Three Dimensional Analysis of the Final Design of Pier Extensions and West Guide Wall to Mitigate Local Scour Risk at the BNSF Railroad Bridge Downstream of the Prado Dam Supplemental Report

June 2016
Three Dimensional Analysis of the Final Design of Pier Extensions and West Guide Wall to Mitigate Local Scour Risk at the BNSF Railroad Bridge Downstream of the Prado Dam Supplemental Report

by

S.A. Lottes, N. Sinha, and C. Bojanowski
Transportation Research and Analysis Computing Center (TRACC)
Energy Systems Division, Argonne National Laboratory

K. Kerenyi
Turner-Fairbank Highway Research Center
Federal Highway Administration
U.S. Department of Transportation

J.A. Sharp
U.S. Army Corps of Engineers
Engineering Research and Development Center Coastal and Hydraulics Lab

June 2016
Table of Contents

1. Introduction and Objectives........................................................................................................... 4
   1.1. Introduction......................................................................................................................... 4
   1.2. Objectives.......................................................................................................................... 5
2. Computational Model Geometries, Physics, and Case Set .......................................................... 7
   2.1. Computational Model Physics ............................................................................................ 9
   2.2. Final Construction Design Analysis Cases........................................................................ 10
3. Results and Discussion.................................................................................................................. 11
   3.1. Effect of Increased Size of Pier Extensions and West Guide Wall Position Changes Due to
        Constructability Issues on the Local Scour Risk...................................................................... 11
   3.2. Modeling Three Dimensional Scour with Final Design of Pier Extensions ...................... 19
      3.2.1. Basic Scour Model Bed Displacement Rate Equations................................................. 20
      3.2.2. Scour Modeling Results............................................................................................... 22
4. Conclusions .................................................................................................................................... 29
5. References ..................................................................................................................................... 31
List of Figures

Figure 1.1: Aerial view of bridge with golf course on left (west) and housing on right (east) Riverside county, CA, 33°52'30.83” N and 117°40'02.92” W. Google Earth. April 4, 2014. Accessed: March 03, 2015. .................................................................................................................. 4

Figure 1.2: View of BNSF bridge from upstream side looking south showing 1938 piers. Source: Riverside county, CA, 33°52'30.83” N and 117°40'02.92” W. Google Earth. April 4, 2014. Accessed: March 03, 2015. .................................................................................................................. 5

Figure 1.3: Existing piers (left) and existing piers with enclosures and extensions (right) .......... 6

Figure 2.1: Computational domain shown in yellow border with surrounding region and different low flow channel paths that were tested upstream of the bridge. Source: Riverside county, CA, 33°52'30.83” N and 117°40'02.92” W. Google Earth. April 4, 2014. Accessed: March 03, 2015. .................................................................................................................. 7

Figure 2.2: Current topology with elevation contours................................................................. 8

Figure 2.3: Recommended west guide wall design and construction design............................... 8

Figure 2.4: Typical computational domain with boundary condition types labeled.................... 9

Figure 3.1: Bed Shear Stress for Original Design Recommendation............................................. 12

Figure 3.2: Constructible Design with Pier Extension Top at 428 foot Elevation ......................... 12

Figure 3.3: Constructible Design with Pier Extension Top at 420 foot Elevation............................ 13

Figure 3.4: Constructible Design with Pier Extension Top at 428 foot Elevation (Regraded lower on east side to a low of 412 foot elevation) .......................................................... 13

Figure 3.5 Constructible Design with Pier Extension Top at 428 foot Elevation (10K flood) ...... 14

Figure 3.6: Velocity magnitude at 424 ft. elevation for pier extensions with top at 428 ft. elevation ........................................................................................................................................... 15

Figure 3.7: Velocity magnitude at 424 ft. elevation for pier extensions with top at 420 ft. elevation ........................................................................................................................................... 15

Figure 3.8: Velocity between pier groups on slice across river at bridge just downstream of pier extensions with tops at 428 foot elevation (30K cfs). .................................................................................. 16

Figure 3.9: Velocity between pier groups on slice across river at bridge just downstream of pier extensions with tops at 420 foot elevation (30K cfs) .................................................................................. 16

Figure 3.10: Mean velocity between flow sections under the bridge with the construction design for two pier extension elevations........................................................................................................... 17
Figure 3.11 Flow rate between flow sections under the bridge with the construction design for two pier extension elevations

