Comparison of Abutment Scour Equations Developed for Alluvial and Cohesive Sediment:
Three Bridge Replacements on Brea Canyon Boulevard over Brea Canyon Channel in Brea, California

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ABSTRACT

Abutment scour calculations assuming alluvial and cohesive material were completed for three bridges on Brea Canyon Blvd over Brea Canyon Channel in Orange County, CA. Preliminary abutment scour calculations assuming alluvial material, resulted in 2.7 to 4.9 meters of theoretical scour. Preliminary geotechnical investigations for the project indicated a layer of Siltstone may limit the theoretical scour but additional investigation was needed to verify the erodibility of the siltstone. Soil samples from each bridge were sent for Erosion Function Apparatus (EFA) testing. Scour was recalculated using equations developed for cohesive bed material with the EFA results, which resulted in abutment scour values of 3.7 to 4.3 meters, only reducing scour at one bridge. This case history demonstrates: 1) the EFA testing quantified the erosivity of the material well, concluding that the material was erosive, 2) Abutment scour equations, regardless of material type, provide conservative results and additional research is needed.

INTRODUCTION

Three bridges are proposed for replacement in Brea, Orange County, California on Brea Canyon Boulevard over Brea Canyon Channel. The existing bridges were constructed between 1920 and 1939 as multi-span slab and T-beam bridges. All three bridges are overtopped during the 100-year event by 0.5 m to 1.1 m, and all of the proposed replacement bridges will pass the 100-year discharge. An equilibrium slope analysis was completed to estimate long-term degradation and determine the thalweg elevation, used as the scour reference elevation, at each new bridge. Preliminary scour calculations for the bridges were completed assuming alluvial material using the National Cooperative Highway Research Program (NCHRP) 24-20 (Ettema et al. 2010) abutment scour equations, recommended in the Hydraulic Engineering Circular (HEC) 18 (Arneson et al. 2012), resulting in 2.7 to 4.9 meters of theoretical abutment scour.

Preliminary geotechnical investigations for the project indicated a layer of Siltstone may limit the theoretical scour. The NCRHP 24-20 equations were developed assuming alluvial material and do not account for cohesive channel bed material. Therefore, soil samples from each bridge location were collected and sent for Erosion Function Apparatus (EFA) testing to determine the critical shear stress of the Siltstone material. The results of this testing were used to recalculate the abutment scour using equations developed for cohesive bed material, the NCHRP 24-15(2) (Briaud et al. 2009) equations, which resulted in abutment scour estimates of 3.7 to 4.3 meters, not a significant difference in theoretical scour.

Two conclusions can be drawn from this case history: 1) the EFA testing quantified the erosivity of the Siltstone material concluding that the material was much more erosive than originally projected, and 2) Both the 24-20 equations (alluvial material) and the 24-15(2) equations (cohesive material) provide conservative abutment scour estimates. Additional refinement and research for abutment scour equations is needed across a variety of material types.

PROJECT BACKGROUND

The existing Brea Canyon Blvd. Bridges 55C0121 (Bridge 1), 55C0122 (Bridge 2), and 55C0123 (Bridge 3) at Brea Canyon Channel in Orange County, California are proposed for replacement. The street name is Brea Boulevard; however, the California Department of Transportation (Caltrans) calls the bridges Brea Canyon Blvd. This paper uses the Caltrans naming for consistency with the bridge inspection reports. Additionally, Caltrans refers to the creek beneath the bridges as Brea Canyon Channel which will be used throughout the report; however; the name of the creek is Brea Canyon Creek.

The existing bridges are located in the northern region of Orange County near the City of Brea, CA, 35 kilometers southeast of Los Angeles, CA as shown in Figure 1 and Figure 2. The existing bridges were constructed in 1920 (Bridge 1), 1930 (Bridge 2), and 1939 (Bridge 3). The original structure at Bridge 1 is a two, 4.1m span cast in place slab supported by unknown foundations. Bridge 2 is a two, 9.1m span T-beam bridge supported with reinforced concrete open-end abutments on concrete piles and Bridge 3 is a three, 9.1m span T-beam bridge also supported with reinforced concrete open-end abutments on concrete piles.

