APPENDIX B
HEC-RAS SEDIMENT TRANSPORT MODELING
Santa Ana River

Submitted To: Orange County Water District
18700 Ward Street
Fountain Valley, California 92708

Submitted By: Golder Associates Inc.
44 Union Boulevard, Suite 300
Lakewood, Colorado 80228

Distribution: HDR Engineering
Orange County Water District

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1.0 INTRODUCTION

Orange County Water District (OCWD), through HDR Engineering, Inc. (HDR), commissioned Golder Associates (Golder) to perform sediment transport modeling for the Prado Dam Sediment Management Demonstration Project Engineering Analysis. A one-dimensional sediment transport model was developed to address the following primary questions related to reaching the primary project goals:

1. What is the anticipated spatial and temporal distribution of deposited, sand-sized replenishment materials in the Lower Santa Ana River (LSAR)?
2. Could the proposed project replenish beach sand?
3. How does the proposed project affect riparian habitat in the LSAR?
4. Could the proposed sediment augmentation project change the gradation of the LSAR bed material in the groundwater recharge reach?

Issues of secondary interest, to be evaluated by the sediment transport modeling, include the following:

1. Do the silt- and clay-sized sediments move through the LSAR?
2. Could the proposed project increase flooding potential, particularly downstream of I-405?
3. Would the project result in increased maintenance requirements at diversion structures?
4. Could the proposed project lessen the effects of channel degradation at Featherly Park?
5. Could the project result in increased scour potential at the levees in the LSAR?
6. What are the measurable effects at critical structures within the LSAR?
7. Could the proposed project result in increased river degradation at the Santa Ana River Interceptor (SARI)?

This report appendix is a revision to the appendix submitted by Golder in February 2011 (Golder 2011). This revision is required due to channel improvements made in the upstream reach of the LSAR. A new HEC-RAS geometry file was created to reflect the channel improvements and provided to Golder in April 2014.

Due to the lack of measured sediment discharge and geometric data over time, the model cannot be fully calibrated to historic conditions. The model was calibrated to the extent practical based on observations during a geomorphic assessment (Attachment A) and information contained in previous reports. However, the model represents what Golder expects will happen in the river during the scenarios modeled and the model can be used to identify areas of potential risk that can be monitored during the demonstration project. The model can be calibrated to observed conditions during sediment re-entrainment at a later date.
2.0 METHOD AND ASSUMPTIONS
The US Army Corps of Engineers (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.1 released January 2010 was chosen to aid in this analysis. The study used both the hydraulic and the sediment transport modules within this software.

The basic requirements for hydraulic calculations in HEC-RAS are channel cross sectional geometry, Manning’s n-values for the cross sections, riverbank locations, and distances between the cross sections, flow information, and flow boundary information. Additional information is required to model bridges and weirs. The HEC-RAS sediment transport module calculates transport capacity for non-cohesive and cohesive soils using hydraulic variables (velocity, flow depth, and shear stress) and sediment properties.

2.1 Geometry
Two geometries have been modeled for this project. The original input geometry used in the 60% design report submitted in February 2011 was obtained from the USACE (Golder 2011). This geometry was broken into two reaches that the USACE developed into two HEC-RAS models to study the effects of different flood events. One model incorporated the reach from Prado Dam downstream to Weir Canyon Road, and the second model continued from Weir Canyon Road to the Pacific Ocean. Golder combined the two existing model reaches to create a continuous model from Prado Dam to the Pacific Ocean. The second geometry was updated due to construction in Reach 9.

Due to construction of channel improvements in Reach 9 downstream of Prado Dam, the geometry was updated to reflect the changes in the upper reach of the HEC-RAS model. Golder obtained the updated geometry in April 2014 and updated the modeling using the new geometry.

Both geometries evaluated were similar. Over the entire reach, 37 bridges are present, ranging from small pedestrian, bicycle, and railroad bridges to multi-lane major freeway bridges. Based on available aerial imagery, the geometry of the existing USACE models appear accurate, except for three structures. The project team added two inflatable rubber dam structures built in 1993, as well as a small grade control structure within the Riverview Golf Course.

The channel generally has three distinct geomorphic reaches shown in Figure B-1 and Table B-1. The first geomorphic reach commences near Prado Dam and ends at North Weir Canyon Road. It is a natural channel with braided and meandering patterns, and a relatively steep longitudinal slope when compared to the remainder of the LSAR. The overbanks are fully vegetated, indicating possible vegetation encroachment after the dam was constructed. Manning’s n values for this reach vary from 0.034 to 0.040 for the main channel and from 0.05 to 0.070 for the overbanks in this reach.
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Table B-1  River Station Boundaries of the Three Geomorphic Reaches

<table>
<thead>
<tr>
<th>Geomorphic Reach</th>
<th>River Station Boundaries</th>
</tr>
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<tr>
<td>Upper Reach</td>
<td>162302-120325</td>
</tr>
<tr>
<td>Recharge Reach</td>
<td>120325-59930</td>
</tr>
<tr>
<td>Lower Reach</td>
<td>59930-760</td>
</tr>
</tbody>
</table>

Figure B-1  Location Map
The second geomorphic unit is the groundwater recharge reach, extending from North Weir Canyon Road to the Garden Grove Freeway. This section has a natural bed, but its banks have been significantly modified. The river in this reach is contained within a regular, trapezoidal channel with a bottom-width of about 325 feet. Several drop and grade-control structures were constructed in this reach to maintain the river slope and prevent undue degradation. Temporary T&L, named for the shape of the levees resembling the letters "T" and "L", and "racetrack" levees constructed by OCWD to enhance infiltration of water into the riverbed (Figure B-2) are also present in the recharge reach. These temporary structures are designed to slow down low flows and allow more infiltration. They are washed away during high flows and were therefore not incorporated in the HEC-RAS model.

Flow diversion structures located in this river reach divert flows to adjacent infiltration ponds. Two of these structures are inflatable rubber dams that were built in 1993 near stations 103085 and 85000. They are approximately 7’ high when inflated. These dams are fully inflated during the storm season when flows are less than 500 cfs. When flows are between 500 and 2,000 cfs they will be partially inflated, and when flows exceed 2,000 cfs they are fully deflated removing the obstruction to the flow in the river. However, water is still allowed to be diverted through open gates. The HEC-RAS model geometry is broken at these inflatable dams to allow for a rating curve to be used to simulate the operation of the dams to properly analyze the sediment transport at these locations. Infiltration and other small diversion points are modeled throughout this reach as well.

The Carbon Canyon Diversion Channel flows into the Santa Ana River in this reach, just downstream of North Glassell St. Currently the flow magnitudes corresponding to the modeled outflows from Prado Dam from this channel are unknown. Other small inflow points along this reach convey flows back to the river from the adjacent infiltration ponds. Numerous storm drains are present along the lower stretches of this reach. Again, the flow contributions corresponding to the modeled flows are unknown. The Manning’s n value is set at 0.030 for the majority of the sections and 0.050 for sections representing the drop structures.
Figure B-2  Example of T&L Levees
The final geomorphic reach is located between the downstream end of the recharge reach (downstream of the State Highway 22) and the Pacific Ocean. This reach has a mild, longitudinal slope and much of the channel geometry has been modified. From just upstream of 17th Street to just upstream of I-405, the channel has a trapezoidal shape with concrete lined sides and bottom. Downstream of this point to just upstream of Adams Street the channel changes to a rectangular channel with concrete lined sides and bottom. Downstream of this point to the PCH the channel takes on a natural soft bottom with concrete lined levees. The extreme downstream reach of the Santa Ana River between the PCH and the Pacific Ocean has natural sides and bottom and is prone to periodic sediment deposition.

Santiago Creek enters the Santa Ana River at the beginning of this reach, and one more major ditch discharges water into the river near the end of the reach, just downstream of the Victoria St. Bridge. The reach has one grade control structure that was not present in the original USACE version of the model. The model used in this study contains this structure. The exact geometry of the grade control structures is not known at this time. Therefore, its geometry was assumed. It has been located near the beginning of this reach just downstream of the Santiago Creek confluence. The Manning’s n value for the channel bed is set at 0.030. At the drop structures, it is set at 0.050. The Manning’s n value for the concrete-lined section was set at 0.014.

2.2 Hydrology

The flow from Prado Dam, controlled by USACE, is the only inflow hydrology used in the HEC-RAS model. Inflows from the catchment downstream of Prado Dam have not been included in the model at this point. Tetra Tech and HDR (2010) indicate that the watershed area contributing to the Santa Ana River downstream of the dam is two orders of magnitude smaller than the watershed area upstream of Prado Dam. Neglecting flows from the downstream part of the catchment is therefore deemed reasonable.

Historical daily flow data from Prado Dam is available from 1980 through the beginning of 2010. These flow records were analyzed to establish five pulse flow scenarios. The pulse flows will be used to model sediment re-entrainment downstream of the dam to augment sediment loads in the SAR. These sediments will be dredged from the Prado Basin and will be placed just downstream of the dam prior to re-entrainment. It is assumed for the purpose of this report that when the pulse flows are released from the reservoir the augmentation sediment will be introduced into the river for conveyance downstream. The pulse flow magnitudes that were selected are 500, 750, 1250, 2,000 and 5,000 cfs. They will be released under controlled conditions over periods of several days until all the augmentation sediment has been re-entrained. In order to determine the fate of the augmented sediment after it has been re-entrained into the river by the pulse flows it was necessary to develop representative flow records in the river for use after the augmentation period simulation.
The historical daily flow data were therefore further analyzed to identify water years representing 75% (Wet), 25% (Dry), and 50% (Median) exceedance water years from October 1 to September 30. The flow records successively representing these conditions are the water years 2002/2003 (wet), 1987/88 (dry), and 1999/2000 (median). The corresponding flow records are shown in Figure B-3 through Figure B-5 below. These flows were added after the pulse flows to create one year of historic daily flow data for use in the simulations. Each of the flow series covers a period of a little over one year, commencing with a pulse flow followed by the full year of average annual daily flow series. The flow and sediment time series used in the various HEC-RAS simulations are contained in Figures B.B.16 to B.B.57, attached to this appendix.