Figure 3.12 Mean velocity between flow sections under the bridge with the construction design comparing current grade to a lower grade on the east side at about 412 feet at the lowest

Figure 3.13 Flow rate between flow sections under the bridge with the construction design comparing current grade to a lower grade on the east side at about 412 feet at the lowest

Figure 3.14: Model domain for scour simulation with large flow computation area and smaller region around piers where scour is computed and the sub-domain remeshed

Figure 3.15 Scour simulation sequence initial topology at the top, two intermediate scour states and final completed scour for 30k cfs flood for case 2 with pier extension top at 428 foot elevation

Figure 3.16 Scour simulation sequence initial topology at the top, two intermediate scour states and final completed scour for 30k cfs flood for case 2 with pier extension top at 420 foot elevation

Figure 3.17 Pier Extension Top at 428 foot elevation, ERDC Experiment results showing bed elevation at end of scour cycle (top), Scour simulation results (bottom)

Figure 3.18 Pier Extension Top at 420 foot elevation, ERDC Experiment results showing bed elevation at end of scour cycle (top), Scour simulation results (bottom)
1. Introduction and Objectives

1.1. Introduction

This report is a supplement to a previous report [ref] covering optimization of wedge shaped pier extensions to streamline large bluff body piers as a local scour countermeasure for the Burlington Northern and Santa Fe (BNSF) Railroad Bridge over the Santa Ana River downstream of Prado Dam in Riverside County, CA. The optimized design was tested in a 1/30 scale physical model at U.S. Army Engineer Research and Development Center (ERDC) in Vicksburg, MS, and the optimized design was used as the base for the construction design. Constructability issues having to do with both materials and site conditions including access underneath the BNSF bridge yielded a construction design that required making the pier extensions wider and either moving the western curve of the west guide wall upstream or changing its geometry.

An aerial view of the bridge of the bridge is shown in Figure 1.1 with a housing development to the right on the east and a golf course that serves as a flood plain on the left to the west.

![Figure 1.1: Aerial view of bridge with golf course on left (west) and housing on right (east) Riverside county, CA, 33°52'30.83" N and 117°40'02.92" W. Google Earth. April 4, 2014. Accessed: March 03, 2015.](image)

The large semi-rectangular piers supporting the railroad tracks of the original bridge can be seen in Figure 1.2. Behind them are two additional cylindrical piers, six feet in diameter, constructed in 1955 to widen the bridge, adding two additional tracks. The low flow channel runs between pier groups 4 and 5, the 2nd and 3rd from west to east, and the river in low flow conditions is visible in Figure 1.2. Pier group numbering is also shown in Figure 1.2. In this picture the east side is on the left and the west side is on right because this view from upstream shows the piers clearly. In most or the figures in the report, the upstream (north) is at the top of the figure with east on the right and west on the left, and for those figures the pier group numbers are the same but run from one on the right to six on the left. The presence of the endangered Santa Ana Sucker fish in the river reach creates constraints on the type of counter measures that can be used to protect the bridge, eliminating most of the usual methods, including riprap, which would create a fish passage problem. Stream bed stabilization that could lead to small drop-offs in grade at the TRACC/BNSF Bridge Scour Countermeasure Analysis
boundaries of the stabilized area with degradation scour also creates a fish passage barrier because the Santa Ana Sucker cannot jump.

The U.S. Army Corps of Engineers (USACE), L.A. District, has developed a new countermeasure design to protect the bridge and satisfy environmental constraints preserving the habitat of the endangered Santa Ana Sucker fish. The proposed design is to encase four central sets of piers with driven sheet pile and to construct triangular concrete pier extensions extending from 50 to 200 feet into the upstream from each pier group tapering from a 26 foot width at the piers to 2 foot at the pier extension nose. The design goal is to shift the potential for local pier scour away from the bridge support piers into the upstream and reduce the local scour at the extension nose by using a narrow upward sloping nose that directs the flow upward. The proposed design had been analyzed with two dimensional flow software, but had not been analyzed with three dimensional computational fluid dynamics (CFD) software capable of accounting for the three dimensional effects in the flow created by the non-uniform river bed and surrounding flood plain topology and structures of varying height in the flow. This report documents a full scale three dimensional CFD study of pier extension and guide wall design alternatives and concludes with design recommendations.