Figure 1. Bridge location map (Google Earth)

Figure 2. Detail of project location. Blue arrow shows flow direction (Google Earth)

All three bridges have had a history of scour at the existing piers and abutments recorded in bridge inspection reports spanning from 1938 to 1981. A large storm event in 1938, resulted in undermining of the Bridge 1 footings, resulting in bridge settlement of 0.61m, 1.2m of scour in the channel at Bridge 2, and up to 1.5 m of scour at Bridge 3. The inspection record in 1980 for Bridge 2 noted the abutment embankment slopes washed out, causing the abutment piles to be exposed for depths of 2.4 m. Bridges 1 and 3 were repaired in 1938 and the channel under bridge 3 was lined with concrete. Scour countermeasures including broken concrete riprap (1939), a steel cutoff wall (1941), and rock riprap (1945) were placed at Bridge 2 to attempt to protect the structure from further scour.

A one-dimensional (1D) unsteady flow model was created using the U.S. Army Corps of Engineers Hydraulic Engineering Center River Analysis System (HEC-RAS) version 5.0.7 for the existing condition based on survey information and LiDAR. The model was run for the 50- and 100-year events, 203 cms and 227 cms, respectively, using hydrographs. The results of the existing conditions model showed Bridge 1 is overtopped by approximately 1.1m for the 100-yr event and approximately 0.9 m for the 50-year event. Bridge 2 is overtopped by approximately 0.5 m for the 100-yr event and Bridge 3 is overtopped by approximately 1.0 m for the 100-yr event and approximately 0.9 m for the 50-yr event. Due to constraints with the roadway alignment and adjacent properties, the proposed bridges must be along the same alignment as the existing bridges; however, the bridges will be lengthened approximately 7.0 m (bridge 1), 17.4 m (bridge 2), and 30.8 m (bridge 3). All of the proposed bridges will be single span precast/prestressed concrete girder bridges and the soffit elevations will be raised 0.3 m for

Bridges 1 and 2 and 0.6 m for Bridge 3. Bridges 1 and 3 will have seat type abutment caps which are supported by 4' diameter Cast-in-Drilled-Hole (CIDH) piles. The remaining bridge, Bridge 2, will also have seat type abutment pile caps. However, the deep foundation for this bridge will be a secant pile wall which consists of alternating primary (unreinforced) and secondary (reinforced) 3' diameter CIDH piles.

The HEC-RAS model was re-run for the proposed bridges. All three proposed bridges will pass the 50-year event discharge, and Bridges 2 and 3 will pass the 100-year without going under pressure flow. Water will be above the soffit elevation of Bridge 3 for less than approximately 10 minutes of the 100-year discharge hydrograph which was determined to be insufficient time for pressure flow scour to develop.

Equilibrium Slope Analysis

The degradation potential and reference elevation for the scour analysis were determined using an Equilibrium Slope Analysis provided by WEST Consultants, Inc. (2021) An Equilibrium Slope Analysis aims to determine the ultimate slope of a channel when sediment transport and sediment supply are in equilibrium. For Brea Canyon Channel, the equilibrium slope was estimated using the equation developed by Mussetter Engineering, Inc. (2008) which computes the maximum stable slope for a given geometry and discharge by combining the relationship between the Froude number with a uniform flow formula. A sensitivity check was performed with 20% higher and lower discharges.

The results of the Equilibrium Slope Analysis indicated the channel is at a state of relative equilibrium and additional channel bed degradation of 0.3 m is possible at each bridge. These results were consistent for the sensitivity analyses. The 0.3m was used as the degradation estimate for bridge design and scour depths should be referenced from the elevations provided in Table 1.

Table 1. Scour reference elevations for each bridge.

Bridge	Reference Elevation (NAVD29, m)
1 (55C0121)	113.7
2 (55C0122)	117.0
3 (55C0123)	122.5

Scour calculations assuming alluvial material

Preliminary scour calculations were completed using the methods outlined in the Hydraulic Engineering Circular (HEC) 18 (Arneson et al. 2012). These calculations were completed for the 100-year event, in lieu of using the check flood for bridge design, as currently required in the State of California by Memos to Designers 16-1 (Caltrans 2017).

