



Appendix F – Hydrology and Hydraulics Report

Draft Environmental Impact Report (SCH#2026010287) June 2026



Hydrology and Hydraulics Report:

Euclid Bridge Replacement Project

Euclid Bridge between State Route 71 and Pine Avenue
City of Chino, CA

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1 INTRODUCTION

The project area includes the existing Chino Creek Bridge (Bridge No. 54-1248), which crosses Chino Creek at Euclid Avenue and is located within Caltrans' right-of-way. The current structure is a two span concrete structure and is also subject to overtopping due to insufficient hydraulic capacity. The roadway widens to four lanes prior to the Pine Avenue intersection to the north, and the SR-71 interchange to the south.

Caltrans currently has ownership and maintenance responsibilities for Euclid Avenue (SR-83). A relinquishment of Euclid Avenue from Caltrans to the City is in process and is anticipated to be completed by early 2026, after which the right of way will be owned by the City. The Caltrans right of way along the corridor is approximately 240' wide, which is anticipated to provide adequate space to construct the project's improvements following the relinquishment to the City.

The Bridge was constructed in 2006 and is relatively new with a long service life remaining, but a new bridge is desired to address flooding issues and provide additional roadway capacity.

Several facilities are located near the project limits, including the Prado Olympic Shooting Park, the Prado Regional Park, and Prado Recreation Dog Park.

This project proposes a new four-lane facility above the 50-year Prado Basin pool elevation. It includes the Euclid Avenue Viaduct, a 76'-6" wide, 2,493' long bridge spanning the Prado Basin and Chino Creek. This viaduct will be a precast, prestressed concrete structure supported by cast-in-drilled hole concrete piles. Additionally, the Pomona Rincon Road Undercrossing, also 76'-6" wide, will be a 106' long, cast-in-place, prestressed concrete box girder, allowing future road widening.

2 CRITERIA

The project must comply with the local drainage requirements from the City of Chino, San Bernardino County Flood Control District, as well as regional drainage requirements set by the Army Corps of Engineers (USACE). Because of the anticipated relinquishment, Caltrans' regional drainage requirements set forth in the latest edition of the Highway Design Manual are deemed not applicable. Specifically:

2.1 Local Drainage Requirements (City of Chino)

Analysis includes calculations for onsite areas. Section 19.07.140 of the City of Chino Municipal Code specifies the following drainage design requirements for development:

- Facilities must be designed to minimize inundation of private properties from storm runoff resulting from a 100-year frequency storm.
- Public streets must be protected from flooding due to runoff from a 10-year frequency storm, in accordance with city standards or an approved equivalent.
- Drainage facilities must be designed in accordance with the City's Master Plan of Drainage and applicable elements of the City's General Plan or any adopted specific plan.

2.2 USACE Los Angeles District Requirements

The City of Chino engaged in discussions with the Reservoir Regulation Section for the Pine Avenue Widening Project upstream of this Project. Discussions led to the adoption of a set of requirements by the USACE Los Angeles District. The Project is to comply with the same set of requirements, including:

- The proposed roadway must clear the 50-year pool elevation of Prado Reservoir (543.41 feet NAVD88; 541 feet NGVD29).
- The project must demonstrate that it will not negatively impact the San Antonio River (SAR) Mainstem federal project by conducting a 100-year flood event with a 50-year impoundment at Prado Reservoir for the proposed condition. Note that the joint probability of such a scenario exceeds the 1% Annual Exceedance Probability (AEP).
- The embankment must not result in backwater effects or increased risks upstream of the proposed embankment along the Chino Creek project. Two separate downstream boundary conditions (normal depth and pool condition) will be used to evaluate potential risks upstream of the proposed alignment.

2.3 Prado Dam Improvement Updates

Improvements to Prado Dam were completed in 2022, resulting in the following conditions, which are not part of the design criteria for this project. Please refer to discussions with the USACE Los Angeles District:

- The crest for Prado Dam and top of road elevation for the SR-71 is at Elev. 594.4 (NGVD29) to contain the probable maximum flood (PMF), estimated at Elev. 589.9 feet (NGVD29). In the future, the ogee crest for the spillway will be raised to Elev. 563 (NGVD29).
- The taking line set by the USACE is located at Elev. 566 (NGVD29), with the County of Orange/Orange County Flood Control District (OCFCD) tasked with acquisition. In lieu

of acquisition of existing improvements, dikes have been and/or are being constructed to Elev. 566 (NGVD29) to protect the Yorba Slaughter Adobe, the Corona Sewage Treatment Plant, the California Institution for Women, the Corona National Housing Tract, the Alcoa Aluminum Plant, and River Road. A dike along SR-71 is also in place. The taking line per Plate 7-45 Prado Dam Filling Frequency Curves corresponds roughly to the 190-200-year event.

- According to Plate 7-45, Prado Dam Filling Frequency Curves for Future Conditions and Operations, the 100-year water surface impoundment elevation is approximately Elev. 553 (NGVD29).

The Euclid Avenue Bridge Replacement project falls within the inundation zone of Prado Basin. Prado Basin is located within the Santa Ana River watershed, which encompasses about 2.260 square miles. The Prado Dam flood control reservoir was constructed in 1941 and is operated by the USACE to provide flood control operations for downstream urban areas in Orange County. Four major watercourses drain into Prado Basin, including Chino Creek, Mill Creek, Santa Ana River, and Temescal Creek. Smaller tributaries include, but are not limited to, Cypress Channel. The Basin reaches its full storage capacity at elevation of 563 feet NGVD29 (or 565.4 NAVD88), which is the crest of its spillway. The Basin is currently undergoing restoration to remove accumulated sediments. As of 2008, the operational storage capacity was 174,000 acre-feet.

The proposed bridge would allow the 100-year storm flow from Chino Creek to pass under the proposed bridge, under no pool condition, without adverse impacts to the upstream and downstream hydraulics of Chino Creek. The intent is for the roadway to remain open during a 50-year inundation event of Prado Basin.

3 REGIONAL HYDROLOGY

The tributary drainage areas for Chino Creek and Cypress Channel were delineated using Digital Elevation Model (DEM) raster files from the United States Geological Survey (USGS) National Elevation Dataset (NED). These delineations were subsequently refined based on hydrology maps from the Master Plan of Drainage for the City of Chino, the Master Plan of Drainage Subarea 1 - Chino Sphere of Influence Chino Agricultural Preserve Area, and the Master Plan of Drainage for the City of Ontario. The tributary areas for Chino Creek and Cypress Channel are approximately 96.71 square miles and 9.1 square miles, respectively. The relevant hydrology maps and an exhibit illustrating the two tributary watershed areas are provided in Attachment 1.

The California Streams shapefile from the California Department of Fish and Wildlife dataset was used to display the stream centerlines within the delineated drainage areas. Additionally, some sections of the stream polylines were modified or adjusted based on the DEM raster file obtained from the USGS.

Rainfall

Grid precipitation raster data was obtained from the NOAA Atlas 14 Precipitation Frequency Estimates - Grid metadata file for the California Zone (Volume 6). This data covers precipitation durations of 5 minutes, 30 minutes, 60 minutes, 3 hours, 6 hours, and 24 hours for 10-, 50, 100, and 200-yr storm frequencies. The Zonal Statistics algorithm from the QGIS version 3.34.6 raster analysis toolbox was used to calculate the mean precipitation depth within the delineated watershed boundaries for all the mentioned durations. The unit step of 5 minutes was selected for the unit rainfalls, based on the lag times of Chino Creek and Cypress Channel.

The hydrographs for the 10-, 50-, 100-, and 200-year storm frequencies for Chino Creek and Cypress Channel were generated using Advanced Engineering Software (AES). These hydrographs will serve as upstream boundary conditions for the 2D, unsteady hydraulic analyses. The methods used to develop these hydrographs are detailed in the San Bernardino County Hydrology Manual (Section E).

Basin Factor

Basin factors of 0.03 for the present condition and 0.015 for the future condition were used to generate the lag time for the tributary areas. The two basin factors were based on Table 7-3 of the Phase II GDM. However, the basin factor 0.03 does not accurately represent current basin conditions which contain more impervious areas and drainage improvements.

Per the direction of USACE, methods in the 1988 Design Memorandum No. 1 Phase II GDM report and methods discussed in Section E of the San Bernardino County Hydrology Manual were applied to calculate the drainage characteristics for Chino Creek and Cypress Channel. Plate 7-1 in the GDM report was used to confirm Chino Creek and Cypress Channel drainage areas are located within subarea “Q”. Subarea “Q” in Table 7-3 was used to select the S-graph, basin n-value, and compare the tabulated drainage characteristics for Chino Creek with the values in the table. Plate 7-1 and Table 7-3 from the GDM report are included as part of Attachment 6.