**Figure B-3**  Wet Year Flow Hydrograph
Figure B-4  Dry Year Flow Hydrograph

Figure B-5  Median Year Flow Hydrograph
Operating procedures for the inflatable dams previously mentioned leads to the diversion of all flow in the river when the flow from Prado Dam is less than 500 cfs. As shown in Figure B-3 through Figure B-5 there are very few times throughout the year where there will be flow in the river below the recharge reach in all three flow scenarios modeled.

### 2.3 Sediment

The sediment file provides the model with sediment data required for execution of the sediment transport simulations. It contains the properties of both bed material and incoming sediment loads. Sediment grain sizes, grain size distributions, sediment mass density, porosity of deposited sediment, bed material properties, and sediment loads flowing into the river are contained in the input files. For this study, the natural sediment load and sediment augmentation load were expressed in tons per day. The magnitude of the natural sediment load into the system was specified using a rating curve relating water flow and sediment load. The augmentation load was specified as a percent concentration in the flowing water.

Cohesive soils are simulated using either non-cohesive sediment transport assumptions (i.e., using the same equations to estimate cohesive sediment transport as are used to estimate non-cohesive sediment transport) or by making use of equations developed by Krone (1962) and Parthanadias (1962), which specifically addresses cohesive sediment erosion, deposition and transport properties. When using the cohesive sediment transport equations by Krone and Parthanadias it is usually necessary to perform specialized testing. In this study, it was assumed that the cohesive sediments can be simulated using non-cohesive sediment transport equations. It is noted that this approach may significantly overestimate the transport of cohesive sediments.

Several equations are available in HEC-RAS to calculate sediment transport, bed sorting, and fall velocity. The equation used in the model is selected on the sediment input screen. Bed sorting calculations are required to simulate how the bed material composition may change over time. For example, if the bed material becomes coarser due to the preferential removal of fine material it may become armored. Once armored it protects the sub-surface sediments and limits the amount of sediment that can be removed from the riverbed for transportation. The fall velocity of the sediment is required to determine its mobility, i.e., how easy is it to settle the material and to re-entrain it once settled on the riverbed.

For this analysis the Engelund-Hansen sediment transport function, the Exner sorting equation, and the Rubey fall velocity equation were selected. The Engelund and Hansen (1967) total load equation was selected because of its simplicity and suitability to calculate sediment transport in sandy river conditions. The equation is a stream-power based relationship using commonly available parameters (e.g., grain size, flow velocity and bed shear stress). It has been identified as one of the most accurate total load sediment transport equations for a wide range of non-cohesive sand sizes (HEC-RAS 2010). Annandale
(2007) showed that the Engelund and Hansen (1967) equation responds appropriately as a total load predictor to changes in total turbulence as opposed to only responding to changes in near-bed turbulence (bed load) or only to turbulence in the water column above the near-bed region (suspended load).

The Exner sorting equation is effective in predicting armoring of the bed material. For this reason, it was selected for the Santa Ana River where armoring has already been found to occur in some river reaches. The Rubey fall velocity method is suitable for silt, sand, and gravel sizes with specific gravities of around 2.65, similar to materials found in the Santa Ana River.

One of the limitations of the HEC-RAS sediment transport modeling software is that specification of only one sediment gradation per cross section is allowed. This means for example that differing gradations cannot be specified for the surface layer and the sub-surface layer at one particular cross-section. This is an important observation because the sediment properties on the surface and sub-surface often differ especially if the channel has armored.

2.4 Quasi-Unsteady Flow

The quasi-unsteady flow file is used to define a flow hydrograph for the HEC-RAS model. The flows are entered as a flow series for a specified duration and are the upstream model boundary condition. A 24-hour duration was used to represent the daily flows. This means that the daily discharges were held constant for periods of 24 hours. Therefore, one year of daily flow data consists of 365 (or 366 in a leap year) different daily flow values.

The computational increment is specified as part of the flow series. This increment determines how often calculations are performed. For example, a computational increment of one hour means that a computation is carried out every simulated hour. Therefore, the calculations are repeated 24 times for every simulated day if the computational increment is one hour. The model is extremely sensitive to this parameter. Long increments can result in model instability, while very short increments may result in more stable models but require very long run times. This parameter was used in model calibration, which is discussed in the next section.

The tide elevation at the Santa Ana River/Pacific Ocean interface was set as the downstream boundary condition. The temperature of the river water must be specified for calculating sediment particle fall velocities. A constant water temperature of 55 degrees Fahrenheit was used for all simulations.
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3.0 MODEL CALIBRATION

Due to the lack of measured sediment discharge and geometric data over time, the model cannot be fully calibrated to historic conditions. The model was calibrated to the extent practical based on observations during a geomorphic assessment (Attachment A) and information contained in previous reports. However, the model represents what Golder expects will happen in the river during the scenarios modeled and the model can be used to identify areas of potential risk that can be monitored during the demonstration project. The model can be calibrated to observed conditions during sediment re-entrainment at a later date.

Before baseline and predictive analyses can be performed, the HEC-RAS model must be calibrated for existing conditions to ensure the model is capable of predicting natural conditions in the river. For this calibration many different scenarios were modeled, combining different computational increments, flows, tidal influences, sediment concentration, incoming sediment gradation, geometry specifications, bed material gradations, allowable depths of scour, and boundary conditions. The project team developed approximately 200 models before developing a calibrated simulation. It should be noted that calibration was conducted on the model before improvements to Reach 9 were made. The new geometry obtained in April 2014 did not affect model calibration results.

Mathematical models that are used to forecast morphologic behavior of rivers are based on numerous assumptions. A systematic approach to model development and calibration is therefore required. The project team employed a three level approach to address these needs, consisting of a geomorphic assessment (Attachment A), basic hydraulic and sediment analyses, and detailed sediment transport modeling supported by a sensitivity analysis. In honoring this approach, the HEC-RAS sediment model development for this project commenced with a field trip to investigate river characteristics. This was followed by basic hydraulic and sediment transport analyses, which was accompanied by systematic model development.

The criterion used to calibrate the HEC-RAS model is based on the Santa Ana River morphology between Prado Dam and the Pacific Ocean. Aerial imagery, field observation, channel geometry and bed material data was used to assess its morphologic stability, which indicated a "quasi-equilibrium" condition. This means that the river experiences only minor morphologic channel changes.

The objective function thus established reflects the present degree of stability of the river. This means that the model parameters that were finally selected were those that reasonably produce a model representing a quasi-equilibrium condition.
3.1 Computational Increment Sensitivity Analysis
As previously stated, model stability is very sensitive to the computational increment. For the calibration runs, computational increments ranged from 15 minutes to 24 hours. It was determined that a 30-minute computational increment provided the best balance between model stability and run time.

3.2 Flow Analysis
After analyzing the historical flow data, a hydrologic year from March 23, 1980 to March 22, 1981 was chosen for the calibration runs. This year was selected because it contains higher than average flows for approximately the first month of the hydrologic year, which is useful for introducing sediment to the system. This flow series was only used in the model calibration. As stated previously, there is not enough data over time to perform a typical calibration using long-term water flow, sediment flow, and geometric data. This flow series is used to ensure the model represents the quasi-equilibrium conditions observed in the river. As discussed in the Hydrology section above, statistical analysis was performed to determine a wet, dry, and median water year to be used in the base and predictive case model runs that are discussed in Sections 4.0 and 5.0 below.

3.3 Tidal Influences
Multiple tide elevations were simulated to determine its effect on erosion and deposition in the lower reaches of the Santa Ana River. Stages of 0, 2, 4, and 8 feet above mean sea level were analyzed. The sensitivity analysis revealed that tidal elevations indicate that the tides have a significant impact on sediment deposition in the most downstream reach of the river. However, when viewed in terms of the total amount of sediment transported through the system it is deemed to have fairly minor effects. The difference in sediment deposition over this tidal range is about 500 tons (Figure B-10) which is a very small percentage of the total re-entrained sediment load. This finding led to the selection of a constant tide level of 0 feet. This was used in both the baseline and predictive analyses.

3.4 Sediment Load
The natural sediment load in the river was calculated using the USACE rating curve from Figure 4.1 in the GDM (USACE 1988). The relevant equation is: \( Q_s = 0.003Q_w^{1.42} \). Select paired water and sediment flows are shown below in Table B-2.
Table B-2  Paired Water and Sediment Flows

<table>
<thead>
<tr>
<th>Qw (cfs)</th>
<th>Qs (tons/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>2.1</td>
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<tr>
<td>250</td>
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<td>500</td>
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<td>54.6</td>
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<tr>
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<td>146.1</td>
</tr>
<tr>
<td>5,000</td>
<td>536.6</td>
</tr>
</tbody>
</table>

The rate by which augmented sediment can be introduced into the river was estimated by conducting a sensitivity analysis. Concentrations of augmented sediment introduced into the river were analyzed for 0.25%, 0.5%, 0.75%, and 1%. No significant difference in the sediment transport characteristics was observed. It was therefore decided to assume that the augmented sediment can be introduced into the Santa Ana River at a concentration of 1%. This allowed calculation of the amount of pulse flow water needed to re-entrain 500,000 yd$^3$ of sediment into the Santa Ana River.