The geotechnical engineers for the project provided sieve analysis of the channel bed material at one location in the channel downstream of the three bridges () and for borings taken adjacent to the channel. The median grain size (D_{50}) resulting from these sieve analyses were below 0.2 mm.

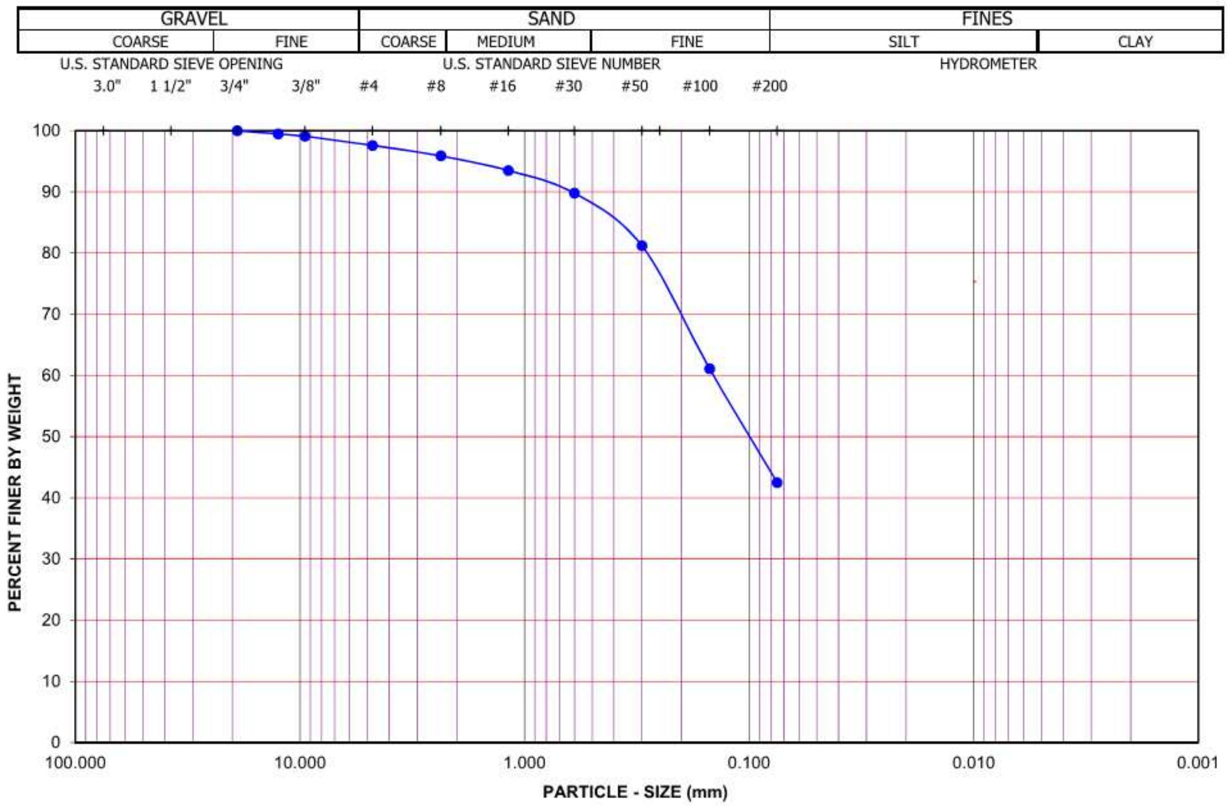

Figure 3. Grain size distribution of the bed material in the channel downstream of the bridges (Leighton 2019).

For contraction scour, HEC-18 notes a reasonable lower limit of the D50 is 0.2 mm as lower grain sizes tend to result in over estimates of the contraction scour. Therefore, this analysis was based on grain sizes of 0.2 mm. The stream is relatively steep through the bridge reach resulting in average upstream velocity in the channel of 2.6 mps for Bridge 1, 4.1 mps is, for Bridge 2, and 2.4 mps for Bridge 3. The Critical Velocity for bed material with a D50 of 0.2 mm is 0.47 mps; thus, the scour conditions will all be Live Bed.