The drainage characteristics for both drainage areas are shown below:

Table 3-1: Hydrology Parameters

Parameters	Chino Creek	Cypress Channel
Length of longest water course “L” (miles)	32.7	8.6
Length of longest water course perpendicular to centroid “Lca” (miles)	12	3.35
Average slope “S” (feet per mile)	270.1	64.1
Present condition - lag time (hours)	2.4	1.17
Future condition - lag time (hours)	1.20	0.59

A sensitivity analysis was previously conducted for the Pine Avenue project, which includes Chino Creek and Cypress Channel sub-watersheds, to illustrate how changes in discharge occur with increases in impervious areas. According to the San Bernardino Hydrology Manual, this analysis used two basin factors, 0.025 and 0.02, reflecting improvements to the drainage systems, reduced vegetation, and increased impervious areas since 1988. No modification to the basin factor is proposed.

The report indicated that as the basin factor decreases by 0.05, the lag time reduces by approximately 20 minutes, while the peak discharge increases by 3% with a basin factor of 0.025 and by 9% with a basin factor of 0.02.

Using the drainage characteristics for subarea “Q” from Table 7-3, the “Valley” S-graph was selected for the Cypress Channel. For the Chino Channel, a combination of “Valley Developed,” “Valley Undeveloped,” “Mountain,” and “Foothill” S-graphs from the San Bernardino Hydrology Manual were used to produce composite unit graphs for the 10-, 50-, 100-, and 200-year existing and proposed condition. The S-graph fractions for Cypress Channel and Chino Creek are listed in the tables below:

Table 3-2: S-graph Fractions for Cypress Channel and Chino Creek Watersheds

Cypress Channel Sub-watersheds	Area (mi ²)	S-graph Fraction
Valley Developed	6.28	0.69
Valley Undeveloped	2.82	0.31

Chino Creek Sub-watersheds	Area (mi ²)	S-graph Fraction
Valley Developed	66.47	0.686
Valley Undeveloped	2.90	0.030
Mountain	26.92	0.278
Foothill	0.42	0.004

The most recent *Southern California Association of Government (SCAG)* database for ArcGIS was used to pull land use information covering the extent of the study. The *Soil Survey Geographic Database (SSURGO)* was used to identify the soil group types within the delineated areas. Land use and soil group types were applied to calculate the impervious values and the storm loss rates. The existing impervious value appears to be 51% for the Cypress Channel and 54% for the Chino Creek sub-watersheds.

The maximum loss rates and low loss rates were produced following methods discussed in Section E.7 of the San Bernardino Hydrology Manual. Both the maximum loss rate and low loss rate were used to determine the effective rainfall for the Chino Creek and Cypress Channel watershed.

Using AES software, the developed and undeveloped valley, mountain, and foothill unit distribution hydrographs were calculated by combining the effective rainfall and the S-graphs of their respective watersheds. Then, the unit distribution hydrographs were multiplied by their valley development fraction to generate a composite hydrograph to find the peak discharge of Chino Creek and Cypress Channel. Peak discharges for the existing condition are summarized in the following table:

Table 3-3: Unit Hydrograph Peak Discharges

Watershed	Peak Discharge (cfs)			
	10-yr	50-yr	100-yr	200-yr
Cypress Channel Watershed	2,879	4,211	4,783	5,631
Chino Creek Watershed	22,942	34,827	42,226	47,501

Refer to the AES computations and results in Attachment 2.

4 LOCAL HYDROLOGY

4.1 Existing Condition

Euclid Bridge

As discussed in the introduction, the existing Euclid Bridge, which spans Chino Creek, is a two-lane roadway running from Pomona Rincon Road to M Combs Way. Due to its insufficient hydraulic capacity, the bridge is susceptible to overtopping and potential closure during larger storm events. This limitation occurs because the bridge was not designed to accommodate the increased flow volumes typically associated with major storms, leading to flooding concerns for both the bridge and the surrounding roadway infrastructure.

The bridge itself is approximately 144.4 feet in length and 46.7 feet in width. The bridge deck features a 2% slope from the centerline to the edges, allowing for drainage of stormwater towards the sides of the bridge. The highest elevation on the bridge deck is assumed to be at the center, with a calculated elevation of 519.52 feet (NAVD 88). The southwest-bound elevation at the centerline is slightly lower, at 519.11 feet (NAVD 88), while the northeast-bound elevation at the centerline is 519.44 feet (NAVD 88). The minimum elevation of the low chord of the bridge is recorded at 511.45 feet (NAVD 88). Concrete barriers are installed along both sides of the bridge and prevent stormwater from flowing directly into Chino Creek. As a result, water must flow along the length of the bridge, either southwestward or northeastward, before eventually draining into Chino Creek. This design limits the natural drainage path, causing stormwater to travel a longer distance before reaching the creek. For more detailed information on the bridge's design and specifications, refer to the as-built drawings provided in Appendix 6.

Under normal conditions and smaller storm events, there is no significant runoff from offsite areas directed toward the existing bridge. However, during larger events, the runoff from upstream areas can accumulate, posing a potential risk of overtopping.

Pomona Rincon Road Crossing

Pomona Rincon Road is situated about 0.4 miles southwest of the existing bridge. Sheet flow from the western half of Euclid Avenue, south of the Euclid Avenue and Pomona Rincon Road intersection, flows northwest into an undeveloped area, eventually reaching a basin with a bottom elevation of 520.7 feet (NAVD 88). The stormwater is then carried northeast through an existing 36-inch CMP storm drain beneath the Euclid Avenue and Pomona Rincon Road crossing, eventually discharging into Chino Creek. The drainage area contributing to the existing storm drain at Pomona Rincon Road (Node 402) spans approximately 13.6 acres.

Runoff from the eastern half of Euclid Avenue flows northeast, where it is collected by a gutter along the east side of the avenue. The runoff then continues northeast along Pomona Rincon Road East, toward the dog park, before ultimately discharging into Chino Creek.

4.2 Proposed Condition

This project involves the construction of a new four-lane transportation facility situated above the 50-year Prado Basin pool elevation, ensuring the infrastructure's resilience against significant flooding. A key feature of the project is the Euclid Avenue Viaduct, a substantial bridge that spans 2,493 feet in length and is 76 feet, 6 inches wide. This viaduct will cross over both the Prado Basin and Chino Creek, providing a critical connection in the transportation network. The

structure will be a precast, prestressed concrete bridge designed for long-term durability and supported by cast-in-drilled hole concrete piles, which will provide a strong foundation capable of withstanding environmental pressures. Additionally, the Euclid Bridge will feature a drainage system that uses scuppers to collect stormwater from the bridge deck. The collected water will then flow through pipes to the nearest bent and discharge at the base of a column into a basin adjacent to Chino Creek.

Street improvements are also planned for Pomona Rincon Road and the north side of the proposed Euclid Bridge, both of which aim to maintain existing drainage patterns while enhancing the system's capacity to manage stormwater more effectively (see Drainage Exhibit for the Proposed Condition in Appendix A) . At Pomona Rincon Road, runoff from Subdrainage Area A will be directed into Chino Creek via Line A. Similarly, runoff from Subdrainage Area B, located at the north side of the Euclid Bridge, will be conveyed through the existing catch basin on the street centerline into Chino Creek. The new design incorporates upgraded storm drain infrastructure (Line A) to accommodate larger stormwater volumes, thereby reducing the potential for flooding and ponding. Additionally, runoff from the improved section of Pomona Rincon Road will be collected by two catch basins (Nodes 602 and 201), which will be connected to Line A for discharge into Chino Creek.

To further manage runoff, storm drain lines B and C are proposed to collect street runoff from the east half of main Euclid Avenue and the west half of the main Euclid Avenue, respectively. Line B will discharge into the eastern slope drain, located between Euclid Avenue and the Euclid Avenue Northbound on-ramp, and Line C will discharge into the existing western basin. The runoff from the slope drain will flow northeast and discharge into an elevated inlet on the north side of the slope drain, which will be connected to Line A. Furthermore, another slope drain will be constructed between the southbound on-ramp of Euclid Avenue and the main Euclid Avenue. This slope drain will collect runoff from the area and direct it northeast into another elevated inlet, which will also connect to Line A.

Runoff from the eastern half of Euclid Avenue, up to where the Euclid Avenue northbound on-ramp begins, will flow northeast and continue along the northbound on-ramp. It will then flow through the gutter along the east side of the avenue, where it will be collected at Node 102. This runoff will subsequently be directed into Line A through a lateral storm drain before being discharged into Chino Creek.

Please refer to the Hydrology map for the locations of the Nodes and Drainage Areas. The table below summarizes the Rational Method results for each hydrology node and the associated drainage area for each node.

