FINAL REPORT

SCOUR ANALYSIS
SUNRISE POWERLINK PROJECT

SAN DIEGO AND IMPERIAL COUNTIES, CALIFORNIA

PREPARED FOR:
SAN DIEGO GAS & ELECTRIC COMPANY

URS PROJECT NO. 27669030.00003

JULY 12, 2010
SCOUR ANALYSIS
SUNRISE POWERLINK PROJECT
SAN DIEGO AND IMPERIAL COUNTIES, CALIFORNIA

Prepared for

San Diego Gas & Electric Company
Ms. Molly Frisbie
8315 Century Park Court, CP-51G
San Diego, CA 92123

URS Project No. 27669030.00003

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San Diego, CA 92108-4314
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July 12, 2010

Ms. Molly Frisbie  
San Diego Gas & Electric Company  
8315 Century Park Court, CP-21G  
San Diego, CA 92123

Subject: Scour Analysis  
Sunrise Powerlink Project  
San Diego and Imperial Counties, California  
URS Project No. 27669030.00003

Dear Ms. Frisbie:

URS Corporation Americas (URS) is pleased to present this final report providing our scour analysis in support of the proposed Sunrise Powerlink project. Our work is intended to assist San Diego Gas & Electric Company (SDG&E) and their consultants with project planning and design and specifically to provide evaluations of potential scour depths to assist with the engineering design of the transmission line structure foundations.

If you have any questions regarding this report, please contact us.

Sincerely,

URS CORPORATION

Matt Moore, R.C.E.  
Senior Project Engineer

Thomas Grace  
Engineer

MM/TG:mv
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<table>
<thead>
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<th>Description</th>
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<td>kilovolt</td>
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<tr>
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SECTION ONE

SECTION 1 INTRODUCTION

1.1 BACKGROUND AND PROJECT DESCRIPTION

The Sunrise Powerlink Project is a proposed 230/500 kilovolt (kV) transmission line that extends from the San Diego Gas & Electric Company (SDG&E) Sycamore Canyon Substation in San Diego County eastward to the SDG&E Imperial Valley Substation in Imperial County. The western portion of the project is a 230 kV transmission line beginning at Sycamore Canyon Substation and extending to the proposed Suncrest Substation located east of Alpine and south of Interstate 8 in the Bell Bluff area. From the Suncrest Substation, a 500 kV transmission line extends eastward, crossing Interstate 8 twice between the Suncrest Substation and the Jacumba area, crossing it again in the Mountain Springs Grade area, and crossing again in the Plaster City area. The eastern terminus of the project is the Imperial Valley Substation (Figure 1).

The overhead transmission line alignment has been divided into 13 sections for reference purposes. The section designations provided to URS Corporation Americas (URS) are from west to east; 4A, 5, 7, 8A, 8B, 8C, 8D, 8E, 9A, 9B, 9C, 10A, and 10B. Section 6 is a proposed underground alignment and is not a part of this study. An alternative route has been analyzed for the western portion of Section 10B. It is our understanding that this Sugarloaf-re-route alternative is intended to help minimize potential environmental impacts in the vicinity of Sugarloaf Mountain near the lower elevations of Mountain Springs Grade and the transition to the desert setting north of the Interstate 8 crossing.

The transmission line includes multiple types of structures, including strain, tangent, angle, and dead end towers, and steel poles. The structure designation of “-1 or -2” indicates that during the planning and early design phase, a structure has been moved from an initial location or the structure type has changed. The structure numbering is unique within the 230 kV and 500 kV portions of the alignment, however, the numbering system starts over at the beginning of the 500 kV line.

Two existing SDG&E transmission lines overlap portions of the proposed Sunrise alignment. The western end of the proposed project (Section 4A) parallels the western end of the existing Sycamore-Creelman transmission line. In addition, from the Jacumba area eastward, the proposed project generally parallels the existing Southwest Powerlink 500 kV Transmission Line (SWPL) to the Imperial Valley Substation (part of Section 9C, Section 10A and 10B).

Structure locations are based on files provided by SDG&E consisting of spreadsheets (staking sheets) and Google Earth files. URS previously performed a geotechnical and geologic hazards investigation for the project and submitted the results in a draft report (Revision 2, dated October 16, 2009).

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this scour analysis was for URS to provide potential scour information to assist SDG&E with project planning and engineering design of transmission line structure foundations. The design of any structure located within a drainage or a wash should address the possible loss of foundation support due to scour. The scope of our work included: reviewing previous URS geotechnical and geologic hazards
investigations for the project; performing hydrologic, hydraulic, and scour depth calculations; and preparing this report. No geotechnical borings were performed for this investigation.

Scour is not a significant hazard for most of the project alignment. Structure sites were specifically located to avoid drainages and the associated scour potential. All of the larger drainages in the western and central portion of the project are spanned, including; San Vicente Creek, the San Diego River, the Sweetwater River, Wilson Creek, Cottonwood Creek, Potrero Creek, Hauser Creek, La Posta Creek, Walker Canyon, Boulder Creek, and Myer Creek. A review of the western and central portion of the alignment was performed and the following structures were selected for specific hydrologic evaluation.

In the desert setting of Section 10A and 10B, the broad desert washes are crossed in such a way that structures are located outside of the active channels. However, some scour potential still exists in this alluvial fan environment. The size, direction and location of the main channels and distributaries can change rapidly during a severe flood event in an alluvial fan. The result is that a flood moving across the upper portion of an alluvial fan may not follow the same flow path, have the same velocity, depth, and distribution of flow. Therefore, several structures that lie within an alluvial fan were analyzed for scour.
SECTION 2 HYDROLOGIC ANALYSIS

Based on our review of the proposed structure locations, hydrologic analyses were conducted to estimate 100-year flood discharges for a total of 15 drainages or washes which contain 26 proposed structure locations, as presented in Table 1 and 2. The structures were selected based on their location within, or nearby, washes and alluvial fan regions that may have scour potential. The Site Plan and Generalized Geologic Maps (Figures 2j, 2l and 2v-2y) show the proposed structure locations. The figures illustrate the analyzed structures. The numbering system for these figures was maintained from the Draft Report – Revision 3, Geotechnical and Geologic Hazards Investigation, Sunrise Powerlink Project, San Diego and Imperial Counties, California, dated March 22, 2010. Figure 3 presents the Key to the Geologic Map. Figures 4A and 4B illustrate approximate locations of the washes and their corresponding drainage areas delineated on the United States Geological Survey (USGS) topographic base maps. The estimated drainage areas for these washes are presented in Table 1. Figure 4C illustrates the hydrology for the Sugarloaf Re-Route alignment and Table 2 illustrates the hydrologic results for the re-route.

For estimating the peak storm event with peak sediment carrying capacity, the 100-year storm event (a storm event with a 0.1 percent chance of occurrence in any given year) was used as a design basis in accordance with the National Flood Insurance Protection (NFIP) program. The use of the 100-year storm event for scour is based on variability of channel hydraulics, channel material, and general complexity of the erosive process. USGS regression equations were used to determine peak discharge from the upgradient watersheds that impact proposed structures with erosion caused by scour. Regional flood frequency equations developed by the USGS were used to estimate 100-year flood discharges in cubic feet per second (cfs) for the ungauged streams (USGS 1977). The regional equations, developed for the California South Lahontan-Colorado Desert Region (Structures EP292-1 through EP324) and South Coast Region (Structures EP54 and EP90-1) that were used in these analyses are:

South Lahontan-Colorado Desert Region: \[ Q_{100} = 1080A^{0.71} \]

South Coast Region: \[ Q_{100} = 1.95A^{0.83}P^{1.87} \]

Where: \( Q_{100} = 100\text{-year flood discharge (cfs)} \)
\( A = \text{Drainage area (in square miles)} \)
\( P = \text{Mean annual precipitation (in inches)} \)

Mean annual precipitation values were obtained from “Mean Annual Precipitation Maps for California Region” prepared by Rantz (1969).

The South Lahontan-Colorado Desert Region regression equation was used in the eastern portion of the proposed power line alignment that lies within the alluvial fan setting. Note that this equation is defined only for basins of 25 square miles or less. For watersheds greater than 25 square miles, the Flood Insurance Study (FIS) for Imperial County, California and Incorporated Areas, dated September 26, 2008, was used for reference to approximate the 100-year flood discharge. Due to the proximity of the structures and the meandering potential of washes, several structures share the same watershed. Detailed input parameters and hydrologic calculations for the stream crossings are...
presented in Appendix A and summarized in Table 1. Table 2 summarizes the hydrologic results for the Sugarloaf Re-Route.