Abutment scour calculations were completed assuming alluvial material, using the National Cooperative Highway Research Program (NCHRP) 24-20 equations for both Condition A (where the abutments are near the main channel) and Condition C (where the abutment fill is assumed to have washed out and the abutments act as piers in the channel). Ultimately, the project team determined the risk to the traveling public of allowing the roadway fill to wash out during a large storm event under Condition C was too great, and a countermeasure to protect the embankment fills, like Rock Slope Protection (RSP) would result in loss of the already limited waterway area. Therefore, discussion of the Condition C results is not included in this paper.

The NCHRP Condition A equations calculate abutment scour as an amplification of the contraction scour as shown in the equation below,

$$y_{max} = \alpha_A y_c$$

where y_{max} is the maximum flow depth resulting from scour, α_A is the amplification factor and y_c is the flow depth including contraction scour calculated using the equation for Live Bed conditions below,

$$y_c = y_1 \left(\frac{q_{2c}}{q_1}\right)^{\frac{6}{7}}$$

where y_1 is the upstream flow depth, q_1 is the upstream unit discharge (discharge divided by channel width) and q_{2c} is the unit discharge in the constricted opening. The abutment scour depth (y_s) is then determined as

$$y_s = y_{max} - y_0$$

where y_0 is the flow depth prior to scour.

Since the contraction scour estimate is based on the relationship of the unit discharge upstream of the bridge and in the contracted bridge reach, these equations are less sensitive to channel velocity. The resulting scour depths for each bridge assuming alluvial material are provided in Table 2.

Table 2. Abutment scour calculations for alluvial material.

Bridge	NCHRP 24-20 Condition A Scour Depth (m)
1 (55C0121)	2.7
2 (55C0122)	4.6
3 (55C0123)	2.7

Geotechnical Considerations and Erosion Function Apparatus (EFA) testing

Geotechnical borings at the locations of the proposed abutments at each of the Brea Canyon Blvd bridges indicated the presence of a layer of Siltstone at elevations 108.2 m (Bridge 1), 118.3 m (Bridge 2), and 119.8 m (Bridge 3) to the depths below the reference elevation shown in Table 3. The presence of this Siltstone layer could indicate the bridge abutments are founded on scour resistant material, or erosion resistant cohesive material. To determine the erosivity and critical shear stress of the supporting material, soil samples from each bridge location were sent to Texas Agricultural and Mechanical (A&M) for Erosion Function Apparatus (EFA) testing.

Table 3. Siltstone depths below the scour reference elevation for each bridge.

Bridge	Reference Elevation (NAVD29, m)	Siltstone Elevation (NAVD29, m)	Depth to Siltstone (m)
1 (55C0121)	113.7	108.2	5.5
2 (55C0122)	117.0	118.3	1.3
3 (55C0123)	122.5	119.8	7.3

The EFA is used to determine the relationship between the shear stress and the erosion rate of the channel bed material. These tests were completed for all of the Brea Boulevard Bridges and the results are summarized by Leighton Consulting, Inc. (2020). According to Briaud et al. 2009, the critical shear stress can be assumed to be the shear stress that results in an erosion rate greater than 0.1 mm/hr.

The EFA results for Bridge 1 are shown in Table 4, for Bridge 2 are in Table 5, and for Bridge 3 are shown in Table 6 provided by Leighton Consulting, Inc. in 2020. The results indicate the critical shear stress is 4.455 Pa for Bridge 1, 5.164 Pa for Bridge 2, and 7.658 Pa for Bridge 3 and the critical velocity is 0.9 mps for Bridges 1 and 2 and 1.18 mps for Bridge 3.

Table 4. EFA results for bridge 1.

Table 1 - Summary of EFA test 11585.005 LB-19 T8 27.5 ft.

Velocity (m/sec)	0.300	0.600	0.900	1.200	1.500	1.980	2.490	3.010	3.580	4.000	4.370
Shear stress (Pa)	0.428	2.025	4.455	10.260	19.125	33.323	66.651	97.396	137.776	172.000	205.292
Erosion rate (mm/hr)	0.100	0.100	0.100	2.609	5.714	8.182	220.000	500.000	480.000	640.000	840.000

Table 5. EFA results for bridge 2.

Table 1 - Summary of EFA test 11585.005 LB-22 T7 26 ft.