Table 4-1: Results from AES Rational Method

Subdrainage Area	Hydrology Nodes (collecting areas)	Drainage Area (ac)	100 yr (cfs)	25-yr (cfs)	10-yr (cfs)	Outlet
A	402 (onsite and offsite)	13	39.46	28.93	22.30	Node 403
	403 (onsite)	14.6	44.4	32.83	25.69	Node 404
	404 (onsite)	18.4	59.98	45	35.59	Node 405
	405 (onsite)	18.5	59.98	45	35.59	Node 406 (Chino Creek)

B	1001 (onsite)	2	6.32	4.8	3.86	Node 1002
	1002(onsite)	3.7	11.98	9.11	7.31	Chino Creek

Subdrainage Area A:

- Node 402 will collect water from approximately 0.9 acres of roadway surfaces, generating around 3.32 cfs of flow during a 100-year storm event. Additionally, Node 402 will collect 36.14 cfs of flow from an offsite drainage area of approximately 11.8 acres.
- Flow from Node 402 will be directed to Chino Creek Basin at Node 405 through a 36” pipe (Line A), passing through Nodes 403 and 404.
- Line A collects water from roadways via 24” laterals A-1 and A-2 along the way. Line A will merge with Lateral A-2 at Node 403 and then with Lateral A-1 at Node 404.
- There will be raised inlets at Nodes 403 and 404 to collect slope drains from off-road areas as well as some street runoff that could not be captured by the proposed laterals.

Subdrainage Area B

- Node 1001 will collect street runoff from the east half of the roadway located northeast of the proposed bridge (approximately 2.4 acres, 7.92 cfs of flow during a 100-year storm event) and direct it to an existing catch basin at Node 1002 through an existing 24” storm drainpipe.
- The west half of the roadway runoff (approximately 5.7 acres, 1.8 cfs of flow during a 100-year storm event) will also be collected at the existing catch basin at Node 1002.

Euclid Bridge

As previously noted, the proposed bridge will drain directly into Chino Creek through a system of scuppers and connected drainage lines designed to convey stormwater efficiently from the bridge deck to the creek.

5 LOCAL HYDRAULICS

5.1 Storm Drain Sizing

WSPGW was also utilized to verify that the storm drain structures could successfully capture and convey 100-year storm flows without becoming a pressurized system. WSPG calculations confirm that a 36" RCP will convey the full 58.28 cfs during a 100-year event.

5.2 Slope Drain Sizing

The Slope Drain will be constructed according to the *County of Orange, OC Public Works Standard Plan 1321*. This plan requires the installation of terrace or down drains every 10 feet of vertical height along the slope to effectively manage stormwater runoff, prevent erosion, and ensure proper water flow to discharge points. The design will help maintain slope stability and enhance the efficiency of the drainage system, particularly during heavy rainfall events.

5.3 Catch Basin Sizing

Hydraulic Toolbox version 5.4.0.0 was used to size the stormwater infrastructure, including inlets and catch basins. All onsite runoff is street flow originating from impervious areas, which generate moderate flow volumes during large storm events. Subdrainage Areas 400 and 401 (the open spaces located to the west of the project site and south of Pomona Rincon Road), draining to Node 402, are the only contributing offsite areas that need to be accommodated.

Standard 1.5-foot-long curb inlets (inlets on grade) with local depressions are proposed along the streets at Euclid Avenue—one located north of the proposed bridge and two south of Pomona Rincon Road. Additionally, two more curb inlets (inlets on grade) are proposed along Pomona Rincon Road: a standard 3-foot-long inlet at Node 405 and a standard 1.5-foot-long inlet at Node 601.

Two 3x3-foot catch basins with local depressions (inlets in sag) will be installed: one at the upstream end of Line A (Node 402), and another with risers at the low point of the west slope drain (Node 403). A 7x7-foot catch basin with local depressions (inlets in sag) and a riser will be installed at the low point of the east slope drain on Euclid Avenue (Node 404). The entire Q_{10} flow will be conveyed by the inlets located along the streets on grade, while the Q_{100} flow will be managed by the inlets situated at the topographic low points (inlets in sag).

For further details on sizing and calculations, please refer to Attachments 3 and 4.

5.4 Outlet Energy Dissipation

Hydraulic Toolbox version 5.4.0.0 was used to size the energy dissipation at the Line A outlet. Based on the flow rate, Riprap Class II is proposed downstream of Line A. The Apron layout will have the following characteristics:

- Apron Length: 14 feet
- Apron depth: 2.61 feet
- Apron width (at end): 19.8 feet

Please refer to Attachments 4 for further details.

6 REGIONAL HYDRAULICS

When the approach embankments of a bridge obstruct the floodplain, localized energy losses occur, causing an elevated water surface upstream of the bridge for a given discharge. Chow (1959), in his *Applied Hydrology Handbook*, explains how flow behaves through constrictions. In subcritical flow conditions, a backwater region forms, increasing the flow depth compared to unconfined conditions. The backwater effect is most pronounced upstream of the constriction, with flow velocities and friction losses escalating as the flow moves toward the bridge. Additional energy losses in this region are attributed to higher velocities and the turbulent exchange of momentum in contracting flows. Further energy losses arise from the drag exerted by bridge piers and abutments, as well as from the expansion reach, where flow velocities increase due to the widening flow area. The total energy loss caused by the bridge occurs between the point of maximum backwater upstream and the point of full flow expansion downstream. To standardize the calculation of energy losses at bridges and culverts, the Federal Highway Administration (FHWA) uses Entrance Loss and Exit Loss Coefficients in its 1985 *Hydraulic Design of Highway Culverts* manual. For sudden flow expansions, such as in typical culverts, FHWA recommends an Exit Loss Coefficient of 1.0.

To assess the backwater effects of the proposed bridge alignment in Prado Basin, two-dimensional hydraulic analyses were performed under both existing and proposed conditions, as outlined in the Criteria Section. The USACE Hydrologic Engineering Center (HEC) River Analysis System (RAS) program was used to run the 2D hydraulic models, determining the areas impacted or improved by the proposed alignment throughout the project's floodplain. Water depth, velocity, and water surface elevation result maps were generated in RAS Mapper, allowing for a comparison of both existing and proposed conditions for Euclid Bridge. The hydraulic analysis was conducted using HEC-RAS version 6.4.1, which can be downloaded from HEC's website (<http://www.hec.usace.army.mil/>).

The assumptions made in building the HEC-RAS hydraulic models are outlined below:

6.1 Topography

Two sets of Topographic Triangulated Irregular Network (TIN) data were used for the analysis. The first set was developed from aerial LiDAR photogrammetry data at a scale of 1" = 100' with 1-foot contours, performed by Digital Mapping, Inc. (DMI) in August 2016. The second set, at a scale of 1" = 40' with 1-foot contours, was also provided by DMI and completed in May 2024. The 2024 set was prioritized as the existing topography, and the areas not covered by this version were supplemented with data from the 2016 set.

Terrain Modifications:

Terrain modification tools from RAS Mapper were used to effectively re-create the trapezoidal thalweg of the low-flow channel for Chino Creek and Cypress Channel.

- Existing contours from the LiDAR photogrammetry are noised from existing vegetation and other structures, resulting in model instability. Modified contours were converted into composite Topographic Triangulated Irregular Network (TIN) and then into a raster.
- Contours along the existing Euclid Avenue roadway at the location of the existing bridge, as well as the contours in the Chino Creek channel from approximately 3,000 feet upstream of Pine Avenue to about 600 feet downstream of Pine Avenue, and from approximately 3,600 feet upstream of Euclid Avenue to about 1,700 feet downstream of

Euclid Avenue, were modified to reflect the low-flow elevation in Chino Creek. The low-flow elevation under the existing bridge is 507.59 feet (NAVD88).

- Contours along a small tributary, approximately 209 feet in length, merging into Chino Creek under the bridge, have been modified.
- In the proposed condition, the surface of the alignment is defined by 1-foot proposed contours, generated for the extents of grading shown on the current 65% Design Plans for Euclid Bridge. The modified contours of the overall project site under existing conditions, along with the proposed alignment contours, were combined to create a TIN (Triangulated Irregular Network) file, which was then used to generate the proposed raster terrain.

6.2 Roughness Coefficient

The roughness coefficients, or n-values, were mapped over the extent of the project's floodplain in the vicinity of Pine Avenue using Cowan's method, as supported by USACE. The method consists of selecting a base roughness value for a straight, uniform, and regular channel (n_0). Then the base n-value was modified based on the effect of cross-section irregularity (n_1), variations of the shape and size of the channel's cross section (n_2), relative effect of obstructions (n_3), type and density of vegetation (n_4), and degree of meandering (m_5). A description of Cowan's method and a map showing the distribution of the roughness coefficient over the extent of the study for the existing and proposed condition are included in Attachment 1. The roughness coefficients for the residential areas and the Big-League Stadium located along the Chino Creek levees were selected based on the Flood Runoff Analysis Engineering Design Manual from the USACE.