### Table 1
Hydrologic Results
Sunrise Powerlink Project

<table>
<thead>
<tr>
<th>Alignment Section</th>
<th>Structure I.D.</th>
<th>Watershed Area (Square Miles)</th>
<th>100-Year Flowrate (cfs)</th>
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<td>1,489</td>
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<td>EP90-1</td>
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<td>EP292-1</td>
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<td>EP295</td>
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<td>7,706</td>
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<td>EP297 EP298</td>
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## Table 2
Hydrologic Results for Sugarloaf Re-Route
Sunrise Powerlink Project

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<th>100-Year Flowrate (cfs)</th>
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<td></td>
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</table>
SECTION 3 HYDRAULIC ANALYSIS

Hydraulic analysis provided the flow depth and flow top width for the scour calculations. Cross-sections were taken at several structure locations with scour potential as shown on Figures 5A and 5B. For this analysis, the normal depth method was used to determine the hydraulic parameters for each cross-section.

Channel cross-section data, required for the hydraulic analyses, were developed based on digital topographic data provided by SDG&E with 2-foot contour intervals. Channel flow top widths and depths were estimated using Manning’s equation, the channel cross section at the proposed structure location, and the approximate channel slope. Channel cross sections and approximate channel slopes were derived from the topographic maps. Channel flow top widths and depths were calculated using Hydraflow Express Extension for AutoCad (Autodesk 2008). Detailed input parameters and hydraulic calculations for the stream crossings are presented in Appendix B and summarized in Table 3.
Table 4 summarizes the hydraulic results for the Sugarloaf Re-Route.

**Table 3**  
Summary of Hydraulic Analysis Results  
Sunrise Powerlink Project

<table>
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<tr>
<th>Alignment Section</th>
<th>Structure I.D.</th>
<th>100-Year Flowrate (cfs)</th>
<th>Flow Depth (feet)</th>
<th>Top Width (feet)</th>
<th>Design Flood Discharge Per Unit Width (cfs/ft)</th>
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Summary of Hydraulic Analysis Results for Sugarloaf Re-Route
Sunrise Powerlink Project

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<th>Top Width (feet)</th>
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SECTION 4  SCOUR ANALYSIS

Scour is defined as the lowering of the stream channel bed due to erosion resulting from high flow velocities during a flood event. The determination of scour depths for midchannel transmission line foundations required the use of empirical relationships that include one or more of the following hydraulic parameters: pier width and skew, flow depth, velocity, and particle size ($D_{50}$) distribution of sediment.

The U.S. Bureau of Reclamation (USBR) publication Technical Guidelines for Computing Degradation and Local Scour (USBR, 1984) lists three empirical methods that can be used to estimate scour depths, including:

- Jain Equation (1981)
- Lacey Regime Equation (1930), and

These methods are commonly used for estimating the burial depths of midchannel facilities. However, due to uncertainties in defining input parameters for the empirical methods and the variability of the results, USBR recommends calculating scour depths using several methods and utilize judgment in averaging the results or selection of the most applicable procedures (USBR, 1984).

The results of this scour depth analysis provide scour depths for the 100-year storm event for structures in alluvial fans which have a definite potential for scour.

Analyses were conducted at proposed structures with scour erosion potential to estimate the scour depth associated with the 100-year design flood event. Scour depths were calculated based on channel flow top widths and depths estimated during the hydraulic analyses and assumed mean grain sizes ($D_{50}$) for bed materials. In the absence of site-specific bed material $D_{50}$ data for individual structures, $D_{50}$ values estimated based on soil survey data and nearby geotechnical borings performed for the Southwest Powerlink alignment were used (Woodward-Clyde Consultants, 1981).

The proposed structures have a foundation diameter which ranges from 4 to 8 feet. For this analysis, we used the maximum diameter of 6 feet which produces the maximum scour depth.

Table 5 presents the maximum 100-year scour depths calculated based on the above methods for each of the wash crossings. Table 6 summarizes the scour results for the Sugarloaf Re-Route.
### Table 5
**Calculated Scour Depths**  
Sunrise Powerlink Project

<table>
<thead>
<tr>
<th>Alignment Section</th>
<th>Structure I.D.</th>
<th>Calculated Scour Depth Below Surface (feet)</th>
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<td>4.3</td>
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<tr>
<td>10B</td>
<td>EP323-1 EP324</td>
<td>8.0</td>
</tr>
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</table>
Table 6
Calculated Scour Depths for Sugarloaf Re-Route
Sunrise Powerlink Project

<table>
<thead>
<tr>
<th>Alignment Section</th>
<th>Structure I.D.</th>
<th>Calculated Scour Depth Below Surface (feet)</th>
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The results of the calculated scour depth for each of the wash crossings are summarized in Table 5 and Table 6.

All structures within the project were reviewed and the foundations of the structures listed have the potential to be subjected to scour.

In addition, an evaluation regarding the potential for the tower and associated structures to induce erosion onto adjacent properties was conducted through the use of aerial photography and property maps. The majority of the structures listed in the above tables are situated on Bureau of Land Management (BLM) land and no nearby property boundaries are present. Based on our evaluation no adjacent properties would be impacted by local erosion induced by the towers. Although the potential for offsite erosion is low, if significant erosion is observed that could have the potential to impact downstream properties, the project area should be stabilized using permanent post-construction stormwater BMP erosion and sediment control BMPs such as vegetation, rock rip-rap, matting, or other appropriate erosion and soil stabilization techniques.
SECTION 5  UNCERTAINTIES AND LIMITATIONS

The recommendations made herein are based on the assumption that topographic or subsurface conditions do not deviate appreciably from those found during our field review, and during the previous and current geotechnical investigations.

Hydrologic and hydraulic engineering is characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect. Scour analysis were performed using approximate grain size ($D_{50}$) values. The $D_{50}$ data used for the scour analyses reflect the general trend of the bed material and sediment type presence along the proposed alignment.

Final design details for the proposed project are not available at this time. The recommendations presented in this report are intended to assist SDG&E and their subconsultants in the project planning and design. The professional judgments and interpretations of the subsurface conditions in the project area, and our understanding of the geologic setting of the project is based on the information provided to us, published literature, and previous studies, referenced in this report.
SECTION SIX

REFERENCES

Autodesk (2008), Hydraflow Express Extension for AutoCAD, Version 6.052 by Autodesk, Inc.

FEMA (1984), Flood Insurance Rate Study (FIS) for Imperial County, California, Federal Emergency Management Agency (FEMA).


USGS (1977), Magnitude and Frequency of Floods in California, Prepared for California Department of Water Resources, California Department of Transportation, Federal Highway Administration.

Woodward-Clyde Consultants (1981), Geotechnical Investigation for Jade to Imperial Valley Substation Segment of the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 213 through 312, PI 21-28, dated June 12, 1981 (WCC Project No. 591251-DES2).
Figures

Figure 1  Vicinity Map
Figure 2l  Site Plan and Generalized Geologic Map
Figure 2v  Site Plan and Generalized Geologic Map
Figure 2w  Site Plan and Generalized Geologic Map
Figure 2x  Site Plan and Generalized Geologic Map
Figure 2y  Site Plan and Generalized Geologic Map

Figure 3  Key to Geologic Map

Figure 4  Hydrologic Work Map

Figure 5  Wash Cross-Sections for Hydraulic Analysis
Proposed Centerline Structure Type
- Original Overhead Alignment
- Underground Alignment
- Supershift Re-Route
- MSG Re-Route
- Miscellaneous Structure Designation

On-Going URS Geotechnical Investigation (2009)
- Seismic Refraction Test Location
- Seismic Refraction Test Designation
- Electrical Resistivity Test Location

Previous Geotechnical Investigation
- Existing Sycamore-Creelman TL Structure
- Existing Sycamore-Creelman TL Structure with Seismic Line
- Existing SWPL TL Structure
- Existing SWPL Tower with Seismic Line
- Existing SWPL Tower with Boring

SOURCES
See Figure 3

SITE PLAN AND GENERALIZED GEOLOGIC MAP
SUNRISE POWERLINK PROJECT
SAN DIEGO & IMPERIAL COUNTIES, CALIFORNIA

CREATED BY: CL DATE: 03-22-10
PM: MEH PROJ. NO: 27669030.00004
FIG. NO: 2v
Sources:


3) Modified from *Preliminary Geologic Map of the Imperial County, California*. Paul K. Morton. 1966


7) Proposed Structures - SDG&E. September 30, October 8, 12 and 15, 2009

8) Freeways/Interstates – ESRI.