Velocity (m/sec)	0.310	0.600	0.900	0.990	1.100	1.210	1.500
Shear stress (Pa)	0.456	2.025	5.164	5.391	6.655	8.053	12.375
Erosion rate (mm/hr)	0.100	0.100	0.100	174.545	360.000	462.857	495.000

Table 6. EFA results for bridge 3.

Table 1 - Summary of EFA test 11585.005 LB-38 T9B 29 ft.

Velocity (m/sec)	0.310	0.600	0.900	1.180	1.300	1.550	1.790	2.000	2.090	2.320
Shear stress (Pa)	0.456	2.025	4.455	7.658	9.295	17.118	34.845	64.000	69.890	86.118
Erosion rate (mm/hr)	0.100	0.100	0.100	0.100	5.455	11.429	105.882	186.667	1140.000	1200.000

Figure 4 shows the graphical results of the erosion rate (mm/hr.) vs. the shear stress for each bridge and indicates the material at Bridges 1 and 3 have medium erodibility and at Bridge 2 has medium to high erodibility.

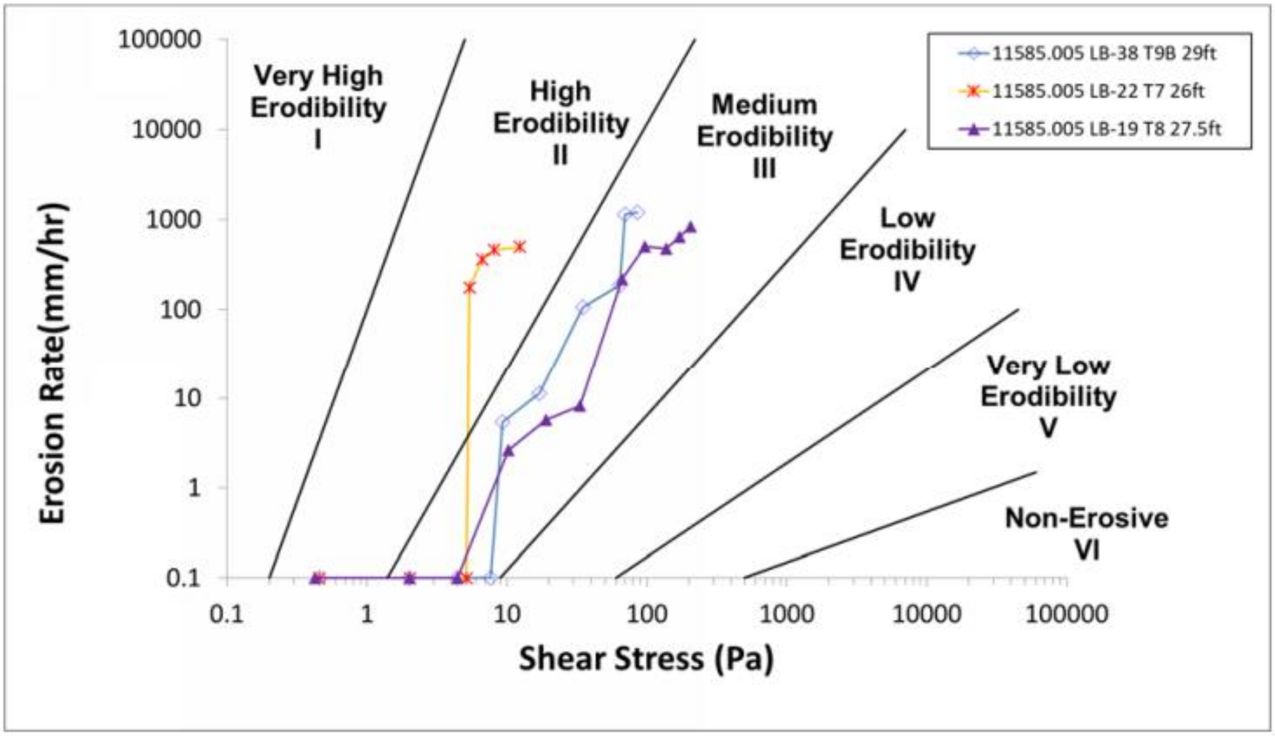

Figure 4. Erosion rate vs. Shear Stress for the material at each bridge (Leighton 2020).