Roughness coefficient shapefiles were created to assign the appropriate n-values along the extent of the proposed and existing alignment. The roughness coefficient shapefiles were imported into RAS mapper to associate the n-values with the terrain files in the model. In addition, n-values were also assigned for the culverts within the embankments based on the shape, material, and roughness of the culverts.

6.3 Two-Dimensional Mesh

2D flow areas were created as part of the geometry for both the existing and proposed conditions in the hydraulic models. The cell size for both 2D flow areas was set to 30 feet by 30 feet. The boundaries of the 2D flow area are defined by South-Central Avenue to the north, 7,000 feet downstream of Euclid Avenue to the south, SR-71 to the west, and Chino-Corona to the east. Upstream, the model extends approximately 7,800 feet beyond the confluence of Chino Creek in Prado Basin.

Hydraulic structures were incorporated into the 2D flow areas using the SA/2D Area Connection Data Editor for both existing and proposed conditions. The internal hydraulic structures were designated as the 2D bridge structure for both the existing and proposed bridge openings. The Weir structure type was assigned to the two existing culverts at Pine Avenue, using the Normal 2D equation domain as the overflow computation method.

Upstream and downstream of the 2D connection geometries, automatically connect to cells at the respective ends. Modifications to the cells near the 2D connection alignments were made as follows:

Existing Condition: The internal connection cell spacing for 2D connections at Pine Avenue and Euclid Avenue was modified to 10 feet by 10 feet along the extent of the alignments, with 2 times near repeat and 20 feet by 20 feet far spacing for the Pine Avenue culvert. For the Euclid Bridge, the spacing was set to 3 times near repeat and 30 feet by 30 feet far spacing. The internal connection cell spacing for the Cypress Bridge was set to 5 feet by 5 feet, with 5 times near repeat and 20 feet of far spacing.

Proposed Condition: The internal connection cell spacing was set to 10 feet by 10 feet along the extent of the 2D connection alignments at Pine Avenue, with 2 times near repeat and 20 feet by 20 feet far spacing. For the Euclid Bridge, the internal connection cell spacing was set to 15 feet by 15 feet along the extent of the 2D connection alignments, with 5 times near repeat and 20 feet by 20 feet far spacing. The internal connection cell spacing for the Cypress Bridge was set to 5 feet by 5 feet, with 5 times near repeat and 20 feet of far spacing.

In the 2D flow area mesh editor, the cell and face tolerance values remained at their default settings, except for the Manning's n-value. The tabulated n-values were assigned to a unique land cover layer and were associated with the terrain in RAS Mapper.

6.4 Boundary Conditions (Hydraulic Scenarios)

Boundary conditions were included in the models as part of the geometry data to introduce external conditions to the study area. The boundary conditions can be drawn along the external boundaries of the 2D mesh to add or remove flow from the 2D flow area. Once all boundary conditions are drawn, the boundary condition types can be selected under the Unsteady Flow Data Editor. The types of boundary conditions can be used to apply water surface elevations and flow data throughout the 2D flow area.

Upstream Boundary Conditions

- Three storm events—10-year, 100-year, and 200-year—were analyzed to compare the results of the existing and proposed models and to identify backwater effects along the alignments. The relative flow hydrographs for each storm event were applied as upstream boundary conditions for Chino Creek and Cypress Channel, respectively.
- Additional boundary conditions were added to the eastern boundary of the model along El Prado Road to allow flow to exit the 2D flow area. The boundary type was set to normal depth with a friction slope of 0.005 foot/foot.

Downstream Boundary Conditions

Five downstream boundary condition scenarios were developed to assess how different conditions affect the floodplain. These scenarios incorporate the future water surface elevations at Prado Dam. The boundary conditions, which were applied to both the existing and proposed models, are outlined as follows:

Scenario 1: The tailwater is set to the normal depth (ND) of the channel, with a friction slope of 0.0035 foot/foot at the downstream boundary. The friction slope is derived from the bed slope using Manning's equation.

Scenario 2: The 10-year inundation pool is applied as a downstream control in Prado Basin. The Filling-Frequency Curves from the 1988 GDM report were used to determine the 10-year water surface elevation, which is 521.4 feet (NAVD88) or 519.0 feet (NGVD29) for future conditions in Prado Basin.

Scenario 3: The 50-year inundation pool is applied as a downstream control in Prado Basin. The Filling-Frequency Curves from the 1988 GDM report were used to determine the 50-year water surface elevation, which is 543.41 feet (NAVD88) or 541.0 feet (NGVD29) for future conditions in Prado Basin.

Scenario 4: The 100-year inundation pool is applied as a downstream control in Prado Basin. The Filling-Frequency Curves from the 1988 GDM report were used to determine the 100-year water surface elevation, which is 555.4 feet (NAVD88) or 553.0 feet (NGVD29) for future conditions in Prado Basin.

Scenario 5: The SPF condition of the inundation pool is applied as a downstream control in Prado Basin. The Filling-Frequency Curves from the 1988 GDM report were used to determine the water surface elevation at the Prado Dam's spillway, which is 565.4 feet (NAVD88) or 563 feet (NGVD29) for future conditions in Prado Basin.

6.5 Bridge Modeling

Existing alignment

The existing bridge alignment at Chino Creek Channel was modeled as a SA/2D connection, with the bridge parameters incorporated based on the as-built drawings (Attachment 6). The following hydraulic structure inputs were applied:

- The embankment station and elevations were set to the existing terrain, and the profile of the bridge and piers was extracted from the as-built drawings.
- The bridge width of 46.75 feet was based on the as-built plans.
- The HEC-RAS model recommends a weir flow coefficient of 2.6 for calculating flow over a typical bridge deck, which was applied in our model. Additionally, the broad crest weir shape was selected to model the roadway/bridge deck.
- Based on the as-built elevations and distances, three 2.5-foot diameter piers were added to the bridge geometry.
- The Headwater Maximum Elevation is set at 574 feet, based on the critical storage elevation of Prado Dam, as detailed in the Prado Dam Filling-Frequency Curves. Specifically, the "Future Conditions Future Operation Curve" was used as the reference curve (see Attachment 6).
- For low flow computations, the momentum method was utilized. Given that the bridge features three circular piers, a drag coefficient (C_d) of 1.2 was applied to account for the resistance created by the piers.
- For low flow computations, the momentum-based method was employed.
- For high-flow computations, the default energy-based method was used, employing the standard step method to model high-flow conditions.

Proposed alignment

The proposed road and bridge alignment at Euclid Avenue extend from Off-ramp 71 to M Combs Way (the entrance to the Prado Lakes and Regional Parks). The embankment and bridge were modeled as an SA/2D hydraulic structure. Based on the 65% improvement plans (Attachment 6), the proposed alignment consists of a bridge along Chino Creek with an assumed low chord elevation of 536.30 feet, NAVD88 (533.88 feet, NGVD29). The following assumptions were applied to the structure data:

- The elevation of the embankment was based on the proposed surface terrain but may be revised during final design.
- The bridge width of 76.5 feet, as specified in the 65% improvement plans, was used to represent the width of the proposed roadway.
- A weir coefficient of 2.6 and a broad crest weir shape were used to model the proposed bridge and roadway.
- Based on the as-built elevations and distances, nineteen 3-foot diameter piers were added to the bridge geometry.
- The Headwater Maximum Elevation is set at 574 feet, based on the critical storage elevation of Prado Dam, as detailed in the Prado Dam Filling-Frequency Curves. Specifically, the "Future Conditions Future Operation Curve" was used as the reference curve (see Attachment 6).
- For low flow computations, the momentum method was utilized. Given that the bridge features three circular piers, a drag coefficient (Cd) of 1.2 was applied to account for the resistance created by the piers.
- For high flow computations, the energy-based method was employed. Specifically, the Energy Equation using the standard step method was used to accurately model flow conditions during high-flow events.