![Figure 1.2: View of BNSF bridge from upstream side looking south showing 1938 piers. Source: Riverside county, CA, 33°52'30.83" N and 117°40'02.92" W. Google Earth. April 4, 2014. Accessed: March 03, 2015.](image)

### 1.2. Objectives

The primary objectives of the computational fluid dynamics (CFD) analysis in this work are to check for significant changes in flow and bed shear stress relating to scour risk in the final construction design compared to the previously recommended design of the pier extensions and west guide wall in the previous report [1].

The pier extensions are designed to protect the four middle sets of pier groups across the channel from the risks of local scour during major floods. The major floods analyzed are a 10 year flood event at 10,000 cfs and a 190 year flood event at 30,000 cfs. Flows for flood events from 100 TRACC/BNSF Bridge Scour Countermeasure Analysis
to 200 years are all expected to be managed by releasing water through the Prado dam at the 30,000 cfs rate. The existing and design conditions were tested using a 1/30 scale model at the U.S. Army Engineer Research and Development Center (ERDC) in Vicksburg, MS. A schematic of the pier groups on the right and pier groups with streamlining pier extensions on the upstream side is shown in Figure 1.3.

Figure 1.3: Existing piers (left) and existing piers with enclosures and extensions (right)
2. **Computational Model Geometries, Physics, and Case Set**

The computational model consists of the full scale three dimensional geometries built for the original optimization work [1].

Figure 2.1 shows that computational domain outlined in yellow with a portion of the surrounding region including the housing developments to the east and golf course to the west. The figure also shows the three courses of the channel that were tested in the original work. Only the existing channel was used in the analysis presented in this supplemental report. Its path is shown in blue.

![Computational domain shown in yellow border with surrounding region and different low flow channel paths that were tested upstream of the bridge. Source: Riverside county, CA, 33°52’30.83” N and 117°40’02.92” W. Google Earth. April 4, 2014. Accessed: March 03, 2015.](image)

The size of the CFD domain was not changed for the current analysis. It was originally selected using a process of expanding an initial model until the assumed boundary conditions on the CFD domain were not influencing the results underneath the bridge. Figure 2.2 shows the fitted surface to the merged point cloud data covering the CFD analytical domain.
Figure 2.2: Current topology with elevation contours

Figure 2.3: Recommended west guide wall design (red) and construction design (blue)
2.1. **Computational Model Physics**

The simulation model that was set up for design optimization analysis covered in the parent report [1] was also used for the analysis work reported here. A newer version of the commercial CFD software STAR-CCM+, version 11.02, was used while the earlier work was done using version 9.06. There were no differences in solvers that would affect the numerical results reported. The newer version has additional capabilities in pre and post processing, such as meshing and plotting capabilities, other enhancements, and some additional physics models that were not used in this work. Commercial CFD software was used because it is widely used in industry for engineering design, each version is benchmarked against a large validation problem set, and it can therefore be considered very reliable for general engineering use. As before to complete the work with a reasonable use of resources and time Unsteady Reynolds Averaged Navier-Stokes (URANS) equations that govern fluid flow were solved using the k-Epsilon turbulence model. The details of the model governing equations, assumptions, uncertainties, boundary conditions, solver controls, and convergence criteria can be found in the full report of the previous work [1].

The computational domain is the same as in previous work, and is shown in Figure 2.1 with boundaries and boundary types labeled.

---

**Figure 2.4**: Typical computational domain with boundary condition types labeled
2.2. Final Construction Design Analysis Cases

The cases analyzed are listed in Table 1. Case 1 is the result from the optimized design from the original report [1]. The final construction design was analyzed with two pier extension heights. One had the top of the pier extensions at an elevation of 428 feet, case 2, and the other had an eight foot lower top pier extension elevation of 420 feet, case 3. The pier extensions in the final construction design are wider than in the recommended design [1], a change that was necessary for constructability. The wider extensions reduce flow area under the bridge increasing the flow velocity between pier groups and also increasing the potential for contraction scour. One option for reducing the impact of the increased width of the pier extensions was to reduce the height of the pier extensions sufficiently to restore most of the total flow area under the bridge for the 30,000 cfs flood. This is case 3 with the pier top elevation of 420 feet. The flow is a complex three dimensional flow, and consequently increasing the flow area in the upper portion of the cross section at the bridge does not necessarily lead to a reduction of the flow velocity at the bed where the contraction created by the pier extensions is unchanged. It was hoped that the oncoming flow would adjust to the larger area above the extensions that present less resistance to flow in a way that would reduce velocity and shear at the bed where there is more flow resistance. Because this option could potentially save some construction costs, it was deemed worth analyzing and testing in the physical model.