Due to the medium to high erosivity of the material at each bridge, the Siltstone is not likely to be resistant to scour and limit theoretical scour depths. However, the material may have some cohesive properties which could not be accounted for because the NCHRP 24-20 abutment scour equations assume alluvial material.

Scour calculations assuming cohesive material

Based on the results of the EFA testing, abutment scour for each bridge was calculated using the methods described in NCHRP 24-15 (2) for cohesive material (Briaud et al. 2009) to determine if the cohesive properties of the abutment support material could reduce the theoretical scour values. The abutment maximum scour depth equation for cohesive material is below

$$\frac{y_{s\,(abut)}}{y_{f1}} = K_1 \cdot K_2 \cdot K_L \cdot K_G \cdot K_p \cdot 243 \cdot Re_{f2}^{-0.28} \cdot \left(1.65 \cdot Fr_{f2} - Fr_{fc}\right)$$

where, y_s is the abutment scour depth near the abutment toe, y_{f1} is the water depth at the toe of the abutment estimated as the flow depth immediately upstream of the abutment toe, K_1 is the correction factor for abutment shape (1.0 for these bridges), K_2 is the correction factor for the abutment skew (0.85 for these bridges), K_G is the correction factor for channel geometry (0.42 for these bridges), K_L is the correction factor for abutment location (1.0 for these bridges), K_p is the correction factor for pressure flow (1.0 was used for this analysis), Re_{f2} is the Reynolds number around the toe of the abutment defined as $Re_{f2} = \frac{V_{f2} \cdot y_{f1}}{\nu}$ where V_{2f} is the velocity around the toe of the abutment, Fr_{f2} is the Froude number around the toe of the abutment defined as $Fr_{f2} = \frac{V_{f2}}{\sqrt{g \cdot y_{f1}}}$, Fr_{fc} is the critical Froude number around the toe of the abutment defined as $Fr_{fc} = \frac{\sqrt{\frac{\tau_c}{\rho}}}{\sqrt{gny_{f1}^{\frac{1}{3}}}}$ where τ_c is the critical shear stress from the EFA testing, ρ is the mass density of water, and n is the Manning's n-value of the material.

The values for velocity, critical shear stress, Fr_{f2}, and Fr_{fc} used in these equations are included in Table 7 for each bridge.

Table 7. NCHRP 24-15(2) variables for each bridge.

Bridge	Velocity (mps)	τ_c (Pa)	Fr_{f2}	Fr_{fc}
1 (55C0121)	2.6	4.455	0.516	0.007
2 (55C0122)	4.1	5.164	0.539	0.009
3 (55C0123)	2.4	7.658	0.482	0.010

The resulting abutment scour depths are inclusive of contraction scour and are shown in Table 8. This method only resulted in less conservative estimates of abutment scour at Bridge 2.

Table 8. Abutment scour calculations for cohesive material.

Bridge	NCHRP 24-15 (2) Cohesive Material Scour Depths (m)
1 (55C0121)	4.3
2 (55C0122)	3.7
3 (55C0123)	3.7

The NCHRP 24-15 (2) equations are highly dependent on the channel velocity. The equations are based on the Froude Number for the channel velocity multiplied by a factor of 1.65. This value is then subtracted from the Froude Number for the critical velocity (calculated from the critical shear stress) without an additional factor. Due to this, these equations are much less sensitive to the critical shear stress and critical velocity of the channel material and are much more sensitive to the channel velocity.

Scour Considerations for Design

As summarized in Table **9**, the NCHRP 24-20 Condition A equations for alluvial material produced scour values of 2.7 m for Bridges 1 and 3 and 4.6 m for Bridge 2, while the NCHRP 24-15 (2) equations produced for cohesive material produced abutment scour depths of 4.3 for Bridge 1 and 3.7 for Bridges 2 and 3, only reducing the theoretical abutment scour depth at Bridge 2.

Table 9. Abutment scour depth comparison between NCHRP 24-20 and NCHRP 24-15(2).