6.6 Unsteady Flow Analysis Settings

The unsteady flow computation settings were set based on multiple test runs to reduce instability issues and improve accuracy of results. Chapter 6.6 of the HEC-RAS 6.4.1 User Manual provides a description of the 2D flow tolerance options and a discussion about the performance model runs if tolerance options are varied. The following 2D flow analysis calculation options were set for the existing and proposed model runs of all flood events:

Table 6-1: HEC-RAS Computational Parameters

Calculation Options	
Theta (0.6-1)	1
Theta Warmup (0.6-1)	1
Water Surface Tolerance (ft)	0.05
Volume Tolerance	0.05
Maximum Iterations	40
Equation Set	Full Momentum (SWE-ELM)
Initial Condition Time (hrs)	0
Initial Condition Ramp Up Fraction (0-1)	0.1
Number of Time Slices	1

In the Unsteady Flow Analysis window, the computation time step, the output time intervals, and the 2D flow calculation options can be set up. All the runs have a simulation time window of 24 hours, with the start date of July 23, 2018, at 0:00 to 24:00. The computation time step was set to 0.5 seconds, and the output time intervals were set to every 10 minutes. A 0.5 second computation time step provided consistent acceptable results since the small intervals help compute the rapid changes along a channel. Larger time steps were not observed due to its inability to calculate accurate hydraulic results at locations changing dynamically. The duration of one model run would range from 5 hours to 7 hours, depending on the number of iterations

performed for the cells within the 2D grid. The number of iterations were not changed as it would increase the duration of the model run.

The 2D Diffusion Wave equation and the theta value of 1 was selected to ensure consistency with the models used for the Pine Avenue improvements project. According to the memorandum (attachment 7), a theta value of 1 provided more stability for the simulation runs. The ramp up time was not applied in the hydraulic models due to instability issues during the warmup time. The water surface and volume tolerance values increased to 0.05 feet to reduce the number of iterations performed in each run.

6.7 Unsteady Model Calibration

Calibration runs were done to test 2D model assumptions and modify the model's parameters with the goal of producing stable and acceptable results. The following steps were implemented for the calibration of the 2D HEC-RAS models:

1. Verify that the same hydrology data for Chino Creek and Cypress Channel were used for the existing and proposed conditions.
2. Review of the existing and proposed surface terrains to remove berms or high grounds effecting flow along the channels.
3. Review and adjust hydraulic structure geometry inputs to address unrealistic results and model errors in water surface profiles around bridge and culverts.
4. Review and adjust roughness coefficient values based on the conditions of the surface and the channel.
5. Inspect 2D flow area where drastic changes in terrain, water surface elevation, or velocities may occur. Additional cells were added to compute results with less errors.

6.8 Results

The results from the 2D unsteady HEC-RAS models were generated to assess the backwater effects and de-flooding time upstream of the existing and proposed roadway alignments. The 2D hydraulic model includes the 10-year, 100-year, and 200-year storm events, with five downstream boundary condition scenarios at Prado Basin. The model was run for both the existing and proposed conditions at Euclid Bridge.

Normal depth scenarios were used to determine whether the proposed alignment would clear the storm events when there is no pool elevation at Prado Reservoir. Additional scenarios with Prado Dam inundation conditions were created to ensure that no significant backwater effects occur upstream of the proposed embankment and that de-flooding time would not be affected.

A total of thirty separate runs (15 existing and 15 proposed scenarios) were performed to generate result maps showing velocity, depth, and water surface elevation of the inundated area under both existing and proposed conditions. These maps were used to identify sections of the embankment that would be flooded or remain open during the storm events. Additionally, the delta (proposed – existing condition) for depth, water surface elevation, and velocity was calculated in HEC-RAS to show the areas that were either improved or impacted by the proposed embankment. The result maps were imported into QGIS as raster files to create the exhibits. Exhibits for the existing condition and proposed condition can be found in Attachment 1 (Hydraulic Exhibits).

Observation cross sections were added to the Ras Mapper to identify changes in flow and volume accumulation along the Chino Creek channel. The time and peak discharges for both

existing and proposed conditions were generated to determine the peak arrival time as flow travels downstream of the Chino Creek channel.

An electronic copy of the HEC-RAS model is provided under Attachment 8.

The results for all 5 scenarios for a 100-year storm event are summarized in tables below for peak discharge, water surface elevation, and flow velocity, comparing the proposed and existing conditions:

6.9 Peak Discharge

Table 6-2: Peak Discharge Comparison for each Inundation Pool Scenarios

Downstream Scenario	Storm Event	Average Percent Change in Peak Discharge at XS 22+315 $(Q_{PR}-Q_{EX})/Q_{EX}$	Delta Peak discharge at STA 22+315 (cfs)	Peak Discharge Delay $T_{peakPR}-T_{peakEX}$ (min)
Scenario 1 Normal Depth	100-year	4%	1562.4	10
Scenario 2 10-year Inundation	100-year	5.52%	2249.4	30
Scenario 3 50-year Inundation	100-year	3.58%	1469.2	20
Scenario 4 100-year Inundation	100-year	-0.40%	-158.3	0
Scenario 5 200-year Inundation	100-year	-12.13%	-3655.4	10

Scenarios 1, 2, and 3 show increases in peak discharge, with Scenario 2 experiencing the largest increase at 5.52%, although all increases remain below 6%. These increases in discharge suggest that, under moderate inundation conditions, the flow rate is higher in the proposed condition compared to the existing one. However, these scenarios also show corresponding delays in peak discharge. Scenario 2 exhibits the most significant delay at 30 minutes, followed by Scenario 3 at 20 minutes, and Scenario 1 at 10 minutes. The combination of increased discharge and delayed peak timing in these scenarios suggests that flood waves in the proposed condition take longer to reach their peak. This could indicate that, under normal depth or moderate inundation conditions, the flow is more spread out or slowed down, causing a longer buildup before the peak is reached.

In contrast, Scenarios 4 and 5 show decreases in peak discharge. Scenario 5, representing the 200-year inundation, experiences the largest reduction at -12.13%. The decrease in peak discharge in these scenarios suggests that more extreme flood events, such as larger inundation scenarios, may have a mitigating effect on peak flow. This could be due to increased storage capacity, overflow, or other flow distribution factors that help absorb or spread out the water more evenly.

For all scenarios, the project results in no raise in peak discharge along the Chino Creek levees.

6.10 Water Surface Elevation (WSE)

Water surface elevations before and after the proposed changes were compared to assess any potential backwater effects. The backwater effect occurs when water levels rise due to an obstruction or restriction in the flow, causing an upstream increase in water surface elevation. This typically happens when flow is slowed or restricted, leading to a buildup of water upstream of the obstruction.

Table 6-3: WSE Comparison for each Inundation Pool Scenarios

Downstream Scenario	Storm Event	Average Water Surface Elevation at STA 22+315		
		Existing (ft)	Proposed (ft)	Delta (ft)
Scenario 1 Normal Depth	100-year	521.40	522.64	1.24
Scenario 2 10-year Inundation	100-year	522.30	524.24	1.95
Scenario 3 50-year Inundation	100-year	543.43	543.49	0.06
Scenario 4 100-year Inundation	100-year	555.41	555.42	0.01
Scenario 5 200-year Inundation	100-year	568.40	568.43	0.03

Scenarios 1 and 2 show larger increases in water surface elevation (1.24 ft and 1.95 ft), indicating higher water levels under proposed conditions, particularly during the 100-year storm with normal depth and 10-year inundation scenarios. However, the increase does not exceed 2 feet. In contrast, Scenarios 3, 4, and 5 show minimal changes (0.01 ft to 0.06 ft), suggesting negligible impacts on water levels for the 50-year, 100-year, and 200-year inundation scenarios.

In summary, Scenarios 1 and 2 suggest potential backwater effects, with Scenario 2 having a higher likelihood due to the larger increase in water elevation. However, the overall impact on flood risk is relatively minor, especially for extreme flood events, indicating that the proposed bridge is unlikely to significantly affect flood risk upstream. Further evaluation may still be necessary for moderate inundation events.

For all scenarios, the project results in no raise in water surface elevation along the Chino Creek levees.

6.11 Velocities

Flow velocities are influenced by changes in surface elevations and downstream controls (n-depth, and impoundment scenarios). The table below shows the delta velocity (proposed (-) existing) just upstream of the existing Euclid bridge alignment for Scenario 1 and Scenario 3.

Table 6-4: Velocity Comparison for each Inundation Pool Scenarios

Downstream Scenario	Storm Event	Delta Average Velocity at XS 22+315 ($\bar{V}_{PR} - \bar{V}_{EX}$) [ft/s]	Delta Maximum Velocity at XS 22+315 ($V_{maxPR} - V_{maxEX}$) [ft/s]
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Scenario 1 Normal Depth	100-year	0.32	2.16
Scenario 2 10-year Inundation	100-year	-0.33	-0.56
Scenario 3 50-year Inundation	100-year	-0.19	-0.29
Scenario 4 100-year Inundation	100-year	0.04	0.13
Scenario 5 200-year Inundation	100-year	0.03	0.12

The average velocities in Chino upstream of the existing bridge did not show a significant increase. The changes in channel velocities are less than 1 ft/sec when comparing the proposed and existing conditions for both average and maximum velocities. The only exception is the 100-year storm event in Scenario 1, where the maximum velocity increases by more than 2 ft/sec under the proposed condition. However, overall, there are no significant impacts on the flow velocities that would affect the operation of Prado Basin.