### GEOLOGIC LEGEND AND SOURCES

**SUNRISE POWERLINK PROJECT**

**SAN DIEGO & IMPERIAL COUNTIES, CALIFORNIA**

<table>
<thead>
<tr>
<th>Legend</th>
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**Alquist Priolo (Earthquake Fault Zone) Faults**

- Accurately Located Fault Trace
- Approximately Located Fault Trace
- Inferred Fault Trace
- Concealed Fault Trace

**Quaternary Faults**

- Accurately Located Fault Trace
- Approximately Located Fault Trace
- Concealed Fault Trace

**Pre-Quaternary Faults**

- Accurately Located Fault Trace
- Approximately Located Fault Trace
- Concealed Fault Trace

**Geologic Contact**

- Approximately Located Geologic Contact

**Approximate location of possible ancient landslide**

Arrows denote direction of possible movement.
Appendix A contains the hydrologic analysis, based upon the USGS regression equations (South Lahontan-Colorado Desert Region and South Coast Region), to determine the 100-year storm event runoff generated from the watersheds.
### Table 1. SUNRISE POWERLINK PROJECT HYDROLOGY CALCULATIONS

<table>
<thead>
<tr>
<th>STRUCTURE I.D.</th>
<th>A (sq.mi.)</th>
<th>P (in)</th>
<th>Q2 (cfs)</th>
<th>Q5 (cfs)</th>
<th>Q10 (cfs)</th>
<th>Q25 (cfs)</th>
<th>Q50 (cfs)</th>
<th>Q100 (cfs)</th>
<th>FIS (Interpolation)</th>
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<td>179</td>
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<td><strong>South Lahontan-Colorado Desert Region</strong></td>
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</tbody>
</table>

1. Equations are defined only for basins of 25 square miles or less in the Northeast and South Lahontan-Colorado Desert regions.

2. For areas greater than 25 square miles, used FIS study (Imperial County, September 2008) that is within the South Lahontan Region to obtain an discharge/area ratio.
Table 2. SUNRISE POWERLINK PROJECT
HYDROLOGY CALCULATIONS FOR THE SUGARLOAF RE-ROUTE

<table>
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<tr>
<th>STRUCTURE I.D.</th>
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<th>Q2 (cfs)</th>
<th>Q5 (cfs)</th>
<th>Q10 (cfs)</th>
<th>Q25 (cfs)</th>
<th>Q50 (cfs)</th>
<th>Q100 (cfs)</th>
<th>FIS (Interpolation)</th>
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<td>139</td>
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<td>78</td>
<td>239</td>
<td>714</td>
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<td>2,017</td>
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1. Equations are defined only for basins of 25 square miles or less in the Northeast and South Lahontan-Colorado Desert regions.
2. For areas greater than 25 square miles, used FIS study (Imperial County, September 2008) that is within the South Lahontan Region to obtain an discharge/area ratio.
California is divided into six hydrologic regions (fig. 1). The regression equations developed for these regions are for estimating peak discharges (QT) having recurrence intervals T that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (A), in square miles; mean annual precipitation (P), in inches; and an altitude index (H), which is the average of altitudes in thousands of feet at points along the main channel at 10 percent, and 85 percent of the distances from the site to the divide. The variables A and H may be measured from topographic maps. Mean annual precipitation (P) is determined from a map in Rantz (1969). The regression equations were developed from peak-discharge records of 10 years or longer, available as of 1975, at more than 700 gaging stations throughout the State. The regression equations are applicable to unregulated streams but are not applicable to some parts of the State (see fig. 1). The standard errors of estimate for the regression equations for various recurrence intervals and regions range from 60 to over 100 percent. The report by Waananen and Crippen (1977) includes an approximate procedure for increasing a rural discharge to account for the effect of urban development. The influences of fire and other basin changes on flood magnitudes are also discussed.

Procedure

Topographic maps, the hydrologic regions map (fig. 1), the mean annual precipitation from Rantz (1969), and the following equations are used to estimate the needed peak discharges QT, in cubic feet per second, having selected recurrence intervals T.

**North Coast Region**

\[
\begin{align*}
Q_2 &= 3.52 A^{0.90} P^{0.89} H^{-0.47} \\
Q_5 &= 5.04 A^{0.89} P^{0.91} H^{-0.35} \\
Q_{10} &= 6.21 A^{0.88} P^{0.93} H^{-0.27} \\
Q_{25} &= 7.64 A^{0.87} P^{0.94} H^{-0.17} \\
Q_{50} &= 8.57 A^{0.87} P^{0.96} H^{-0.08} \\
Q_{100} &= 9.23 A^{0.87} P^{0.97}
\end{align*}
\]

**Northeast Region**
Sierra Region

\[
\begin{align*}
Q_2 &= 22 A^{0.40} \\ 
Q_5 &= 46 A^{0.45} \\ 
Q_{10} &= 61 A^{0.49} \\ 
Q_{25} &= 84 A^{0.54} \\ 
Q_{50} &= 103 A^{0.57} \\ 
Q_{100} &= 125 A^{0.59}
\end{align*}
\]

Central Coast Region

\[
\begin{align*}
Q_2 &= 0.24 A^{0.88} p^{1.58} H^{-0.80} \\ 
Q_5 &= 1.20 A^{0.82} p^{1.37} H^{-0.64} \\ 
Q_{10} &= 2.63 A^{0.80} p^{1.25} H^{-0.58} \\ 
Q_{25} &= 6.55 A^{0.79} p^{1.12} H^{-0.52} \\ 
Q_{50} &= 10.4 A^{0.78} p^{1.06} H^{-0.48} \\ 
Q_{100} &= 15.7 A^{0.77} p^{1.02} H^{-0.43}
\end{align*}
\]

South Coast Region

\[
\begin{align*}
Q_2 &= 0.0061 A^{0.92} p^{2.54} H^{-1.10} \\ 
Q_5 &= 0.118 A^{0.91} p^{1.95} H^{-0.79} \\ 
Q_{10} &= 0.583 A^{0.90} p^{1.61} H^{-0.64} \\ 
Q_{25} &= 2.91 A^{0.89} p^{1.26} H^{-0.50} \\ 
Q_{50} &= 8.20 A^{0.89} p^{1.05} H^{-0.41} \\ 
Q_{100} &= 19.7 A^{0.88} p^{0.84} H^{-0.33}
\end{align*}
\]

South Lahontan-Colorado Desert Region

\[
\begin{align*}
Q_2 &= 7.3 A^{0.30} \\ 
Q_5 &= 53 A^{0.44} \\ 
Q_{10} &= 150 A^{0.53} \\ 
Q_{25} &= 410 A^{0.63} \\ 
Q_{50} &= 700 A^{0.68} \\ 
Q_{100} &= 1080 A^{0.71}
\end{align*}
\]

In the North Coast region, use a minimum value of 1.0 for the altitude index (H). Equations are defined only for basins of 25 mi² or less in the Northeast and South Lahontan-Colorado Desert regions.

Reference


**Additional Reference**


Figure 1. Flood-frequency region map for California. ([PostScript file of Figure 1.](#))
Figure 1. Flood-frequency region map for California.
Appendix B contains the Hydraflow results for the hydraulic analysis. Channel flow velocities and depths were estimated using Manning’s equation, the channel cross section at the proposed structure and the approximate channel slope.
User-defined

Invert Elev (ft) = 2845.00
Slope (%) = 2.10
N-Value = 0.040

Highlighted

Depth (ft) = 2.61
Q (cfs) = 1,982
Area (sqft) = 319.13
Velocity (ft/s) = 6.21
Wetted Perim (ft) = 255.28
Crit Depth, Yc (ft) = 2.59
Top Width (ft) = 255.05
EGL (ft) = 3.21

(Sta, El, n)-(Sta, El, n)...
(0.00, 2860.00)-(107.00, 2848.00, 0.040)-(150.00, 2845.00, 0.040)-(215.00, 2848.00, 0.040)-(302.00, 2848.00, 0.040)-(339.00, 2846.00, 0.040)-(422.00, 2846.00, 0.040)-(482.00, 2848.00, 0.040)-(659.00, 2860.00, 0.040)

Calculations

Compute by: Known Q
Known Q (cfs) = 1982.00

Depth (ft) = 2.61
Q (cfs) = 1,982
Area (sqft) = 319.13
Velocity (ft/s) = 6.21
Wetted Perim (ft) = 255.28
Crit Depth, Yc (ft) = 2.59
Top Width (ft) = 255.05
EGL (ft) = 3.21

(Sta, El, n)-(Sta, El, n)...
(0.00, 2860.00)-(107.00, 2848.00, 0.040)-(150.00, 2845.00, 0.040)-(215.00, 2848.00, 0.040)-(302.00, 2848.00, 0.040)-(339.00, 2846.00, 0.040)-(422.00, 2846.00, 0.040)-(482.00, 2848.00, 0.040)-(659.00, 2860.00, 0.040)
P292-1

User-defined

- Invert Elev (ft) = 725.50
- Slope (%) = 2.00
- N-Value = 0.040

Highlighted

- Depth (ft) = 0.99
- Q (cfs) = 189.00
- Area (sqft) = 55.98
- Velocity (ft/s) = 3.38
- Wetted Perim (ft) = 107.30
- Crit Depth, Yc (ft) = 0.92
- Top Width (ft) = 107.28
- EGL (ft) = 1.17