The region with the greatest scour risk is around pier group five. The existing low flow river channel runs between pier groups four and five and the flow coming off of the flood plain to the west is funneled between pier group 5 and the west guide wall. Creating more cross section flow area at the east pier groups by lowering the flood plain in that area was also analyzed because doing that was relatively inexpensive. This is case 4, and it was run to see if creating more flow area on the east side would reduce scour potential on the west side. That case was not run in the physical model where it would have been much more expensive to try.

Case 5 uses the final construction design for a lower flow flood of 10,000 cfs. It was run simply to check the bed shear with a significantly lower flow flood and verify that it is much lower than for high flow floods as was the case in the initial study [1].

Table 1: Simulation Case Set

<table>
<thead>
<tr>
<th>Cases</th>
<th>Topology</th>
<th>Extension Length</th>
<th>Extension Top Elevation</th>
<th>West Guide Wall</th>
<th>Discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing</td>
<td>~100 ft</td>
<td>428 ft.</td>
<td>Topology conforming</td>
<td>30,000 cfs</td>
</tr>
<tr>
<td>2</td>
<td>Existing</td>
<td>~133 ft</td>
<td>428 ft.</td>
<td>Modified topology conforming</td>
<td>30,000 cfs</td>
</tr>
<tr>
<td>3</td>
<td>Existing</td>
<td>~133 ft</td>
<td>420 ft.</td>
<td>Modified topology conforming</td>
<td>30,000 cfs</td>
</tr>
<tr>
<td>4</td>
<td>Regraded lower on east side</td>
<td>~133 ft</td>
<td>428 ft.</td>
<td>Modified topology conforming</td>
<td>30,000 cfs</td>
</tr>
<tr>
<td>5</td>
<td>Existing</td>
<td>~133 ft</td>
<td>428 ft.</td>
<td>Modified topology conforming</td>
<td>10,000 cfs</td>
</tr>
</tbody>
</table>
3. Results and Discussion

3.1. Effect of Increased Size of Pier Extensions and West Guide Wall Position Changes Due to Constructability Issues on the Local Scour Risk

The unsteady flow governing equations for the cases in Table 1 were solved until a near steady bed shear distribution was reached. Color plots of the bed shear for these cases are shown in the following figures. The shear stress scale was clipped at 15 Pa so that very small areas of much higher shear do not wash out the detail on the bed around the pier extensions and piers. Only the section of the computational domain in the vicinity of the BNSF bridge is shown in shear stress distribution color plots because that is the area of interest.

Figure 3.1 shows the bed shear distribution computed using the design recommendation [1], which was used as the starting point for the construction design. Figure 3.2 shows a color plot of the bed shear using the construction design with the pier extension top elevation at 428 feet. The increased width of the pier extensions reduces the width of the space between pier group 5 and the west guide wall. This reduction results in a higher velocity of water through that zone and an increase in bed shear between the guide wall and pier group 5. The increased bed shear is visible as a red patch of color just to the left of the cylindrical piers next to the west guide wall in Figure 3.2. That increase in bed shear, which is above the critical bed shear, yields an increase in contraction scour in that zone. This increase in contraction scour between pier group 5 and the west guide wall based on the construction design were observed in testing at the US Army Corps of Engineers ERDC facility in Vicksburg MS before this study was done, and the foundation depths in the construction design were adjusted to accommodate the increase in contraction scour.

The bed shear for the 30,000 cfs flood with eight foot lower pier extensions, providing for flow cross section area in the upper (near surface) part of the flow at the bridge is shown in Figure 3.3. The desired reduction in bed shear between pier group 5 and the west guide wall for this case did not occur. Compare the shear in that area to that of Figure 3.2; there is not a significant difference. With a lower pier extension top the pier extensions on both of the east most pier groups two and three are below grade in the current site topology and do not show in the figure.

