and prototype experimental test results have been correlated with current theory.

FOR THE COMMANDER:

[Signature]

ROBERT L. HERNDON
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-1601, 1 July 1970
Engineering and Design
HYDRAULIC DESIGN OF FLOOD CONTROL CHANNELS

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Chapter 5
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Appendix D
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Appendix E
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Appendix F
Report on Standardization of Riprap Gradations

Appendix G
Velocity Estimation Based on Field Observations

Appendix H
Examples of Stone Size Calculations

Appendix I
Notation
Chapter 1
Introduction

1-1. Purpose

This manual presents procedures for the design analysis and criteria of design for improved channels that carry rapid and/or tranquil flows.

1-2. Scope

Procedures are presented without details of the theory of the hydraulics involved since these details can be found in any of various hydraulic textbooks and publications available to the design engineer. Theories and procedures in design, such as flow in curved channels, flow at bridge piers, flow at confluences, and side drainage inlet structures, that are not covered fully in textbooks are discussed in detail with the aid of Hydraulic Design Criteria (HDC) charts published by the US Army Engineer Waterways Experiment Station (USAEWES). The charts and other illustrations are included in Appendix B to aid the designer. References to HDC are by HDC chart number. The use of models to develop and verify design details is discussed briefly. Typical calculations are presented to illustrate the principles of design for channels under various conditions of flow. Electronic computer programming techniques are not treated in this manual. However, most of the basic hydraulics presented herein can be adapted for computer use as illustrated in Appendix D.

1-3. References

References are listed in Appendix A.

1-4. Explanation of Terms

Abbreviations used in this manual are explained in the Notation (Appendix I). The symbols employed herein conform to the American Standard Letter Symbols for Hydraulics (American Society of Mechanical Engineers 1958) with only minor exceptions.

1-5. Channel Classification

In this manual, flood control channels are considered under two broad classifications: rapid- and tranquil-flow channels. The most important characteristics that apply to rapid and tranquil flows are listed below:

a. Velocities. Rapid flows have supercritical velocities with Froude numbers greater than 1 (F > 1), and tranquil flows have subcritical velocities with Froude numbers less than 1 (F < 1).

b. Slopes. Invert slopes in general are greater than critical slopes (S > Sₐ) for rapid flow and less than critical slopes (S < Sₐ) for tranquil flow.

c. Channel storage. Channel storage is usually negligible in rapid flow, whereas it may be appreciable in natural rivers with tranquil flow.

d. Discharge. All discharges are normally confined within the channel for rapid flow (no overbank flow).

Other characteristics such as standing waves, surges, and bed configuration that differ under the influence of rapid- or tranquil-flow conditions should be recognized and considered as the occasion demands. Rapid and tranquil flows can occur within a longitudinal reach of a channel with changes in discharge, roughness, cross section, or slope. Channel improvements may bring about changes in flow characteristics.

1-6. Preliminary Investigations for Selection of Type of Improvement

The investigation required in selecting the type of channel improvement to be adopted involves three considerations: physical features of the area, hydraulic and hydrologic aspects, and economy.

a. Physical features. The topography of the area controls in a general way the channel alignment and invert grades. Of prime importance, also, are width of available right-of-way; location of existing channel; and adjacent existing structures, such as bridges, buildings, transportation facilities, utility structures, and outlets for local drainage and tributaries. Invert slopes may be controlled by elevations of existing structures as well as by general topography, elevations at ends of improvements, and hydraulic features.

b. Historical and observed elements. The flow characteristics noted in historical records and indicated from detailed observation of existing conditions will usually be basic to the selection of type of improvement or design. With the flood discharges determined, the interdependent factors that determine improvement methods and general channel alignment are slope of invert, width and depth of flow, roughness coefficient, the presence or nature of aggradation and degradation processes, debris transportation, bank erosion, cutoffs, and bar formations.
c. Preliminary layout. A preliminary map or aerial mosaic of the area showing the topography and other control factors to a scale satisfactory for plotting the center line of the channel should be obtained. A scale of 1 inch (in.) to 100 feet (ft) with 2-ft-contour interval is suggested, although judgment based on local conditions should be used. A preliminary profile should be prepared that will show all pertinent elevations of the ground and existing structures along the banks and along the center line of the proposed channel.

d. Preliminary alternative designs. From a study of the preliminary plan, profiles, and available widths, tentative channel cross sections are adopted. These are generally rectangular or trapezoidal sections. Low velocity flows can usually be carried in natural-bottom trapezoidal channels with or without stone-revetted side slopes. High-velocity flows normally would be carried in concrete-lined channels. Preliminary hydraulic analyses of the proposed channels are then made with a view toward establishing the most efficient channel improvement from the standpoint of hydraulic efficiency and economic feasibility.

e. Economy. Approximate cost estimates are prepared, including costs of channel construction, appurtenant works and bridges, and rights-of-way. It may be necessary to consider several channel alignments, cross sections, and construction materials before the least-cost design consistent with sound engineering principles is determined. Assured performance, consistent with project formulation based on sound engineering judgment, is a necessary part of economic consideration. With an optimum general design thus tentatively established, and provided the cost is economically feasible for the project as a whole, the detailed hydraulic design is presented in Chapter 2.
Chapter 3
Riprap Protection

Section I
Introduction

3-1. General

The guidance presented herein applies to riprap design for open channels not immediately downstream of stilling basins or other highly turbulent areas (for stilling basin riprap, use HDC 712-1, Plates 29 and 30). The ability of riprap slope protection to resist the erosive forces of channel flow depends on the interrelation of the following factors: stone shape, size, weight, and durability; riprap gradation and layer thickness; and channel alignment, cross-section, gradient, and velocity distribution. The bed material and local scour characteristics determine the design of toe protection which is essential for riprap revetment stability. The bank material and groundwater conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful bank protection. Riprap protection for flood control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance or replacement (see ER 1110-2-1150). While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised as deemed necessary to provide a practical protection design for the specific project conditions.

3-2. Riprap Characteristics

The following provides guidance on stone shape, size/weight relationship, unit weight, gradation, and layer thickness. Reference EM 1110-2-2302 for additional guidance on riprap material characteristics and construction.

a. Stone shape. Riprap should be blocky in shape rather than elongated, as more nearly cubical stones “nest” together best and are more resistant to movement. The stone should have sharp, angular, clean edges at the intersections of relatively flat faces. Stream rounded stone is less resistant to movement, although the drag force on a rounded stone is less than on angular, cubical stones. As rounded stone interlock is less than that of equal-sized angular stones, the rounded stone mass is more likely to be eroded by channel flow. If used, the rounded stone should be placed on flatter side slopes than angular stone and should be about 25 percent larger in diameter. The following shape limitations should be specified for riprap obtained from quarry operations:

(1) The stone shall be predominantly angular in shape.

(2) Not more than 30 percent of the stones distributed throughout the gradation should have a ratio of \( a/c \) greater than 2.5.

(3) Not more than 15 percent of the stones distributed throughout the gradation should have a ratio of \( a/c \) greater than 3.0.

(4) No stone should have a ratio of \( a/c \) greater than 3.5.

To determine stone dimensions \( a \) and \( c \), consider that the stone has a long axis, an intermediate axis, and a short axis, each being perpendicular to the other. Dimension \( a \) is the maximum length of the stone, which defines the long axis of the stone. The intermediate axis is defined by the maximum width of the stone. The remaining axis is the short axis. Dimension \( c \) is the maximum dimension parallel to the short axis. These limitations apply only to the stone within the required riprap gradation and not to quarry spalls and waste that may be allowed.

b. Relation between stone size and weight. The ability of riprap revetment to resist erosion is related to the size and weight of stones. Design guidance is often expressed in terms of the stone size \( D_n \), where % denotes the percentage of the total weight of the graded material (total weight including quarry wastes and spalls) that contains stones of less weight. The relation between size and weight of stone is described herein using a spherical shape by the equation

\[
D_n = \left( \frac{6W_m}{\pi G_f} \right)^{y/3}
\]

where

\[ D_n = \text{equivalent-volume spherical stone diameter, ft} \]

\[ W_m = \text{weight of individual stone having diameter of } D_n \]
\( \gamma_s \) = saturated surface dry specific or unit weight of stone, pcf

Plate 31 presents relations between spherical diameter and weight for several values of specific or unit weight. Design procedures for determining the stone size required to resist the erosive forces of channel flow are presented in paragraph 3-5 below.

c. Unit weight. Unit weight of stone \( \gamma_s \) generally varies from 150 to 175 pcf. Riprap sizing relations are relatively sensitive to unit weight of stone, and \( \gamma_s \) should be determined as accurately as possible. In many cases, the unit weight of stone is not known because the quarry is selected from a list of approved riprap sources after the construction contract is awarded. Riprap coming from the various quarries will not be of the same unit weight. Under these circumstances, a unit weight of stone close to the minimum of the available riprap sources can be used in design. Contract options covering specific weight ranges of 5 or 10 pcf should be offered when sufficient savings warrant.

d. Gradation.

(1) The gradation of stones in riprap revetment affects the riprap’s resistance to erosion. Stone should be reasonably well graded throughout the in-place layer thickness. Specifications should provide for two limiting gradation curves, and any stone gradation as determined from quarry process, stockpile, and in-place field test samples that lies within these limits should be acceptable. Riprap sizes and weights are frequently used such as \( D_{90} \) (min), \( D_{90} \) (max), \( W_{90} \) (min), etc. The D or W refers to size or weight, respectively. The number is the percent finer by weight as discussed in b above. The (max) or (min) refers to the upper or lower limit gradation curves, respectively. Engineer Form 4794-R is a standard form for plotting riprap gradation curves (Plate 32). The gradation limits should not be so restrictive that production costs would be excessive. The choice of limits also depends on the underlying bank soils and filter requirements if a graded stone filter is used. Filters may be required under riprap revetments. Guidance for filter requirements is given in EM 1110-2-1901. Filter design is the responsibility of the Geotechnical Branch in each District.

(2) Standardized gradations having a relatively narrow range in sizes (\( D_{90}/D_{15} \) of 1.4-2.2) are shown in Table 3-1. Other gradations can be used and often have a wider range of allowable sizes than those given in Table 3-1. One example is the Lower Mississippi Valley Division (LMVD) Standardized Gradations presented in Appendix F. The LMVD gradations are similar to the gradations listed in Table 3-1 except the LMVD \( W_{90} \) (max) and \( W_{90} \) (max) weights are larger, which can make the LMVD gradations easier to produce. Most graded ripraps have ratios of \( D_{90}/D_{15} \) less than 3. Uniform riprap (\( D_{90}/D_{15} < 1.4 \)) has been used at sites in the US Army Engineer Division, Missouri River, for reasons of economy and quality control of sizes and placement.

(3) Rather than a relatively expensive graded riprap, a greater thickness of a quarry-run stone may be considered. Some designers consider the quarry-run stone to have another advantage: its gravel- and sand-size components serve as a filter. The gravel and sand sizes should be less by volume than the voids among the larger stone. This concept has resulted in considerable cost savings on large projects such as the Arkansas and Red River Navigation Projects. Not all quarry-run stone can be used as riprap; stone that is gap graded or has a large range in maximum to minimum size is probably unsuitable. Quarry-run stone for riprap should be limited to \( D_{90}/D_{15} \leq 7 \).

(4) Determining optimum gradations is also an economics problem that includes the following factors:

(a) Rock quality (durability under service conditions)

(b) Cost per ton at the quarry (including capability of quarry to produce a particular size)

(c) Number of tons required

(d) Miles transported

(e) Cost of transportation per ton-mile

(f) Cost per ton for placement

(g) Need for and cost of filter

(h) Quality control during construction (it is easier to ensure even coverage with a narrow gradation than with a wide gradation)

(i) Number of different gradations required. Sometimes cost savings can be realized by using fewer gradations.

See EM 1110-2-2302 for further discussion of these factors.
<table>
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<tr>
<th>Limits of Stone Weight, lb/ft^3, for Percent Lighter by Weight</th>
<th>D_{15}(min)</th>
<th>D_{50}(min)</th>
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<td>50</td>
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<td>54</td>
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</table>

Specific Weight = 155 pcf

| * 9              | 36        | 15        | 11         | 7         | 5          | 2         | 0.37      | 0.53      |
| 12               | 86        | 35        | 26         | 17        | 13         | 5         | 0.48      | 0.70      |
| 15               | 169       | 67        | 50         | 34        | 25         | 11        | 0.61      | 0.88      |
| 18               | 292       | 117       | 86         | 58        | 43         | 18        | 0.73      | 1.06      |
| 21               | 463       | 186       | 137        | 93        | 69         | 29        | 0.85      | 1.23      |
| 24               | 691       | 276       | 205        | 138       | 102        | 43        | 0.97      | 1.40      |
| 27               | 984       | 394       | 292        | 197       | 146        | 62        | 1.10      | 1.59      |
| 30               | 1,350     | 540       | 400        | 270       | 200        | 84        | 1.22      | 1.77      |
| 33               | 1,797     | 719       | 532        | 359       | 266        | 112       | 1.34      | 1.96      |
| 36               | 2,331     | 933       | 691        | 467       | 346        | 146       | 1.46      | 2.11      |
| 42               | 3,704     | 1,482     | 1,098      | 741       | 549        | 232       | 1.70      | 2.47      |
| 48               | 5,529     | 2,212     | 1,638      | 1,106     | 819        | 346       | 1.95      | 2.82      |
| 54               | 7,873     | 3,149     | 2,335      | 1,575     | 1,168      | 492       | 2.19      | 3.17      |

Specific Weight = 165 pcf

| * 9              | 39        | 15        | 11         | 8         | 6          | 2         | 0.37      | 0.53      |
| 12               | 92        | 37        | 27         | 18        | 14         | 5         | 0.48      | 0.70      |
| 15               | 179       | 72        | 53         | 36        | 27         | 11        | 0.61      | 0.88      |
| 18               | 309       | 124       | 92         | 62        | 46         | 19        | 0.73      | 1.06      |
| 21               | 491       | 196       | 146        | 98        | 73         | 31        | 0.85      | 1.23      |
| 24               | 733       | 293       | 217        | 147       | 109        | 46        | 0.97      | 1.40      |
| 27               | 1,044     | 417       | 309        | 209       | 155        | 65        | 1.10      | 1.59      |
| 30               | 1,432     | 573       | 424        | 286       | 212        | 89        | 1.22      | 1.77      |
| 33               | 1,906     | 762       | 565        | 381       | 282        | 119       | 1.34      | 1.94      |
| 36               | 2,474     | 990       | 733        | 495       | 367        | 155       | 1.46      | 2.11      |
| 42               | 3,929     | 1,571     | 1,164      | 766       | 592        | 246       | 1.70      | 2.47      |
| 48               | 5,684     | 2,346     | 1,738      | 1,173     | 869        | 367       | 1.95      | 2.82      |
| 54               | 8,350     | 3,340     | 2,474      | 1,670     | 1,237      | 522       | 2.19      | 3.17      |

Specific Weight = 175 pcf

Notes:
2. The maximum limits at the W_{50} and W_{15} sizes can be increased as in the Lower Mississippi Valley Division Standardized Gradients shown in Appendix F.
e. Layer thickness. All stones should be contained within the riprap layer thickness to provide maximum resistance against erosive forces. Oversize stones, even in isolated spots, may result in riprap failure by precluding mutual support and interlock between individual stones, causing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller size stone. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness:

1. It should not be less than the spherical diameter of the upper limit $W_{50}$ stone or less than 1.5 times the spherical diameter of the upper limit $W_{50}$ stone, whichever results in the greater thickness.