(Sta, El, n)-(Sta, El, n)... 
(0.00, 740.00)-(33.00, 732.00, 0.040)-(44.00, 730.00, 0.040)-(165.00, 728.00, 0.040)-(275.00, 726.00, 0.040)-(305.00, 725.50, 0.040)-(335.00, 726.00, 0.040)-(418.00, 728.00, 0.040)-(532.00, 730.00, 0.040)

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<tr>
<td>729.00</td>
<td>-3.50</td>
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Section

Sta (ft)
EP293 & EP294

User-defined
Invert Elev (ft) = 698.00
Slope (%) = 3.50
N-Value = 0.040

Calculations
Compute by: Known Q
Known Q (cfs) = 2163.00

Highlighted
Depth (ft) = 1.19
Q (cfs) = 2,163
Area (sqft) = 326.61
Velocity (ft/s) = 6.62
Wetted Perim (ft) = 350.94
Crit Depth, Yc (ft) = 1.34
Top Width (ft) = 350.92
EGL (ft) = 1.87

(Sta, El, n)-(Sta, El, n)...
(0.00, 700.00)-(203.00, 698.00, 0.040)-(401.00, 698.00, 0.040)-(455.00, 700.00, 0.040)

User-defined
Invert Elev (ft) = 596.00
Slope (%) = 2.60
N-Value = 0.040

Calculations
Compute by: Known Q
Known Q (cfs) = 7706.00

Highlighted
Depth (ft) = 1.90
Q (cfs) = 7,706
Area (sqft) = 919.99
Velocity (ft/s) = 8.38
Wetted Perim (ft) = 556.47
Crit Depth, Yc (ft) = 2.15
Top Width (ft) = 556.40
EGL (ft) = 2.99

(Sta, El, n)-(Sta, El, n)...
(0.00, 600.00)-(153.00, 598.00, 0.040)-(271.00, 596.00, 0.040)-(342.00, 596.00, 0.040)-(593.00, 596.00, 0.040)-(683.00, 596.00, 0.040)-(717.00, 598.00, 0.040)-(751.00, 600.00, 0.040)-(768.00, 600.00, 0.040)-(772.00, 598.00, 0.040)-(852.00, 598.00, 0.040)-(921.00, 600.00, 0.040)
EP297 & EP298

User-defined
Invert Elev (ft) = 508.00
Slope (%) = 2.20
N-Value = 0.040

Calculations
Compute by: Known Q
Known Q (cfs) = 1193.00

Highlighted
Depth (ft) = 0.33
Q (cfs) = 1,193
Area (sqft) = 478.98
Velocity (ft/s) = 2.49
Wetted Perim (ft) = 1484.01
Crit Depth, Yc (ft) = 0.28
Top Width (ft) = 1484.01
EGL (ft) = 0.43

(Sta, El, n)-(Sta, El, n)... 
(0.00, 510.00)-(80.00, 508.00, 0.040)-(1499.00, 508.00, 0.040)-(1813.00, 510.00, 0.040)
EP299 & EP300-1

User-defined
Invert Elev (ft) = 488.00
Slope (%) = 1.50
N-Value = 0.040

Highlighted
Depth (ft) = 2.36
Q (cfs) = 10,933
Area (sqft) = 2602.96
Velocity (ft/s) = 4.20
Wetted Perim (ft) = 2895.72
Crit Depth, Yc (ft) = 1.92
Top Width (ft) = 2895.66
EGL (ft) = 2.63

Calculations
Compute by: Known Q
Known Q (cfs) = 10933.00

(Sta, El, n)-(Sta, El, n)...
(0.00, 500.00)-(16.00, 498.00, 0.040)-(89.00, 496.00, 0.040)-(176.00, 494.00, 0.040)-(230.00, 492.00, 0.040)-(414.00, 492.00, 0.040)-(428.00, 498.00, 0.040)
-(444.00, 492.00, 0.040)-(465.00, 490.00, 0.040)-(2438.00, 490.00, 0.040)-(2598.00, 488.00, 0.040)-(3265.00, 488.00, 0.040)-(3336.00, 490.00, 0.040)-(3452.00, 49
-(3556.00, 494.00, 0.040)-(3710.00, 496.00, 0.040)-(3858.00, 498.00, 0.040)-(3966.00, 500.00, 0.040)

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Sta (ft)
**Channel Report**

Hydraflow Express Extension for AutoCAD® Civil 3D® 2009 by Autodesk, Inc.

**EP313**

**User-defined**

- Invert Elev (ft) = 312.00
- Slope (%) = 1.60
- N-Value = 0.040

**Calculations**

- Compute by: Known Q
- Known Q (cfs) = 1406.00

**Highlighted**

- Depth (ft) = 1.09
- Q (cfs) = 1,406
- Area (sqft) = 290.62
- Velocity (ft/s) = 4.84
- Wetted Perim (ft) = 277.83
- Crit Depth, Yc (ft) = 0.97
- Top Width (ft) = 277.25
- EGL (ft) = 1.45

(Sta, El, n)-(Sta, El, n)...

(0.00, 318.00)-(8.00, 316.00, 0.040)-(14.00, 314.00, 0.040)-(21.00, 312.00, 0.040)-(249.00, 312.00, 0.040)-(266.00, 314.00, 0.040)-(277.00, 312.00, 0.040)-(305.00, 312.00, 0.040)-(309.00, 314.00, 0.040)-(315.00, 316.00, 0.040)-(322.00, 318.00, 0.040)
Channel Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2009 by Autodesk, Inc.

Friday, Jul 2 2010

EP316-2

User-defined

Invert Elev (ft) = 266.00
Slope (%) = 2.60
N-Value = 0.040

Highlighted

Depth (ft) = 1.56
Q (cfs) = 1,915
Area (sqft) = 304.50
Velocity (ft/s) = 6.29
Wetted Perim (ft) = 281.82
Crit Depth, Yc (ft) = 1.62
Top Width (ft) = 281.38
EGL (ft) = 2.17

Calculations

Compute by: Known Q
Known Q (cfs) = 1914.70

(Sta, El, n)-(Sta, El, n)...
(0.00, 276.00)-(23.00, 268.00, 0.040)-(188.00, 268.00, 0.040)-(192.00, 266.00, 0.040)-(275.00, 266.00, 0.040)-(357.00, 268.00, 0.040)-(360.00, 268.00, 0.040)
-(447.00, 266.00, 0.040)-(473.00, 266.00, 0.040)-(521.00, 268.00, 0.040)-(554.00, 270.00, 0.040)-(579.00, 276.00, 0.040)

Elev (ft) | Depth (ft)
--- | ---
278.00 | 12.00
276.00 | 10.00
274.00 | 8.00
272.00 | 6.00
270.00 | 4.00
268.00 | 2.00
266.00 | 0.00
264.00 | -2.00
-100 | 0 100 200 300 400 500 600 700

EGL (ft) = 2.17
Top Width (ft) = 281.38
Vel (ft/s) = 6.29
H (ft) = 1.56
Q (cfs) = 1,915
A (sqft) = 304.50
Wet Perim (ft) = 281.82

Known Q (cfs) = 1914.70

(0.00, 276.00)-(23.00, 268.00, 0.040)-(188.00, 268.00, 0.040)-(192.00, 266.00, 0.040)-(275.00, 266.00, 0.040)-(357.00, 268.00, 0.040)-(360.00, 268.00, 0.040)
-(447.00, 266.00, 0.040)-(473.00, 266.00, 0.040)-(521.00, 268.00, 0.040)-(554.00, 270.00, 0.040)-(579.00, 276.00, 0.040)
**EP323-1 & EP324**

### User-defined
- **Invert Elev (ft)** = 168.00
- **Slope (%)** = 1.00
- **N-Value** = 0.040

### Calculations
- **Compute by:** Known Q
- **Known Q (cfs)** = 38791.00

### Highlighted
- **Depth (ft)** = 3.74
- **Q (cfs)** = 38791
- **Area (sqft)** = 5330.30
- **Velocity (ft/s)** = 7.28
- **Wetted Perim (ft)** = 1946.93
- **Crit Depth, Yc (ft)** = 3.31
- **Top Width (ft)** = 1946.58
- **EGL (ft)** = 4.56

### Elev (ft) Section

- **Sta (ft)**
  - 162.00
  - 168.00
  - 174.00
  - 180.00
  - 186.00
  - 192.00
  - 198.00
  - 204.00

- **Depth (ft)**
  - 0.00
  - 6.00
  - 12.00
  - 18.00
  - 24.00
  - 30.00
  - 36.00

---

**Channel Report**

Hydraflow Express Extension for AutoCAD® Civil 3D® 2009 by Autodesk, Inc.