### 3.5 Incoming Sediment Gradation

An incoming sediment gradation was calculated by taking the average of the two samples collected for each of the three boreholes (B3, B12, and B16) for a total of six samples taken from Prado Basin in March 2010. The percent passing for each size fraction was averaged across the six samples to develop the incoming sediment gradation. This gradation is classified as fine sand and is assumed to be the material that will be dredged from the Prado Basin and re-entrained to the Santa Ana River downstream of Prado Dam. Figure B-6 shows the assumed gradation for the calibration, natural, and augmented sediment load. It is recommended that additional samples are taken in the recommended dredging area using continuous sampling techniques to obtain a better representation of sediment that will be dredged and re-entrained downstream.
Figure B-6  Incoming Sediment Load Gradation
3.6 Geometry Specifications

The model calibration required sensitivity analyses related to model geometry. The geometric sensitivity analysis considered interpolated cross sections and development of sediment pass-through nodes. The models originally developed by the USACE and obtained in April 2014 used several interpolated cross sections throughout the length of the study reach. For steady flow computations (without sediment transport), this can increase model stability. However, for sediment transport computations major instabilities can occur when multiple interpolated cross-sections are used in reaches with constant slopes and cross sectional geometry. Therefore, most of the interpolated sections were removed from the model.

Pass-through nodes can be valuable when analyzing sediment transport systems, because they prevent sediment from depositing or eroding. During model calibration, several locations were experiencing large deposition rates that did not reflect conditions observed in the field. In some cases, these deposition areas led to model run failure. As such, pass-through nodes were added to reflect observed quasi-equilibrium conditions. Locations where implementation of this approach was necessary usually included areas near bridges or drop structures. Several pass-through nodes were used in the final calibration of the model and remain in the base case and predictive assessments.

3.7 Bed Material Gradations

Several sediment samples in the Santa Ana River below Prado Dam were collected in 2008. These samples and the resulting gradations were used in the calibration analysis. It is noted that the HEC-RAS model allows specification of only one gradation per cross section. It was therefore necessary to select one sample at cross sections where more than one gradation was available. In such cases, to be consistent, Golder selected the sample gradation that was taken in the thalweg (i.e., the deepest part of the river channel). Table B-3 relates sample points and the corresponding cross sections (also see Figure B-1). The gradations of the sediment collected at each sample point are presented in Attachment B.
### Table B-3  Sample Point Number and River Station

<table>
<thead>
<tr>
<th>Sample Point</th>
<th>River Station</th>
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<tbody>
<tr>
<td>SS10R</td>
<td>152780</td>
</tr>
<tr>
<td>SS9R</td>
<td>147873.3</td>
</tr>
<tr>
<td>SS5RSur</td>
<td>139329.3</td>
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<td>SS4RSur</td>
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</tr>
<tr>
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</tr>
</tbody>
</table>

### 3.8 Allowable Depths of Scour and Erodible Bed Limits

HEC-RAS allows a set of maximum scour depths and erodible bed limits (horizontal extents of allowable scour) to be input that may be applied during any one simulation. This can be used as a calibration factor to ensure that the simulated bed scour depths resemble observed scour. A sensitivity analysis was conducted for the SAR model to determine the value of the maximum allowable simulated scour depth for each simulation. The scour depth limits that were tested ranges from 0 to 6 feet. The bank stations were used as boundaries for the erodible bed limits.

In all the sensitivity runs conducted, the maximum selected scour depth was achieved at many locations within a short amount of time. This implies that the model calculates scour depths of up to 6 feet within a year. Field observations reveal that the river has not historically degraded at a rate of 6 feet per year in any location, which indicates that the simulated scour depths from these sensitivity model runs do not represent actual field observations.

The Tetra Tech and HDR (2010) scour study of the SARI indicates an average observed scour depth of 6.7 feet over 30 years, implying an average annual rate of about 0.25 feet per year. Golder therefore used this value, i.e., 0.25 feet per year, as the maximum allowable scour depth that could be applied at any location for the simulations conducted (duration = 1 year). No scour was allowed in the concrete lined sections or in the sections representing drop structures. It is understood that “forcing” the model to a maximum allowable depth of scour does not necessarily represent model calibration, however, as previously mentioned, the goal of the calibration is to replicate the quasi-equilibrium condition observed in...
the field. The bank stations remain as the erodible bed limits throughout the base case and predictive model runs.

3.9 Calibration Summary

The sediment model was calibrated to the extent possible using available information taking the following into account:

- The flow data used for calibration is represented by flows for the hydrologic year 1980.
- The computational increment was set at 30 minutes, balancing the need for model stability and run-time requirements.
- Many interpolated cross-sections were removed from the original USACE models to improve model stability for sediment transport simulations.
- Based on the sensitivity analysis it was decided to set the average tide elevation at the Pacific Ocean at 0 feet.
- The cross sections along channelized portions of the river channel are trapezoidal with flat beds. Therefore, no minor channels, bars, islands, or other geomorphic features are included in those reaches.
- The temporary T&L levees and “racetrack” levees in the recharge reach, constructed of sand, were not simulated. These features fail rapidly during large flow events and are not considered to affect sediment discharge.
- Hydraulic structures such as bridges and drop structures were defined as pass-through features. This means that sediment was not allowed to deposit at these structures. If this was not done the Exner equation would have incorrectly calculated unrealistic values of scour or deposition that has not been observed.
- Based on the history of the Santa Ana River since construction of Prado Dam, the scour limit was set at 0.25 feet. This means that the maximum calculated scour allowed at any cross-section is 0.25 feet for the entire simulation period. Erodible bed limits were set using bank stations from the model.
- The natural sediment load was calculated using a sediment-rating curve developed by the USACE for the SAR.
- The gradation of sediments originating from Prado Dam was determined using the samples collected by Golder from three boreholes in the Prado Basin during March 2010.
- The bed-material sediment gradations specified at each section represent sediment properties of the bed in the thalweg of the river (i.e., the lowest cross-sectional elevation).
- The magnitude of the augmentation load and the amount of water required to re-entrain the sediment into the river were determined by assuming a 1% sediment concentration.
- The Engelund and Hansen equation is used to estimate the sediment transport capacity; the Ruby equation to calculate fall velocities, and the Exner equation to calculate bed sorting.
- Lateral inflows and diversion of flows to OCWD’s infiltration ponds have been omitted. The project team believes that such omission does not significantly affect the simulation results as it relates to transport of the augmented sediment.
Following the demonstration project, the model will be updated with data obtained during sediment re-entrainment.

### 3.10 Calibration Results

#### 3.10.1 Upstream Geomorphic Reach

The upstream reach is located between River Stations 160818 and 120325. Currently, Prado Dam acts as a sediment trap resulting in only suspended, fine-grained sediment released into the downstream reach. This creates sediment-hungry water, which has led to river degradation. Although the river channel in the upstream segment historically experienced incision, conditions have stabilized somewhat due to channel armoring. The channel is deemed to be in a quasi-equilibrium condition as reflected by the model result presented in Figure B-7, indicating no substantial net degradation or aggradation.

![Figure B-7 Cumulative Sediment Passing Along the Upper Segment](image)
3.10.2 Recharge Reach

The recharge reach is located between River Station 120325 and 59930. The sediment transport capacity in the recharge reach is large compared to the natural sediment supply, indicating potential degradation. However, the grade control structures and, in particular, the drop structure constructed in 1993 retain a measure of quasi-equilibrium in this reach. The rubber dam structures, when inflated, ponds water, and results in large amounts of deposited sediment upstream of it. The captured sediment sizes range from silt to gravel with the highest amount of deposition by weight occurring with very fine to medium sand size fractions. During model calibration, it was assumed that the existing rubber dam was fully inflated throughout the model run. Therefore, it may lead to greater amounts of deposited sediment throughout the recharge reach. Figure B-8 is an overview of the entire section illustrating the significant deposition of sediment just upstream of the rubber dam structure for the calibration run.\(^1\)

\(^1\) Note: For the graphs representing hydraulics and sediment transport characteristics, the abscissa represents the upstream end of the river to the right and the downstream end to the left. Therefore, if the amount of cumulative sediment is lower on the left hand side of the graph than on the right hand side, it means that sediment must have deposited (or removed) within the reach. Negative slopes of curves on the graph indicate erosion of the riverbed or addition of sediments, while positive slopes of curves on the graph represent deposition of transported sediment or removal of sediments.
Figure B-8  Reduction in Sediment Load near the Inflated Rubber Dam Structure
3.10.3 Lower Geomorphic Reach

The lower geomorphic reach is located between River Stations 59930 and 760. The section between station 53380 and station 19400 is concrete-lined. At section 22050, the slope of the channel changes and becomes milder.

Excess sediment transport capacity exists in the reach upstream of section 22050. This is evidenced by the almost horizontal slope of the mass cumulative sediment curve upstream of the slope change (i.e., to the right of the note indicating the slope change on Figure B-9) and by the bare concrete channel bed throughout that reach as observed in the field. Downstream of section 22050 sediment deposition occurs (evidenced by the positive slope of the cumulative mass plot to the left of the note indicating the slope change location in Figure B-9). This deposition is due to the natural reduction in riverbed slope, continuing up to about station 6656. Downstream of this station tidal influence significantly affects sediment transport. Figure B-10 illustrates that, for assumed zero tidal elevation, significant sediment deposition is evidenced by the sudden drop in cumulative sediment mass. Such deposition of sediment is observed in the field, confirming the model result. Figure B-10 illustrates the effect of the tide elevation on sediment deposition downstream of section 22050 for other tidal surface elevations of 0, 4, and 8 feet.
Figure B-9  Cumulative Sediment Passing Along the Lower Segment

Figure B-10  Effect of Different Tide Elevation on Sediment Deposition
3.11 Hydraulics

A riverbed will degrade or aggrade depending on the relative balance between sediment transport capacity and the amount of sediment available for transport. If the sediment transport capacity is greater than the amount of sediment within the river, the riverbed will erode and, therefore, degrade. If the sediment transport capacity of the flowing water is not large enough to transport the available sediment, sediment in the water column will fall out and could lead to aggradation of the riverbed. This process is not dictated by a single flow, but is a cumulative effect of multiple flows within the river.