2. The thickness determined by (1) above should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement. At one location in the US Army Engineer Division, Missouri River, divers and sonic sounders were used to reduce the underwater thickness to 1.25 times the dry placement thickness.

Section II

Channel Characteristics

3-3. Side Slope Inclination

The stability of riprap slope protection is affected by the steepness of channel side slopes. Side slopes should ordinarily not be steeper than 1V on 1.5H, except in special cases where it may be economical to use larger hand-placed stone keyed well into the bank. Embankment stability analysis should properly address soils characteristics, groundwater and river conditions, and probable failure mechanisms. The size of stone required to resist the erosive forces of channel flow increases when the side slope angle approaches the angle of repose of a riprap slope protection. Rapid water-level recession and piping-initiated failures are other factors capable of affecting channel side slope inclination and needing consideration in design.

3-4. Channel Roughness, Shape, Alignment, and Gradient

As boundary shear forces and velocities depend on channel roughness, shape, alignment, and invert gradient, these factors must be considered in determining the size of stone required for riprap revetment. Comparative cost estimates should be made for several alternative channel plans to determine the most economical and practical combination of channel factors and stone size. Resistance coefficients (Manning’s $n$) for riprap placed in the dry should be estimated using the following form of Strickler’s equation:

$$n = K \left[ D_{50}(\text{min}) \right]^{0.6}$$  \hspace{1cm} (3-2)

where

- $K = 0.036$, average of all flume data
- $= 0.034$ for velocity and stone size calculation
- $= 0.038$ for capacity and freeboard calculation

$D_{50}(\text{min})$ = size of which 90 percent of sample is finer, from minimum or lower limit curve of gradation specification, ft

The $K$ values represent the upper and lower bounds of laboratory data determined for bottom riprap. Resistance data from a laboratory channel which had an irregular surface similar to riprap placed underwater show a Manning’s $n$ about 15 percent greater than for riprap placed in the dry. Equation 3-2 provides resistance losses due to the surface roughness of the riprap and does not include form losses such as those caused by bends. Equation 3-2 should be limited to slopes less than 2 percent.

Section III

Design Guidance for Stone Size

3-5. General

Riprap protection for open channels is subjected to hydrodynamic drag and lift forces that tend to erode the revetment and reduce its stability. Undermining by scour beyond the limits of protection is also a common cause of failure. The drag and lift forces are created by flow velocities adjacent to the stone. Forces resisting motion are the submerged weight of the stone and any downward and lateral force components caused by contact with other stones in the revetment. Stone availability and experience play a large part in determining size of riprap. This is particularly true on small projects where hydraulic parameters are ill-defined and the total amount of riprap required is small.
3-6. Design Conditions

Stone size computations should be conducted for flow conditions that produce the maximum velocities at the riprapped boundary. In many cases, velocities continue to increase beyond bank-full discharge; but sometimes backwater effects or loss of flow into the overbanks results in velocities that are less than those at bank-full. Riprap at channel bends is designed conservatively for the point having the maximum force or velocity. For braided channels, bank-full discharges may not be the most severe condition. At lesser flows, flow is often divided into multiple channels. Flow in these channels often impinges abruptly on banks or levees at sharp angles.

3-7. Stone Size

This method for determining stone size uses depth-averaged local velocity. The method is based on the idea that a designer will be able to estimate local velocity better than local boundary shear. Local velocity and local flow depth are used in this procedure to quantify the imposed forces. Riprap size and unit weight quantify the resisting force of the riprap. This method is based on a large body of laboratory data and has been compared to available prototype data (Maynord 1988). It defines the stability of a wide range of gradations if placed to a thickness of 1D_{100}(max). Guidance is also provided for thickness greater than 1D_{100}(max). This method is applicable to side slopes of 1V on 1.5H or flatter.

a. Velocity estimation. The characteristic velocity for side slopes V_{ss} is the depth-averaged local velocity over the slope at a point 20 percent of the slope length from the toe of slope. Plate 33 presents the ratio V_{ss}/V_{avg}, where V_{avg} is the average channel velocity at the upstream end of the bend, as a function of the channel geometry, which is described by R/W, where R is the center-line radius of bend and W is the water-surface width. V_{avg}, R, and W should be based on flow in the main channel only and should not include overbank areas. The trapezoidal curve for V_{ss}/V_{avg} shown in Plate 33 is based on the STREM model described in Bernard (1993). The primary factors affecting velocity distribution in riprap-lined, trapezoidal channel bendways are R/W, bend angle, and aspect ratio (bottom width/depth). Data in Maynord (1992) show a trapezoidal channel having the same bottom width but side slopes ranging from 1V:1.5H to 1V:3H to have the same maximum V_{ss}/V_{avg} at the downstream end of the bend. Plate 33 should be used for side slopes from 1V:3H to 1V:1.5H. For straight channels sufficiently far (>5W) from upstream bends, large values of R/W should be used, resulting in constant values of V_{ss}/V_{avg}. Very few channels are straight enough to justify using V_{ss}/V_{avg} < 1. A minimum ratio of V_{ss}/V_{avg} = 1 is recommended for side slopes in straight channels. Rock stability should be checked for both side slopes and the channel bottom. In bendways, the outer bank side slope will generally require the largest rock size. In straight reaches, the channel bottom will often require the largest stone size. Velocities in the center of a straight channel having equal bottom and side slope roughness range from 10 to 20 percent greater than V_{avg}. Plate 34 describes V_{ss} and Plate 35 shows the location in a trapezoidal channel bend of the maximum V_{ss}. Velocity downstream of bends decays at approximately the following rate: No decay in first channel width downstream of bend exit; decay of V_{ss}/V_{avg} = 0.1 per channel width until V_{ss}/V_{avg} = 1.0. Plate 36 shows the variation in velocity over the side slope in a channel. The straight channel curve in Plate 36 was found applicable to both 1V:2H and 1V:3H side slopes. The bend curve for R/W = 2.6 was taken from a channel having strong secondary currents and represents a severe concentration of high velocity upon the channel side slope. These two curves represent the extremes in velocity distribution to be expected along the outer bank of a channel bend having a riprap side slope from toe of bank to top of bank. Knowing V_{ss} from Plate 33, the side slope velocity distribution can be determined at the location of V_{ss}. An alternate means of velocity estimation based on field observation is discussed in Appendix G. The alpha method (Appendix C), or velocities resulting from subsections of a water-surface profile computation, should be used only in straight reaches. When the alpha method is used, velocity from the subsection adjacent to the bank subsection should be used as V_{ss} in design of bank riprap.

b. Stone size relations. The basic equation for the representative stone size in straight or curved channels is

\[
D_{30} = S f C_{e} C_{d} \left[ \frac{\gamma u}{\gamma_{w} \gamma_{e}} \right]^{2} \left( \frac{V}{\sqrt{K_{g} d}} \right) ^{2.5}
\]  

(3-3)

where

\[
D_{30} = \text{riprap size of which 30 percent is finer by weight,}
\]

length
\( S_f \) = safety factor (see c below)  
* \( C_f \) = stability coefficient for incipient failure,  
\[ D_{50}/D_{15} = 1.7 \text{ to } 5.2 \]  
= 0.30 for angular rock  
* = 0.375 for rounded rock  
\( C_v \) = vertical velocity distribution coefficient  
= 1.0 for straight channels, inside of bends  
= 1.283 - 0.2 \log (R/W), outside of bends (1 for \( R/W > 26 \))  
= 1.25, downstream of concrete channels  
= 1.25, ends of dikes  
\( C_T \) = thickness coefficient (see d(1) below)  
* = 1.0 for thickness = 1D_{90}(max) or 1.5 D_{50}(max), whichever is greater  
* = local depth of flow, length (same location as \( V \))  
\( \gamma_w \) = unit weight of water, weight/volume  
* \( V \) = local depth-averaged velocity, \( V_{95} \) for side slope riprap, length/time  
\( K_1 \) = side slope correction factor (see d(1) below)  
\( g \) = gravitational constant, length/time\(^2\)  
* Some designers prefer to use the traditional \( D_{50} \) in riprap design. The approximate relationship between \( D_{50} \) and \( D_{95} \) is \( D_{95} = D_{50} \left(D_{95}/D_{15}\right)^{1/3} \). Equation 3-3 can be used with either SI (metric) or non-SI units and should be limited to slopes less than 2 percent.  

\( c. \) Safety factor: Equation 3-3 gives a rock size that should be increased to resist hydrodynamic and a variety of nonhydrodynamic-imposed forces and/or uncontrollable physical conditions. The size increase can best be accomplished by including the safety factor, which will be a value greater than unity. The minimum safety factor is \( S_f = 1.1 \). The minimum safety factor may have to be increased in consideration for the following conditions:  

(1) Imposed impact forces resulting from logs, uprooted trees, loose vessels, ice, and other types of large floating debris. Impact will produce more damage to a lighter weight riprap section than to a heavier section. For moderate debris impact, it is unlikely that an added safety factor should be used when the blanket thickness exceeds 15 in.  

(2) The basic stone sizing parameters of velocity, unit weight of rock, and depth need to be determined as accurately as possible. A safety factor should be included to compensate for small inaccuracies in these parameters. If conservative estimates of these parameters are used in the analysis, the added safety factor should not be used. The safety factor should be based on the anticipated error in the values used. The following discussion shows the importance of obtaining nearly correct values rather than relying on a safety factor to correct inaccurate or assumed stone sizing parameters. The average velocity over the toe of the riprap is an estimate at best and is the parameter to which the rock size is the most sensitive. A check of the sensitivity will show that a 10 percent change in velocity will result in a nearly 100 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 30 percent change in the riprap thickness. The riprap size is also quite sensitive to the unit weight of the rock to be used: a 10 percent change in the unit weight will result in a 70 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 20 percent change in the riprap thickness. The natural variability of unit weight of stone from a stone source adds to the uncertainty (EM 1110-2-2302).  

* The rock size is not nearly as sensitive to the depth parameter.  

(3) Vandalism and/or theft of the stones is a serious problem in urban areas where small riprap has been placed. A \( W_{50}(min) \) of 80 lb should help prevent theft and vandalism. Sometimes grouted stone is used around vandalism-prone areas.  

(4) The completed revetment will contain some pockets of undersized rocks, no matter how much effort is devoted to obtaining a well-mixed gradation throughout the revetment. This placement problem can be assumed to occur on any riprap job to some degree but probably more frequently on jobs that require stockpiling or additional handling. A larger safety factor should be considered with stockpiling or additional hauling and where placement will be difficult if quality control cannot be expected to address these problems.  

(5) The safety factor should be increased where severe freeze-thaw is anticipated.
d. **Applications.**

(1) The outer bank of straight channels downstream of bends should be designed using velocities computed for the bend. In projects where the cost of riprap is high, a channel model to indicate locations of high velocity might be justified. Equation 3-3 has been developed into Plate 37, which is applicable to thicknesses equal to *1D* 165 max, *γ* of 165 pcf, and the *S* of 1.1. Plate 38 is used to correct for values of other than *γ* of 165 pcf (when *D* 10 is determined from Plate 37). The *K* 1 side slope factor is normally defined by the relationship of Carter, Carlson, and Lane (1953)

\[
K_1 = \frac{1 - \sin^2 \theta}{\sin^2 \phi}\tag{3-4}
\]

where

- \(\theta\) = angle of side slope with horizontal
- \(\phi\) = angle of repose of riprap material (normally 40 deg)

Results given in Maynord (1988) show Equation 3-4 to be conservative and that the repose angle is not a constant 40 deg but varies with several factors. The recommended relationship for *K* 1 as a function of *θ* is given in Plate 39 along with Equation 3-4 using \(\phi = 40\) deg. Using the recommended curve for side slope effects, the least volume of rock per unit length of bank line occurs on a 1V-1.5H to 1V-2H side slope. Also shown on Plate 39 is the correction for side slope when *D* 30 is determined from Plate 37. Correction for the vertical velocity distribution in bends is shown in Plate 40. Testing has been conducted to determine the effects of blanket thickness greater than *1D* 165 (max) on the stability of riprap. Results are shown in Plate 40. The thickness coefficient *C* 2 accounts for the increase in stability that occurs when riprap is placed thicker than the minimum thickness of *1D* 165 (max) or 1.5 *D* 50 (max), whichever is greater.

(2) The basic procedure to determine riprap size using the graphical solution of this method is as follows:

(a) Determine average channel velocity (HEC-2 or other uniform flow computational methods, or measurement).

(b) Find *V* 20 using Plate 33.

(c) Find *D* 50 using Plate 37.

(d) Correct for other unit weights, side slopes, vertical velocity distribution, or thicknesses using Plates 38 through 40.

(e) Find gradation having *D* 50 (min) ≥ computed *D* 50. Alternately Equation 3-3 is used with Plates 39 and 40 to replace steps (c) and (d).

(3) This procedure can be used in both natural channels with bank protection only and prismatic channels having riprap on bed and banks. Most bank protection sections can be designed by direct solution. In these cases, the extent of the bank compared to the total perimeter of the channel means that the average channel velocity is not significantly affected by the riprap. The first example in Appendix H demonstrates this type of bank protection.

(4) In some cases, a large part of the channel perimeter is covered with riprap; the average channel velocity, depth, and riprap size are dependent upon one another; and the solution becomes iterative. A trial riprap gradation is first assumed and resistance coefficients are computed using Equation 3-2. Then the five steps described in (2) above are conducted. If the gradation found in paragraph (e) above is equal to the assumed trial gradation, the solution is complete. If not, a new trial gradation is assumed and the procedure is repeated. The second example in Appendix H demonstrates this type of channel riprap.

(5) In braided streams and some meandering streams, flow is often directed into the bank line at sharp angles (angled flow impingement). For braided streams having impinged flow, the above stone sizing procedures require modification in two areas: the method of velocity estimation and the velocity distribution coefficient *C* 2. All other factors and coefficients presented are applicable.