Friday, Jul 2 2010
SUGARLOAF RE-ROUTE EP282

User-defined

- Invert Elev (ft) = 892.00
- Slope (%) = 3.25
- N-Value = 0.040

Calculations

- Compute by: Known Q
- Known Q (cfs) = 5140.00

Highlighted

- Depth (ft) = 1.99
- Q (cfs) = 5140.00
- Area (sqft) = 630.30
- Velocity (ft/s) = 8.15
- Wetted Perim (ft) = 468.58
- Crit Depth, Yc (ft) = 2.22
- Top Width (ft) = 468.47
- EGL (ft) = 3.02

(Sta, El, n)-(Sta, El, n)... (-40.00, 898.00)-(21.00, 892.00, 0.040)-(186.00, 892.00, 0.040)-(470.00, 894.00, 0.040)-(679.00, 896.00, 0.040)-(1019.00, 898.00, 0.040)
**SUGARLOAF RE-ROUTE EP283**

**User-defined**
- Invert Elev (ft) = 858.00
- Slope (%) = 3.28
- N-Value = 0.040

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 2017.00

**Highlighted**
- Depth (ft) = 0.50
- Q (cfs) = 2,017
- Area (sqft) = 495.06
- Velocity (ft/s) = 4.07
- Wetted Perim (ft) = 1026.26
- Crit Depth, Yc (ft) = 0.52
- Top Width (ft) = 1026.25
- EGL (ft) = 0.76

(Sta, El, n)-(Sta, El, n)... (0.00, 860.00)-(122.00, 858.00, 0.040)-(237.00, 858.00, 0.040)-(800.00, 858.00, 0.040)-(1076.00, 858.00, 0.040)-(1243.00, 860.00, 0.040)-(1367.00, 862.00, 0.040)

---

**Elevation Table**

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**Graph**

- X-axis: Sta (ft) from -100 to 1500
- Y-axis: Elev (ft) from 857.00 to 863.00
- Section line graph with depth values indicated.
Channel Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2009 by Autodesk, Inc.

Re-Route EP284

User-defined
- Invert Elev (ft) = 776.00
- Slope (%) = 3.20
- N-Value = 0.040

Calculations
- Compute by: Known Q
- Known Q (cfs) = 2017.00

Highlighted
- Depth (ft) = 1.75
- Q (cfs) = 2,017
- Area (sqft) = 278.96
- Velocity (ft/s) = 7.23
- Wetted Perim (ft) = 245.85
- Crit Depth, Yc (ft) = 1.92
- Top Width (ft) = 245.81
- EGL (ft) = 2.56

(Sta, El, n)-(Sta, El, n)...
(0.00, 780.00)-(69.50, 778.00, 0.040)-(161.00, 776.00, 0.040)-(234.00, 776.00, 0.040)-(340.00, 778.00, 0.040)-(440.00, 780.00, 0.040)
# SUGARLOAF RE-ROUTE EP285

**User-defined**
- Invert Elev (ft) = 716.00
- Slope (%) = 3.33
- N-Value = 0.040

**Highlight**
- Depth (ft) = 1.01
- Q (cfs) = 2,017
- Area (sqft) = 345.31
- Velocity (ft/s) = 5.84
- Wetted Perim (ft) = 431.79
- Crit Depth, Yc (ft) = 1.10
- Top Width (ft) = 431.78
- EGL (ft) = 1.54

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 2017.00

(Sta, El, n)-(Sta, El, n)...  
(0.00, 718.00)-(191.00, 716.00, 0.040)-(443.00, 716.00, 0.040)-(608.00, 718.00, 0.040)

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**Elev (ft) Depth (ft)**

**Sta (ft)**
Re-Routep EP286

User-defined
- Invert Elev (ft) = 660.00
- Slope (%) = 5.00
- N-Value = 0.040

Calculations
- Compute by: Known Q
- Known Q (cfs) = 6706.00

Highlighted
- Depth (ft) = 3.42
- Q (cfs) = 6.706
- Area (sqft) = 487.02
- Velocity (ft/s) = 13.77
- Wetted Perim (ft) = 228.28
- Crit Depth, Yc (ft) = 4.00
- Top Width (ft) = 228.13
- EGL (ft) = 6.37

(Sta, El, n)-(Sta, El, n)... 
(0.00, 664.00)-(45.00, 662.00, 0.040)-(80.00, 660.00, 0.040)-(140.00, 660.00, 0.040)-(200.00, 662.00, 0.040)-(258.00, 664.00, 0.040)

Elev (ft) Depth (ft)

Elev (ft)

Section

Depth (ft)
Re-Route EP287

**User-defined**
- Invert Elev (ft) = 616.00
- Slope (%) = 3.57
- N-Value = 0.040

**Calculations**
- Compute by: Known Q
- Known Q (cfs) = 962.00

(Sta, El, n)-(Sta, El, n)...
(0.00, 618.00)-(141.00, 616.00, 0.040)-(210.00, 616.00, 0.040)-(256.00, 618.00, 0.040)

**Highlighted**
- Depth (ft) = 1.22
- Q (cfs) = 962.00
- Area (sqft) = 153.76
- Velocity (ft/s) = 6.26
- Wetted Perim (ft) = 183.10
- Crit Depth, Yc (ft) = 1.35
- Top Width (ft) = 183.07
- EGL (ft) = 1.83
Re-Route EP288

**User-defined**
- Invert Elev (ft) = 568.00
- Slope (%) = 2.35
- N-Value = 0.040

**Highlighted**
- Depth (ft) = 1.00
- Q (cfs) = 962.00
- Area (sqft) = 186.00
- Velocity (ft/s) = 5.17
- Wetted Perim (ft) = 214.04
- Crit Depth, Yc (ft) = 0.99
- Top Width (ft) = 214.00
- EGL (ft) = 1.42

**Calculations**
Compute by: Known Q
Known Q (cfs) = 962.00

(Sta, El, n)-(Sta, El, n)...  
(0.00, 570.00)-(68.00, 568.00, 0.040)-(226.00, 568.00, 0.040)-(270.00, 570.00, 0.040)
Re-Route EP289

User-defined

Invert Elev (ft) = 524.00
Slope (%) = 2.00
N-Value = 0.040

Highlighted

Depth (ft) = 2.64
Q (cfs) = 10,888
Area (sqft) = 1293.13
Velocity (ft/s) = 8.42
Wetted Perim (ft) = 638.37
Crit Depth, Yc (ft) = 2.71
Top Width (ft) = 638.24
EGL (ft) = 3.74

(Sta, El, n)-(Sta, El, n)...
(0.00, 530.00)-(221.00, 526.00, 0.040)-(318.00, 524.00, 0.040)-(567.00, 524.00, 0.040)-(710.00, 524.00, 0.040)-(731.00, 526.00, 0.040)-(805.00, 526.00, 0.040)-(864.00, 528.00, 0.040)-(1185.00, 528.00, 0.040)-(1269.00, 530.00, 0.040)

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Sta (ft)

User-defined
Invert Elev (ft) = 508.00
Slope (%) = 2.00
N-Value = 0.040

Calculations
Compute by: Known Q
Known Q (cfs) = 10888.00

Highlighted
Depth (ft) = 2.84
Q (cfs) = 10,888
Area (sqft) = 1734.93
Velocity (ft/s) = 6.28
Wetted Perim (ft) = 1317.20
Crit Depth, Yc (ft) = 2.81
Top Width (ft) = 1317.18
EGL (ft) = 3.45

(Sta, El, n)-(Sta, El, n)...
(0.00, 512.00)-(237.00, 510.00, 0.040)-(687.00, 508.00, 0.040)-(950.00, 510.00, 0.040)-(1022.00, 510.00, 0.040)-(1088.00, 510.00, 0.040)-(1353.00, 510.00, 0.040)
Appendix C contains the soils data from SWPL (Woodward Clyde, 1981) and scour analysis results. The scour analysis was performed using the USBR technical guidelines for Computing Degradation and Local Scour (USBR, 1984). The analyses were conducted at proposed structures with scour erosion potential to estimate the scour depth associated with the 100-year design flood event.
<table>
<thead>
<tr>
<th>Location Identification</th>
<th>Depth of Scour Below Streambed (ft)</th>
<th>Multiplying Factor</th>
<th>Mean Depth at Design Discharge (ft)</th>
<th>Design Discharge (cfs)</th>
<th>Lacey's Silt Factor</th>
<th>Mean Grain Size of Bed Material (mm)</th>
<th>Blench's &quot;Zero Bed Factor&quot;</th>
<th>Depth of Scour Below Streambed (ft)</th>
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<th>Depth of Scour Below Streambed (ft)</th>
<th>Pier Size (ft)</th>
<th>Flow Depth (ft)</th>
<th>Threshold Froude Number</th>
<th>Threshold Velocity, from Figure 12 (ft/s)</th>
<th>Acceleration due to Gravity (ft/s²)</th>
<th>Average Scour Depth Below Streambed (ft)</th>
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### SUNRISE POWERLINK PROJECT
#### SUGARLOAF RE-ROUTE SCOUR DEPTH CALCULATIONS