Bridge	NCHRP 24-20 Condition A Scour Depth (m)	NCHRP 24-15 (2) Cohesive Material Scour Depths (m)
1 (55C0121)	2.7	4.3
2 (55C0122)	4.6	3.7
3 (55C0123)	2.7	3.7

The NCHRP 24-15 (2) report compares the results of the abutment scour depths in cohesive materials to those determined using other abutment scour equations like the NCHRP 24-20 equations. The 24-15(2) report notes only NCHRP Condition B, where the abutments are set back from the main channel and scour occurs in the floodplain, are comparable to the 24-15(2) equations. The results of the comparison indicate the two methods agreed for spill through abutments, but did not agree for wingwalls in front of the abutments. The Brea Canyon Blvd bridges are all located near the main channel (Condition A) and will have wingwall in front of the abutments. Therefore, the applicability of the NCHRP 24-15 (2) equations is limited even if they account for the cohesive nature of the channel material.

Due to the limitations of the NCHRP 24-15 (2) equations and because the NCHRP 24-20 equations are recommended in HEC-18, the results of the NCHRP 24-20 equations were used for the proposed bridge design.

The abutment scour equations using both methods produced significant theoretical scour depths. Deep abutment foundations are required for the proposed bridges to design for this theoretical scour. Bridges 1 and 3 will have seat type abutment caps which are supported by 4’ diameter Cast-in-Drilled-Hole (CIDH) piles. The remaining bridge, Bridge 2, will also have seat type abutment pile caps. However, the deep foundation for this bridge will be a secant pile wall which consists of alternating primary (unreinforced) and secondary (reinforced) 3’ diameter CIDH piles. Cross sections of each proposed bridge are included in Figure 5, Figure 6, and Figure 7.

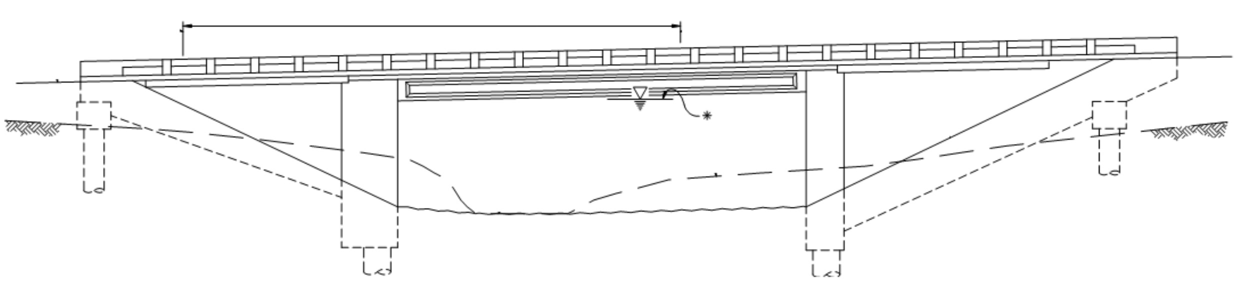

Figure 5. Proposed Bridge 1 profile view. Finish grade channel shown with solid line and original grade with dashed lines.

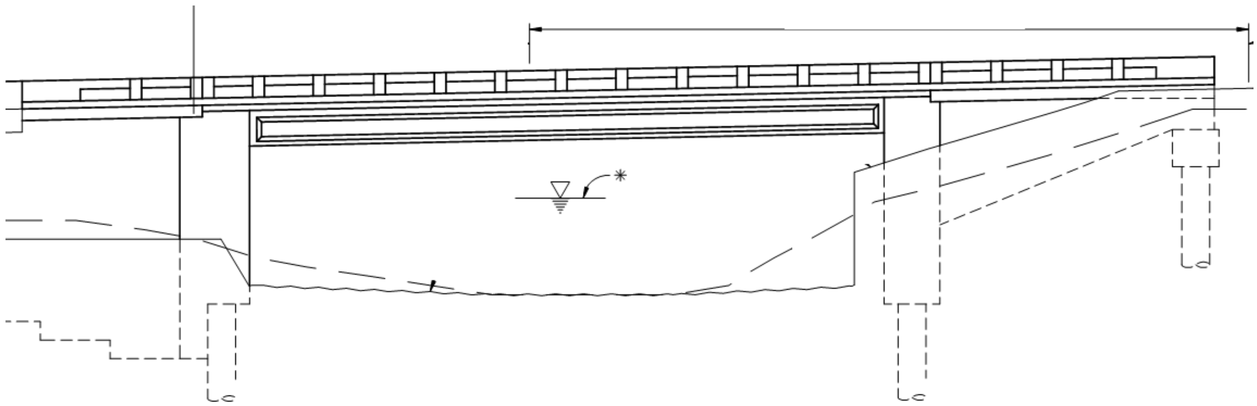

Figure 6. Proposed Bridge 2 profile view. Finish grade channel shown with solid line and original grade with dashed lines.