Figure 3.4 shows bed shear distribution for case 4 where the higher initial elevation was regraded on the east to a low of 412 foot elevation to provide more flow area under the bridge. This change does reduce bed shear significantly on the east side, but unfortunately has little effect on the west side between pier group 5 and the west guide wall. The bed shear there actually appears to be a little higher in that location than for the other cases.
Figure 3.1: Bed Shear Stress for Original Design Recommendation

Figure 3.2: Constructible Design with Pier Extension Top at 428 foot Elevation
Figure 3.3: Constructible Design with Pier Extension Top at 420 foot Elevation

Figure 3.4: Constructible Design with Pier Extension Top at 428 foot Elevation (Regraded lower on east side to a low of 412 foot elevation)
The bed shear for a 10,000 flood is shown in Figure 3.5. As expected bed shear is much lower over nearly all of the area than it is in the 30,000 cfs flood cases. In this case shear between pier group 5 and the west guide wall is much reduced most likely because there is much less flow coming off of the west flood plain that is funneled between pier group 5 and the west guide wall. There is a high shear spot just west of the upstream nose of the pier extension at pier group 5. This would yield some local scour in that location. The higher shear there is likely the result of some flow from the low flow channel crossing slightly west across the nose and separating. This would not happen in the larger floods because flow coming off of west flood plain merging into the main channel tends to push the flow a bit eastward in that case. Some local scour at the pier extension tips is not of concern because that was part of the intention of this countermeasure design: to shift any local scour that would occur upstream away from the piers supporting the bridge.

Figure 3.5 Constructible Design with Pier Extension Top at 428 foot Elevation (10K flood)

Figure 3.6 and Figure 3.7 are velocity magnitude color plots for case 2 with top of pier extensions at 428 foot elevation and case 3 with tops of pier extensions at 420 foot elevation respectively. The plots are on a horizontal slice through the domain at an elevation of 424 feet, which is four feet above the top of the extensions in case 3, so they do not show up in that plot. The velocity magnitudes are very similar at this elevation for these cases, although they are a little lower on the east side as expected. They appear nearly the same between pier group 5 and west guide wall, which is likely due to nearly the same amount of water coming off of the west flood plain that must be conveyed through that zone.
Figure 3.6: Velocity magnitude at 424 ft. elevation for pier extensions with top at 428 ft. elevation

Figure 3.7: Velocity magnitude at 424 ft. elevation for pier extensions with top at 420 ft. elevation
The water velocity is color plotted on two cross sections for the 428 foot and 420 foot pier extension top elevations respectively in Figure 3.8 and Figure 3.9 respectively. The vertical slices are between the upstream cylindrical pier and the rectangular piers cutting through a small portion of the rectangular piers. These plots clearly show significantly higher zones between pier group 5 and the west guide wall, and even beyond it because the west guide wall is overtopped. The local topology and 3D characteristics of the flow have even produced a higher velocity near the bed than in the middle. Without the west guide wall overtopping, the velocity here would be even higher. Armoring the ground behind the west guide wall should protect it from scour. Water coming off the west flood plain contributes to the high velocities in this zone. On the east side lowering the pier extension tops to provide more cross section flow area does appear to lower the velocity between pier groups on the east. This difference is likely enhanced by not having an east flood plain so there is no significant amount of flow from the east that must merge back into the high flow channel at the bridge on the east side.

![Figure 3.8: Velocity between pier groups on slice across river at bridge just downstream of pier extensions with tops at 428 foot elevation (30K cfs)](image)

![Figure 3.9: Velocity between pier groups on slice across river at bridge just downstream of pier extensions with tops at 420 foot elevation (30K cfs)](image)

Plots of section mean velocity and volume flow rate for the cases with 428 foot and 420 foot elevation pier extension tops are shown in Figure 3.10 and Figure 3.11. Section 1 is between the east guide wall on the right and pier group 1; section 2 is between pier group 1 and pier group 2, and so on, with section 5 being the zone between pier group 5 and the west guide wall. Figure 3.10 shows that lowering the tops of the pier extensions yields a slight reduction in mean velocity.
between the sections on the east, but does not lower the velocity at section 5, which was of primary interest. Figure 3.11 shows that lowering the pier extension height does not yield any significant redistribution of flow between sections.

![Velocity between Pier-Sections](image1)

**Figure 3.10** Mean velocity between flow sections under the bridge with the construction design for two pier extension elevations

![Discharge between Pier-Sections](image2)

**Figure 3.11** Flow rate between flow sections under the bridge with the construction design for two pier extension elevations

The same type of velocity and volume flow sections plots comparing case 2 and case 4 with a regraded and lowered high flow channel bed around the eastern pier groups are shown in Figure...
3.12 and Figure 3.13. The first shows that velocity is indeed lowered through the east side sections, but that has no effect on section 5, where a lower velocity is most desired. The second shows that there is some redistribution of flow through the east sections, but none at section 5 where a lower volume flow would help reduce scour there.