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Figure 12. - Suggested competent mean velocities for significant bed movement of cohesionless materials, in terms of grain size and depth of flow (after Neill, 1973).
### TABLE 2.3
#### SUMMARY OF LABORATORY TESTING RESULTS

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<th>Moisture Content(%)</th>
<th>Dry Density (lb/ft³)</th>
<th>Liquid Limit(%)</th>
<th>Plasticity Index(%)</th>
<th>% Passing #200 Sieve</th>
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<th>Geologic** Formation</th>
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<td>Pcp</td>
<td>60/1&quot;</td>
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<tr>
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<td></td>
<td>CL-CH</td>
<td>Pcp</td>
<td>60/1&quot;</td>
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</tbody>
</table>

Selected samples indicate tower sites within project area and less than 10 feet below ground surface.
<table>
<thead>
<tr>
<th>Sample No</th>
<th>Tower Site</th>
<th>Depth (ft)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (lb/ft³)</th>
<th>Liquid Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>% Passing #200 Sieve</th>
<th>USCS Symbol</th>
<th>Geologic** Formation</th>
<th>Direct Shear Test Results (lb)</th>
<th>Blow Count/ft</th>
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<td>SM-SC</td>
<td>Ql</td>
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<td>15-3</td>
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<td>Ql</td>
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<td>97</td>
<td>SM</td>
<td>Pcp</td>
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<td>700</td>
</tr>
</tbody>
</table>

* Boring made near tower site(s) specified.
** See Section 3.1.

Selected samples indicate tower sites within project area and less than 10 feet below ground surface.
GRAIN SIZE DISTRIBUTION CURVES
MIGUEL-IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

SAMPLE | CLASSIFICATION AND SYMBOL | *LL | *PI
---|---|---|---
1-4 | Silty sand (SM) | - | -
2-1 | Silty sand (SM) | - | -
3-2 | Silty sand (SM) | - | -
5-1 | Silty sand (SM) | - | -
6-5 | Silty sand (SM) | - | -

*LL - Liquid Limit
*PI - Plasticity Index
### Grain Size Distribution Curves

**SAMPLE** | **CLASSIFICATION AND SYMBOL** | **LL** | **PI**
---|---|---|---
7-4 | Silty clay (CL) | 48 | 31
9-3 | Silty sand (SM) | non | plastic
10-4 | Silty clay (CL) | 43 | 27
11-5 | Silty sand (SM) | non | plastic
12-1 | Clayey sand (SC) | 32 | 15

**LL** - Liquid Limit  
**PI** - Plasticity Index

---

**Notes:**
- The table contains data for different soil samples, classified by their grain size distribution,
- The sample codes (7-4, 9-3, 10-4, 11-5, 12-1) correspond to specific locations or identifiers.
- The classification includes terms like 'silty clay' and 'clayey sand', with their respective liquid limit (LL) and plasticity index (PI) values.
- The LL values range from 32 to 48, while the PI values range from 15 to 31.
- The soil types are further classified as 'non' and 'plastic' as noted in the PI column.
### Grain Size Distribution Curves

**SAMPLE** | **CLASSIFICATION AND SYMBOL** | **LL** | **Pl**
---|---|---|---
13-1 | Silty sand to sandy silt (SM-ML) | 26 | 4
13-4 | Silty sand to sandy silt (SM-ML) | 25 | 2
13-7 | Sandy to silty clay (CL) | 40 | 25
14-4 | Silty sand (SM) | - | -
15-2 | Silty to clayey sand (SM-SC) | 21 | 5

*LL = Liquid Limit
*Pl = Plasticity Index

**MIGUEL-IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)**

**DRAWN BY:** ch | **CHECKED BY:** jw | **PROJECT NO:** 591251-DES2 | **DATE:** 6-3-81 | **FIGURE NO:** A-3

**WOODWARD-CLYDE CONSULTANTS**
Memorandum

Project: Sunrise Powerlink
Subject: Mitigation Measures H-6a and WQ-APM-10 for Scour Analysis
Date: July 30, 2010
To: Anne Coronado (Aspen Environmental Group)
From: Kevin Fisher (Horizon Water and Environment)
Ken Schwarz (Horizon Water and Environment)

(1) INTRODUCTION

In support of the environmental assessment of the Sunrise Powerlink Project (project), Aspen Environmental Group (Aspen) requested staff from Horizon Water and Environment (Horizon) to review the following report:

Scour Analysis: Sunrise Powerlink Project, San Diego and Imperial Counties, California prepared for San Diego Gas & Electric Company prepared by URS Corporation Americas (URS), URS project number 27669030.00003

More specifically, Horizon was asked to review the Scour Analysis report with a focus toward the project’s conformance with Mitigation Measures WQ-APM-10 and H-6a. Horizon has also provided general comments on the report content (as necessary). This memorandum summarizes Horizon’s findings. Please note, this memorandum does not provide quality assurance or confirmation of hydrologic or hydraulic calculations, estimations, or simulations presented in the Scour Analysis report. Those results and findings are the responsibility of the registered professional engineer of record for the report.

(2) Conformance with Mitigation Measures

Measure WQ-APM-10 requires that:
At locations where the project would cross below or pass adjacent to streams with erodible bed or banks, the burial depth shall be extended below the estimated 100-year depth of scour for that stream, or located at a sufficient distance from the bank as to avoid erosion that can reasonably be expected to occur during the life of the project; and

Mitigation Measure H-6a requires that:
A determination of towers requiring scour protection under WQ-APM 10 shall be made during the design phase by a registered professional engineer with expertise in river mechanics. All towers within the project shall be reviewed by the river mechanics engineer and the foundations of those towers determined to be subject to scour or lateral movement of a stream channel shall be protected by burial beneath the 100-year scour depth, setbacks from the channel bank, or bank protection as determined by the river mechanics engineer. An evaluation shall also be made regarding the
potential for the tower and associated structures to induce erosion onto adjacent property. Should the potential for such erosion occur, the tower location shall be moved to avoid this erosion, or erosion protection (such as rip rap) provided for the adjacent property. This evaluation, and associated scour/erosion protection design plans, shall be submitted to the CPUC for review and approval 60 days prior to the initiation of construction of the towers.

The report provides estimated local scour depths for the 100–year flood event for 15 drainages or washes which contain 26 proposed transmission line support structures. The estimated local scour depths are based on three equations; the results of these equations provide a range of potential maximum scour depths for the 100–year flood event (See Appendix C). Page 4-1, last paragraph, 1st sentence states that “Table 5 presents the maximum 100-year scour depths calculated based on the above methods for each of the wash crossings.” Table 5 does not provide the estimated maximum 100-year scour depths, rather Table 5 presents the average of the maximum scour depth results derived from the three methods. It is recommended that Table 5 be revised to present the scour results from all three methods to provide the range of maximum scour depths estimated for each structure. The revised Table 5 can also include the average result (as is currently provided).

The report provides no interpretation of the scour results and no guidance to the design engineer regarding how best to apply the scour analysis to structural design. For example, at RP324 estimated maximum local scour ranges from 5.5 to 17.2 feet below bed surface, with a mean of 8.0 feet. Thus, to satisfy the requirements of Mitigation Measures WQ-APM-10 and H-6a (i.e., burial depth shall be extended below the estimated 100-year depth) the design engineer will either need to (1) use the maximum value provided in the report (e.g., 17.2 feet for RP324), (2) provide a sound, engineering-based explanation as to why the average value (e.g., 8.0 feet for RP324) adequately satisfies the mitigation measures and represents the probable maximum scour depth for the 100–year flood, or (3) conduct additional analysis to derive an alternative maximum scour depth associated with the 100-year flood event.

The report does not comment on potential sediment transport conditions (including channelized mudflows) that could also destabilize structures.

While the scour analysis methodology is based on an estimation process using input streamflow (hydrology) and in-stream hydraulic parameters (including width, depth, velocity, bed size material, etc.), evaluating (or interpreting) the calculated scour results would be aided by some discussion of observed or historic channel scour in the study area. In other words, can we better understand potential channel scour in the project area from past observations? If so, how do the results presented in Table 5 compare to past observations. It is noted that the collapse of the Interstate 5 double bridge over Arroyo Pasajero, near Coalinga, CA on March 10, 1995 was due to the excessive forces of scour and mudflows which undermined the concrete piers beneath the bridges.

The report concludes that “…no adjacent properties would be impacted by local erosion induced by the towers.” However, in accordance with Mitigation Measures H-6a, the report should provide some interpretation of the lateral stability (i.e., planform alignment) of the drainages in the vicinity of the transmission lines. Review and interpretation of historical aerial photographs may aid in this analysis. The report should also provide some interpretation of the potential for contraction scour (primarily resulting from natural changes in channel geometry) at each location.