(a) The major challenge in riprap design for braided streams is estimating the imposed force at the impingement point. Although unproven, the most severe bank...
* attack in braided streams is thought to occur when the water surface is at or slightly above the tops of the midchannel bars. At this stage, flow is confined to the multiple channels that often flow into or "impinge" against bank lines or levees. At lesser flows, the depths and velocities in the multiple channels are decreased. At higher flows, the channel area increases drastically and streamlines are in a more downstream direction rather than into bank lines or levees.

(b) The discharge that produces a stage near the tops of the midchannel bars is \( Q_{mb} \). \( Q_{mb} \) is probably highly correlated with the channel-forming discharge concept. In the case of the Snake River near Jackson, Wyoming, \( Q_{mb} \) is 15,000-18,000 cfs, which has an average recurrence interval of about 2-5 years. Using cross-section data to determine the channel area below the tops of the midchannel bars and \( Q_{mb} \) allows determination of the average channel velocity at the top of the midchannel bars, \( V_{mb} \).

(c) Field measurements at impingement sites were taken in 1991 on the Snake River near Jackson, Wyoming, and reported in Maynord (1993). The maximum observed ratio \( V_{ss}/V_{mb} = 1.6 \), which is almost identical to the ratio shown in Plate 33 for sharp bendways having \( R/W = 2 \) in natural channels, and this ratio is recommended for determining \( V_{ss} \) for impinged flow. The second area of the design procedure requiring modification for impinged flow is the velocity distribution coefficient \( C_v \), which varies with \( R/W \) in bendways as shown in Plate 40. Impinged flow areas are poorly aligned bends having low \( R/W \), and \( C_v = 1.25 \) is recommended for design.

(6) Transitions in size or shape may also require riprap protection. The procedures in this paragraph are applicable to gradual transitions where flow remains tranquil. In areas where flow changes from tranquil to rapid and then back to tranquil, riprap sizing methods applicable to hydraulic structures (HDC 712-1) should be used. In converging transitions, the procedures based on Equation 3-3 can be used unaltered. In expanding transitions, flow can concentrate on one side of the expansion and design velocities should be increased. For installations immediately downstream of concrete channels, a vertical velocity distribution coefficient of 1.25 should be used due to the difference in velocity profile over the two surfaces.

\[ D_{30} = \frac{1.95 \cdot S^{0.55} \cdot q^{2.9}}{g^{1.5}} \quad (3-5) \]

where

- \( S \) = slope of bed
- \( q \) = unit discharge

Equation 3-5 is applicable to thickness = 1.5 \( D_{100} \), angular rock, unit weight of 167 pcf, \( D_{50}/D_{15} \) from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope with no tailwater. The following steps should be used in application of Equation 3-5:

1. Estimate \( q = Q/b \) where \( b = \) bottom width of chute.
2. Multiply \( q \) by flow concentration factor of 1.25. Use greater factor if approach flow is skewed.
3. Compute \( D_{30} \) using Equation 3-5.
4. Use uniform gradation having \( D_{50}/D_{15} \leq 2 \) such as Table 3-1.
5. Restrict application to straight channels with side slope of 1V:2.5H or flatter.

The guidance for steep slope riprap generally results in large riprap sizes. Grouted riprap is often used instead of loose riprap in steep slope applications.

3-8. Revetment Top and End Protection

Revetment top and end protection requirements, as with all channel protective measures, are to assure the project benefits, to perform satisfactorily throughout the project economic life, and not to exceed reasonable maintenance.
costs. Reference is made to ER 1110-2-1405, with emphasis on paragraph 6c.

a. Revetment top. When the full height of a levee is to be protected, the revetment will cover the freeboard, i.e., extend to the top of the levee. This provides protection against waves, floating debris, and water-surface irregularities. Similar provisions apply to incised channel banks. A horizontal collar, at the top of the bank, is provided to protect against escaping and returning flows as necessary. The end protection methods illustrated in Plate 41 can be adapted for horizontal collars. Plate 36 provides general guidance for velocity variation over channel side slopes that can assist in evaluating the economics of reducing or omitting revetment for upper bank areas. Revetment size changes should not be made unless a sufficient quantity is involved to be cost effective. Many successful revetments have been constructed where the top of the revetment was terminated below the design flow line. See USACE (1981) for examples.

b. Revetment end protection. The upstream and downstream ends of riprap revetment should be protected against erosion by increasing the revetment thickness T or extending the revetment to areas of nonerosing velocities and relatively stable banks. A smooth transition should be provided from where the revetment begins to the design riprap section. The key-in section should satisfy filter requirements. The following guidance applies to the alternative methods of end protection illustrated in Plate 41.

(1) Method A. For riprap revetments 12 in. thick or less, the normal riprap layer should be extended to areas where velocities will not erode the natural channel banks.

(2) Method B. For riprap revetments exceeding 12 in. in thickness, one or more reductions in riprap thickness and stone size may be required (Plate 41) until velocities decrease to a nonerosing natural channel velocity.

(3) Method C. For all riprap revetments that do not terminate in nonerosing natural channel velocities, the ends of the revetment should be enlarged, as shown in Plate 41. The decision to terminate the revetment in erosive velocities should be made with caution since severe erosion can cause the revetment to fail by progressive flanking.

c. Length. Riprap revetment is frequently carried too far upstream and not far enough downstream of a channel bend. In a trapezoidal channel, the maximum velocities along the outer bank are often located in the straight reach immediately downstream of the bend for relatively large distances downstream. In a natural channel, the limit of protection on the downstream end should depend on where the flow crosses to the opposite bank, and should consider future bar building on the opposite bank, resulting in channel constriction and increased velocities. Guidance is generally lacking in this area, but review of aerial photographs of the subject location can provide some insight on where the crossover flow occurs. Model tests in a sand bed and bank flume (USACE 1981) were conducted to determine the limits of protection required to prevent scour that would lead to destruction of the revetment. These tests were conducted in a 110-deg bend having a constant discharge. The downstream end of the revetment had to be 1.5 channel widths downstream of the end of the bend. Geomorphic studies to determine revetment ends should be considered.

Section IV
Revettment Toe Scour Estimation and Protection

3-9. General

Toe scour is probably the most frequent cause of failure of riprap revetments. This is true not only for riprap, but also for a wide variety of protection techniques. Toe scour is the result of several factors, including these three:

a. Meandering channels, change in cross section that occurs after a bank is protected. In meandering channels the thalweg often moves toward the outer bank after the bank is protected. The amount of change in cross section that occurs after protection is added is related to the erodibility of the natural channel bed and original bank material. Channels with highly erodible bed and banks can experience significant scour along the toe of the new revetment.

b. Meandering channels, scour at high flows. Bed profile measurements have shown that the bed observed at low flows is not the same bed that exists at high flows. At high flows the bed scours in channel bends and builds up in the crossings between bends. On the recession side of the flood, the process is reversed. Sediment is eroded from the crossings and deposited in the bends, thus obscuring the maximum scour that had occurred.

c. Braided channels. Scour in braided channels can reach a maximum at intermediate discharges where flow in the channel braids attacks banks at sharp angles.
Note that local scour is the mechanism being addressed herein. When general bed degradation or headcutting is expected, it must be added to the local scour. When scour mechanisms are not considered in the design of protection works, undermining and failure may result. *Plate 42 may be used for depth of scour estimates. The design curve in Plate 42 represents an upper limit for scour in channels having irregular alignments. For bendways having a relatively smooth alignment, a 10 percent reduction from the design curve is recommended. Neill (1973) provides additional information on scour depth estimation.

3-10. Revetment Toe Protection Methods

Toe protection may be provided by two methods:

a. Extend to maximum scour depth. Place the lower extremity below the expected scour depth or found it on nonerodible material. These are the preferred methods, but they can be difficult and expensive when underwater excavation is required.

b. Place launchable stone. Place sufficient launchable stone to stabilize erosion. Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. As the attack and resulting erosion occur below the stone, the stone is undermined and rolls/slides down the slope, stopping the erosion. This method has been widely used on sand bed streams. Successful applications include:

   (1) Windrow revetments: riprap placed at top of bank.

   (2) Trench-fill revetments: riprap placed at low water level.

   (3) Weighted riprap toes: riprap placed at intersection of channel bottom and side slope.

Trench-fill revetments on the Mississippi River have successfully launched to protect for a vertical scour depth of up to 50 ft. On gravel bed streams, the use of launchable stone is not as widely accepted as in sand bed streams. Problems with using launchable stone in some gravel bed rivers may be the result of underestimating stone size, scour depth, or launchable stone volume because the concept of launchable stone has been successful on several gravel bed rivers.

3-11. Revetment Toe Protection Design

The following guidance applies to several alternative methods of toe protection illustrated in Plate 43.

a. Method A. When toe excavation can be made in the dry, the riprap layer may be extended below the existing groundline a distance exceeding the anticipated depth of scour. If excavation quantities are prohibitive, the concept of Method D can be adapted to reduce excavation.

b. Method B. When the bottom of the channel is nonerodible material, the normal riprap should be keyed in at streambed level.

c. Method C. When the riprap is to be placed underwater and little toe scour is expected (such as in straight reaches that are not downstream of bends, unless stream is braided), the toe may be placed on the existing bottom with height \( a \) and width \( c \) equal to \( 1.5T \) and \( 5T \), respectively. This compensates for uncertainties of underwater placement.

d. Method D. An extremely useful technique where water levels prohibit excavation for a toe section is to place a launchable section at the toe of the bank. Even if excavation is practicable, this method may be preferred for cost savings if the cost of extra stone required to produce a launched thickness equal to or greater than \( T \) plus the increase shown in Table 3-2 is exceeded by the cost of excavation required to carry the design thickness \( T \) down the slope. This concept simply uses toe scour as a substitute for mechanical excavation. This method also has the advantage of providing a "built-in" scour gage, allowing easy monitoring of high-flow scour and the need for additional stone reinforcement by visual inspection of the remaining toe stone after the high flow subsides or by surveyed cross sections if the toe stone is underwater. It is readily adaptable to emergency protection, where high flow and the requirement for quick action make excavation impractical. Shape of the stone section before launching is not critical, but thickness of the section is important because thickness controls the rate at which rock is released in the launching process. For gradual scour in regular bendways, the height of the stone section before launching should be from 2.5 to 4.0 times the bank protection thickness (T). For rapid scour in impinged flow environments or in gravel bed streams, the stone section height before launching should be 2.5 to 3.0 T. In
Table 3-2
Increase in Stone Volume for Riprap Launching Sections

<table>
<thead>
<tr>
<th>Vertical Launch Distance, ft(^1)</th>
<th>Dry Placement</th>
<th>Underwater Placement</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 15</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>&gt; 15</td>
<td>50</td>
<td>75</td>
</tr>
</tbody>
</table>

Note: \(^1\) From bottom of launch section to maximum scour.

any case, the thinner and wider rock sections represented by the lower values of thickness have an apparent advantage in that the rock in the stream end of the before-launch section has a lesser distance to travel in the launching process. Providing an adequate volume of stone is critical. Stone is lost downstream in the launching process; and the larger the scour depth, the greater the percentage of stone lost in the launching process. To compute the required launchable stone volume for Method D, the following assumptions should be used:

1. Launch slope = 1V on 2H. This is the slope resulting from rock launched on noncohesive material in both model and prototype surveys. Launch slope is less predictable if cohesive material is present, since cohesive material may fall in large blocks.

2. Scour depth = existing elevation - maximum scour elevation.

3. Thickness after launching = thickness of the bank revetment T.

To account for the stone lost during launching and for placement underwater, the increases in stone volume listed in Table 3-2 are recommended. Using these assumptions, the required stone volume for underwater placement for vertical launch distance less than 15 ft = 1.5T times launch slope length

\[
= 1.5T \times \text{scour depth times } \sqrt{5}
\]

\[
= 3.35T \times (\text{scour depth})
\]

Add a safety factor if data to compute scour depth are unreliable, if cohesive bank material is present, or if monitoring and maintenance after construction cannot be guaranteed. Guidance for a safety factor is lacking, so to some extent it must be determined by considering consequences of failure. Widely graded ripraps are recommended because of reduced rock voids that tend to prevent leaching of lower bank material through the launched riprap. Launchable stone should have \(D_{50}/D_{15} \geq 2\).

3-12. Delivery and Placement

Delivery and placement can affect riprap design. See EM 1110-2-2302 for detailed guidance. The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the most stable riprap revetment because the long axes of the riprap particles are oriented perpendicular to the bank. It is the most expensive method except when stone is unusually costly and/or labor unusually cheap. Steeper side slopes can be used with hand-placed riprap than with other placing methods. This reduces the required volume of rock. However, the greater cost of hand placement usually makes machine or dumped placement methods and flatter slopes more economical. Hand placement on steep slopes should be considered when channel widths are constricted by existing bridge openings or other structures when rights-of-way are costly. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone. Stone should not be dropped from an excessive height or dumped and spread as this may result in the same undesirable conditions. However, in some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method. Smooth, compact riprap sections have resulted from compacting the placed stone sections with a broad-tracked bulldozer. This stone must be quite resistant to abrasion. Thickness for underwater placement should be increased by 50 percent to provide for the uncertainties associated with this type of placement. Underwater placement is usually specified in terms of weight of stone per unit area, to be distributed uniformly and controlled by a “grid” established by shoreline survey points.

Section V
Ice, Debris, and Vegetation

3-13. Ice and Debris

Ice and debris create greater stresses on riprap revetment by impact and flow concentration effects. Ice attachment to the riprap also causes a decrease in stability. The Cold Regions Research Engineering Laboratory, Hanover, NH, should be contacted for detailed guidance relative to ice
effects on riprap. One rule of thumb is that thickness should be increased by 6-12 in., accompanied by appropriate increase in stone size, for riprap subject to attack by large floating debris. Riprap deterioration from debris impacts is usually more extensive on bank lines with steep slopes. Therefore, riprapped slopes on streams with heavy debris loads should be no steeper than 1V on 2.5H.

3-14. Vegetation

The guidance in this chapter is based on maintaining the riprap free of vegetation. When sediment deposits form lowflow berms on riprap installations, vegetation may be allowed on these berms under the following conditions: roots do not penetrate the riprap; failure of the riprap would not jeopardize project purposes prior to repairs; and the presence of the berm and vegetation does not significantly reduce the discharge capacity of the project. For riprap areas above the 4 or 5 percent exceedence flow line, consideration may be given to overlaying the riprap with soil and sod to facilitate maintenance by mowing rather than by hand or defoliants. This may be particularly appropriate for riprap protecting against eddy action around structures such as gate wells and outlet works in levees that are otherwise maintained by mowing.

Recognizing that vegetation is, in most instances, inimical to riprap installations, planned use of vegetation with riprap should serve some justifiable purpose, be accounted for in capacity computations, be controllable throughout the project life, have a strengthened riprap design that will withstand the additional exigencies, and account for increased difficulty of inspection.