Figure 3.12 Mean velocity between flow sections under the bridge with the construction design comparing current grade to a lower grade on the east side at about 412 feet at the lowest

Figure 3.13 Flow rate between flow sections under the bridge with the construction design comparing current grade to a lower grade on the east side at about 412 feet at the lowest
3.2. Modeling Three Dimensional Scour with Final Design of Pier Extensions

Argonne TRACC staff has been working with the Federal Highway Administration for a number of years to develop a three dimensional scour simulation methodology. This effort has been enabled by various advanced automated meshing capabilities and moving mesh capabilities that have been developed for CFD software over the past decade to primarily to enable analysis of fluid structure interaction (FSI) problems. Scour can be viewed and modeled as a FSI problem where flow forces on the bed act through shear and turbulent fluctuations to entrain sediment and displace the bed downward. More details and background of these 3D scour simulation techniques is given in reference [4]. This past winter the capability to compute scour at field scale with complex bed topologies and fairly complex pier and other flow obstruction geometries was developed. This was achieved by programming the bed displacement in an external Python program that uses libraries optimized for 2.5D mapping operations.

The scour computation methodology assumes a quasi-steady flow in the sense that the scour time scale is much longer than the flow time scale. In a series of bed displacement scour iterations, a converged quasi-steady flow is computed for the current bed topology using URANS equations, the same as other analysis presented in this work, see [1] for details. A bed erosion rate is then computed based on a Van Rijn [5] entrainment function, given below, as a function of shear stress above the Shields critical shear stress for the sediment size and an additional term accounting for turbulent fluctuations caused by large eddies. This additional term is a function of the turbulent kinetic energy (TKE) or “k” in the k-Epsilon turbulence model. An effective time step for bed displacement is set to achieve a maximum target bed displacement that depends on the length scale of an estimated maximum scour depth. In this application that maximum target displacement is about 3 inches. The time step for this maximum displacement is around 6 minutes in this BNSF bridge scour case and it can grow longer as the bed shear drops toward the critical value.

The computed bed displacement rate distribution over the bed is exported in a file that is read by the Python program that performs the bed displacement on a tessellation of the bed that typically contains 50,000 to 100,000 facets or more, with a finer mesh in areas of interest. The Python bed displacement program also reads the previous bed geometry file mesh, and outputs a displaced, deformed bed for every scour iteration. The newly incrementally deformed bed is then reimported into the CFD flow field meshing and solver software, and the 3D flow domain is remeshed and a new quasi-steady flow state is computed.

This whole process has been automated to iterate under the control of a Java language control program interface available for the CFD software to perform successive scour iteration until the scour reaches an asymptotic state with nearly zero scour rate, i.e. no significant ongoing sediment transport other than migration of bed forms. The methodology includes a sand slide model that is based on angle of repose. In the cases tested here about 100 scour iterations is sufficient to scour down to an equilibrium scour depth of about 18 feet.

Two BNSF cases were run at full scale with 2 mm and 4.5 mm sediment. The 2 mm was an initial estimate. Data obtained from core samples taken at the site indicated that the mean sediment diameter was around 4.5 mm. Both of these sizes produced scour depths that were close to each other. This is likely because the initial bed shear stress for the 30,000 cfs flood is a factor.
between five and ten times critical over large portions of the bed. As listed in the previous report [1] all of the assumptions and uncertainties are the same for these scour simulations because they rely on the same CFD solution methods and physics models (the effects of vegetation are not accounted for, etc.).

The ERDC physical model tests were run at 1/30 scale and the scour depths were scaled up with a correction factor. A number of the assumptions and conditions for the physical model tests are different than for the full scale scour simulation. In spite of these differences, the scour simulations were remarkably close to the scaled up ERCD results.