Sunrise Powerlink Project
(3) General Comments on Report Content and Quality

Section 2, Hydrologic Analysis

Page 2-1, 2nd paragraph, 1st sentence: “For estimating the peak storm event with peak sediment carrying capacity, the 100-year storm event (a storm event with a 0.1 percent chance of occurrence in any given year) was used as a design basis in accordance with the National Flood Insurance Protection (NFIP) program”

Comments:
(1) A 100-year “storm event” is not necessarily equivalent to a 100-yr flood discharge or flood event. We recommend the author refer to a 100-yr flood or discharge event. This change in terminology applies throughout the document.
(2) A 100-year “storm event” has a 1.0 percent chance of occurrence in any given year, not a 0.1 percent chance.
(3) It is not clear how or why the NFIP is applicable to this analysis.

Page 2-1, 2nd paragraph, 2nd sentence: The use of the 100-year storm event for scour is based on variability of channel hydraulics, channel material, and general complexity of the erosive process.

Comment:
(1) It is not clear from the report that the use of the 100-year flood event as the design discharge for scour analysis was based on “variability of channel hydraulics, channel material, and general complexity of the erosive process.” Rather, the selection of the 100-year flood event for scour analysis was more likely based on the requirements of mitigation measures and standard engineering practice.

Table 1, Fourth column header: 100-year flowrate
Comment:
(1) This would more accurately be described as “estimated 100-year peak discharge”.

Page 3-1, 2nd paragraph, 2nd sentence: Channel flow top widths and depths were estimated using Manning’s equation, the channel cross section at the proposed structure location, and the approximate channel slope.

Comment:
(1) This would more accurately be stated as “Manning’s equation was used to estimate flow top width and depth based on the channel geometry and slope derived from the digital topographic map.”
Comment: The author does not indicate the Manning’s n value used for the hydraulic analysis. In Appendix B it is apparent that an “n” value of 0.040 was applied for all drainages. The author should provide a statement regarding how and why this value was selected.

Section 4, Scour Analysis

Page 4-1, 1st paragraph, 1st sentence: Scour is defined as the lowering of the stream channel bed due to erosion resulting from high flow velocities during a flood event.

Comment:
(1) This statement characterizes stream degradation, but the analysis only considers local scour i.e., erosion of the bed around a pier or foundation which is the result of the structure obstructing flow. The hydraulics resulting in local scour differs from streambed degradation.

Page 4-1, 1st paragraph, 2nd sentence: “...and particle size ($D_{50}$) distribution of sediment.”

Comment:
(1) The $D_{50}$ notation is not properly used here as it is not a notation to express “particle size”, rather the mean grain size of a given sample. The sentence would be more accurate if it read “...and mean particle size $D_{50}$.”

Page 4-1, last paragraph, 1st sentence and Table 5: Table 5 presents the maximum 100-year scour depths calculated based on the above methods for each of the wash crossings.

Comment:
(1) Table 5 does not provide maximum 100-year scour depths, rather (as described above) the average maximum scour depth derived from the three methods. It is recommended that Table 5 be revised to include the range of scour depths estimated for each structure, or report the results of all three methods and the mean.

Appendix B, Hydraulic Analysis

EP90-1: 100-yr discharge reported in the hydrologic calculations (2051 cfs) does not match the Hydroflow input (1982 cfs).

EP313: 100-yr discharge reported in the hydrologic calculations (1364 cfs) does not match the Hydroflow input (1406 cfs).

EP323-1 & EP324: 100-yr discharge reported in the hydrologic calculations (38,784 cfs) does not match the Hydroflow input (38,791 cfs).
Technical Memorandum

Date: August 16, 2010

To: Molly Frisbie, PE, San Diego Gas & Electric Company

From: Matt Moore and Tom Grace, URS Corporation

Subject: Sunrise Powerlink Scour Analysis – Response to Comments

URS reviewed the Horizon Water and Environment comments on the Sunrise Powerlink Scour Analysis Report. Comment responses are provided below. The format includes the original comment along with the URS response.

Conformance with Mitigation Measures

Comment (1):
The report provides estimated local scour depths for the 100–year flood event for 15 drainages or washes which contain 26 proposed transmission line support structures. The estimated local scour depths are based on three equations; the results of these equations provide a range of potential maximum scour depths for the 100–year flood event (See Appendix C). Page 4-1, last paragraph, 1st sentence states that “Table 5 presents the maximum 100-year scour depths calculated based on the above methods for each of the wash crossings.” Table 5 does not provide the estimated maximum 100-year scour depths, rather Table 5 presents the average of the maximum scour depth results derived from the three methods. It is recommended that Table 5 be revised to present the scour results from all three methods to provide the range of maximum scour depths estimated for each structure. The revised Table 5 can also include the average result (as is currently provided).

Response to Comment (1):
URS agrees that Table 5 provides the average scour depth from the three methods as provided in Appendix C of the report. Page 4-1, last paragraph, 1st sentence would more accurately read “Table 5 presents the average scour depths calculated based on the above methods for each of the wash crossings.” See discussion below for the recommended scour depths for design purposes.

Comment (2):
The report provides no interpretation of the scour results and no guidance to the design engineer regarding how best to apply the scour analysis to structural design. For example, at RP324 estimated maximum local scour ranges from 5.5 to 17.2 feet below bed surface, with a mean of 8.0 feet. Thus, to satisfy the requirements of Mitigation Measures WQ-APM-10 and H-6a (i.e., burial depth shall be extended below the estimated 100-year depth) the design engineer will either need to (1) use the maximum value provided in the report (e.g., 17.2 feet for RP324), (2) provide a sound, engineering-based explanation as to why the average value (e.g., 8.0 feet for RP324) adequately satisfies the mitigation measures and represents the probable maximum scour depth for the 100–year flood, or (3) conduct additional analysis to derive an alternative maximum scour depth associated with the 100-year flood event.

Response to Comment (2):
URS recommends use of the average scour depth from the three methods for structural design in most cases with the exception where one scour estimation method provides a large discrepancy with two of the other methods (thereby providing a large skew in the average scour calculation). The discussion below provides justification of the use of engineering judgment in the selection of recommended scour depths.
The United States Bureau of Reclamation (USBR) document “Computing Degradation and Local Scour,” dated January 1984 provides guidance for estimating local scour for mid-channel structures. There are numerous equations/methods for calculating local scour for structures in washes/drainage channels. The procedures in the USBR document recommend calculating scour using at least two techniques and apply engineering judgment in selecting an average or most reliable method.

The approach selected for this scour analysis was to utilize two regime-based approaches (Lacey and Blench) and one rational equation (Jain). In many cases, an average of the three methods was used to determine the recommended design scour depth. All empirical equations produce estimates of scour depths based upon varying input parameters and assumptions. Note that the Blench equation and Jain equation scour results (regime and rational equations, respectively) are generally closely matched, while the Lacey equation (regime equation) produces values typically higher than the other two equations. Use of the Blench and Jain equations would typically be appropriate to determine the probable maximum local scour depth. However, the Lacey equation was utilized as an additional equation to provide a higher level of certainty in the estimated scour depths, where appropriate. In some cases the Lacey Equation provided scour estimates well in excess of the results of the Blench and Jain equation. In those cases the scour analysis results of the Lacey Equation, in addition to those of the Blench and Jain equations, were taken under engineering consideration in determination of the final recommended scour depth (for example Structure IDs EP295/296, EP 299/300-1, EP 323-1/324). In our opinion, the approach utilized provides a sound engineering estimate of the probable maximum scour for the 100-year peak flood discharge.

Comment (3):
The report does not comment on potential sediment transport conditions (including channelized mudflows) that could also destabilize structures.

Response to Comment (3):
The scour analysis was based upon the estimated 100-year peak flood discharges. These peak discharges were estimated utilizing the USGS regression equations and/or FEMA flood discharge/frequency tables and charts. Typically the USGS regression tables and FEMA flood frequency/discharge charts/tables are based upon stream gage data. Mudflow/debris flow conditions have the potential to increase the total flow due to bulking caused by additional sediment and debris in the flood discharge. However, the estimated peak 100-year flood discharges are based upon documented regression equations and/or FEMA discharge/frequency tables/charts so they account for natural bulking. The scour analysis does not account for a particular situation for mudflows on lateral loading and/or specific debris impact to a particular structure.

Comment (4):
While the scour analysis methodology is based on an estimation process using input streamflow (hydrology) and in-stream hydraulic parameters (including width, depth, velocity, bed size material, etc.), evaluating (or interpreting) the calculated scour results would be aided by some discussion of observed or historic channel scour in the study area. In other words, can we better understand potential channel scour in the project area from past observations? If so, how do the results presented in Table 5 compare to past observations. It is noted that the collapse of the Interstate 5 double bridge over Arroyo Pasajero, near Coalinga, CA on March 10, 1995 was due to the excessive forces of scour and mudflows which undermined the concrete piers beneath the bridges.