Results indicate that:

- In Scenarios 1 and 2, water surface elevations rise by 1.24 ft and 1.95 ft, respectively, indicating potential backwater effects, especially in Scenario 2. These increases suggest that the proposed conditions may slow down the flow, causing a buildup of water upstream. However, these increases do not exceed 2 feet, and the overall impact on flood risk is considered minor.
- At station 22+315, the delay in peak discharge is most pronounced in Scenario 2 (30 minutes), followed by Scenario 3 (20 minutes), and Scenario 1 (10 minutes). This suggests that floodwater takes more time to reach peak flow under moderate inundation conditions. In contrast, Scenarios 4 and 5 show minimal delays (0 to 10 minutes), indicating that peak flow is reached more quickly during extreme inundation events.
- Average velocities in Chino upstream of the existing bridge did not increase significantly, with changes in velocities being less than 1 ft/sec between the proposed and existing conditions for both average and maximum velocities, except in Scenario 1 (100-year storm and normal depth downstream boundary condition), where the maximum velocity increases by more than 2 ft/sec. Despite this, the overall impact on flow velocities is considered minor, with no significant effects on the operation of Prado Basin.

The 2D hydraulic analysis demonstrates that the proposed bridge shows minimal hydraulic impacts in the 50-year inundation of Prado Reservoir. The backwater effects observed in the 100-year peak discharge scenario with normal depth as a downstream control are within a manageable threshold.

For all scenarios, the project results in no increase in velocities along the Chino Creek levees.

7 BRIDGE SCOUR ANALYSIS

A bridge scouring analysis was performed to determine the level of protection that should be considered in the design of both the piers and the embankments. The analysis is based on the final alignment of Euclid Avenue and the bridge design retained by the City during the 65% progress set and the grading plans. The vertical alignment of the bridge is dependent on the following criteria:

1. The *Caltrans Highway Design Manual* suggests that (1) the 10-year WSE be at least two feet below the low chord of the bridge and/or culvert; (2) the low chord of bridge will be above the 100-year WSE during a normal depth scenario.
2. The USACE requires that the top of deck be above the 50-year inundation pool elevation.
3. The design flood for encroachments shall not be less than the flood with a 2-percent chance of being exceeded in any given year.

7.1 Methodology

7.1.1 Soils Characteristics

The geotechnical report prepared by Converse Consultants (January 23, 2017) for the Pine Avenue Extension project was used to estimate the bed material in Chino Creek and determine the grain size, based on the information from boring No. BH-6, which is closest to the Chino Creek location (see Attachment 6 for the report). The shallow top layers consist of clayey silt and alluvium, underlain by alternating layers of sandy clay and silty sand, beginning around 7.5 feet below ground surface (bgs). Groundwater was encountered at 12 feet bgs. According to the report, the bed material is primarily composed of sand (77%) and gravel (22%), with an insignificant content of silt and clay. The average grain size diameter (D50) is 0.09 mm, and the 95th percentile grain size diameter (D95) is 2.36 mm.

7.1.2 General Scour and Armoring

According to HEC 20, general scour begins when a bed sediment particle at critical condition moves due to any slight increase in hydraulic forces acting on it. The critical condition for incipient motion of a bed sediment particle occurs when the impelling drag forces acting on the particle are equally resisted by frictional forces. In addition, armoring of the channel bed can occur when at least five percent of the bed material is larger than the critical size (i.e., D₉₅).

As input data for the analysis, a D₉₅ sediment particle diameter of 2.36 mm was obtained from geotechnical sampling (see Attachment 6). Equation 6.16 from HEC 20 shown below was then utilized to determine the depth of degradation or scour required to form the armor layer with a conservative layer thickness three times the critical particle size for stability.

The depth of degradation was subsequently calculated to be negligible (0.0006 ft) before an armor layer is established.

$$Y_s = y_a \left(\frac{1}{P_c} - 1 \right)$$

Equation 6.16 HEC 20 Stream Stability at Highway Structures

where:

Y_s = Depth of degradation or scour required to form the armor layer, ft

y_a = Thickness of the armor layer, ft

P_c = Percent of material coarser than the critical particle size expressed as a decimal fraction

The above equation assumes an infinite supply of bed sediment in Chino Creek. Due to the presence of the debris basins upstream (see the exhibit 1 in attachment 1 for all the basins locations) that essentially captures a majority of coarse-grained sediments, it is likely that Chino Creek is more sediment-barren than rich. For more accurate results, models with Long Term Sediment Transport equations (for example, HEC-6T) could be used to calculate watershed sediment yields and estimate the amount of bed material transported through Chino Creek over a series of storm events. Results from the model compute a volumetric yield that can translate into a long-term vertical migration value provided a finite sediment supply. Due to the extensive modeling requirements of such simulation, long-term impacts were calculated based on the empirical armoring equation included in HEC-20.

7.1.3 Short-term scour

The main types of scours include contraction scour, local scour (Pier and Abutment Scours), debris scour, and general aggradation/degradation scour.

Contraction scour is generally described as scour that occurs during floods when there is a higher capacity for sediment transport in the bridge opening than what the upstream sediment supply can replace. Within contraction scour, there are the following subsets:

Clearwater and Live-bed Contraction Scour

According to HEC 18, two types of contraction scour can occur in the study area: live-bed or clearwater scour. The type of scour depends on the average cross-sectional velocity during peak discharge. If the average velocity at peak discharge exceeds the calculated critical velocity (as shown in Equation 6.1 below), live-bed scour will occur; otherwise, clearwater scour will be observed.

The width of the floodway varies along the study area: approximately 2,691 ft upstream of the bridge opening (~80 ft upstream at XS 22315), 1,578 ft at the bridge opening, and 2,067 ft downstream. Profile lines were drawn perpendicular to flow vectors to analyze the flow dynamics, revealing noticeable contraction and expansion. These findings support the use of a contraction analysis.

The HEC-RAS model was carefully examined, and the upstream profile line was divided into three sections to refine the analysis for the 100-year event normal depth scenario. These sections include a 708-foot Right Overbank (ROB) located south of the bridge, a 1,608-foot Main Channel, and a 374-foot Left Overbank (LOB) situated to the north of the bridge. This segmentation was implemented to provide more detailed results based on the varying flow patterns along the cross-section.

As shown in Table 7-1, all three sections were identified as experiencing live-bed scour during both storm events. This suggests a potential for upstream sediment to be deposited in the downstream cross-sections. The critical velocity is calculated based on the following equation (*Equation 6.1 HEC 18 Evaluating Scour at Bridges*):

$$V_c = K_u y^{1/6} D^{1/3}$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/s

y = Average depth of flow upstream of the bridge, ft

D = Particle size for V_c , ft

D_{50} = Particle size in a mixture of which 50 percent are smaller, ft

Ku = 11.17

Table 7-1: Average and Critical Cross Sectional Velocities (100-year Event Normal Depth Scenario)

	ROB	Main Channel	LOB
100-yr average velocity (fps)	1.07	3.44	2.48
100-yr critical velocity (fps)	0.94	1.10	0.83

Allowable Velocity Approach (Overbanks)

An allowable velocity approach was used to determine whether scour would occur on the creek overbanks. Aerial imagery as well as multiple site visits to Chino Creek confirm that the banks of the creek are heavily vegetated. The National Engineering Handbook (NEH) specifies an allowable velocity approach in which the maximum permissible canal velocities for various channel boundary materials are tabulated. According to NEH Chapter 8 Table 8-4, the allowable velocity for grass-lined (vegetated) sandy silt channels is 6.0 to 7.0 ft/s. Therefore, it was concluded that degradation of the overbanks is unlikely to occur.

Contraction Scour (Main Channel)

The contraction scour mechanism was considered to have potential scour impact on the existing downstream channel due to the placement of the proposed bridge upstream. As the proposed bridge opening will form a choke point (contraction), where most floodwaters would have to pass through, it was assumed that scour would occur as a result downstream. Once it was determined that the live-bed scour is the primary mechanism of scour in the creek (as explained above), the scour potential in the main channel was examined. The primary driving variable to determine scour depth in the below equation is the ratio between the cross-sectional widths and flows across them (*Equations 6.2 and 6.3 HEC 18 Evaluating Scour at Bridges – Laursen Live-Bed Equation*).