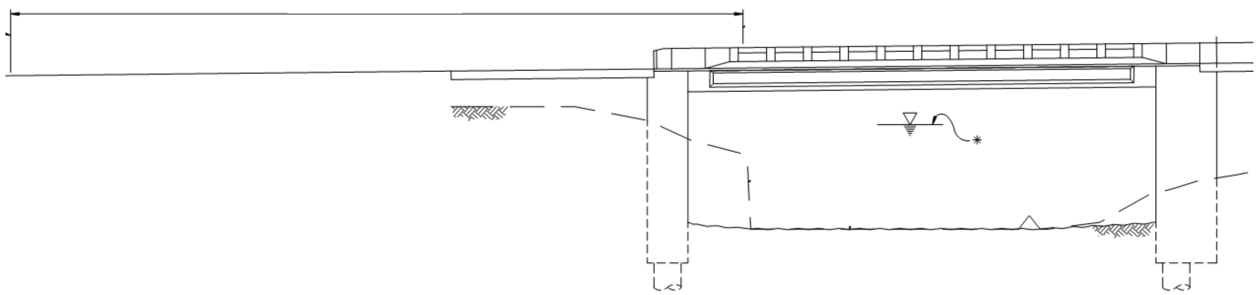

Figure 7. Proposed Bridge 3 profile view. Finish grade channel shown with solid line and original grade with dashed lines.

As shown in Table 10, comparing the abutment scour elevation to the top of the abutment elevation shows the bridge abutments will be 7.9 m to 11.6 m tall.

Table 10. Scour reference elevations for each bridge.

Bridge	Reference Elevation (NAVD29, m)	Abutment Scour Elevation for Design (NAVD29, m)	Approximate Top of Abutment Elevation (NAVD29, m)	Abutment Height (m)
1 (55C0121)	113.7	111	118.9	7.9
2 (55C0122)	117	112.4	124.0	11.6
3 (55C0123)	122.5	119.8	130.5	10.7

Conclusions

Theoretical abutment scour was determined using two methods, the NCHRP 24-20 equations assuming alluvial material, and the NCHRP 24-15(2) methods for cohesive material, for three bridge replacement projects on Brea Canyon Blvd over Brea Canyon Channel in Brea, California. Scour calculations were completed for each bridge using the NCHRP 24-20 equations assuming alluvial material which resulted in scour depths of 2.7 m for Bridges 1 and 3 and 4.6 m for Bridge 2.

Geotechnical investigations at the bridges indicated the abutments will be founded on Siltstone which could have erosion resistant or cohesive properties. The material at each bridge was tested for erosivity, and to determine the critical shear stress and velocity of the material, with the Erosion Function Apparatus (EFA).

Results of the EFA testing indicated the material at the bridges was more erosive than originally envisioned. The material at Bridges 1 and 3 was determined to have medium erodibility and at Bridge 2 was determined to have medium to high erodibility.

The results of the EFA testing were used in the NCHRP 24-15 (2) equations which were developed for cohesive material. These equations produced abutment scour depths of 4.3 for Bridge 1 and 3.7 for Bridges 2 and 3. These depths are only less conservative than the NCHRP 24-20 equations for Bridge 2 and were more conservative at Bridges 1 and 3. Ultimately, the results of the NCHRP 24-20 equations were used for the bridge design. With these scour depths, the proposed bridges will have tall, expensive abutments with deep abutment foundations. The approximate abutment heights required, measured from the top of the abutment to the scour elevation range from 7.9 m to 11.6 m.

Both the NCHRP 24-20 and 24-15(2) reports indicate the need for additional research into, and methods for calculating, abutment scour in a wider number of scenarios. The results of the analysis for the Brea Canyon Creek bridges highlights the particular need for further research into abutment scour for bridges near the main channel with potentially cohesive bed material.

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