Section VI
Quality Control

3-15. Quality Control

Provisions should be made in the specifications for sampling and testing in-place riprap as representative sections of revetment are completed. Additional sample testing of in-place and in-transit riprap material at the option of the Contracting Officer should be specified. The primary concern of riprap users is that the in-place riprap meets specifications. Loading, transporting, stockpiling, and placing can result in deterioration of the riprap. Coordination of inspection efforts by experienced staff is necessary. Reference EM 1110-2-2302 for detailed sampling guidance and required sample volumes for in-place riprap.
ATTACHMENT 6A
SOME OBSERVATIONS RELATIVE TO THE
PERFORMANCE OF FLOW-THROUGH ROCK DRAINS
(CAMPBELL, 1989)
SOME OBSERVATIONS RELATIVE TO THE PERFORMANCE OF FLOW-THROUGH ROCK DRAINS

by

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INTRODUCTION

In Sept 1986, an International Symposium on Flow-Through Rock Drains was held in Cranbrook, British Columbia. At the Cranbrook Symposium, Campbell (1986) addressed some of the concerns that have been expressed, regarding the long-term performance of rock drains. The 1986 paper presented arguments and data which showed that some of the concerns that have been expressed, are not supported by field and laboratory data that have been collected.

The author has been requested to provide an update of data and information that have been obtained, subsequent to the Cranbrook Symposium. This paper presents a brief summary of some field observations that have been made at rock drains, together with an assessment of what these observations indicate with respect to rock drain performance.
PARTICLE BREAKAGE DUE TO HIGH STRESSES

John Wilkins, an Engineer who worked with the Tasmanian Hydroelectric Authority, was one of the pioneer investigators of the parameters governing turbulent flow through coarse rock (Wilkins, 1956). The metric version of the results of Wilkins' investigations, is as follows:

\[ V_v = 5.28(eD/7.5)^{0.5}S^{0.5} \] (Equation 1)

Where:

- \( V_v \) is the average velocity of flow-through the voids within the mass of the coarse rock fragments.
- \( D \) is the mean stone size (metres)
- \( e \) is the void ratio
- \( S \) is the hydraulic gradient through the rock drain

The term inside the brackets in equation 1 represents the hydraulic radius, that is, the mean area through which flow takes place, divided by the area of the wetted surface on the flow boundaries. Equation 1 shows that the rate of flow-through a rock drain, per unit area of the wetted cross section, is governed by the mean size of the rock fragments, and by the void ratio of the mass of the rock fragments comprising the rock drain.

When the zone of coarse segregated rock comprising a rock drain becomes covered, the weight of the waste rock above the drain must be carried by transfer of load, at the points of contact between the adjacent rock fragments. Broken rock is angular. Consequently, the contact areas between the adjacent blocks of coarse rock are small. These small contact areas result in high stresses at the points of contact, which in turn result in rock crushing and rock fracture.

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Both crushing and fracture of the rock result in a reduction in the mean size of the rock fragments and a reduction in the void ratio, relative to what would have been observed before the rock drain materials became covered. Reduction in particle size, and in void ratio result in a reduction in the hydraulic radius. Equation (1) shows that a reduction in the hydraulic radius results in a reduction in the through-flow capacity of the rock drain.

There is seldom an opportunity to observe the effect of rock fracture and point-to-point crushing within a rock drain after it has become covered. However, one such opportunity became available at the Fording Coal Ltd., Fording River Operations during the winter of 1987, as No. 1 Spoil, was being excavated.

No. 1 Spoil was the first waste rock dump developed at the Fording Coal property. Development of this spoil was required to permit commencement of the open pit mining operations on the property. It was known that the spoil was located above coal reserves, and that its removal would be required at a later date.

In January, 1987, a shovel breakdown occurred as excavation was in progress in an area where the base of No. 1 Spoil is in contact with the natural foundation on which it rests. The short interruption in the mining activity provided an opportunity to examine the condition of the zone of coarse segregated rock at the base of the No. 1 Spoil. Temperatures at the time were below freezing, and the small amount of cohesion provided by minor ice in the waste rock permitted the excavated spoil face to stand nearly vertical.

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April, 1989

The condition of the rocks comprising the coarse segregated zone at the base of the No. 1 Spoil, after it had been covered for about 12 years by 50 metres of waste rock fill, is illustrated in photos 1, and 2. Fracturing of individual rocks, as a result of high stresses at the points of contact between adjacent blocks, is evident in the photographs. However, the zone of coarse rock at the base of the dump remained porous.

POTENTIAL PARTICLE MIGRATION WITHIN A DUMP

In reference 1, gradation curves for a laboratory model waste dump are presented. The distribution of particle sizes in the vertical direction, as obtained from the model dump, constitutes a well graded filter, and indicates that downward migration of particles within the body of a waste rock dump cannot occur. A similar conclusion is indicated by the gradation curves from field trials conducted by Nichols (1986), and by the gradation of the rock sizes on the face of the Crows Nest Resources' West Line Creek Dump, as presented in Reference 1.

Examination of the waste rock at the base of the No. 1 Spoil did not reveal any evidence of the presence of fines that could have originated from segments of the dump above the segregated zone at the base. The January '87 inspection of No. 1 Spoil provided field data that supports the conclusion that downward particle migration within the body of a waste rock dump is precluded.

BEDLOAD AND SUSPENDED SEDIMENT

During the winter of 1986-87, a rock fill was advanced across the North Fork of Rose Creek near Faro, Yukon. This rock fill was constructed to provide access to a

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proposed new mining area. At the location where it crosses the bottom of the North Fork valley, the roadway fill is approximately 60 metres high.

The fill for the roadway, at the location of the creek crossing, consists of calcium silicate, a hard metamorphic rock. The rock fill was placed by end dumping from roadway level. Segregation of the large fragments of the waste rock, in the course of their transit down the face of the fill, and the accumulation of these coarse fragments at the toe of the advancing fill, constitutes a rock drain. This rock drain conducts the flows in the North Fork of Rose Creek through the base of the roadway fill.

The North Fork rock drain first operated in the spring of 1987, when it passed a maximum flow of 7 cubic metres per second, as measured on the downstream side of the rock drain.

Concerns have been expressed about the potential clogging of rock drains, as a result of deposition of suspended sediments within the drain, or as a result of deposition of bedload at the inlet end of the drain. Inspection of the North Fork rock drain, in May, 1987, provided some information germane to both of these concerns.

The presence of even the most pervious rock drain within a drainage course represents an impedance to flow, relative to the conditions in the open channel prior to development of the rock drain. Thus for quasi-equilibrium to be maintained, the area of the gross wetted cross section within the drain must be greater than the area of the wetted cross section in the open channel leading to the drain. If the rate of flow in the open channel above the rock drain were to remain constant, the water level in the rock drain would rise to a level such that the combination of gross wetted cross section and the

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hydraulic gradient, would result in a rate of discharge though the drain equal to the rate of flow in the open channel.

The fact that the free water surface in the rock drain must rise to accommodate the flow, implies that a pool must form at the inlet end of the rock drain. This is true for all rock drains.

As the water in the open channel enters the pool upstream of the inlet to a rock drain, the flow velocities decrease abruptly. As a result of this reduction in velocity, the bedload sediments that may be in transit along the bottom of the stream are deposited at the inlet to the pool. It is this mechanism that is responsible for the formation of deltas at the locations where rivers and streams enter lakes and oceans. Since the bedload sediments are deposited at the upstream end of the pool, they are not transported to the inlet of the rock drain, and do not result in blockage of the inlet to the drain.

When the North Fork rock drain was inspected in May 1988, it was evident that the pool on the upstream end of the drain had been at a higher level than it was at the time of the inspection. The level to which the pool had previously risen is indicated by the wood debris on the upstream face of the rock fill, as shown in Photo No. 3.

Within the range of fluctuation in the pool surface level, fine-grained sediment had been deposited on the upper surface of boulders at the inlet end of the drain. This sediment is visible on the surface of the boulders shown in Photo No. 3. The grain size of the sediments that had settled on the surface of the boulders, is an indication of the size of the suspended solids that had been transported to the inlet of the rock drain. A size

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analyses of a sample of the sediment, collected from the surface of the boulders, showed that all of the particles were smaller than 0.074 mm; the No. 200 US Standard Sieve size.

By use of equation (1), the velocity of flow through the void spaces within the rock drain is estimated to have been approximately 0.1 metres per second. The settling velocity of a 0.074 mm size particle would be approximately 3.4 mm metres per second, or about 1/30th of the average velocity of flow through the drain. In a turbulent flow field, particles having a settling velocity of about 3 per cent of the flow velocity, would not settle within the drain; these particles would be swept through the rock drain with the flow.

The field observations that the author has made subsequent to the September '86 Cranbrook Symposium on through-flow rock drains provide confirmation of some of the conclusions stated in reference 1. These conclusion are:

1. End dumping results in a distribution of particle sizes grading from fine to progressively coarser proceeding from the level of the dump platform toward the base of the dump. This gradual reduction in particle sizes constitutes a well-graded filter that precludes downward migration of particles within the body of a dump.

2. During periods of significant stream discharge, a pool will always be present at the inlet end of a rock drain. This pool serves to trap bedload at the location where the stream inter the pool. Consequently deposition of bedload does not result in blockage at the inlet to the rock drain.

3. The pool at the inlet end of a rock drain also results in deposition of suspended sediment. The data collected at the North Fork rock drain, as well as data for the Swift Creek rock drain (reported in reference 1) indicate that potential deposition of suspended sediment is not a factor that would result in reduction in the through-flow capacity of a rock drain.

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April, 1989

REFERENCES


DBC/gg
2/dc-pape
PHOTOGRAPH No 1

Illustrating rock fracturing as the result of high contact stresses, within the zone of coarse segregated rock at the base of Fordling Coal's No 1 Spoil. This zone had been buried for approximately 12 years, by about 50 meters of waste rock.

The marks on the measuring tape are at 1 decimeter spacing. The rock on the left hand side of the measuring tape has split vertically, and crushing is indicated by the series of closely-spaced vertical fractures in the region to the left of the 2 decimeter mark. Vertical fractures are also evident on the right side of the 1 decimeter mark.

PHOTOGRAPH No 2

Rock fracture, as a result of stresses imposed by the weight of the overlying rock fill, is indicated by the series of inclined cracks that intersect the upper portion of the block, in the region to the right of the 2 decimeter mark on the measuring tape.
Illustrating the size of the rocks at the upstream side of the North Fork rock drain. The wood debris at the level of the man's hardhat indicates a previous level of the pool on the upstream side of the rock drain. Silt had settled onto the surface of the boulders at the time of previous high water. The size of the silt particles is an indication of the size of the suspended solids that had been transported to the upstream end of the rock drain. A grain size analysis showed that all of the silt was finer than 0.074 mm. All particles of this size would have been transported through the rock drain.

Showing the flows exiting the downstream toe of the North Fork rock drain. The rate of discharge is between 3 and 4 cubic metres per second.
ATTACHMENT 6B
RESERVOIR ROUTING THROUGH SUCCESSIVE ROCKFILL DETENTION DAMS
(SAMANI, J.M.V.; AND SOLIMANI, A., 2007)
Reservoir Routing through Successive Rockfill Detention Dams

J. M. V. Samani and M. Heydari

ABSTRACT

Rock has been advantageously employed in hydraulic structures such as rockfill dams, gabion weirs and drain works. One rockfill dam applications can be flood control in watershed management. The objectives of building rockfill detention dams are flow storage for a specific period and lowering of the outflow hydrograph. As this type of dam consists of coarse particles, seepage flow will deviate from Darcy's law and mostly be turbulent. Under the practical conditions of watershed management, it might be necessary to build successive rockfill dams, where a final outflow hydrograph with lower peak flows and longer duration is needed. Due to their reciprocal effects, the hydraulics of successive rockfill detention dams are complicated. This paper describes a routing flow model through successive rockfill dams considering the storage among them and their effects on each other. In the developed model, the velocity has been introduced to the 1-D continuity equation as an exponential relationship between Reynolds number (Re) and the Darcy-Weisbach friction factor (f). By introducing the inflow hydrograph and rockfill characteristics as input data to the model, the outflow hydrograph can be determined through the storage routing method. The results of the developed model show good agreement with the experimental data collected for this investigation. The results show that the degree of peak reduction of the routed hydrograph depends on the number of successive rockfill dams, the distance between them, the average size of the rockfill material, and the dam dimensions.

Keywords: Non-Darcy flow, Reservoir routing, Rockfill dams.

INTRODUCTION

Many hydraulic structures have been constructed to utilize the water from rivers, but almost all of them have been made of concrete or steel in order to utilize the water to the utmost limit. These kinds of structures interrupt the natural flow of the river and reduce the auto-purification effect of the river. Therefore such structures have had a negative influence on the habitat environment of the river. With such circumstances as a background, nature-friendly river designing has been attracting attention in recent years. Rock can be used to build gabions, spillways, and groins (Stephenson, 1979) and a rockfill dam made of rocks is expected to be a suitable structure for the river.

Rockfill dam application is an economic and useful method for flood management purposes when suitable rock is available. Rockfill dams can be designed satisfactorily when the hydraulics of flow through the rockfill dam are known. This type of dam consists of coarse particles, and so the flow will deviate from Darcy's law and be mostly turbulent. This means that the relationship between the flow velocity, \( V \), and the hydraulic gradient, \( i \), is nonlinear. Various researchers have proposed the following

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nonlinear relationships (Herrera and Felton, 1991):
\[ i = A'V^B \]  \hspace{1cm} (1)
\[ i = A'V + B'V^2 \]  \hspace{1cm} (2)

where \( A, A', B, \) and \( B' \) are coefficients depending on the rock and fluid characteristics. Equations (1) and (2) were proposed by Prony in 1804 and Forchheimer in 1901, respectively (Li et al., 1998). Other researchers suggested relationships between Reynold's number (Re) and the Darcy-Weisbach friction factor (f) in the following forms (Herrera and Felton, 1991):
\[ f = a Re^b \]  \hspace{1cm} (3)
\[ f = \frac{a'}{Re} + b' \]  \hspace{1cm} (4)

where \( a, a', b, \) and \( b' \) are also coefficients which depend on the rock and fluid characteristics. Reynold's number is defined as:
\[ Re = \frac{V(d - \sigma)}{v} \]

where \( d \) is the average size of rock particles, \( \sigma \) is the standard deviation of the rock size distribution and \( v \) is kinematic viscosity. If the Reynold's number is written in terms of \( V \), the Darcy-Weisbach friction factor can be expressed in the form of Equations (1) and (2), respectively. Various researchers, such as Ergun (1989), Wilkins (1956), Ward (1964), Leps (1973), McCorquodale et al. (1978), Stephenson (1979), Herrera and Felton (1991), Hansen et al. (1995), Binghum et al. (1998), and Kataraman and Ramp (1998) proposed the above equations in their research. Findikakis and Tu (1985) introduced an equation to simulate flood routing by computing the flow profile through a rock dump using the continuity equation. A review of the different relationships proposed by various researchers was undertaken by Samani et al. (2003). They proposed 1-D and 2-D models for flow through rockfill dams (Samani et al., 2003). In their 1-D model, the following relationship has been used:
\[ f = 54.0 Re^{-0.677} \]  \hspace{1cm} (5)

In the present paper, a model is proposed to solve the problem of routing flow through successive rockfill dams considering the storage among rockfill dams reservoirs and their reciprocal effects on each other. The model is based on the 1-D continuity equation, which employs Eq.5 within the storage routing method.