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k1}$$

$$y_s = y_2 - y_0$$

where:

- y_1 = Average depth in the upstream main channel, ft (m)
- y_2 = Average depth in the contracted section, ft (m)
- y_o = Existing depth in the contracted section before scour, ft (m) (see Note 7)
- Q_1 = Flow in the upstream channel transporting sediment, ft³/s (m³/s)
- Q_2 = Flow in the contracted channel, ft³/s (m³/s)
- W_1 = Bottom width of the upstream main channel that is transporting bed material, ft (m)
- W_2 = Bottom width of main channel in contracted section less pier width(s), ft (m)
- k_1 = Exponent determined below

V_o/T	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

- V_o = $(g_o/\Delta)^{1/2} = (gy_1 S_1)^{1/2}$, shear velocity in the upstream section, ft/s (m/s)
- T = Fall velocity of bed material based on the D_{50} , m/s (Figure 6.8)
For fall velocity in English units (ft/s) multiply T in m/s by 3.28
- g = Acceleration of gravity (32.2 ft/s²) (9.81 m/s²)
- S_1 = Slope of energy grade line of main channel, ft/ft (m/m)
- g_o = Shear stress on the bed, (lb/ft²) (Pa (N/m²))
- Δ = Density of water (1.94 slugs/ft³) (1000 kg/m³)

The peak flow values from both the upstream profile line and the profile line at the bridge opening (where the contraction occurs) were used in the live-bed contraction scour calculations. The wider channel upstream of the bridge opening (2,693 ft) compared to the narrower opening (1,530 ft) indicates a contraction of the floodway. This contraction is associated with higher flow rates, as shown in Table 7-2 below. The upstream section has lower flows per unit width than the contraction section, suggesting that velocities increase at the bridge opening due to the narrowing of the floodplain.

This velocity increase enhances the creek's sediment transport capacity, leading to erosion and scour rather than sedimentation. Supporting this, the results from the spreadsheet in Attachment 5 show negligible aggradation (0.0006 ft) and significant degradation (4.85 ft) in the main channel of the creek during the 100-year event.

Table 7-2: Cross Sectional Flows

	Main Channel U/S	Main Channel in Contraction section
100-year flow (cfs)	42,330	42,327
100-year flow per unit width (cfs/ft)	26.32	27.66

Pier and Abutment Scour

According to HEC-18, the primary mechanisms driving local scour are turbulence and vortex formation, which occur due to the disruption of flow caused by obstructions such as bridge piers and abutments. When flow interacts with these structures, complex flow patterns develop, leading to the erosion of bed material around the obstruction.

For local scour at bridge piers, several factors come into play. These include the characteristics of the bed material (such as sediment size and composition), the configuration of the bed, and the flow characteristics (e.g., velocity and turbulence intensity). Additionally, the fluid properties

(such as water density and viscosity) and the geometry of the pier and its footing are critical factors in determining the extent of scour. The shape and size of the pier, as well as its interaction with the flow, influence the strength of the vortices and turbulence generated, which, in turn, impacts the scouring process.

Scour at bridge abutments is influenced by different factors, primarily the geometry and location of the abutment within the channel or floodplain. The alignment of the abutment with the flow direction, its shape, and its proximity to the channel banks play significant roles in the development of scour. As flow encounters the abutment, it may redirect or accelerate, leading to erosion of sediment along the structure's sides.

Importantly, local scour tends to progress vertically to reach the geotechnical failure depth first, which is the depth at which the soil or foundation material can no longer support the structure. Once this critical depth is reached, scour can then extend laterally, eroding the surrounding bed material and potentially undermining the stability of the structure. The rate at which lateral scour occurs depends on various factors, including the stability of the bed material, the flow regime, and the duration of exposure to erosive forces.

The HEC-18 pier scour equation (*Equations 7-1 HEC 18 Pier Scour at Bridges*) is recommended for both live-bed and clear-water pier scour.

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43} \quad (7.1)$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$$\begin{aligned} y_s &\leq 2.4 \text{ times the pier width (a) for } Fr \leq 0.8 \\ y_s &\leq 3.0 \text{ times the pier width (a) for } Fr > 0.8 \end{aligned} \quad (7.2)$$

In terms of y_s/a , Equation 7.1 is:

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 \left(\frac{y_1}{a} \right)^{0.35} Fr_1^{0.43} \quad (7.3)$$

where:

- y_s = Scour depth, ft (m)
- y_1 = Flow depth directly upstream of the pier, ft (m)
- K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
- K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
- K_3 = Correction factor for bed condition from Table 7.3
- a = Pier width, ft (m)
- L = Length of pier, ft (m)
- Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
- V_1 = Mean velocity of flow directly upstream of the pier, ft/s (m/s)
- g = Acceleration of gravity (32.2 ft/s²) (9.81 m/s²)

The length of the pier was determined based on the group of cylinders representing the pier shape. The proposed bridge will have 17 piers and two abutments. Each pier will be circular, with a diameter of 36 inches, resulting in a total pier length of 51 feet. Both the abutments and piers will be oriented parallel to the flow direction, while the roadway deck will be skewed at a 72-degree angle to the river centerline.

For the pier scour calculations, correction factors of 1, 1, and 1.1 were applied for K_1 , K_2 , and K_3 , respectively. K_1 and K_2 account for flow dynamics and the shape of the pier array, while K_3 is used to adjust for site-specific conditions, such as sediment type or flow obstructions. Based on these parameters, the calculated scour depth at the pier is approximately 5 feet. For detailed calculations, please refer to Attachment 5.

Abutment scour was calculated using both the HIRE and Froehlich abutment scour equations. The HIRE equation is applicable when the ratio of abutment length to flow depth exceeds 25, while the Froehlich equation applies when this ratio is less than or equal to 25. In this case, the HIRE equation was applied to both the south and north abutments, resulting in calculated scour depths of 13 feet at the south abutment and 10 feet at the north abutment. Based on maximum velocity results from the HEC-RAS hydraulic model, the terminal supports, Pier 4 on the south side and Abutment 19 on the north side, exhibit flow acceleration consistent with abutment-like behavior. Although Pier 4 is not an actual abutment in the final bridge design, it was used as a surrogate location for scour estimation due to its hydraulic performance in the model configuration. The HIRE abutment scour equation from HEC-18 was applied as follows:

8.6.2 HIRE Abutment Scour Equation

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE) can also be used for estimating abutment scour (FHWA 2001). This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The modified equation, referred to herein as the HIRE equation, is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4 \text{ Fr}^{0.33} \frac{K_1}{0.55} K_2 \quad (8.2)$$

8.7

where:

- y_s = Scour depth, ft (m)
- y_1 = Depth of flow at the abutment on the overbank or in the main channel, ft (m)
- Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment
- K_1 = Abutment shape coefficient (from Table 8.1)
- K_2 = Coefficient for skew angle of abutment to flow calculated as for Froehlich's equation (Section 8.7.1)

For detailed calculations, please refer to Attachment 5.

7.1.4 Long-term Sediment Transport

Vertical Change Scour Analysis

Long term sediment transport analysis was carried out through both sediment transport functions and watershed sediment yield equations.

Because different sediment transport functions were developed under different conditions, a wide range of results can be expected from one function to the other. The most common sediment transport functions were evaluated for applicability, compared to the characteristics of Chino Creek around the proposed Euclid Bridge.

The transport models that were evaluated are:

- Ackers-White (Flume): A function for total sediment load, separating fine sediment transport (related to turbulent fluctuations) and coarse sediment transport (related to net

grain shear). It's applicable to non-cohesive sands >0.04 mm, based on over 1000 flume experiments with sediment beds like plane, rippled, and dune forms. The function doesn't apply to upper-phase transport with Froude numbers >0.8.

- Engelund-Hansen (Flume and Field): A total load predictor for sandy rivers with significant sediment load. Based on flume data for sediments between 0.19 and 0.93 mm, it's been widely tested and matches field data well.
- Laursen-Copeland (Field): A total sediment load predictor combining qualitative analysis, original experiments, and additional data, with applicability for particle sizes ranging from 0.011 to 29 mm.
- Meyer-Peter Müller (Flume): A bed load transport function based on experimental data for rivers with coarse sediment. The transport rate depends on the shear stress difference between the grain and the critical shear stress. It's applicable for sediments between 0.4 and 29 mm, with sediment specific gravity from 1.25 to >4.0, and can be used for well-graded sediments and non-plane bed forms.
- Yang (Field - Sand): Developed under the idea that unit stream power dominates total sediment concentration, based on flume and field data. It covers sediment sizes from 0.062 to 7.0 mm, with sediment concentrations from 10 ppm to 585,000 ppm, across various channel widths, depths, velocities, temperatures, and slopes.

From the above equations, the Engelund-Hansen and Meyer-Peter Müller equations were not applicable due to our $D_{50} = 0.09$ mm particle size. Detailed equations and calculations using the methods mentioned above are provided in Attachment 5. The summary results are as follows:

Table 7-3: Existing vs Proposed Sediment Transport Rates

Transport Function	Existing 100-yr sediment load, lb/s	Proposed 100-yr sediment load, lb/s
Ackers White (flume)	106,314	160,309
Laursen-Copeland (Field)	15,291.28	15,733.61
Yang (field-sand) *	2,118 ppm	1,929 ppm
*Note: The Yang formula provides the total sediment concentration in parts per million (ppm).		