One way of expressing the relative magnitude of the sediment transport capacity of the flowing water is to relate it to flow velocity, which can be quantified by making use of the Hjulstrom Curve (Hjulstrom 1935). From that curve, it is possible to identify ranges of flow velocities that will mobilize varying sediment sizes. Figure B-11 and Figure B-12 illustrate potentials to erode and deposit different particle sizes along the Santa Ana River reach.

Figure B-11 illustrates that gravel is expected to mobilize for all the simulated flows in the concrete channel in the lower geomorphic reach. For flows greater than 5,000 cfs, cobbles will erode in the concrete reach. The conditions in the recharge reach are different, and gravel will erode when flows are greater than approximately 250 cfs. Velocities are not high enough to erode cobbles for any of the modeled flow in the recharge reach. In the canyon reach, the erosion potential varies by cross section. However, in many locations mobilization of gravels may occur for all modeled flow events.

Anticipated sediment deposition form an important part of this study. Its characteristics may be gleaned from Figure B-12, which indicates that the potential for gravel and cobbles to deposit in the recharge reach is greater than in the upstream and lower geomorphic reaches. However, it is noted that finer materials, i.e., finer than medium gravel-sized material, are not likely to deposit in this reach. This observation provides a guideline on how the recharge reach might be managed to maximize sediment deposition.
Figure B-11  Susceptibility to Erode Based on Steady Flows

Figure B-12  Susceptibility to Deposit Based on Steady Flows
Figure B-13 illustrates Froude Number vs. river station to assist in better understanding the flow regime. Froude numbers in excess of 1 represents super-critical or shooting flow, while those less than one represent sub-critical or tranquil flow. It is noted that the river sections with natural erodible riverbeds are generally sub-critical, with the average Froude number in the recharge reach approximating about 0.4. The evenly spaced peaks in Froude number in the recharge reach represent flow conditions at the grade control structures and are localized. Some of the peaks in the other sections may occur in the general vicinity of bridges.

The Froude number in the concrete channel approaches and even exceeds critical flow conditions for most discharges. It averages about 0.75 to 0.80 in the concrete channel reach, indicating unstable and dangerous flow conditions. High Froude numbers in the concrete channel exceed values of 2, indicating very rapid flows with high sediment transport capacity.
Figure B-13  Froude Number Summary Based on stead Flows
4.0 BASELINE ASSESSMENT DOWNSTREAM OF PRADO DAM

Using the calibrated model parameters set forth above, baseline scenarios were simulated to develop understanding of system characteristics. The baseline model consists of the following:

- **Geometry**: Calibrated model geometry
- **Hydrology**: The three selected hydrologic sequences, representing dry, median and wet years
- **Sediment**: Sediment loads were calculated using the rating curve developed by the USACE (1988) for each of the three water year flow series analyzed

For the base case, pulse flows that will be used to re-entrain sediment were not modeled. Therefore, only the three water year flow series were used in the simulations to analyze base case conditions. The sediment load used is only the natural sediment load coming from Prado Dam during these three flow series. The inflow hydrographs and sediment loads used for the wet year base case scenario is presented in Attachment B (Figure B.B.16 and B.B.17). The dry year is shown in Figures B.B.28 and B.B.29 and the median year is shown in Figures B.B.40 and B.B.41.

4.1 Results

4.1.1 Cumulative Mass

For each of the three baseline models the cumulative mass flowing into each section vs. river station was plotted by particle size at the end of the year. Those plots are shown in Attachment C (Figure B.C.1 to Figure B.C.3). The three plots are vastly different in volumes of sediment being carried through the system; however, locations where deposition or erosion occurs are generally consistent between the three models. Three main deposition points are present in each model for all size fractions. One is an inverse slope just upstream of Weir Canyon Rd. (station 121000), another is an inverse slope just downstream of the Talbert Ave. Bridge (station 29000), and the third is the tidal zone.

The model results indicate that, in general for the wet year model, the sand load continues to increase in a downstream direction until approximately the end of the recharge reach with a major deposition at the inverse slope at Weir Canyon Road. From the end of the recharge reach onward, the sand load roughly remains constant until the inverse slope at the Talbert Ave. Bridge. The sand load increases again until it eventually reaches the tidal zone, where it deposits.

For the dry year model, sand moving through the system follows the same pattern as far as deposition areas as the wet year model except that the maximum sand load occurs upstream of the rubber dam at the Five Coves. From this point, it gradually deposits to the end of the recharge reach and stays fairly constant until it encounters the inverse slope at the Talbert Ave. Bridge. The median year model is very
similar to the wet year model as far as sand load pattern and depositional areas but is much lower in volume.

For all three base models, the gravel load increases downstream of the Weir Canyon Rd. and remains fairly constant throughout the recharge reach before gradually depositing throughout the lower geomorphic reach.

Cobbles are only transported in the upstream geomorphic reach in all models. Silt particles in suspension remain consistent in the upstream and recharge reaches and gradually deposit throughout the lower reach. About the same volume of silt particles incoming to the upstream reach are transported to the ocean. Clay particles are transported throughout the entire reach, although the volume is relatively small.

4.1.2 Degradation and Aggradation
Total erosion and deposition for the three base case models is shown in Figure B-14 through Figure B-16 for the base case scenarios. From an overall point of view, it is noted that the general trend within and upstream of the recharge reach resembles degradation for the wet, dry, and median year scenario. The upstream geomorphic reach shows erosion hovering around the 0.25-foot degradation limit input to the model. Deposition occurs in all three models between Gypsum Canyon Rd. and a local road at the Green River Golf club in the upstream reach at station 147000. Deposition in this area is not apparent from the cumulative mass figures described in Section 4.1.1. Locally sediment deposits of between 0.5 and 1.5 feet occur in the recharge reach, depending on the run. The concrete lined channel experiences no simulated scour (as expected). Deposits occur in the downstream reach again with depositions of 1 to 3.5 feet at the Talbert Avenue Bridge. Tidal influences cause another deposit of less than 1.0 foot.
Figure B-14  Total Erosion or Deposition for the Wet Year Base Case

Figure B-15  Total Erosion or Deposition for the Dry Year Base Case
Figure B-16  Total Erosion or Deposition for the Median Year Base Case
5.0 PREDICTIVE ASSESSMENT

The predictive assessment was used to evaluate the effects of augmenting sediment loads by introducing 500,000 yd$^3$ of sediment into the SAR.

5.1 Location of Sediment Introduction

Locations and methods to introduce the augmented sediment load are still under consideration. From a modeling point of view, it is assumed that the sediment will be introduced into the SAR at a location just downstream of Prado Dam.

5.2 Sediment Concentration

The sensitivity analysis revealed that minor changes in depositional patterns occurred when using alternative initial sediment concentrations of 0.25%, 0.50%, 0.75%, and 1% at the point where the sediment is introduced. Based on engineering judgment, it was deemed reasonable to assume that a sediment concentration of 1% would be appropriate for the re-introduction of sediment. This is a conservative assumption, based on engineering judgment, of a sediment concentration that will not cause sediment to fall out of suspension immediately after re-entrainment and create a dam in the river. As this assumption may affect acceptable levels of sediment concentration in the river water it may be varied to optimize the project.

5.3 Hydrology and Sediment Augmentation

The historical flows range between 100 and 5,000 cfs. The hydrology used for the predictive runs are the same years of data run for the three respective base case models with the addition of the pulse flows and corresponding durations added to the beginning of each run when sediment augmentation is assumed to take place. At this time, the pulse flow range that was used in the simulations varies: 500, 750, 1,250, 2,000, and 5000 cfs. It should be noted that OCWD does not anticipate dam operations will be modified to provide pulse flows as modeled here. Sediment re-entrainment will take place throughout normal dam operations as flows allow for a period of 72 hours followed by a period of 24 hours without re-entrainment. The pulse flows with re-entrainment were modeled to show the minimum amount of time re-entrainment can take place and the effects on the re-entrained sediment after a reasonable amount of time of normal flow conditions. The natural sediment load is still assumed to be flowing from the dam during re-entrainment flows and is added to augmented sediment loads.

The duration of these flows was dictated by the time required to re-entrain 500,000 yd$^3$ of sediment at a 1% concentration by weight. The corresponding sediment load rates in tons/day, assuming a 1% concentration by weight, are provided in Table B-4. The estimated times required to transport 500,000 yd$^3$ of sediment for each of the discharges, following the 72 hours on/24 hours off re-entrainment schedule, are shown in Table B-5.
Table B-4  Sediment Load at 1% Concentration by Weight

<table>
<thead>
<tr>
<th>Discharge (cfs)</th>
<th>Sediment Load at 1% Concentration by Weight (tons/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>13,620</td>
</tr>
<tr>
<td>750</td>
<td>20,430</td>
</tr>
<tr>
<td>1,250</td>
<td>34,050</td>
</tr>
<tr>
<td>2,000</td>
<td>54,480</td>
</tr>
<tr>
<td>5,000</td>
<td>136,200</td>
</tr>
</tbody>
</table>

Table B-5  Durations to Deplete 500,000 yd³ of Sediment for Six Selected Discharges

<table>
<thead>
<tr>
<th>Discharge (cfs)</th>
<th>Duration (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>57.4</td>
</tr>
<tr>
<td>750</td>
<td>37.9</td>
</tr>
<tr>
<td>1,250</td>
<td>22.4</td>
</tr>
<tr>
<td>2,000</td>
<td>13.8</td>
</tr>
<tr>
<td>5,000</td>
<td>5.3</td>
</tr>
</tbody>
</table>

The graphs relating incoming sediment versus time are presented in Attachment B (alternate graphs in Figure B.B.16 to Figure B.B.51).