Response to Comment (4):
Historical scour/flooding analysis was not included in the scope of the current study. Rather, the scour analysis was performed per site specific conditions utilizing standard scour analysis equations. The recommended scour
depth is based upon the average local scour depth at each location utilizing several scour prediction equations. Scour of man-made structures in the project area is not well documented and in our opinion, the theoretical evaluation performed is the most appropriate methodology.

Comment (5):
The report concludes that “...no adjacent properties would be impacted by local erosion induced by the towers.” However, in accordance with Mitigation Measures H-6a, the report should provide some interpretation of the lateral stability (i.e., planform alignment) of the drainages in the vicinity of the transmission lines. Review and interpretation of historical aerial photographs may aid in this analysis. The report should also provide some interpretation of the potential for contraction scour (primarily resulting from natural changes in channel geometry) at each location.

Response to Comment (5):
Scour calculations were determined with the use of the maximum depth in the channel section generally perpendicular (in line with the floodplain) at the proposed structures, so any lateral migration of the main channel is implicitly considered in the evaluation (assuming approximately the same channel depth after channel migration). Contraction scour calculations are typically applied at bridge constriction locations and were not applied in this case. Scour calculations assume a single pier condition with no contraction scour component.

General Comments on Report Content and Quality

Section 2, Hydrologic Analysis

Page 2-1, 2nd paragraph, 1st sentence: “For estimating the peak storm event with peak sediment carrying capacity, the 100-year storm event (a storm event with a 0.1 percent chance of occurrence in any given year) was used as a design basis in accordance with the National Flood Insurance Protection (NFIP) program”

Comment (6):
A 100-year “storm event” is not necessarily equivalent to a 100-yr flood discharge or flood event. We recommend the author refer to a 100-yr flood or discharge event. This change in terminology applies throughout the document.

Response to Comment (6):
100-year flood (1-percent annual chance of occurrence) peak discharges were used in the analysis. It is understood that a 100-year rainfall storm event does not necessarily equate to a 100-year peak discharge. URS accepts the terminology change from “100-year storm event” to "100-year peak discharge event."

Comment (7):
A 100-year "storm event" has a 1.0 percent chance of occurrence in any given year, not a 0.1 percent chance.

Response to Comment (7):
URS agrees with the comment. Scour calculations were based upon the estimated 100-year peak discharge (100-year flood event or 1-percent chance of occurrence in any given year).

Comment (8):
It is not clear how or why the NFIP is applicable to this analysis.
Federal Emergency Management Agency/National Flood Insurance Program (FEMA/NFIP) Flood Insurance Study documentation was utilized for estimates of flood discharges where available. In areas where FEMA/NFIP estimates or information were not available the USGS regression equations were utilized to estimate peak flood event discharges.

Page 2-1, 2nd paragraph, 2nd sentence: The use of the 100-year storm event for scour is based on variability of channel hydraulics, channel material, and general complexity of the erosive process.

Comment (9):  
It is not clear from the report that the use of the 100-year flood event as the design discharge for scour analysis was based on “variability of channel hydraulics, channel material, and general complexity of the erosive process.” Rather, the selection of the 100-year flood event for scour analysis was more likely based on the requirements of mitigation measures and standard engineering practice.

Response to Comment (9):  
Use of the 100-year peak flood discharge was based upon standard engineering practice and recommendations provided in the United States Bureau of Reclamation (USBR) document “Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation,” January 1984, pages 31 and 32. The sentence in question was based on the USBR document, as excerpted below:

“The first step in local scour study for design of a structure is selection of design flood frequency. Reclamation criteria for design of most structures shown in Table 6 (shown below) varies from a design flood estimated on a frequency basis from 50 to 100 years. This pertains to an adequate waterway for passage of the floodflow peak. The scour calculations for these same structures are always made for a 100-year flood peak. The use of the 100-year flood peak for scour is based on variability of channel hydraulics, bed material, and general complexity of the erosive process.”

The sentence in question could be more accurately stated as: “For estimating the peak storm event with peak sediment carrying capacity, the estimated 100-year peak flood discharge (a storm event discharge with a 1-percent chance of occurrence in any given year) was used as a design basis in accordance with standard engineering practice and as referenced in the United States Bureau of Reclamation document ‘Computing Degradation and Local Scour,’ dated January 1984 (page 32), upon which the scour analysis methodology and approach is based.”
Table 6. Classification of scour equation for various structure designs

<table>
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<tr>
<th>Equation type</th>
<th>Scour</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Natural channel for restrictions and bends</td>
<td>Siphon crossing or any buried pipeline. Stability study of a natural bank. Waterway for one-span bridge.</td>
</tr>
<tr>
<td>B</td>
<td>Bankline structures</td>
<td>Abatments to bridge or siphon crossing. Bank slope protection such as riprap, etc. Spur dikes, groins, etc. Pumping plants. Canal headworks.</td>
</tr>
</tbody>
</table>

Table 1, Fourth column header: 100-year flowrate

Comment (10):
This would more accurately be described as “estimated 100-year peak discharge”.

Response to Comment (10):
URS agrees that this can be described as the estimated 100-year peak discharge and updated in the report.

Page 3-1, 2nd paragraph, 2nd sentence: Channel flow top widths and depths were estimated using Manning’s equation, the channel cross section at the proposed structure location, and the approximate channel slope.

Comment (11):
This would more accurately be stated as “Manning’s equation was used to estimate flow top width and depth based on the channel geometry and slope derived from the digital topographic map.”

Response to Comment (11):
URS agrees that this can be stated as “Manning’s equation was used to estimate flow top width and depth based on the channel geometry and slope derived from the digital topographic map.”

Comment (12):
The author does not indicate the Manning’s n value used for the hydraulic analysis. In Appendix B it is apparent that an “n” value of 0.040 was applied for all drainages. The author should provide a statement regarding how and why this value was selected.
Response to Comment (12):
The Manning’s roughness coefficient used for the analysis of calculating estimated depths and velocities was 0.04. Manning’s roughness coefficients are dependent upon site conditions and are estimated based on review of aerial photography and field conditions. Values for the site locations could range from a minimum of 0.025 to 0.04 (based upon selection of Manning’s ‘n’ values in Table 5-6 of ‘Open Channel Hydraulics’, Chow, 1959). A Manning’s ‘n’ value of 0.04 was chosen to provide a conservative scour estimate because the equations utilized to estimate the scour are largely based upon the flow depth in the channel, with larger flow depths providing greater scour.

Section 4, Scour Analysis

Page 4-1, 1st paragraph, 1st sentence: Scour is defined as the lowering of the stream channel bed due to erosion resulting from high flow velocities during a flood event.

Comment (13):
This statement characterizes stream degradation, but the analysis only considers local scour i.e., erosion of the bed around a pier or foundation which is the result of the structure obstructing flow. The hydraulics resulting in local scour differs from streambed degradation.

Response to Comment (13):
The scour analysis was based on local scour conditions at the proposed structure location (Equation Type C per the USBR “Computing Degradation and Local Scour” report, page 40 – see Table 6 from the USBR document above under Comment Response 9).

Page 4-1, 2nd sentence: “...and particle size (D_{50}) distribution of sediment.”

Comment (14):
The D_{50} notation is not properly used here as it is not a notation to express “particle size”, rather the mean grain size of a given sample. The sentence would be more accurate if it read “...and mean particle size D_{50}.”

Response to Comment (14):
URS agrees with this comment.

Page 4-1, last paragraph, 1st sentence and Table 5: Table 5 presents the maximum 100-year scour depths calculated based on the above methods for each of the wash crossings.

Comment (15):
Table 5 does not provide maximum 100-year scour depths, rather (as described above) the average maximum scour depth derived from the three methods. It is recommended that Table 5 be revised to include the range of scour depths estimated for each structure, or report the results of all three methods and the mean.

Response to Comment (15):
URS agrees that Table 5 provides the average maximum scour depth and not the overall maximum scour depth. See responses to comments 1 and 2 for further discussion.
Appendix B. Hydraulic Analysis

General response to the following comments indicates that while there were minor discrepancies in the Hydroflow input/output and the scour calculations, the recommended design scour depth will not be revised significantly.

Comment (16):
EP90-1: 100-yr discharge reported in the hydrologic calculations (2051 cfs) does not match the Hydroflow input (1982 cfs).

Response to Comment (16):
It has been verified that 2,051 cfs is correct.

Comment (17):
EP313: 100-yr discharge reported in the hydrologic calculations (1364 cfs) does not match the Hydroflow input (1406 cfs).

Response to Comment (17):
It has been verified that 1,364 cfs is correct.

Comment (18):
EP323-1 & EP324: 100-yr discharge reported in the hydrologic calculations (38,784 cfs) does not match the Hydroflow input (38,791 cfs).

Response to Comment (18):
It has been verified that 38,784 cfs is correct.