In general, the 100-yr proposed conditions show a higher transport potential around the bridge due to higher velocity than in the existing condition. Depending on sediment availability from upstream, this may indicate long term channel degradation in the vicinity of the proposed bridge.

Also, Sediment yield analysis was used to estimate the amount of bed material that would be transported through Chino Creek. The event-based erosion predictor, Modified Universal Soil Loss Equation – MUSLE was used. The transport functions used above derive a sediment yield rate for a given area based on cross sectional velocity, whereas MUSLE predicts event-based sediment yield using the following equation (English units):

$$Y = 95 \times (V \times Q_p)^{0.56} K \times LS \times C \times P$$

Where,

Y = sediment yield (tons)

V = event runoff volume (acre-ft)

Q_p = event peak runoff rate (cfs)

K = soil erodibility factor

LS = hillslope length-slope factor

C = cover management factor

P = erosion management practice factor

For this study, the 100-year runoff events were analyzed. K and LS were obtained from the California State Water Resources Control Board (SWRCB). C was determined based on the pervious land-uses within the drainage area and referenced against C values used for the MUSLE equation in the Rancho Mission Viejo 2013 Runoff Management Plan. Flow volumes and peak rates (Q and Q_p) were obtained from the hydrologic models. The erosion management practice factor, P , was assumed to be 1.0.

Table 7-4: Sediment Yield

Parameters	Values (100-Year)	Unit
V	38,139.00	acre-ft
Q_p	42,329.67	cfs
K	0.28	-
LS	0.83	-
C	0.95	-
P	1.00	-
Y	3,007,262.00	tons
Annualized yield, tons	30,072.62	tons/100 year

Lateral migration

The approximate location of the Chino Creek banks was tracked using aerial imagery sourced from the USGS EarthExplorer tool. Imagery from the years 1947, 1966, 2014, and 2022 for the project location was imported into QGIS for analysis. The georeferencing tool in QGIS was then used to scale, rotate, and align the imported imagery with the basemap, using a series of reference points such as street intersections and prominent physical landmarks to ensure accurate alignment. Polygons were created to delineate the creek banks on each image, and these delineations were assigned to layers corresponding to the imagery year. This process allowed for an effective analysis of the creek bank movement over time, from 1947 to 2022.

Historically, the creek shifted about 129 feet in a southerly direction from 1947 to 1966. Following this period, and even before significant development activities—such as the construction of the El Prado Golf Course and the Inland Empire Utilities Agency Water Treatment Plant—Chino Creek maintained a relatively stable watercourse. However, subsequent manmade channel modifications, particularly in the reaches upstream of Euclid Avenue, have

confined the creek to its current channel. These modifications have significantly restricted the creek's natural lateral movement, stabilizing its course in those areas.

Despite these upstream modifications, the creek's boundaries near Euclid Avenue have continued to expand. As shown in the exhibits in Attachment 5, the creek's expansion from 2014 to 2022 was estimated at approximately 53 feet (~6.6 ft/year) on the north embankment and 36 feet (~4.5 ft/year) on the south embankment. This ongoing expansion indicates that the creek is still undergoing dynamic changes, primarily driven by scouring and major storm events.

7.2 Results

The short-term and long-term scouring of Chino Creek in the vicinity of the proposed Euclid Bridge was investigated using hydraulic analysis and aerial imagery to aid in the design of the structure. It was determined that vertical changes due to scouring have a high potential to cause significant risks to the foundation of the bridge as well as sediment erosion and deposition downstream. This is especially true for large storm events in the proposed condition as the added imperviousness is likely to result in higher velocities and larger scouring capacity of Chino Creek.

Following the industry-standard methodology outlined in HEC-18 (FHWA, 2012), the total scour depth at the bridge includes the following components:

1. *Long-Term Aggradation/Degradation:* An estimated long-term aggradation/degradation depth of approximately 0.015 feet was predicted based on incipient motion analysis. The analysis indicates that the critical shear stress of the bed material is unlikely to be exceeded during storm events up to the 2-year recurrence interval. With a D_{95} of 2.36 mm, the existing armor layer is expected to provide sufficient resistance against erosive forces from minor storm events, resulting in negligible degradation (estimated at 0.0008 feet).
2. *Contraction Scour:* Contraction scour within the main channel is estimated at approximately 0.35 feet due to the narrowing of the floodway at the bridge crossing. In the south overbank, an estimated 0.15 feet of scour is expected, and approximately 0.1 foot of scour is expected at the north overbank. During the 100-year flood event, the analysis confirms that the bed material will become fully suspended, contributing to increased contraction scour potential.
3. *Local Scour at Piers:* Approximately 5 feet of local scour is expected at the piers due to the eddy viscosities generated at the interface between the piers and the flow.
4. *Abutment Scour:*
 - South Abutment: In the model results, Pier 4 serves as a surrogate location for the south abutment. Flow acceleration near this pier results in conditions consistent with abutment scour. The estimated scour depth at this location is 13 feet.
 - North Abutment: The north abutment is represented in the model by Abutment 19. The hydraulic model indicates flow constriction and acceleration at this location, resulting in an estimated abutment scour depth of 10 feet.

Therefore, the bridge should be designed to accommodate a total scour depth of approximately 7.35 feet in the main channel (0.35 ft of approach flow contraction scour + 7 ft of pier scour), 18.15 feet at the south surrogate abutment location (Pier 4) (0.15 ft of contraction scour + 5 ft of pier scour + 13 ft of abutment scour), and 15.1 feet at Abutment 19 (north side), the actual north abutment (0.1 ft of contraction scour + 5 ft of pier scour + 10 ft of abutment scour).

Although lateral migration of the creek does not seem to be a cause of major concern, it would be prudent to maintain the channelization upstream to minimize prospects of migration and contain Chino Creek within its natural flow path before it reaches the proposed bridge site. The subsequent bridge should be designed to accommodate for the scouring that is expected to occur for a large event such as a 100-year storm in addition to long-term sediment transport effects. In a large storm event, the bridge will be subject to inundation and overtopping (i.e. submergence, horizontal impact loading, and buoyancy). This will be addressed in the Bridge Type Selection Report for the 95% submittal.

8 CONCLUSION

The results from the hydrologic and hydraulic analysis have been performed to validate the design of the bridges, culverts, and storm drain infrastructure that will be incorporated into the Euclid bridge replacement and Euclid Avenue improvements. For this project to be feasible, the design and plans of the proposed Euclid bridge need to comply with stipulations on the city, county, state, and national level.

For the local drainage requirements, the proposed Euclid Bridge demonstrated that the adjacent areas, golf course, and street itself would not become flooded in the 100-year storm event and would successfully convey both onsite and offsite runoff to natural drainage channels with minimal impact during a 10-year storm event.

For regional drainage requirements, the proposed Euclid Bridge demonstrated that the embankments would not interfere with the operation of Prado Reservoir and Dam downstream nor compromise the integrity of the Chino Creek levees upstream. This was done through the 2D HEC-Ras model that concluded that there were insignificant backwater effects, the top of bridge cleared the 50-year inundation levels, and the low chord of the bridge would meet the necessary clearance criteria for the 10- and 100-year hydrographs.

Additionally, scour calculations were conducted to support the structural design of the bridge, ensuring its resilience against both short-term and long-term scour conditions. The analysis concluded that the bridge piers should be designed for a vertical scour depth of 7.35 feet. For the abutments, a vertical scour depth of 18.15 feet was recommended for the surrogate south abutment (Pier 4), and 15.1 feet for the north abutment (Abutment 19). Lateral scour was also evaluated and determined to be significant, warranting further consideration in the final design.

It can thus be concluded that this project fully adheres to the necessary conditions set out by the aforementioned entities appropriate for the 95% plan.

9 REFERENCES

1. *Southern California Association of Government (SCAG) database*
2. *Soil Survey Geographic Database (SSURGO)*
3. *County of Orange, OC Public Works Department Standard Plans*
4. *Caltrans Highway Design Manual, Chapter 820: Cross Drainage, July 1, 2020*
5. *Evaluating Scour at Bridges, fifth Edition, Hydraulic Engineering Circular No. 18 (HEC 18), April 2012*
6. *Stream Stability at Highway Structures, fourth Edition, Hydraulic Engineering Circular No. 20 (HEC 20), April 2012*
7. *Part 654 Stream Restoration Design National Engineering Handbook (NEH), United States Department of Agriculture, Threshold Channel Design, August 2007*