5.4 Results

5.4.1 Cumulative Mass

The simulation results are contained in Attachment D. Figure B.D.1 to Figure B.D.15 illustrates the mass balance of sediment discharge over the entire length of the SAR downstream of Prado Dam. From those graphs, it is seen that the general progression of the cumulative sediment mass differs from the base case and is fairly consistent between the three yearly flow scenarios. For the base case, the simulated sediment load generally increases between the dam and the end of the recharge reach, except for the silt and clay load that remains constant throughout the model and for the cobble load that only exists over a relatively small distance along the upstream end of the SAR.

5.4.1.1 Sand Load

From Figure B.D.1 to Figure B.D.15 for the predictive case it is seen that the cumulative load of sand gradually decreases through the upstream reach with a large deposit at the inverse slope at Weir Canyon Road for all scenarios.
The wet year model sand load generally remains consistent through the recharge reach with discreet deposition points downstream of the Imperial Highway Rubber Dam (station 102000), upstream of the Lincoln Ave. Bridge (station 84400), and at the Garden Grove Freeway. Bridge (station 60000). In the lower reach, the sand load remains constant through the concrete lined reach and then deposits in the tidal zone.

The sand load in the dry year models gradually decreases throughout the recharge reach with less noticeable discreet deposition points than the wet model. Like the wet year model, the dry year model sand load remains constant through the concrete lined section and deposits in the tidal zone. The sand load through the recharge reach in the median year model is similar to the dry year model with a gradual reduction in load. A large deposition area is present downstream of the Imperial Highway Rubber Dam (station 102000) in the median year model. In general, the lower the pulse flow at the beginning of the run for all water years, the more sand is deposited in the recharge reach and the less is transported through to the tidal zone.

5.4.1.2 Other Sediment Size Fractions
In general, the silt load remains consistent through all wet year scenarios with small discreet depositions in lower pulse flow models downstream of the Imperial Highway Rubber Dam (station 102000). The dry and median year models show a more gradual deposition in the silt load through the recharge reach at lower pulse flows. Higher pulse flows do not result in much deposition through the recharge reach. As with the sand load, the silt load remains constant through the concrete lined reach and deposits in the tidal zone with a portion of the load flowing through to the ocean. Clay, gravel, and cobble loads behave similar to the base load models.

5.4.2 Degradation and Aggradation
Total changes in invert elevations along the river for the predictive case are shown in Figure B-17 through Figure B-19. Major deposition areas in the predictive models include:

- An area between Gypsum Canyon Rd. and a local golf course road (station 148000) with depositions around 4 feet maximum in the wet year models and 2.5 feet maximum in the dry and median year models
- Upstream of Weir Canyon Rd. (station 121000) with depositions of about 3 to 3.5 feet maximum in all model years
- Between the SPT Railroad Bridge and the Katella Ave. Bridge at a transition from a 320 to 270 feet channel bottom width (station 72000) of about 3.5 feet maximum for the wet year model and about 6.5 feet maximum for the dry and median year models
- Upstream of the Garden Grove Freeway. Bridge (station 60000) with depositions of about 5 feet maximum for the wet and dry year and 7 feet maximum for the median year
- At the inverse slope downstream of the Talbert Ave. Bridge (station 29000) of about 5 feet maximum for all three model years.
Other areas of deposition are estimated to be present throughout the modeled reach but are generally limited to 2.5 feet or less.

Some locations of deposition are present in both the base case and in the predictive scenarios. It can be assumed, from analyzing Figures B-14 to B-16 against Figures B-17 to B-19, that the sediment re-entrainment is not responsible for the total deposition shown at these particular locations. The difference between these figures is the sediment added to the system through re-entrainment. A normal process of any river is active deposition and erosion in a balanced manner. Therefore, it is likely a geomorphic trend that at locations where significant deposition takes place, there would be some degree of deposition under normal water and sediment flow conditions.

Deposition levels could be considered conservative due to several assumptions made in the model setup. These include:

- The sediment re-entrainment site and storage at the re-entrainment. This is calculated from available borehole data. There is a possibility that there will be more fines in the sediment for re-entrainment that will be passed through the system more readily than sand size particles.
- The presence of sediment passes through nodes. By using pass through nodes to aid in stability, the model does not show deposition in some areas that would likely see deposition in reality. This could lead to more deposition in other locations than may occur in the field.

Even though deposition levels reported may be conservative, the model can provide guidance in selecting deposition-monitoring points during the demonstration project.

### 5.4.3 Sediment Concentration

The simulated sediment concentrations immediately after sediment augmentation (October 1), at March 1, and at the end of simulation (September 30) are shown in Figure B-20 through Figure B-28. Within the upstream and recharge reaches the concentrations hover around 10,000 mg/l for all modeled flow scenarios immediately after sediment re-entrainment. The concentration on October 1 steadily drops off in the lower reach before dropping to near 0 downstream of the end of the recharge reach. As the models progress through the year, the general trend of the sediment concentration through the reach remains the same but the concentration decreases as time goes on.

### 5.4.4 Spatial and Temporal Distribution of Deposited Sediment

The simulated temporal and spatial distributions of deposited sand for the predictive case are presented in Figure B.D.16 to Figure B.D.45. These figures shows the distribution of sand-sized sediment particles in tons for each River Station on October 1 (just after augmentation) and on September 30 (at the end of the simulation). These figures indicate that generally sand deposits throughout the LSAR immediately
after re-entrainment but is transported out of the recharge reach during the period of only the natural sediment load. Sediment that has deposited in the upstream geomorphic reach as well as in the tidal zone immediately after re-entrainment is generally observed to remain throughout the model run. In general, the lower the pulse flow at the beginning of the model run, the more sediment remains in the recharge reach after the model run.
Figure B-17  Total Erosion or Deposition for the Wet Year Predictive Scenarios

Figure B-18  Total Erosion or Deposition for the Dry Year Predictive Scenarios
Figure B-19  Total Erosion or Deposition for the Median Year Predictive Scenarios

Figure B-20  Sediment Concentration Immediately After Introduction of Sediment for the Wet Year Predictive Scenarios
Figure B-21  Sediment Concentration at March 1 for the Wet Year Predictive Scenarios

Figure B-22  Sediment Concentration at End of Simulation for the Wet Year Predictive Case
Figure B-23  Sediment Concentration Immediately After Introduction of Sediment for the Dry Year Predictive Case

Figure B-24  Sediment Concentration at March 1 for the Dry Year Predictive Scenarios
Figure B-25  Sediment Concentration at End of Simulation for the Dry Year Predictive Case

Figure B-26  Sediment Concentration Immediately After Introduction of Sediment for the Median Year Predictive Case
Figure B-27  Sediment Concentration at March 1 for the Median Year Predictive Scenarios

Figure B-28  Sediment Concentration at End of Simulation for the Median Year Predictive Case
5.4.5 Releases of Fines to the Ocean

Figure B-29 to Figure B-31 indicates that fine-grained sediment remains consistent throughout the modeled reach until reaching the tidal zone for the wet year model with a few discreet depositions and releases between 100,000 and 150,000 tons of fines to the ocean. The median and dry year models show a more gradual deposition of fines throughout the modeled reach with releases to the ocean of between 25,000 and 100,000 tons.
Figure B-29  Weight in Tons of Clay and Silt in Suspension for the Wet Year Predictive Case
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Figure B-30  Weight in Tons of Clay and Silt in Suspension for the Dry Year Predictive Case

Figure B-31  Weight in Tons of Clay and Silt in Suspension for the Median Year Predictive Case
5.4.6 Flood Potential

The increased flood potential resulting from the proposed project was considered by simulating floods through the system prior to and after implementation of the augmentation project. The simulations incorporate two worst-case assumptions, and are therefore very conservative. The first assumption is that the flood will occur immediately after termination of the pulse flows. The second assumption is that no sediment transport will occur during the flood. The first assumption is statistically conservative. The second assumption is technically unrealistic and is therefore very conservative. Figures B.D.46 to B.D.60 show the effect of the new bed geometry, i.e., incorporating the changes due to aggradation and degradation resulting from sediment augmentation, on the water surface elevation (WSEL) for the sections with levees of the LSAR (i.e., downstream of River Station 120810). It also shows the locations and magnitude of negative freeboard in the sections with levees.

The cross section geometry used is the resulting geometry just after sediment augmentation, i.e., immediately after termination of the pulse flows. This is a conservative scenario as it is deemed unlikely that a major flood will occur concurrently with sediment augmentation activities. The information in these figures was created by running the design flow through the LSAR with existing geometry to establish a baseline and with the altered geometry immediately after sediment augmentation. The design flow used for the flood potential simulations is shown below in Table B-6:
### Table B-6  Design Flow used for Flood Potential Simulations

<table>
<thead>
<tr>
<th>River Station</th>
<th>Discharge (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>162302.5</td>
<td>30000</td>
</tr>
<tr>
<td>161331.7</td>
<td>31000</td>
</tr>
<tr>
<td>160675.5</td>
<td>32000</td>
</tr>
<tr>
<td>156544.8</td>
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</tr>
<tr>
<td>152402.5</td>
<td>33500</td>
</tr>
<tr>
<td>150463.6</td>
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</tr>
<tr>
<td>144546.3</td>
<td>35000</td>
</tr>
<tr>
<td>142790.3</td>
<td>35500</td>
</tr>
<tr>
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<td>36000</td>
</tr>
<tr>
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<td>36500</td>
</tr>
<tr>
<td>121986.6</td>
<td>37000</td>
</tr>
<tr>
<td>106494.2</td>
<td>38000</td>
</tr>
</tbody>
</table>

In both cases, i.e., for the base case and the predictive case, it is assumed that the river has rigid boundaries (bed and banks), as is normally done in flood studies. Flood simulations were completed for conditions after each of the pulse flows that were considered (i.e., pulse flows of 500, 750, 1,250, 2,000, and 5,000 cfs).