**Model Development and Solution**

**Flow Rating Equation**

To develop the flow rating equation, it was shown (Samani et al., 2003) that:
\[ V = \alpha i^{b+2} \]  \hspace{1cm} (6)

where
\[ \alpha = \left( \frac{2gV^b}{a(d - \sigma)^{b+1}} \right) \]  \hspace{1cm} (7)

In Eq.7, \( a \) and \( b \) are Eq.3 coefficients, \( v \) is kinematic viscosity, \( d \) is average size of rock particles, \( g \) is acceleration due to gravity and \( \sigma \) is the standard deviation of the rock size distribution. Combining Eq.6 with the continuity equation and defining \( i \) as \( \frac{dh}{ds} \), yields:
\[ Q = \alpha \left( \frac{\frac{dh}{ds}}{h \cdot w} \right)^{\frac{1}{b+2}} \]  \hspace{1cm} (8)

where \( Q \) is outflow rate through dam, \( h \) is water depth inside the rockfill dam, \( w \) is the width of flow cross section, \( x \) is the longitudinal coordinate in the flow direction. Integrating Eq.8 between the limits \( H_{wp} \) to \( H_{down} \) for \( h \) and zero to \( D \) for \( x \) gives the following:
\[ Q = \left( \frac{1}{D} \right)^{\frac{1}{b+2}} \frac{\alpha \cdot w}{(3 + b)^{b+2}} \left( H_{wp} - H_{down} \right)^{\frac{1}{b+2}} \]  \hspace{1cm} (9)

where, according to Figure 1, \( D \) is defined according to Sharma (1991) as:
\[ D = L - 0.7S_t \]
\[ S_t = H_{wp} \cot \beta \]

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| \( l_f \) | 0.4f |
| \( \phi \) | 0.2f |
| \( \gamma \) | 1.1f |
| \( \theta \) | 77, 72 |
\[ D = L - 0.7H_{up} \cot \beta \]  

(10)

In Eq.10, \( \beta \) is the angle of the upstream and downstream face of the dam with the horizontal direction, \( H_{up} \) and \( H_{down} \) refer to dam upstream and downstream water depths, respectively, and \( L \) is dam length according to flow direction. Eq.9 is the flow rating equation for 1-D flow through rockfill dams (Samani et al., 2003).

**Reservoir Routing**

Figure 2 shows a number of rockfill dams successively located along the same path. During a flood, flow volume is stored among the successive reservoirs and, accordingly, the outflow hydrograph is lowered significantly. Due to the successive storages, the routed outflow hydrograph will experience a reduced peak and the time required to reach a safe peak will increase. The following discussion shows the set of equations needed for simulating the flow through successive dams and reservoirs.

In this investigation, it is assumed that the flow in the reservoir has no significant velocity.

The basic equation for flow routing is:

\[ I - O = \frac{dS}{dt} \]  

(11)

where \( I \) is reservoir inflow rate, \( O \) is reservoir outflow rate, and \( \frac{dS}{dt} \) is storage variation with respect to time. The finite difference form of Eq.11 for the first reservoir is:

\[ \frac{I_{i+1}^{(0)} + I_{i}^{(0)}}{2} - \frac{O_{i+1}^{(0)} + O_{i}^{(0)}}{2} = \frac{S_{i+1}^{(0)} - S_{i}^{(0)}}{\Delta t} \]  

(12)

where \( i \) and \( i+1 \) indicate successive time steps for a time increment equal to \( \Delta t \) and according to Figure 2. The superscript refers to the reservoir number. For the second reservoir, the routing equation becomes:

\[ \frac{I_{i+1}^{(0)} + I_{i}^{(0)}}{2} - \frac{O_{i+1}^{(0)} + O_{i}^{(0)}}{2} = \frac{S_{i+1}^{(0)} - S_{i}^{(0)}}{\Delta t} \]  

(13)

Figure 2. Routing through successive detention rockfill dams.
where $I_i^{(3)} = O_i^{(1)}$ and $I_r^{(3)} = O_r^{(1)}$

Substituting the outflow from reservoir No.1 for the inflow to reservoir No.2 in Eq.13 yields:

$$\frac{O_i^{(1)} + O_i^{(3)}}{2} = \frac{O_r^{(2)} + O_r^{(3)}}{2} = \frac{S_i^{(2)} - S_i^{(3)}}{\Delta t}$$ (14)

Eq.11 can be written for the third reservoir as:

$$\frac{I_i^{(3)} + I_r^{(3)}}{2} - \frac{O_i^{(3)} + O_r^{(3)}}{2} = \frac{S_i^{(3)} - S_i^{(2)}}{\Delta t}$$ (15)

and as $I_i^{(2)} = O_i^{(1)}$ and $I_r^{(2)} = O_r^{(1)}$, Eq.15 becomes:

$$\frac{O_i^{(1)} + O_i^{(3)}}{2} - \frac{O_r^{(2)} + O_r^{(3)}}{2} = \frac{S_i^{(2)} - S_i^{(3)}}{\Delta t}$$ (16)

In the same manner it is possible to extend the above concepts to $P$ successive dams as follows:

$$\frac{O_i^{(P-1)} + O_i^{(P)}}{2} - \frac{O_r^{(P-2)} + O_r^{(P)}}{2} = \frac{S_i^{(P-1)} - S_i^{(P)}}{\Delta t}$$ (17)

Last equation can be written as:

$$O_i^{(P)} = Q(H_r^{(P-1)} - H_i^{(P)})$$ (18)

and $O_r^{(P)} = Q(H_i^{(P)})$

where $Q$ is flow rate and $Q(H_r^{(P-1)})$ is downstream flow rating relationship.

The general form of flow rating equation, Eq.9, for calculating $O$ of each of the rockfill dams is:

$$O = \left( \frac{1}{D^2} \right)^\frac{1}{2} \frac{\alpha w_i}{b+3} \left( H_i^{(0)} - H_i^{(3)} \right)^\frac{1}{2}$$ (19)

where $H_i^{(0)}$ and $H_r^{(0)}$ refer rockfill upstream and downstream water depths, respectively. Therefore if $P = 3$, four equations (Eq.12, Eq.14, Eq.16 and Eq.18) will be available to be solved for four unknowns ($H_i^{(1)}, H_i^{(2)}, H_i^{(3)}$ and $H_r^{(3)}$), and for $P$ dams, $P+1$ unknowns need to be calculated.

In four equations (Eq.12, Eq.14, Eq.16 and Eq.18), $S_i^{(1)}, O_i^{(1)}, S_i^{(2)}, O_r^{(2)}$, $S_i^{(3)}$ and $O_i^{(3)}$ are the unknowns which give us:

$$O_i^{(0)} = f(H_i^{(0)}, H_r^{(0)})$$ (20)

$$S_i^{(0)} = f(H_i^{(0)})$$ (21)

$$O_r^{(2)} = f(H_r^{(2)}, H_i^{(3)})$$ (22)

$$S_i^{(2)} = f(H_r^{(2)})$$ (23)

$$O_i^{(3)} = f(H_i^{(3)}, H_r^{(4)})$$ (24)

$$S_i^{(3)} = f(H_i^{(3)})$$ (25)

$$O_r^{(4)} = f(H_i^{(4)})$$ (26)

Therefore, the actual unknowns are $H_i^{(1)}, H_i^{(2)}, H_i^{(3)}$ and $H_r^{(4)}$.

**Solution**

By using of Eq.19 for each of the dams and Eq.18 as the downstream flow rating relationship for $P$ dams, the generalized equations of the problem are summarized below as:

$$\frac{I_i^{(1)} + I_i^{(3)}}{2} - \frac{O_i^{(1)} + O_i^{(3)}}{2} = \frac{S_i^{(1)} - S_i^{(3)}}{\Delta t}$$ (12)

$$\frac{O_i^{(1)} + O_i^{(3)}}{2} - \frac{O_r^{(2)} + O_r^{(3)}}{2} = \frac{S_i^{(2)} - S_i^{(3)}}{\Delta t}$$ (14)

$$\frac{O_i^{(2)} + O_i^{(4)}}{2} - \frac{O_r^{(2)} + O_r^{(4)}}{2} = \frac{S_i^{(3)} - S_i^{(4)}}{\Delta t}$$ (16)

$$\frac{O_i^{(P-1)} + O_i^{(P)}}{2} - \frac{O_r^{(P-2)} + O_r^{(P)}}{2} = \frac{S_i^{(P-1)} - S_i^{(P)}}{\Delta t}$$ (17)

$$O_i^{(P)} = Q(H_i^{(P-1)})$$ (18)

where the unknowns are $H_i^{(1)}, H_i^{(2)}, H_i^{(3)}, \ldots, H_i^{(P-1)}$. To solve this model for the $P+1$ unknown, the following information is needed

- Equation 19 for each dam,
- Inflow hydrograph for the first reservoir,
- Volume-elevation relationship for each reservoir,
- Rockfill characteristics for each dam, and,
- Downstream flow rating relationship.

**Numerical Procedure**

To solve the previously listed set of equations a computer program was developed on the basis of the Gauss-Seidel iterative method algorithm and it can be defined as follows:

1. Set initial condition for $H_i^{(1)}, H_i^{(2)},$
$H^{(2)}, \ldots, H_i^{(P+1)}$. In the inflow hydrograph, the initial condition $t=t_1$ and $Q=Q_1$ is known. Therefore water depths for $(i=1)$ are calculable by using Eq.27.

$$Q(t^{(1)}) = Q^{(2)} = Q^{(3)} = \ldots, Q^{(P+1)} = \ldots$$

(27)

1. Read inflows $I_i^{(1)}, I_i^{(2)}$ for the selected $dt$ from the inflow hydrograph.

2. For running the iterative method, $H_i^{(1)}, H_i^{(2)}, \ldots, H_i^{(P+1)}$ are assumed.

The best assumption for $H_i^{(1)}, H_i^{(2)}, \ldots, H_i^{(P+1)}$ is $H_i^{(1)}, H_i^{(2)}, H_i^{(3)}, \ldots, H_i^{(P+1)}$ respectively,

4. Calculate $S_i^{(1)}, S_i^{(2)}, \ldots, S_i^{(P)}$ using the information from step 1 and the volume-elevation relationship.

5. Calculate $O_i^{(1)}, O_i^{(2)}, \ldots, O_i^{(P)}$ using the information from steps 1 and 2 and Eq.19.

6. Calculate $S_i^{(1)}, S_i^{(2)}, \ldots, S_i^{(P)}$ using the information from step 3 and the volume-elevation relationship for each reservoir.

7. Calculate $O_i^{(1)}, O_i^{(2)}, \ldots, O_i^{(P)}$ using the information from step 3 and Eq.19.

8. Calculate $H_i^{(P+1)}$ using Eq.18.

9. Solve for $O_i^{(2)}, O_i^{(3)}, \ldots, O_i^{(P)}$ using Equations 12, 14, 16, 17, respectively.

10. Compare the results of steps 9 and 7 and repeat steps 3 to 9 until convergence occurs.

The numerical procedure can be conducted provided that $a$ and $b$ in Eq.3 are known. Due to the sensitivity of $H_i$ to the range of average size of rock material particles less than 20 mm (Samani et al., 2003), a calibration for $a$ and $b$ using the experimental data collected in this investigation has been conducted.

Experimental Data

Experimental data were needed for the calibration and validation of the mathematical model. The experiments were conducted in a laboratory channel 10.0 m long, 0.3 m wide, and 0.45 m high. Different cases were investigated as follows by changing the characteristic rockfill particle diameter, rockfill dam length, distance between rockfill dams, and number of rockfill dams with $\beta = 90$ degrees:

- Two rockfill dams with average particle size of 14.5 mm, length of 0.4 m and distance between dams of 0.4 m;
- Two rockfill dams with average particle size of 14.5 mm, length of 0.4 m and distance between dams of 0.75 m;
- Two rockfill dams with average particle size of 21.0 mm, length of 0.5 m and distance between dams of 0.5 m;
- Two rockfill dams with average particle size of 21.0 mm, length of 0.5 m and distance between dams of 1.0 m;
- Three rockfill dams with average particle size of 14.5 mm, length of 0.4 m and distance between dams of 0.75 m;
- Three rockfill dams with average particle size of 21.0 mm, length of 0.5 m and distance between dams of 1.0 m.

The inflow hydrograph to the first rockfill reservoir was measured by a triangular weir installed at the beginning of the reservoir. This hydrograph was established with a programmable electrical valve. The outflow hydrograph was measured using a downstream channel rating curve. For measuring water level variation along the channel, each reservoir and downstream channel, a number of sensitive digital point gauges were installed. Each point gauge was equipped with memory storage to record water levels during the routing procedure. To get average particle size about 20.0 mm, particle rocks were sifted (sieved) through two sieves with openings of 21.0 mm and 19.0 mm, respectively.

To hold rockfill dams in their positions each rockfill was equipped by a thin galvanized basket. Finally, the output of the experiment was six routed outflow hydrographs which were used in calibrating and validating the mathematical model.
Model Verification, Calibration and Evaluation

For verifying the model, the following steps were taken by an assumed inflow hydrograph and assumed characteristics of successive rockfill dams:
a) Checking the computer program by imposing equal water elevations for the upstream and downstream levels of each of the dams. This condition introduced a zero flow rate through the rockfill dam and, if the water levels in all reservoirs are equal, the outflow of the successive rockfill dams will be zero, too.
b) Applying a constant inflow rate shows a stable water level for each reservoir indicating steady state flow conditions.
c) The model flood routing results show that the volume of the first reservoir inflow hydrograph is equal to the outflow hydrograph volume of each of the rockfill dams.
d) Applying the model for a very short length between successive rockfill dams considering the small separation between dams will introduce outflow hydrographs very close to the inflow hydrographs in terms of magnitudes and duration.

The conclusion from the tests was positive indicating the validity of the mathematical model.

Calibration for Eq.3 was conducted using 50% of the collected data and a nonlinear optimization program. The results are $a = 54.0$ and $b = -0.077$.