3.2.1. Basic Scour Model Bed Displacement Rate Equations

The model uses a variable critical shear stress based on local bed slope from [Brors – 1999]

\[ \tau_c = \frac{\sin(\gamma + \phi)}{\sin(\phi)} \tau_{cfb} \]

\( \tau_{cb} \) is the flat bed critical shear stress and \( \phi \) is the angle of repose of the sand, taken to be 33 degrees. The angle \( \gamma \) is the angle of the bed in the direction of the shear stress with respect to the horizontal given by:

\[ \gamma = \cos^{-1}\left(\frac{\tau_{bed} \cdot \mathbf{g}}{\|\tau_{bed}\| \|\mathbf{g}\|}\right) - \frac{\pi}{2} \]

The river bed displacement rate due to scour is given by a Van Rijn [5] pickup function enhanced with an additional TKE shear term accounting for turbulent fluctuations above the URANS computed bed shear:

\[ \frac{dh}{dt} = 0.00033 \sqrt{\frac{\Delta g d_{50} d^{0.3}}{\theta_s}} \left(\max(0, \frac{\tau_{bed} + \tau_{TKE} - \tau_c}{\tau_c})\right)^{1.5} \]

\[ \Delta = \frac{(\rho_s - \rho)}{\rho} \]

\[ d_s = d_{50} \left(\frac{\Delta g}{\nu^2}\right)^{1/3} \]

\[ \theta_s = 1 - \gamma \]

\( \rho_s \) and \( \rho \) are the densities of the sediment and the water respectively
\( \tau_c \) is the critical shear stress, \( \tau_{bed} \) is the bed shear stress at a point
\( \tau_{TKE} \) is the extra shear generated by turbulent fluctuations
\( g \) is the acceleration due to gravity,
\( d_{50} \) is the median sand diameter, \( \gamma \) is the porosity of the sand, and
\( \nu \) is the kinematic viscosity of water
Variable time step with target maximum bed displacement:

\[ \Delta t = \frac{\Delta y_{\text{max}}}{\frac{dh}{dt}} \]

where \( \Delta y_{\text{max}} \) is the user specified maximum displacement for a scour iteration step.

The scour rate is applied normal to the bed according to:

\[ E_r = \left( \frac{dh}{dt} \right) \frac{A}{\|A\|} \]

Figure 3.14: Model domain for scour simulation with large flow computation area and smaller region around piers where scour is computed and the sub-domain remeshed
3.2.2. Scour Modeling Results

A sequence of four developing scour elevation color plots for case 2, the final construction design with the pier extension top elevation at 428 feet for the 30,000 cfs flood conditions is shown in Figure 3.15. The top scene is the initial condition with existing site topology. With the right most pier extension buried The east most pier extension with top elevation at 428 feet is buried below grade at the initial conditions. Below this scene there are two successively deeper intermediate scour profiles for the bed. In the second frame, the top of the east most pier extension is exposed and portions of the pile encasements are beginning to show. In the 3rd frame most of the pile encasement at pier group 5 has been uncovered. The 4th frame shows the final scour condition for the flood. The deepest scour near the piers occurs just to the left of the pier group 5 encasement between that group and the west guide. This area was identified as having the highest scour risk potential based on its initial elevation and high bed shear stress in the initial quasi-steady state CFD analysis.

A corresponding sequence of developing scour elevation color plots for case 3, which has the pier extension top elevations lowered to 420 foot elevation is shown in Figure 3.16. In this case the tops of both of the east most pier extensions are below grade at the initial condition. In the 2nd frame the east most pier extension top is beginning to be uncovered, and the pile encasement in the west most pier group is also beginning to be uncovered. The scour process is seen to continue in the 3rd frame, and the finish scour condition is shown in the 4th frame. For this case the pier extension top of pier group 3 (2nd from the right) is not completely uncovered due to its lower initial elevation. The scour depths around pier group 5 next to the west guide wall are very similar to those of the case 2, the final construction design. The scour depth immediately downstream of the pile encasement of pier group 5 is however slightly deeper than that of case 2. This result demonstrates that lowering the pier extension tops to create more flow area in the upper portion of the 30,000 cfs flood flow does not reduce conditions leading to scour at the bed in this location. This result may be partly due to the large amount of water coming off the west flood plain and rejoining the main flow on the west side. Eddy sheading from flow over the lower top of the pier extensions may also generate some additional turbulence related scour for this case.