The maximum WSEL increase in the levee reach is between 4.0 and 6.0 feet at a few locations including downstream of Weir Canyon Rd. (station 120300), near the bicycle bridge and road bridge at the Imperial Highway (station 105000), near the Five Coves rubber dam (station 86000), between Interstate 5 and the Garden Grove Freeway (station 61000), between the Warner Ave. bridge and the Slater Ave. bridge (station 33000), and at a slope change downstream of the Interstate 405 bridge (station 23000). WSEL increases are generally more in the downstream reach and less in the upstream reach as pulse flows become larger. Locations where it is estimated that water could potentially flow out of the channel banks are:

- Station 116498 between Weir Canyon Rd. and the Imperial Hwy.
- Station 106987 Upstream of a bicycle bridge
- Station 85999 At a flat slope downstream of the Glassell St. Bridge
- Station 61000 Between Interstate 5 and the Garden Grove Freeway

In general, the pulse flow of 500 cfs may increase flooding risk only in the upstream reach (stations 116498 and 106987). Pulse flows of 750 and 1,250 cfs may increase the greatest flooding risk throughout the four locations where flooding is seen. Pulse flows of 2,000 cfs have a decreased risk of flooding in all locations, and pulse flows of 5,000 cfs do not create a flooding risk.
As described above, the flood risk modeling is conservative and likely over-estimating flood risks due to several factors including:

- All 500,000 yd$^3$ of sediment are re-entrained in a short amount of time (less than one season)
- The extreme event would occur immediately after sediment re-entrainment before normal flows would redistribute deposited sediment throughout the reach
- The extreme event would not redistribute deposited sediment during the event

The simulations are designed to provide information regarding where to monitor for the development of conditions that could increase flood risks and decision makers to take corrective actions. That said, the locations mentioned above with an increased risk of flooding after sediment deposition will be monitored more closely during the demonstration project for increased deposition and the potential for water surface elevation increases.
6.0 CONCLUSION

This report appendix is a revision to the appendix submitted by Golder in February 2011 (Golder 2011). This revision was required due to channel improvements made in the upstream reach of the LSAR. A new HEC-RAS geometry file was created to reflect the channel improvements and provided to Golder in April 2014. Generally, the revision did not affect the sediment transport throughout the LSAR. As previously discussed, the model was not calibrated to historic data in the traditional sense. However, the model does reflect the quasi-equilibrium condition of the river as observed during field visits. The model can be used as a means to help guide monitoring location decisions. After conducting monitoring during the demonstration project, data collected can be used to verify and update the model where required. As discussed in the Introduction, there are four primary and seven secondary questions to be addressed by the sediment transport model results.

1. **What is the anticipated spatial and temporal distribution of deposited sand-sized replenishment materials in the LSAR?**

   The simulated spatial distribution of deposited sand-sized sediment changes with time. The currently simulated spatial distribution of deposited sand immediately after sediment augmentation differs from the distribution at the end of one year.

   For all simulated flow scenarios, immediately following the augmentation period, sand-sized particles are simulated to deposit in several reaches along the LSAR. This includes a large amount of deposited sand in the recharge reach. However, during the course of a year with the river subject to average flows the simulation indicates that sand initially deposited in the recharge reach will likely move to the far downstream reach, i.e., the tidal zone of the LSAR. This is the case even though the total number of days where there is flows capable of carrying sediment farther downstream is small.

   It is pointed out that these results may not reflect actual long-term conditions. The reason or this is that only one year of flows was simulated. Golder is of the opinion that the Santa Ana River will reach a new equilibrium condition with continued sediment augmentation in the long term and that it might lead to a complete change in the composition of the sand-sized material in the LSAR.

2. **Could the proposed project replenish beach sand?**

   Sand sized particles are currently predicted to ultimately deposit in the far downstream reaches of the LSAR. The preliminary model indicates that the sand-sized particles might not reach the beach. However, it is noted that the current model only reflects one year of flow and not long-term conditions.

   It is Golder’s opinion that long-term sediment augmentation will likely result in a new equilibrium river state. Over the long-term such a new equilibrium state will likely lead to more sand moving down the river towards the beach as sediment augmentation operations continue. In such a case, it is possible that the augmentation project could replenish beach sand.

3. **How does the project affect riparian habitat in the LSAR?**

   The current preliminary simulations indicate that clay and silt move through the LSAR to the ocean. Sand-sized particles deposit throughout the river reach, with current simulations indicating that much of it ends up in the upstream reach and the tidal zone at the end of the simulation period. Gravel is more evenly distributed throughout the system, while cobbles are mainly deposited in the upper reaches of the river.
Generally, silt- and clay-sized particles are often undesirable for riparian habitat. The current simulations indicate that clay-sized particles will be transported to the ocean and will not deposit over a wide area. More desirable particle sizes like sand and gravel may be more prevalent under augmentation conditions. More sand may be present in the recharge reach and in the tidal zone and the amounts of gravel throughout the river reach will likely increase.

4. **Could the proposed sediment augmentation project change the gradation of the LSAR bed material in the groundwater recharge reach?**

The model results show that between 2 and 3 feet of additional sand will likely deposit within the recharge reach immediately after sediment re-entrainment. This sand could change the overall particle size distribution of the bed material in the recharge reach. The modeling indicates that some re-entrainment flows may provide better performance than others. One aim might be to minimize the amount of sediment deposited in the flat river reach downstream of station 22050 (towards the ocean) and maximize the amount of sediment deposited in the recharge reach (between stations 120325 and 60129). Another objective might be to minimize flood impacts. The final selection of a desirable range of re-entrainment flows will likely also have to consider the desire of OCWD to minimize the amount of sediment deposition in the settling ponds adjacent to the river. Consideration of the results presented in this Appendix and its Annexure can be used to select preferred re-entrainment flow magnitudes.

Golder is of the opinion that sediment augmentation can lead to a more favorable particle size distribution that can result in increased permeability if an appropriate sediment augmentation strategy is followed. Optimization of such a strategy is the subject of the demonstration program.

The replies to the secondary questions are as follows:

1. **Do the silt- and clay-sized sediments move through the LSAR?**

   Fines (clay and silt sized particles) experience very little deposition as they move through the LSAR. Under augmentation conditions, it is predicted that about one quarter to one third of the fines introduced to the LSAR will be released to the Pacific Ocean in the wet year model and between 10 and 15% in the dry and median year models. The fines that do deposit generally gradually deposit throughout the entire modeled reach. The wet year models suggest that about 100,000 to 150,000 tons of fine-grained sediment will be released to the ocean. The dry and median year models predict releases somewhere around 25,000 to 100,000 tons over the model year.

2. **Could the proposed project result in increased flooding potential, particularly downstream of I-405**

   Given the conservative assumptions used in this assessment, the maximum increase in water surface elevation for the design flood in the levied reach is about 4 to 6 feet and occurs at a few locations within this reach. The larger increases in WSEL in the upstream and recharge reaches occur under lower pulse flows and under larger pulse flows for the largest WSEL increases seen in the lower reach. Downstream of I-405, WSEL increases after sediment re-entrainment are up to 6 feet under high pulse flow scenarios but the increase in WSEL does not lead to any flooding. Other locations between Weir Canyon Rd. and the Imperial Highway do experience flooding due to WSEL increases after sediment re-entrainment.
3. **Would the project result in increased maintenance requirements at diversion structures?**

Maintenance requirements considered include bank protection, scour downstream of structures, sedimentation and riverbank / levee overtopping. The potential for increased maintenance depends on the magnitudes of the flows that will be used to re-entrain the augmented sediment. If the selected flows are much higher than flows normally occurring in the LSAR, it might result in greater maintenance requirements due to increased scour at some structures and possible bank erosion in some river reaches.

Adding additional sediment to the river might also result in increased maintenance needs. This is particularly true if flows with high sediment concentrations are diverted to the infiltration ponds operated by OCWD. Some of the drop structures might also accumulate more sediment, but it is deemed unlikely that the amounts of deposited sediment will be so much greater as to result in significantly higher maintenance cost at the latter.

The increased amounts of deposited sediment in the most downstream river reach, i.e., the lower geomorphic reach, may lead to increased maintenance cost if it is deemed necessary to remove such deposits. Currently, the prediction is that the amount of freeboard in this river reach is more than enough to prevent flooding during design flows except at the few locations discussed above. This implies that it may not be necessary to remove the increased amounts of deposited sediment from this reach except where the risk of flooding is greater. Alternatively, levee protection or other flood protection measures can be provided where there is flooding potential.

4. **Could the proposed project lessen the effects of channel degradation at Featherly Park be reversed?**

The model shows continued degradation occurring in Featherly Park in unlined portions of the LSAR.

5. **Could the project result in increased scour potential at the levees in the LSAR?**

At this stage, the potential for increased scour at the levees has not been calculated. The scour limit of 0.25 feet set in the simulation model prevents realistic assessment of levee scour potential. However, it is Golder’s opinion, at this stage, that it is unlikely that increased scour would be experienced, except if very high re-entrainment flows are selected. If the range of the selected re-entrainment flows resemble flow conditions normally experienced in the LSAR, it is unlikely that increased scour would occur. Rather, the increased amounts of sediment introduced into the LSAR will likely decrease the potential for levee scour.

6. **What are the measurable effects at critical structures within the LSAR?**

There are a few areas where measurable results can provide feedback as to the success of the demonstration project. These areas include:

A. Cannot have excessive deposition which leads to increased flood risks especially downstream of I-405

B. No more than 8 feet of deposition can be allowed at Rock Canyon Weir

C. The deposition of sand can be measured at the Pacific Coast Highway bridge

D. Degradation at Featherly Park can be observed

7. **Could the proposed project result in increased river degradation at the SARI line?**

The current constraints used in the simulation model might not fully account for maximum possible scour. At this stage, it is not deemed reliable to make conclusions from the simulation model to predict scour at the SARI line. However, based on professional experience, it is Golder’s opinion that such scour will not be exacerbated if the selected
re-entrainment flows remain within the ranges normally experienced in the river. If this requirement can be satisfied, it is likely that the scour will actually decrease. This is because the sediment load in the river will increase when the sediment loads are augmented. Such an increase in the sediment load is likely to lead to aggradation of the riverbed rather than degradation.
7.0 CLOSING

Golder appreciates the opportunity to be of service on this project.