The conclusions from the tests were positive indicating the validity of the mathematical model.

For the validation of the model, the complete data were used as below:
The model has been applied for the six routing cases so that the mathematical model can be evaluated. Figures 3 to 8 show the routed measured and calculated hydrographs. The agreement between the measured and calculated hydrographs is quite reasonable. The $R^2$ (regression coefficient) of all the figures is more than 91%, proving the validity of the mathematical model.

Sensitivity

At this stage, where the mathematical model has been validated, the sensitivity associated with the important model parameters can be evaluated. Sensitivity was tested by assuming an inflow hydrograph and successive rockfill dams. This investigation shows that each parameter has a different effect on $\Delta Q\%$ and $\Delta T\%$, where $\Delta Q\%$ is the percentage of the difference between the peaks of the last outflow hydrograph and the first inflow hydrograph relative to the first inflow hydrograph peak and $\Delta T\%$ is the re-

Figure 3. Two rockfill dams with a average particle size of 14.5 mm, length of 0.4 m and distance of 0.4 m ($R^2=97\%$).
Figure 4. Two rockfill dams with an average particle size of 14.5 mm length of 0.4 m and distance between dams of 0.75 m ($R^2=97\%$).

Figure 5. Two rockfill dams with a average particle size of 21.0 mm length of 0.5 m and distance between dams of 0.5 m ($R^2=96\%$).

Figure 6. Two rockfill dams with a average particle size of 21.0 mm length of 0.5 m and distance between dams of 1.0 m ($R^2=96\%$).

lated percentage difference in the time to peak, respectively. Table 1 demonstrates the results of the sensitivity of the parameters. Each of the investigated parameters has a
different level of sensitivity.

Among the investigated parameters, $d_{50}$ is the most important parameter compared to the others. The larger the $d_{50}$, the bigger the outflow hydrograph peak and the shorter the related period will be. Longer $L$ implies more head losses, a lower outflow peak and a longer relative time difference between peaks. For the constant $L$, $\beta$ would have the same effect as $L$ meaning that larger $\beta$ results in more head losses and outflow hydrograph dampening.

**CONCLUSION**

In the present study, a model has been presented to solve the problem of flow routing through successive rockfill detention dams. The power law relationship between the Darcy-Weisbach friction factor and Reynolds number (Eq.3) was calibrated using a non-linear optimization program. Results showed $a = 54.0$ and $b = -0.077$. The model has been verified and validated with meas-
Table 1. Sensitivity of the parameters on $\Delta Q\%$ and $\Delta T\%$.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$L$ (m)</th>
<th>$d_{50}$ (m)</th>
<th>$\beta$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter range</td>
<td>25 to 100</td>
<td>0.025 to 0.1</td>
<td>45 to 90</td>
</tr>
<tr>
<td>$\Delta Q%$ changes</td>
<td>-53.18 to -64.42</td>
<td>-65.17 to -50.94</td>
<td>-53.73 to -63.11</td>
</tr>
<tr>
<td>$\Delta T%$ changes</td>
<td>69 to 106.4</td>
<td>100 to 63</td>
<td>78 to 103</td>
</tr>
</tbody>
</table>

The success of reservoir storages cause the routed outflow hydrograph to experience a greater reduction and lag time in reaching a safe peak flow magnitude. The model considers the reciprocal effects of reservoirs on each other. The sensitivity associated with the important model parameters shows that $d_{50}$ is the most important parameter affecting the routing process.

Appendix

The following symbols are used in this paper:

- $A, B, A', B'$ = Empirical coefficients;
- $a, b, a', b'$ = Empirical coefficients;
- $d$ = Diameter of rockfill particle;
- $d_{50}$ = Average size of the rockfill material;
- $D$ = A parameter that can be calculated from Equation (10);
- $f$ = Weisbach coefficient;
- $g$ = Acceleration due to gravity;
- $h$ = Hydraulic head;
- $H_1$ = Upstream water depth across the rockfill dam;
- $H_2$ = Downstream water depth across the rockfill dam;
- $i$ = Hydraulic gradient across the rockfill dam;
- $L$ = Base of the rockfill dam;
- $Q$ = Flow rate;
- $Q_i$ = Reservoir inflow rate;
- $Q_o$ = Reservoir outflow rate;
- $Re$ = Reynolds’ number;
- $S$ = Reservoir storage;
- $t$ = time;
- $V$ = Velocity;
- $x$ = The longitudinal direction of the dam;
- $w$ = Dam width (perpendicular to flow direction).

$\alpha$ = Coefficient;

$\beta$ = The angle of the upstream or downstream of the dam face relative to the horizontal direction;

$\beta_1$ = The angle of the upstream face of the dam with the horizontal direction;

$\Delta Q\%$ = The percentage of the difference between the peak flows of the last outflow hydrograph and the first inflow hydrograph relative to the first inflow hydrograph peak;

$\Delta T\%$ = The time percentage difference between the time to peak of the last outflow hydrograph and the first inflow hydrograph relative to the first inflow hydrograph peak;

$\sigma$ = Standard deviation of rock material;

$\nu$ = Kinematic viscosity.

REFERENCES

روندیابی سیل در مخازن سد‌های تأخیری پارس‌نگی متوالی

چ.م. و. سامانی و. حیدری

چکیده

سنگ بنوان یکی از مصالح سودمند در سازه‌های هیدرولیکی نظیر سد‌های پارس‌نگی، گویایی نیز در عملیات زهکشی استفاده می‌شود. سد‌های پارس‌نگی، بیشتر در منابع حوضه‌های آبریز و کنترل سیلاب استفاده می‌شوند. هدف از این سد‌ها ایجاد ذخیره آب و کاهش دی‌ب یک حد اکثر سیل ورودی به مخزن می‌باشد. از آنگاهیکه چرایان شوری، از محیط‌هایی متفاوت بودن آن‌ها می‌پیامدها، لذا قانون دارسی در این میکرو‌ها اعتبار خود را از دست می‌دهد. در شرايط عملي، ممكن است ساختن سد‌های پارس‌نگی متوالی ضرورت داشته باشد تا هما چاکره روندیابی جريان بهتر كنند. پانل اذان متقابل این سد‌ها به هم‌نظری هیدرولیک آنها پیچیده می‌گردد. این مقاله در طرح روکش جریان در سد‌های پارس‌نگی متوالی بر اساس مبتهبی که در این سد‌ها و اثرات متقابل با لحاظ که ورود جریان بین سد و اثرات متقابل بین آنها، اثره می‌دهد. در این مدل، سرعت را به مدل یک‌دیمی‌پیوسته جریان به شکل تابع نمایی علیه عدد رانژ و شرایط دارسی-ویاپسایس معرفی می‌گردد. با تعریف هیدروگراف ورودی و مشخصات سیستم ورود استفاده پونان ورودی مدل، هیدروگراف روندیابی شده قابل محاسبه خواهد بود. نتایج مدل اندازه‌گیری با داده‌های آزمایشگاهی این تحقیق نشان می‌دهد. افزایش مقدار سد‌ها، افزایش سد اصلی بین آن‌ها، میزان اندازه سدگ، دانه‌ها و ابعاد سد‌ها باعث کاهش دراکو دیپ سیلاب خواهد شد.
ATTACHMENT 6C
UNCERTAINTY ANALYSIS OF ROUTED OUTFLOW IN ROCKFILL DAMS
(SAMANI, J.M.V; AND SOLIMANI, A., 2008)
Uncertainty Analysis of Routed Outflow in Rockfill Dams

J. M.V. Samani* and A. Solimani

ABSTRACT

Detention rockfill dams are an easy and common tool for flood control. Due to their coarse pores, the flow in void spaces is turbulent and non-Darcy. Different relationships introduced by researchers are used to define the hydraulics of the flow within the rockfill materials. The present research is aimed at gaining a better understanding of the difference among these relationships and the sources of uncertainty associated with the different parameters of each of the relationships. To examine the importance of various factors on the uncertainty of the outflow hydrograph, sensitivity analysis was conducted. For this purpose, a rockfill mass was provided, fifteen random samples of the mass selected, and then the physical characteristics of the material were measured or estimated. Also, some flood routing tests have been conducted. In these tests a physical model of a dam was installed and downstream water level was measured for different outflow rates. While the downstream water level was considered as certain variable but other parameters were seen as stochastic (stochastic parameters are considered as random variables) and outflow discharge as an output uncertain parameter. Uncertainty analysis has been conducted for different points of the outflow hydrograph by employing available methods. The results show that the Samani et al. and McCorquodale et al. relationships have the lowest and highest uncertainty, respectively. The sensitivity analysis demonstrates different levels of sensitivity accompanied each of the relationship parameters which results in different effects on the total uncertainty of the relationships.

Keywords: First-Order Variance Estimation (FOVE) Method, Flood Routing, Harr's Method, Latin Hypercube Sampling (LHS) Method, Rockfill Dam.

INTRODUCTION

Uncertainties may arise due to natural variations in the phenomenon being considered, or to an incompleteness of our understanding. Uncertainties may also arise from the inaccurate characterization of important parameters or variables. Hence, engineering practice is frequently associated with decision making under uncertainty. The physical or numerical models, developed and used to simulate natural phenomena, are often in reality probabilistic, and hence, subject to analysis by rules of probability theory. Identifying the components of uncertainty related the physical phenomenon and quantifying them, can therefore improve decision making and the results (Haung, 1986; Mercer, 1975).

One of the common and most economic methods for flood mitigation used in watershed management is rockfill dams. The fact that flood mitigation through rockfill dams is an uncertain phenomenon, raises questions about the reliability and credibility of the relationships involved. As this type of dam consists of coarse particles, the flow deviates from Darcy's law resulting in turbulence in the void spaces. This means that the relationship between the flow velocity, V, and its hydraulic gradient, i, is a nonlinear one. Different researchers have proposed different nonlinear relationships that give various outputs.

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McCorquodale et al. (1978) introduced the following equations:

$$i = \frac{4.6v}{g\text{m}^2} \left( \frac{\rho - 0.79}{\rho_{\text{m}}^2} \right) V^2$$  for $R_w \leq 125$ or $R_p > 500$

and

$$i = \frac{70v}{g\text{m}^2} \left( \frac{1 + f_f/f_s}{\rho_{\text{m}}^2} \right) V$$  for $R_w > 125$ or $R_p > 500$  \hspace{1cm} (1)

where $R_w$ and $R_p$, Reynolds numbers, are defined as below:

$$R_w = \frac{\nu m_0^{1/2}}{v}$$  \hspace{1cm} (2)

$$R_p = \frac{v m}{\nu n^{1/2}}$$

Based on the data collected by McCorquodale et al. the specific Reynolds numbers ($R_w$ and $R_p$) are dimensionless variables selected by them to define the constrains of his defined relationships.

Stephenson’s (1979) relationship is

$$i = \frac{500\nu}{g n^2} \left( \frac{K_f}{n^2 g d} \right) V^2$$  \hspace{1cm} (3)

and Adel’s relationship is (Ahmed and Sunada, 1969)

$$i = \frac{160\nu(1-n)}{g n d_i^3} V + \frac{2.2}{g n d_i^3} V^2$$  \hspace{1cm} (4)

In the above equations, $i$ is the hydraulic gradient, $V$ is flow velocity, $d$ is the average diameter of rock, $g$ is acceleration due to gravity, $K_f$ is the friction coefficient in the turbulent flow region, $n$ is porosity, $f_f$ is the friction factor, $d_{15}$ is particle diameter where 15% of the total particles weight are smaller, $f_s$ is the friction factor between large particles and the instrument wall, $f_c$ is the Darcy-Weisbach coefficient, $m$ is the pores effective hydraulic radius, and $m$ refers to the hydraulic radius of pores.

Samani et al. (2003) introduced the following equation:

$$i = \frac{V}{\alpha}$$  \hspace{1cm} (5)

in which

$$\alpha = \left( \frac{2k \nu^b}{\alpha d_{50} - \sigma^2} \right)^{1/2}$$  \hspace{1cm} (6)

where $d_{50}$ is particle diameter size where 50% of the total particles' weight is smaller, $a$ and $b$ are empirical coefficients of the equation related to the flow and particles characteristics, and $\sigma$ is the standard deviation of particles.

Several methods of uncertainty analysis have been developed and applied in water resource engineering. The most widely used methods are first-order variables estimating (FOVE), Harr’s Probabilistic Point Estimation method and Monte Carlo Simulation (MCS) (Ang and Tang, 1984). FOVE is based on linearizing the functional relationship that relates a dependent random variable and a set of independent random variables by Taylor series expansion (Yen et al., 1986). This method has been applied in several water resource and environmental engineering problems involving uncertainty. Examples include storm sewer design (Tang and Yen, 1972), ground-water flow estimation (Dettinger and Wilson, 1981), prediction of dissolved oxygen (Burges and Lettenmaier, 1975; Chadderton et al., 1982), subsurface flow and contaminant transport estimation (Sitar et al., 1987), and water surface profile of a buried stream flowing under coarse material (Hansen and Bari, 2002). In Harr’s method, the average and variance of probabilistic variables and their correlations are used (more details are introduced in Tung, 1993). If there are $N$ variables, the number of cases (points) will be $2N$ which is considered an important advantage compared to the point estimate method proposed by Rosenbluth (1981). In cases when obtaining the derivatives is too complicated, Harr’s method is considered as a good substitute for the FOVE method. Herr’s method has been used in studying the spatial variation of river bed scouring (Yeh and Tung, 1993) and for uncertainty analysis incorporating marginal distribution (Chang and Yang, 1997). In MCS, stochastic inputs are generated from their probability distributions and are then entered into empirical or analytical models of underlying physical process involved in generating stochastic outputs. Then, the generated outputs are analyzed statistically to quantify the uncertainty of the output. Several examples of uncertainty analysis by MCS can be found in wa-
MATERIALS AND METHODS

The uncertainty of predicted routed flow hydrographs depends on the physical and hydraulic parameters of the flow relationships. The physical parameters can be measured or estimated by providing a big mass of rockfill material. The hydraulic parameters, however, can not be measured unless an experiment is conducted. For this purpose, a small rockfill material, from the big mass, has been used to build a small physical dam model to be used in flow rating relationship measurements.