Bed elevation contours labeled with elevations in the near vicinity of pier groups 4 and 5 and the west guide wall are shown in for end scour simulations are shown in Figure 3.17 and Figure 3.18. Figure 3.17 shows results for the 428 foot pier extension top elevation, and Figure 3.18 shows results for the 420 foot pier extension top elevation. Shown above in schematics are preliminary results of scaled up scour elevations from corresponding physical model tests conducted at ERDC. While there are some variations between these two sets of values, they are remarkably close having been obtained through two different means of obtaining the scoured topology for the 30,000 cfs flood conditions.
Figure 3.15 Scour simulation sequence initial topology at the top, two intermediate scour states and final completed scour for 30k cfs flood for case 2 with pier extension top at 428 foot elevation
Figure 3.16 Scour simulation sequence initial topology at the top, two intermediate scour states and final completed scour for 30k cfs flood for case 2 with pier extension top at 420 foot elevation.
Figure 3.17 Pier Extension Top at 428 foot elevation, ERDC Experiment results showing bed elevation at end of scour cycle (top), Scour simulation results (bottom)
Figure 3.18 Pier Extension Top at 420 foot elevation, ERDC Experiment results showing bed elevation at end of scour cycle (top), Scour simulation results (bottom)
4. Conclusions

The construction design includes changes to the recommended design necessary for constructability. These changes include a widening the pier extensions and shifting the west guide wall a small distance upstream. These changes result in an increased contraction scour of approximately two feet primarily between the west guide wall and pier group 5. This increase is mitigated in the final construction design by deepening the foundation of the pier extensions and the pile encasements.

- **The CFD analysis supports the conclusion that the construction design meets the primary goals of the project.**

  These goals were (1) to eliminate nearly all of the local, large scour risk in the 30,000 cfs flood at the rectangular bluff body piers supporting the bridge and displace the remaining greatly reduced local scour risk upstream to the pier extension nose tips away from the pier supports of the railroad bridge, and (2) to avoid placing anything in the channel that would impede fish passage, such as riprap or concrete armoring of the bed under the bridge that could lead to a large sudden drop off at the end of the armored section due to long term head cut.

- **Options analyzed that were intended to reduce contraction scour around pier group 5 on the west would not significantly reduce contraction scour at that location and are therefore not recommended.**

  Increasing the flow cross section area at the pier extensions by lowering their top elevation by eight feet from a 428 to 420 foot elevation does not significantly reduce velocity or bed shear between pier group 5 and the west guide wall where the deepest scour occurs. Regrading the higher elevation ground on the east side in the vicinity of the piers and extensions lowering grade level to a minimum of around 412 feet in elevation to increase flow area under the bridge during floods does not significantly affect volume flow or velocity on the west side between pier group 5 and the west guide wall for the 30,000 cfs flood.

- **The 3D CFD moving bed scour bed simulation results are remarkably close to the scaled up physical model scour tests and therefore increase confidence in those results and add supporting data for the other conclusions.**

  While a number of the assumptions in the physical model tests and the CFD based scour simulations are the same, such as not accounting for erosion resistance of vegetation, there are a number of differences that make the two types of tests fundamentally different in their method for determining the scour distribution caused by a flood. Primary examples of these differences are that the physical model by necessity must be run as a scale model, in this case 1/30 scale. The scour depth results are then scaled back up with a correction factor. Since water is used as the fluid, various non-dimensional parameters do not scale equally. For free surface flows, Froude similitude between model and full scale is normally used, while the Reynolds number is assumed to be large enough to produce similar turbulence behavior in the model and full scale condition. Scaled testing of scour with sediment entrainment and transport introduces additional issues in choosing an appropriate size sand for the tests. In spite of these challenges carefully designed and performed physical model tests do produce good conservative scour results.
In CFD based scour simulation, on the other hand, the model is run at full field scale, so there are no scaling issues to deal with. In addition, sediment size from core samples is used directly in the entrainment rate function, so issues in choosing an appropriate sediment or sand size are minimized. The limitations of using CFD to compute scour at full scale include the need to employ turbulence models in order to run the model in a reasonable amount of time on currently available parallel computer cluster resources. Accuracy is also limited by the accuracy of the sediment entrainment rate function. Developing these functions is still an active area of research, especially for cohesive soils, and there are other limitations and uncertainties in numerical modeling.

Given the differences in physical modeling and CFD scour simulation applied to determine as accurately as currently possible with currently available modeling technology the final scour depth distribution at field scale for given flood conditions, a close correspondence in the results of applying the two approaches increases confidence in both of them.
5. References