GOLDER ASSOCIATES INC.

Craig P. Baxter, PE
Project Engineer

George W. Annandale, D.Ing., PE
Principal

CPB/GWA/rjg
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8.0 REFERENCES


U.S. Army Corps of Engineers (USACE). 1988. Santa Ana River Design Memorandum No. 1: Phase II GDM on the Santa Ana River Mainstem Including Santiago Creek, Volume 3: Lower Santa Ana River (Prado Dam to Pacific Ocean)

1.0 SITE VISIT ASSESSMENT

A site visit was conducted on February 2010 at the Prado Dam Basin and the Santa Ana River downstream of Prado Dam to the Pacific Ocean. The objective was to obtain information pertaining to the present hydraulic, sediment, and geomorphic conditions of the system. The observations and collected data were also used to develop sediment transport simulation models.

1.1 Prado Basin

The Prado Dam Basin is characterized by significant sedimentation and encroachment of vegetation (Figure A-1). Some vegetation has been cleared, principally for agricultural use. The bed material of tributaries flowing into the reservoir consists mostly of sand-sized material. Additionally, large quantities of debris including garbage, uprooted trees, and branches were observed just upstream of the River Road Bridge (Figure A-2).

Figure A-1: Prado Basin Vegetation Encroachment
1.2 Downstream of Prado Dam

The objective of visiting the river downstream of Prado Dam was to obtain baseline information for the hydraulics, sediments, and geomorphic conditions. During the site visit, three sub-reaches were identified:

- The upstream reach extending from the Prado Dam to the recharge area;
- The recharge area; and
- The lower reach between the end of the recharge area and the Ocean.

Each of these reaches is described below.

1.2.1 Upstream reach.

The upstream reach has several sections with distinctive characteristics. The upstream section, from the dam outlet structure to the golf course property line, has a relatively flat slope compared to the remainder of the upstream reach. The floodplain is covered with riparian vegetation; the banks are moderately incised and vegetated islands dot the main channel. No significant bed forms were identified in this reach. The bed material could not be inspected due to flow releases from Prado Dam at the time of the site visit. However, bed material analyses have been performed by Engineering and Hydrosystems and Golder Associates in 2007 and 2009.

The river bed slope increases in the next segment extending along the entire length of the golf course. This section is characterized by significant river bed incision, and during the site visit the water was
confined inside the main channel banks. The river valley is crossed by a high railroad bridge and two access bridges to the golf course. River bank stabilization was under construction during the field trip. The construction activities extended both upstream and downstream of the access bridge to the Golf Course along the Santa Ana River. Hydraulic controls identified along this reach include man-made and natural bed controls. Figure A-3 illustrates the general condition and shape of the river in this area and the general bank and hydraulic characteristics of the reach.

![Figure A-3: Santa Ana River Looking Upstream from Entrance Road to Green River Golf Course](image)

The river section between the golf course and the upstream end of the recharge area contains a large floodplain, with several flow splits forming natural islands. Riparian vegetation is mostly concentrated near the river bank. Overbank sediment deposits were frequently observed, which indicates overtopping of the river banks with water flowing onto the floodplain during flood events. This reach ends at a drop structure a short distance downstream of Weir Canyon Road. The river bed upstream of the drop structure is silted-in up to the top of the drop structure.

No flow diversions were detected throughout this reach. However several storm drainages into the river have been identified on both sides of the river. An important drainage is located on the left bank of the Santa Ana River commencing just downstream of the Prado Dam outlet structures. This tributary may produce significant inflow of both water and sediment during extreme events

**1.2.2 Recharge Area Reach**

The recharge area reach extends from the drop structure immediately downstream of the Weir Canyon Road Bridge to the drop structure immediately downstream of California 22 Highway. Several man made
hydraulic structures were identified throughout the recharge area that confines and controls the water and sediment flow. The structures include:

- Bank stabilization with concrete and/or riprap;
- Drop structures constructed with concrete;
- Two inflatable rubber dam structures;
- Small grade control structures;
- Storm drainage;
- Diversion dams;
- Infiltration ponds adjacent to the main channel, and
- Temporary, small detention ponds between T&L levees to enhance infiltration.

During the field trip it was observed that all flows from Prado Dam and urban runoff were either diverted to infiltration ponds adjacent to the river or temporarily dammed in the Santa Ana River for infiltration into the river bed. The principal diversion structures are located downstream of Imperial Highway Bridge and downstream of Glassell St. Bridge (Figure A-4). At the diversion structure downstream of Glassell St. the flow was completely diverted. It appeared that part of the diverted flow was returned to the Santa Ana River to be infiltrated further downstream.

The rubber dam structures downstream of the Imperial Highway and Glassell St. were constructed in 1993 and currently have only minor deposition of sediment on their upstream sides (Figure A-5). The inflatable dams may be partially lowered during flows greater than 500 cfs to enable downstream transport of sediment and to reduce the amount of solids flowing into the off-channel infiltration ponds. At flows greater than 2,000 cfs, the inflatable dams are fully deflated. Above 2,000 cfs additional intertie tubes through the SAR levees divert small amounts of flow.

Downstream of the diversion structures the flow is significantly reduced (Figure A-6). The bed material consisted predominantly of sand with small amounts of cobbles. The bed material contains insignificant amounts of silt and clay (Figure A-7).
Figure A-4: Rubber Dam Structure Downstream of Glassell St.

Figure A-5: Sediment Deposition Downstream of Glassell St. and Upstream of the Rubber Dam Structure
Figure A-6: Recharge Area Downstream of Orangewood Ave. Looking Downstream Showing Remaining Flow in River Detained by T&L Levees

Figure A-7: Upstream of Chapman Ave. Bridge Looking Downstream Showing Mostly Sand Bed Material
1.2.3 **Lower Reach**

The lower reach (i.e. from the downstream end of the recharge reach to the ocean) has segments with distinctive geomorphic and hydraulic characteristics. The river reach between the Highway 22 drop structure and the Santa Ana River Trail pedestrian bridge is unlined. Floods are controlled by levees and several grade control structures are present along the reach. The Riverview Golf Course is located in part of this river reach, within the main channel and floodplain. Bed material is mostly sandy with small percentages of gravel (Figures A-8 and A-9). The main channel bed forms predominantly consist of dunes (Figure A-8). Bank erosion was also identified.

![Figure A-8: Bed Material Upstream of Memory Ln. Bridge Looking Downstream](image-url)
Figure A-9: Bed material is mostly sandy, with some gravel

Both the river bed and banks are lined with concrete between the Riverview Golf Course and the Mesa Verde Country Club (Figure A-10). The channel has a trapezoidal shape and contains a low-flow channel. No significant sediment deposition was observed throughout this reach, indicating high sediment transport capacity. However, sediment deposition was observed downstream of I-405, near the Mesa Verde Country Club.

Figure A-10: Lined Channel Section at W. Edinger Ave. Looking Downstream
Downstream of Mesa Verde Country Club the Santa Ana River continues with concrete lined banks and a natural, unlined channel bed. The channel bed consists mostly of sand, with a frequent presence of bars. The change in river bed morphology from dunes to bars is deemed to be associated with the reduction in river bed slope and the influence of tides. As the Santa Ana River flows into the ocean downstream of the West Coast Highway, the channel becomes narrow and the river bed becomes incised due to a large amount of sediment deposition at the beach (Figure A-11).

Figure A-11: Aerial View of Santa Ana River at the Pacific Ocean
ATTACHMENT B
INPUT PARAMETER/PROJECT BACKGROUND FIGURES AND TABLES
Figure B.B.1

SS10R

Cumulative % Passing

Particle Size (mm)
SS9R

Figure B.B.2
Figure B.B.3
SS4RSur

Cumulative % Passing vs. Particle Size (mm)

Figure B.B.4
Figure B.B.6
Figure B.B.7
Figure B.B.8
Figure B.B.10
Figure B.B.12
Figure B.B.13
Figure B.B.14
Figure B.B.15
Base Case Wet Year Flow Hydrograph From Dam

Figure B.B.16
Base Case Wet Year Sediment Inflow

Simulation Date

Sediment Inflow (tons/day)

Figure B.B.17
500 cfs Pulse Flow Wet Year Hydrograph From Dam

Inflow (cfs)

Simulation Date

500 cfs Pulse Flow Wet Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

750 cfs Pulse Flow Wet Year Hydrograph From Dam

Inflow (cfs)

Simulation Date

Figure B.B.20
750 cfs Pulse Flow Wet Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

Figure B.B.21
1,250 cfs Pulse Flow Wet Year
Hydrograph From Dam

Simulation Date

Inflow (cfs)

1,250 cfs Pulse Flow Wet Year
Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date


Figure B.B.23
2,000 cfs Pulse Flow Wet Year Hydrograph From Dam

Inflow (cfs)

Simulation Date

2,000 cfs Pulse Flow Wet Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date


Figure B.B.25
5,000 cfs Pulse Flow Wet Year Hydrograph From Dam

Inflow (cfs)

Simulation Date


5,000 cfs

Pulse Flow

Wet Year

Hydrograph

From Dam

Figure B.B.26
5,000 cfs Pulse Flow Wet Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

Base Case Dry Year Sediment Inflow

Simulation Date

Sediment Inflow (tons/day)

500 cfs Pulse Flow Dry Year Hydrograph From Dam

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Inflow (cfs)

Simulation Date


Figure B.B.30
500 cfs Pulse Flow Dry Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date