Flow Rating Relationships

The objective of this study is to determine the uncertainty of outflow routed hydrographs resulting from different relationships, i.e., equations 1, 2, 3, 4, and 5. For this purpose, by considering \( i = \frac{-dH}{dx} \) and integrating each relationship for the limits \( x = 0 \) to \( D \) and \( H = H_1 \) to \( H_2 \), the following relationship among \( Q, H_1 \) (dam upstream water level) and \( H_2 \) (dam downstream water level) is obtained:

\[
P \left( \frac{M^2}{2} \left( H_2^2 - H_1^2 \right) - BQ^2 \left( H_2 - H_1 \right) \right) = -L \quad (7)
\]

where \( Q \) is the flow rate, \( P \) is \( \frac{-w^2}{M^2} \), \( M \) is \( CQw \), \( w \) is dam width and \( D \) is calculated according to Sharma (1991). The amount of \( D \) is less than \( L \) for Trapezoidal rockfill dams and equal to \( L \) for rectangular ones. According to McCorquodale et al.’s, Stephenson’s and Adel’s relationships introduced above, equation (7) would have a general form providing that \( B \) and \( C \) are defined as the following:
According to the McCorquodale et al. relationship:
\[
C = \frac{70\nu}{g n m'^2} \quad \text{and} \quad B = \frac{0.27(1 + (f_s / f_o))}{g n m'^2} \quad (7a)
\]
According to the Stephenson relationship:
\[
C = \frac{800 \nu}{g n d'^2} \quad \text{and} \quad B = \frac{K_v}{g n d}
\quad (7b)
\]
According to the Adel relationship:
\[
C = \frac{160 \nu (1 - n)^2}{g n d'^2} \quad \text{and} \quad B = \frac{2.2}{g n d}
\quad (7c)
\]

The flow rating relationship for Samani et al. is different from the other relationships where it is as follows (Samani et al., 2003):
\[
Q = \left( \frac{1}{D} \right)^{1/2} \frac{\alpha w}{(3 + b)^{1/2}} \left( H_{10}^2 + H_{10}^2 \right)^{1/2} \quad (8)
\]
in which D and L are equal in rectangular rockfill dams.

**Hydraulic Parameters**

In this analysis, \( H_2 \) and Q are considered as certain and resultant uncertain variables, respectively. Defining \( H_2 \) would mean a certain \( H_2 \) for a specific Q. The physical rockfill dam model of 66 cm length, 30 cm width, and 33 cm height has been installed in a same width of 9 m flume. Table 1 shows the range of hydraulic parameters measurements used in the experiment that would be used in the routing calculation.

**Physical Parameters**

In order to evaluate the physical parameters of the different relationships, 15 random samples from the mass provided were selected and used. Table 2 shows the grain size distribution of the rockfill material. The size distribution curve, \( d_{50}, d_{10}, d_{95}, d_{15}, d_{60}, d_{n}, d, m, m' \) and \( \sigma \) of each sample have been determined, where \( d_{50} \) is the average size diameter and \( d \) is the harmonic average size. Different subscripts for notation d refer to the percentages of total particle weight that are smaller than the related d. Table 3 shows the average size, standard deviation, and

**Table 1.** \( H_2 \) and Corresponding Q of experiment.

<table>
<thead>
<tr>
<th>Q (m³ s⁻¹)</th>
<th>( H_2 ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00046</td>
<td>0.029</td>
</tr>
<tr>
<td>0.0006</td>
<td>0.032</td>
</tr>
<tr>
<td>0.00105</td>
<td>0.044</td>
</tr>
<tr>
<td>0.0015</td>
<td>0.059</td>
</tr>
<tr>
<td>0.0026</td>
<td>0.084</td>
</tr>
<tr>
<td>0.0015</td>
<td>0.076</td>
</tr>
<tr>
<td>0.00098</td>
<td>0.049</td>
</tr>
</tbody>
</table>

The coefficient of variation of the parameters of the 15 samples.

It is necessary to say that the measurement of parameters was carried out by several persons and several times in order that the human error can be assumed to be minimized.

In this research, the parameters \( K_v, f_e/f_o, a, \) and \( b \) for each sample were estimated by taking into account the suggestions given by relationship developers. Table 4 shows the estimated values of the parameters.

**Routing and Uncertainty Analysis**

In this research the following storage routing equation has been employed:
\[
I - O = \frac{\Delta S}{\Delta t} \quad (9)
\]
where I and O indicate the flow rates of the inflow and outflow hydrographs, respectively, and \( \Delta S \) is the storage within the time \( \Delta t \). According to equation (9), the inflow hydrograph will be routed in the reservoir

**Table 2.** Particle size distribution of rockfill mass.

<table>
<thead>
<tr>
<th>%</th>
<th>Sieve size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>32</td>
</tr>
<tr>
<td>96.8</td>
<td>24</td>
</tr>
<tr>
<td>68.8</td>
<td>18</td>
</tr>
<tr>
<td>26.5</td>
<td>12.7</td>
</tr>
<tr>
<td>3.5</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>4</td>
</tr>
</tbody>
</table>
Table 3. Statistical characteristics of different parameters.

<table>
<thead>
<tr>
<th></th>
<th>( d_{0} )</th>
<th>( d_{15} )</th>
<th>( d_{50} )</th>
<th>( d_{90} )</th>
<th>( d_{m} )</th>
<th>( d )</th>
<th>( e )</th>
<th>( \sigma )</th>
<th>( m )</th>
<th>( m' )</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
</tr>
<tr>
<td>10</td>
<td>11</td>
<td>13.74</td>
<td>15</td>
<td>15</td>
<td>13</td>
<td>0.67</td>
<td>0.00039</td>
<td>0.0034</td>
<td>0.0025</td>
<td>0.40</td>
<td>Aver.</td>
</tr>
<tr>
<td>0.49</td>
<td>0.38</td>
<td>0.83</td>
<td>0.75</td>
<td>0.60</td>
<td>0.77</td>
<td>0.034</td>
<td>0.000008</td>
<td>0.000199</td>
<td>0.000236</td>
<td>0.013</td>
<td>St. Dev.</td>
</tr>
<tr>
<td>0.05</td>
<td>0.04</td>
<td>0.06</td>
<td>0.05</td>
<td>0.04</td>
<td>0.06</td>
<td>0.05</td>
<td>0.02</td>
<td>0.06</td>
<td>0.11</td>
<td>0.03</td>
<td>Vari.</td>
</tr>
<tr>
<td>0.22</td>
<td>0.45</td>
<td>0.69</td>
<td>0.79</td>
<td>0.60</td>
<td>0.45</td>
<td>-1.053</td>
<td>-0.37</td>
<td>0.03</td>
<td>0.06</td>
<td>-1.114</td>
<td>Skew.</td>
</tr>
</tbody>
</table>

upstream the rockfill dam (the physical model) and then a routed outflow hydrograph is introduced downstream of the dam. The outflow hydrograph is accompanied with uncertainty which is the final objective of this procedure.

The methods selected for the analysis are FOVE, Harr, and LHS. In this analysis, \( d_{0} \), \( d_{50} \), \( d_{15} \), \( d_{90} \), \( d_{m} \), \( d \), \( e \), \( m \), \( m' \), \( \sigma \), \( a \), \( b \), \( f/e/f_o \), and \( K \) are considered as stochastic inputs, \( H_2 \) as a certain input and \( Q \) as an uncertain output.

FOVE is a simple, effective, and precise method especially when the relationship of input and output variables is linear. This method does not take into account the probability distribution of variables which might be considered as a disadvantage. This method uses a Taylor series to linearize the relationship between output and input variables. The following shows the function:

\[
Y = g(X) = g(X_1, X_2, ..., X_n)
\]  

(10)

where \( Y \) is the function of \( N \) stochastic variables, \( X \). The Taylor series is written as

\[
Y = g(x_0) + \sum_{i=1}^{n} \frac{\partial g}{\partial x_i} (X_i - x_0) + \frac{1}{2} \sum_{i=1}^{n} \sum_{j=i+1}^{n} \frac{\partial^2 g}{\partial x_i \partial x_j} (X_i - x_0)(X_j - x_0)
\]  

(11)

and, considering the first two terms and neglecting higher order terms of the above series, gives the following:

\[
Y \approx g(x_0) + S_x C(X) S_x
\]  

(12)

where, \( S_x = \frac{\partial g}{\partial X} \) is the sensitivity coefficient vector. According to equation (12), the average is \( E[Y] \approx g(x_0) + S_x C(X) S_x \) and the variance is \( Var[Y] \approx S_x C(X) S_x \), where \( \mu \) is average matrix and \( C(X) \) is the matrix of stochastic variables, \( X \). Assuming \( x_0 = \mu \), then:

\[
E[Y] = g(\mu) \text{ and } Var[Y] = S_x C(X) S_x
\]  

(12a)

where \( S \) is the vector of sensitivity coefficients at \( x_0 = \mu \). When the stochastic variables are all independent, the variance can be calculated from \( Var[Y] = S C(X) S \) where \( D \) is the diagonal matrix of stochastic variables variance, i.e.,

\[
D = diag(\sigma_1^2, \sigma_2^2, ..., \sigma_n^2)
\]

The different stages of the FOVE method can be summarized as:

- Identifying input stochastic (physical) parameters,
- Applying the Taylor series, and,
- Calculating the average and variance of flow rates of different relationships (models).

Harr's method is similar to the FOVE method because it uses the two first order moments of stochastic variables and not the probability distribution, but it is easier in terms of calculations. It is considered as a good substitute for FOVE when it is dealing with complex derivatives. The different stages of Harr's method can be summarized as:

Table 4. Statistical Characteristics of Estimated Parameters.

<table>
<thead>
<tr>
<th></th>
<th>( A )</th>
<th>( \Delta )</th>
<th>( f/e/f_o )</th>
<th>( K )</th>
</tr>
</thead>
<tbody>
<tr>
<td>54</td>
<td>-0.074</td>
<td>1.54</td>
<td>3.1</td>
<td>Average</td>
</tr>
<tr>
<td>4.47</td>
<td>0.014</td>
<td>0.036</td>
<td>0.139</td>
<td>St. Deviation.</td>
</tr>
<tr>
<td>0.083</td>
<td>-0.194</td>
<td>0.023</td>
<td>0.045</td>
<td>Variance</td>
</tr>
</tbody>
</table>
- Identifying input physical parameters of each of the relationships and calculating its correlation matrix,
- Decomposition of the correlation matrix (CO) to the eigen vectors matrix and eigen values matrix (with MATLAB software)

\[ CO = V L \Lambda V^T \]  \hspace{1cm} (13)

where \( V = (v_1, v_2, \Lambda, v_n) \) is the eigen vectors matrix and \( \Lambda = \lambda_1, \lambda_2, \ldots, \lambda_n \) is the eigen value diagonal matrix,
- Calculating 2N intersection points where this couple of points are calculated from the following equation:

\[ X_{iz} = \mu \pm \sqrt{N} \]  \hspace{1cm} (14)

where \( \mu \) = Mean; \( \sigma_i \) = Standard deviation of \( i \)th stochastic input; \( N \) = Number of inputs; \( V \) = Eigen vectors matrix,
- Calculating \( Y_{i2} = g (X_{i2}) \) and \( Y_{i2}^2 = g^2 (X_{i2}) \) for \( i = 1, 2, \ldots, N \) where \( Y_i = \) Model output and then calculating \( \bar{Y}_i = Y_{i2} + Y_{i2}^2 \) and

\[ Y_i = \frac{Y_i^2 + Y_i}{2} \]

- Calculating the average and variance of different model outputs:

\[ E(Y) = \frac{\sum Y_i \lambda_i}{\sum \lambda_i} \]

\[ E(Y^2) = \frac{\sum Y_i^2 \lambda_i}{N} \]

\[ Var(Y) = E(Y^2) - E^2(Y) \]  \hspace{1cm} (15)

- Computing model uncertainty with the coefficient of variation. For an elaborate discussion on the FOVE and Harr’s methods the reader is referred to Hosseini (2000).

LHS is an effective method especially in dealing with nonlinear relationships. Its main disadvantage is the need for a probability distribution of variables. The probability distribution of \( Q \) can be estimated as follows: (1) obtain a random set of size \( n \) of the stochastic inputs from the corresponding probability distribution using LHS; (2) follow the necessary steps to route the inflow hydrograph and obtain the outflow hydrograph. For ease of calculation, just seven points of the routed outflow hydrograph are used in this analysis; (3) analyze statistically the outflows (7 points) to determine its probability distribution and its basic statistics such as mean, standard deviation, coefficient of variation, and coefficient of skewness.

In using MCS to generate the stochastic inputs referred to in Step 1 above, normally a large data set, for instance, \( n = 1000 \), is generated from the probability distribution of each input. The probability distribution of each parameter was normal, log-normal, uniform, and log-Pierson. For more details on MCS the reader is referred to Rief (1988).

An alternative to MCS sampling that reduces the number of sets of generated inputs and consequently the number of generated outputs is the LHS method. The basic concept of LHS lies in generating random numbers of a stochastic input over its range in a stratified manner, such as the overall variability of the given stochastic input can reasonably be delineated by limited sample size. The properties of LHS are discussed by McKay (1988) and McKay et al. (1979). In the MCS or LHS procedures, all stochastic inputs are assumed to be independent.

Sometimes, when a large number of stochastic inputs are involved in determining the output, sensitivity analysis may be carried out to determine the degree of influence of each stochastic input on the output uncertainty, \( C_i \). In FOVE, \( C_i \) is calculated by the following:

\[ C_i = \frac{S_i^2 \sigma_i^2}{\sigma_i^2} \]  \hspace{1cm} (16)

\[ i = 1, 2, \ldots, N \]
Table 5. Uncertainty analysis results using FOVE for independent input parameters.

<table>
<thead>
<tr>
<th>H (m)</th>
<th>E (Q) Var (Q) C.V (Q)</th>
<th>E (Q) Var (Q) C.V (Q)</th>
<th>E (Q) Var (Q) C.V (Q)</th>
<th>E (Q) Var (Q) C.V (Q)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.032</td>
<td>0.00856 2.5×10^{-3} 0.03 0.00053 2.6×10^{-6} 0.10 0.00053 7.2×10^{-6} 0.05 0.00055 7×10^{-6} 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.044</td>
<td>0.00992 5.8×10^{-3} 0.03 0.00086 4.1×10^{-6} 0.07 0.00086 1.1×10^{-6} 0.04 0.00089 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.059</td>
<td>0.00126 8.8×10^{-4} 0.02 0.00135 6.2×10^{-7} 0.06 0.00137 1.7×10^{-6} 0.03 0.0014 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.064</td>
<td>0.00223 2.2×10^{-4} 0.07 0.00198 9×10^{-7} 0.05 0.00223 2.8×10^{-6} 0.02 0.00231 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.076</td>
<td>0.00192 1.2×10^{-4} 0.02 0.00102 5.2×10^{-8} 0.07 0.0020 2.5×10^{-6} 0.02 0.00202 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.049</td>
<td>0.00097 5.9×10^{-5} 0.03 0.00085 6×10^{-7} 0.08 0.00104 1.3×10^{-6} 0.04 0.00106 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.043</td>
<td>0.00084 5.2×10^{-6} 0.03 0.0008 4×10^{-7} 0.08 0.00083 1.1×10^{-6} 0.04 0.00085 0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

where N is the number of stochastic input parameters, \( \sigma_i^2 \) is variance of ith input parameter, \( \sigma_Y \) is variance of output parameter, \( S_i \) is parameter sensitivity coefficient, and \( Y = f(x_1, x_2, ..., x_N) \).

In Harr’s method and LHS, a linear regression relationship between x’s, input parameters, and output, Y, can be considered, as the following

\[
Y = a_0 + \sum_{i=1}^{N} a_i x_i + e
\]

(19)

where \( a_0 \) is the interception value of the line with the y axis, \( a_i \) refers to regression coefficients that show the sensitivity coefficients, and e indicating the model error. Due to the dimensional problem, it is recommended to centralize the output parameter and then, by standardizing \((Y-Y)\) and input parameters, the regression can be conducted. In this case, coefficients will indicate the output variation for a variation of input parameter equal to one standard deviation. Then \( C_i \) values which indicates the uncertainty of input parameter can be calculated from the following relationship:

\[
C_i = \frac{SSR_i}{SSR} R_i^2 \text{ for } i=1,2,...,m \tag{20}
\]

In which SSR, is the summation of square values of the ith input stochastic parameter from the regressed line and SSR is the summation of SSR, for independent input parameters. For more detailed the reader is referred to McKay (1988).

Steps in the Procedure

The uncertainty in predicting routed flow hydrographs depends on the stochastic input parameters of the flow relationships; in this research four of the relationships, i.e. McCorquodale et al., Stephenson, Adel, and Samani et al., are investigated. The different stochastic inputs have been obtained by conducting an experimental procedure. For this a big mass of rockfill material was provided. Fifteen random samples of the mass have been selected and the related stochastic input parameters were measured or estimated. The data collected was employed for the uncertainty and sensitivity analyses.

From the same rockfill mass, a small

Table 6. Uncertainty analysis results using Harr for independent input parameters.

<table>
<thead>
<tr>
<th>McCorquodale et al.</th>
<th>Stephenson</th>
<th>Adel</th>
<th>Samani et al.</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (m)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------</td>
<td>-----------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>0.032</td>
<td>0.00656 5×10^{-3} 0.04 0.00053 2×10^{-5} 0.10 0.00053 8×10^{-6} 0.10 0.00054 7×10^{-6} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.044</td>
<td>0.00954 4×10^{-3} 0.03 0.00086 6×10^{-6} 0.10 0.00086 2×10^{-5} 0.10 0.00086 2×10^{-5} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.059</td>
<td>0.01068 3×10^{-4} 0.04 0.00134 1×10^{-5} 0.09 0.00137 5×10^{-6} 0.09 0.00135 5×10^{-6} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.085</td>
<td>0.02323 1×10^{-4} 0.13 0.00198 4×10^{-7} 0.09 0.00225 1×10^{-6} 0.09 0.00224 1×10^{-6} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.077</td>
<td>0.01913 5×10^{-5} 0.04 0.01002 1×10^{-7} 0.09 0.01199 1×10^{-6} 0.09 0.01196 1×10^{-6} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.049</td>
<td>0.00927 1×10^{-5} 0.04 0.00803 6×10^{-6} 0.09 0.00104 3×10^{-5} 0.10 0.00103 3×10^{-5} 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.043</td>
<td>0.00656 5×10^{-6} 0.04 0.00053 3×10^{-6} 0.10 0.00053 8×10^{-6} 0.10 0.00054 7×10^{-6} 0.05</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
physical dam was built and installed in a laboratory flume and then, by introducing different inflows, different corresponding downstream water levels, $H_2$, were identified. By introducing a hypothetical inflow hydrograph and conducting the routing calculation, routed outflow hydrograph was obtained. For the uncertainty analysis, $H_2$ has been regarded as certain input parameter and other parameters as stochastic. The analysis was conducted for seven $H_2$ values corresponding to flow rates covering the useful range of the outflow hydrograph, 0.032, 0.044, 0.095, 0.084, 0.076, 0.049, and 0.043 m. After gathering the certain and stochastic inputs, FOVE, Harr and LHS were employed for determining the uncertainty of the routed outflow hydrograph and then sensitivity analysis was applied to the different flow relationships to see the relative importance of stochastic inputs in estimating the variability of the output, routed outflow rate.

RESULTS

The results of calculating the outflow discharge uncertainty considering the input parameters dependent and independent parameters were very close to each other, therefore just the independent ones are introduced. The results of uncertainty analysis for different relationships are shown in Tables 5, 6, and 7 and Figure 1 (a, b and c). It can be concluded that:

If the Coefficient of Variation (C.V.) is considered as the indicator of uncertainty, LHS sampling and FOVE show the highest and the lowest uncertainty results, respectively, and Harr's method falls in between. As an example, the average C.V.'s of depth 0.084 for the different methods are 0.12, 0.9, and 0.04, respectively. Therefore, applying LHS sampling would mean a more uncertain environment and more reliable design.

By averaging the results of the three methods, the routed outflow hydrograph calculated by Samani et al., McCorquodale et al., Adel, and Stephenson, relationships show uncertainty of 0.05, 0.5, 0.6, and 0.08, respectively. The results of the first three relationships are almost the same which means using any of those relationships would make no significant difference.

Results of McCorquodale et al.'s relationship for $R_e < 500$ show high uncertainty; for instance for $H_2 = 0.084$, the uncertainty of outflow using LHS is 0.21 which is 3.5 times the average C.V. of the outflow hydrograph. In case of $R_e < 500$ it gives the less uncertainty among the other relationships.

The sensitivity analysis shows that the parameters $m$ and $n$ in McCorquodale et al.'s relationship introduce the greatest uncertainty, at 0.72 and 0.25, respectively, among the other parameters where in Stephenson's, the parameters $d$ and $n$ expose the highest sensitivity of 0.85 and 0.8, respectively. In Adel's relationship, $d_s$ with 0.88 and $n$ with 0.12 show the highest influence on the routed outflows. Finally, in Samani et al.'s a and $d_{50}$ as the most important parameters, shows the influence of 0.77 and 0.12, respectively.

Table 7. Uncertainty analysis results using MCS with LHS.

<table>
<thead>
<tr>
<th>$H_2$ (m)</th>
<th>McCorquodale et al.</th>
<th>Stephenson</th>
<th>Adel</th>
<th>Samani et al.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E (Q) Var (Q) C.V (Q)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
<td>E (Q) Var (Q) C.V (Q)</td>
</tr>
<tr>
<td>0.032</td>
<td>0.00056 3x10^{-8}</td>
<td>0.03 0.00053 3x10^{-8}</td>
<td>0.10 0.00056 3x10^{-8}</td>
<td>0.10 0.00056 3x10^{-8}</td>
</tr>
<tr>
<td>0.044</td>
<td>0.00094 8x10^{-8}</td>
<td>0.03 0.00085 6x10^{-8}</td>
<td>0.09 0.0009 8x10^{-8}</td>
<td>0.10 0.00092 7x10^{-8}</td>
</tr>
<tr>
<td>0.059</td>
<td>0.00145 2x10^{-7}</td>
<td>0.03 0.00134 1x10^{-7}</td>
<td>0.09 0.00143 2x10^{-7}</td>
<td>0.09 0.00144 2x10^{-7}</td>
</tr>
<tr>
<td>0.084</td>
<td>0.00232 2x10^{-6}</td>
<td>0.21 0.00198 3x10^{-6}</td>
<td>0.09 0.00225 5x10^{-6}</td>
<td>0.09 0.00223 5x10^{-6}</td>
</tr>
<tr>
<td>0.076</td>
<td>0.00192 3x10^{-6}</td>
<td>0.03 0.00102 9x10^{-6}</td>
<td>0.09 0.00208 4x10^{-6}</td>
<td>0.09 0.00209 4x10^{-6}</td>
</tr>
<tr>
<td>0.049</td>
<td>0.00097 8x10^{-6}</td>
<td>0.03 0.00085 6x10^{-6}</td>
<td>0.09 0.00109 1x10^{-6}</td>
<td>0.10 0.00109 9x10^{-6}</td>
</tr>
<tr>
<td>0.043</td>
<td>0.00084 7x10^{-6}</td>
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<td>0.09 0.00088 7x10^{-6}</td>
<td>0.10 0.00088 6x10^{-6}</td>
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</tbody>
</table>
CONCLUSION

In flood routing, routed hydrographs have an uncertainty that depends on different physical stochastic parameters. Four of the relationships, namely of Samani et al., Stephenson, Adel, and McCorquodale et al. are employed to reflect the hydraulics of the flow through rockfill dams. In this analysis, FOVE, Harr's method and the LHS have been used to evaluate the uncertainty of the routed hydrograph. The results of the analysis show that Samani et al.'s Stephenson's and Adel's relationships are almost the same and this means using any of those relation-
ships make no significant difference. Also, applying LHS sampling means more reliable design. The sensitivity analysis shows that the parameters m and n in McCorquodale et al.'s relationship, d and n in Stephenson's, $d_{15}$ and n in Samani et al.'s show the highest influence on the routed outflows among other parameters.

**Notations**

- $a =$ Empirical coefficient of the equation related to flow and particle characteristics;
- $b =$ Empirical coefficient of the equation related to flow and particle characteristics;
- $C_i =$ Indicates the uncertainty of the input parameter;
- $d =$ Average diameter of rock;
- $d_{15} =$ Particle diameter where 15% of the total particles' weight is smaller;
- $d_{50} =$ Particle diameter size where 50% of the total particles' weight is smaller;
- $f =$ Friction factor;
- $f_r =$ Friction factor between large particles and the instrument wall;
- $f_d =$ Darcy-Weisbach coefficient;
- $g =$ Acceleration due to gravity;
- $H_1 =$ Dam upstream water level;
- $H_2 =$ Dam downstream water level;
- $I =$ Hydraulic gradient;
- $I =$ Inflow rate;
- $K_r =$ Friction coefficient in the turbulent flow region;
- $L =$ Length of dam base;
- $m =$ Refers to pores hydraulic radius;
- $m' =$ Pores effective hydraulic radius;
- $n =$ Porosity;
- $O =$ Outflow rate;
- $S =$ Storage;
- $t =$ Time;
- $V =$ Flow velocity;
- $\sigma =$ The standard deviation of particle;
- $Q =$ Flow rate;
- $SSR =$ The summation of SSR;
- $SSR_i =$ The summation of square values of the ith input stochastic parameter;
- $w =$ Dam width.

**REFERENCES**


آنالیز عدم قطعیت هیدروگراف خروجی روند یاپی سیل در سد پارسه‌سیگی

ج.م. و. سامانی و ع. سلیمانی

چکیده

سدهای پانزده گذشته تاکنون یکی از روشهای ارزان برای کاهش خسارات سیل هستند که از مواد سنگی ساخته شده‌اند. هم‌ریزیک به دلیل وجود خلل و فرج برگزیده از قانونی دارسی تعبیه شده که تحقیقات قبیل انجام گرفته‌اند که به چهره رابطه مکان کوزک و همکاران، استیمپس، آدل و همکاران بیشتری جذب آرای جریان در محیط سنگزرهای دارند. پس ضروری به نظر می‌رسد که تجزیه و تحلیل عدم قطعیت و تجزیه و تحلیل حساسیت بر روی این روابط صورت گیرد با توصیف بهتری از میزان پرداختگی نتایج حاصل از استفاده از این روابط بخش آیه و سهم پارامترهای ورودی در عدم قطعیت خروجی این روابط مشخص گردد. بدین منظور یکی از هزینه‌های بهبود و سنجش‌های تنظیم در تهیه و پیامدها می‌توانند تصادفی انتخاب شد و مشخصات فیزیکی سرعت با هریک از روابط تجربی برای هر یک از نمونه‌ها اندازه‌گیری یا تخمین زده شد. همچنین از این نویسه سلیمانی در فلزات آزمایشگاهی ساخته شد و ارتفاع آب در بالاتر (H2) و پایین‌تر (H1) سردر دو دی‌های مختلف اندازه‌گیری گردید و به عنوان نتیجه قلمی سایر پارامترها به عنوان متغیر قلمی به عنوان مدل نرم‌افزار LHS و یا به عنوان محاسبه به روش‌هایی مشابه به عنوان پارامتر بر اساس از نظر گرفته شدند. تجزیه و تحلیل عدم قطعیت پیوسته فن برای یک هیدروگراف خروجی انجام گردیده، سپس از روش‌هایی مشابه به عنوان گروه LHS در (FOVE) و روش نرم‌افزاری نیز کاربرد بیشتری نمی‌بیند.

جبهه رابطه با تجزیه و تحلیل عدم قطعیت به کار گرفته شاند. بطور کلی تابعی می‌دهند که رابطه سالمی و همکاران به ترتیب دارای نماینده و پیش‌نیازهای عدم قطعیت می‌باشند.

همچنین تابعی تجزیه و تحلیل حساسیت نشان می‌دهد که پارامترهای مختلف در تعیین عدم قطعیت حسی خروجی تأثیر دارند.
ATTACHMENT 6D
LEPS CALCULATION WORKSHEET
Formulas for Flow Velocity through a Rock Drain [Source: Flow Through Rockfill (Leps, 1973)]

LogD (in)  LogWm^{0.5} (in/sec)
-0.124939  1.000000
0.301030  1.204120
0.778151  1.447158
0.903090  1.505150
1.380211  1.763428
1.681241  1.924279

\[ y = 0.0223x^2 + 0.4779x + 1.059 \]
\[ R^2 = 1 \]

\[ K_2 = 0.9546D^{(0.4779 - 0.0223\log D)} \text{ (ft/sec)} \]
Where $D =$ inches

\[ V = K_2^{(0.54)} = 0.9546D^{(0.4779 - 0.0223\log D)}(0.54) \text{ (ft/sec)} \]

<table>
<thead>
<tr>
<th>HMS ID</th>
<th>Drain Length</th>
<th>Rock Size (D)</th>
<th>Slope (i)</th>
<th>Velocity (ft/sec)</th>
<th>Travel Time (Min)</th>
<th>Travel Time (Hr)</th>
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</thead>
<tbody>
<tr>
<td>North-N2</td>
<td>7780</td>
<td>14</td>
<td>0.03</td>
<td>0.47</td>
<td>273.48</td>
<td>4.56</td>
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<td>North-N3</td>
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<td>0.47</td>
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<td>4.26</td>
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<tr>
<td>South-S1</td>
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<td>0.47</td>
<td>294.64</td>
<td>4.91</td>
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<td>South-S1C</td>
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<td>14</td>
<td>0.03</td>
<td>0.47</td>
<td>371.20</td>
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<td>South-S2A</td>
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<td>0.47</td>
<td>412.15</td>
<td>6.87</td>
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