Figure B.B.31
750 cfs Pulse Flow Dry Year Hydrograph From Dam

Inflow (cfs)

Simulation Date


Figure B.B.32
750 cfs Pulse Flow Dry Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

1,250 cfs Pulse Flow Dry Year Hydrograph From Dam
1,250 cfs Pulse Flow Dry Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

2,000 cfs Pulse Flow Dry Year Hydrograph From Dam

Inflow (cfs)

Simulation Date


Figure B.B.36
2,000 cfs Pulse Flow Dry Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date


Figure B.B.37
5,000 cfs Pulse Flow Dry Year Hydrograph From Dam

Figure B.B.38
5,000 cfs Pulse Flow Dry Year Sediment Inflow

Simulation Date

Sediment Inflow (tons/day)

Base Case Median Year Flow Hydrograph From Dam

Inflow (cfs)

Simulation Date

Base Case Median Year Sediment Inflow

Simulation Date

Sediment Inflow (tons/day)

500 cfs Pulse Flow Median Year Hydrograph From Dam

Inflow (cfs)

Simulation Date

500 cfs Pulse Flow Median Year Sediment Inflow

Simulation Date

Sediment Inflow (tons/day)
750 cfs Pulse Flow Median Year Hydrograph From Dam


Simulation Date

Inflow (cfs)

0 1000 2000 3000 4000

Figure B.B.44
Figure B.B.45

750 cfs Pulse Flow Median Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

1,250 cfs Pulse Flow Median Year Hydrograph From Dam
1,250 cfs Pulse Flow Median Year Sediment Inflow

Sediment Inflow (tons/day)
Simulation Date


Figure B.B.47
2,000 cfs Pulse Flow Median Year Hydrograph From Dam

Inflow (cfs)

Simulation Date

2,000 cfs Pulse Flow Median Year Sediment Inflow

Sediment Inflow (tons/day)

Simulation Date

5,000 cfs Pulse Flow Median Year Hydrograph From Dam
5,000 cfs Pulse Flow Median Year Sediment Inflow
Cumulative Mass In vs. River Station for Wet Year Base Case Assessment

Figure B.C.1
Cumulative Mass In vs. River Station for Median Year Base Assessment

- Total Clay
- Total Silt
- Total Sand
- Total Gravel
- Total Cobbles
Cumulative Mass In vs. River Station for 500 cfs Pulse Flow Wet Year Predictive Assessment

Figure B.D.1
Cumulative Mass In vs. River Station for 750 cfs Pulse Flow Wet Year Predictive Assessment

Figure B.D.2
Cumulative Mass In vs. River Station for 1,250 cfs Pulse Flow Wet Year Predictive Assessment

Figure B.D.3
Cumulative Mass In vs. River Station for 2,000 cfs Pulse Flow Wet Year Predictive Assessment

- Clay
- Total Silt
- Total Sand
- Total Gravel
- Total Cobbles

Figure B.D.4
Cumulative Mass In vs. River Station for 5,000 cfs Pulse Flow Wet Year
Predictive Assessment

Figure B.D.5
Cumulative Mass In vs. River Station for 500 cfs Pulse Flow Dry Year Predictive Assessment

Figure B.D.6
Cumulative Mass In vs. River Station for 750 cfs Pulse Flow Dry Year Predictive Assessment

- Clay
- Total Silt
- Total Sand
- Total Gravel
- Total Cobbles

Figure B.D.7
Cumulative Mass In vs. River Station for 1,250 cfs Pulse Flow Dry Year Predictive Assessment

Figure B.D.8
Cumulative Mass In vs. River Station for 2,000 cfs Pulse Flow Dry Year Predictive Assessment

Figure B.D.9
Cumulative Mass In vs. River Station for 5,000 cfs Pulse Flow Dry Year Predictive Assessment

- **Clay**
- **Total Silt**
- **Total Sand**
- **Total Gravel**
- **Total Cobbles**

Figure B.D.10
Cumulative Mass In vs. River Station for 500 cfs Pulse Flow Median Year Predictive Assessment

Figure B.D.11
Cumulative Mass In vs. River Station for 750 cfs Pulse Flow Median Year Predictive Assessment

Figure B.D.12
Cumulative Mass In vs. River Station for 1,250 cfs Pulse Flow Median Year Predictive Assessment

Figure B.D.13
Cumulative Mass In vs. River Station for 2,000 cfs Pulse Flow Median Year Predictive Assessment

Figure B.D.14
Cumulative Mass In vs. River Station for 5,000 cfs Pulse Flow Median Year Predictive Assessment

Figure B.D.15
Sand Size Particles Deposited on October 1
500 cfs Wet Year Predictive Scenario
Sand Size Particles Deposited on September 30
500 cfs Wet Year Predictive Scenario

Figure B.D.17
Sand Size Particles Deposited on October 1
750 cfs Wet Year Predictive Scenario
Sand Size Particles Deposited on September 30
750 cfs Wet Year Predictive Scenario
Sand Size Particles Deposited on October 1
1,250 cfs Wet Year Predictive Scenario

Figure B.D.20
Sand Size Particles Deposited on September 30
1,250 cfs Wet Year Predictive Scenario

Figure B.D.21
Sand Size Particles Deposited on October 1
2,000 cfs Wet Year Predictive Scenario

Figure B.D.22
Sand Size Particles Deposited on September 30
2,000 cfs Wet Year Predictive Scenario

Figure B.D.23
Sand Size Particles Deposited on October 1
5,000 cfs Wet Year Predictive Scenario
Sand Size Particles Deposited on September 30
5,000 cfs Wet Year Predictive Scenario
Sand Size Particles Deposited on October 1
500 cfs Dry Year Predictive Scenario

Figure B.D.26
Figure B.D.27

Sand Size Particles Deposited on September 30
500 cfs Dry Year Predictive Scenario
Sand Size Particles Deposited on October 1
750 cfs Dry Year Predictive Scenario

Figure B.D.28
Sand Size Particles Deposited on September 30
750 cfs Dry Year Predictive Scenario

Figure B.D.29
Sand Size Particles Deposited on October 1
1,250 cfs Dry Year Predictive Scenario

Figure B.D.30
Sand Size Particles Deposited on September 30
1,250 cfs Dry Year Predictive Scenario
Sand Size Particles Deposited on October 1
2,000 cfs Dry Year Predictive Scenario

Figure B.D.32
Sand Size Particles Deposited on September 30
2,000 cfs Dry Year Predictive Scenario
Sand Size Particles Deposited on October 1
5,000 cfs Dry Year Predictive Scenario

Figure B.D.34
Sand Size Particles Deposited on September 30
5,000 cfs Dry Year Predictive Scenario

River Station

Sand Deposited (tons)
Sand Size Particles Deposited on October 1
500 cfs Median Year Predictive Scenario

Figure B.D.36
Sand Size Particles Deposited on September 30
500 cfs Median Year Predictive Scenario

Figure B.D.37
Sand Size Particles Deposited on October 1
750 cfs Median Year Predictive Scenario

Figure B.D.38
Sand Size Particles Deposited on September 30
750 cfs Median Year Predictive Scenario

Figure B.D.39
Sand Size Particles Deposited on October 1
1,250 cfs Median Year Predictive Scenario

Figure B.D.40
Sand Size Particles Deposited on October 1
2,000 cfs Median Year Predictive Scenario

Figure B.D.42
Sand Size Particles Deposited on September 30
2,000 cfs Median Year Predictive Scenario

Figure B.D.43
Sand Size Particles Deposited on October 1
5,000 cfs Median Year Predictive Scenario

River Station

Sand Deposited (tons)
Sand Size Particles Deposited on September 30
5,000 cfs Median Year Predictive Scenario

Figure B.D.45
Effects of Design Flow From Pre-Depositional Case
500 cfs Pulse Flow Wet Year Simulation

Water Surface Change (ft)

Freeboard (ft)

River Station

Figure B.D.46
Effects of Design Flow From Pre-Depositional Case
750 cfs Pulse Flow Wet Year Simulation

Figure B.D.47
Effects of Design Flow From Pre-Depositional Case
1,250 cfs Pulse Flow Wet Year Simulation

Figure B.D.48
Effects of Design Flow From Pre-Depositional Case
2,000 cfs Pulse Flow Wet Year Simulation

Figure B.D.49
Figure B.D.50

Effects of Design Flow From Pre-Depositional Case
5,000 cfs Pulse Flow Wet Year Simulation

River Station

Water Surface Change (ft)

Freeboard (ft)

Negative Freeboard

Water Surface Change

Figure B.D.50
Effects of Design Flow From Pre-Depositional Case
500 cfs Pulse Flow Dry Year Simulation

Figure B.D.51
Effects of Design Flow From Pre-Depositional Case
750 cfs Pulse Flow Dry Year Simulation

Figure B.D.52
Effects of Design Flow From Pre-Depositional Case
1,250 cfs Pulse Flow Dry Year Simulation

Figure B.D.53
Effects of Design Flow From Pre-Depositional Case
2,000 cfs Pulse Flow Dry Year Simulation

Figure B.D.54
Effects of Design Flow From Pre-Depositional Case
5,000 cfs Pulse Flow Dry Year Simulation
Effects of Design Flow From Pre-Depositional Case
500 cfs Pulse Flow Median Year Simulation

Figure B.D.56
Effects of Design Flow From Pre-Depositional Case
750 cfs Pulse Flow Median Year Simulation

Figure B.D.57
Effects of Design Flow From Pre-Depositional Case
1,250 cfs Pulse Flow Median Year Simulation

Water Surface Change (ft)
Freeboard (ft)

River Station

Figure B.D.58
Effects of Design Flow From Pre-Depositional Case
2,000 cfs Pulse Flow Median Year Simulation

Figure B.D.59
Effects of Design Flow From Pre-Depositional Case
5,000 cfs Pulse Flow Median Year Simulation

Figure B.D.60
At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.