Carmel River Floodplain Restoration and Environmental Enhancement Project

March 2019

Technical Studies that are Bound Separately

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Hydrology Reports

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Prepared For:

United States Fish and Wildlife Service

Monterey County Resource Management Agency

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Anticipated Changes to Downstream BFE Memo

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MEMO

To: Josh Harwayne (Denise Duffy & Associates)

From: Edward Ballman, P.E.

Date: August 26, 2015

Subject: Anticipated Changes in Downstream Base Flood Elevations Due to the Carmel River Floodplain Restoration and Environmental Enhancement Project

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At your request Balance Hydrologics has reviewed the hydraulic modeling analyses prepared earlier for the Carmel River Floodplain Restoration and Environmental Enhancement Project (CRFREE) in order to characterize anticipated changes in downstream flood elevations. The focus of this review has been on Base Flood Elevations (BFEs), also known as 100-year flood elevations, since they are the primary regulatory standard for County ordinances and regulations under the National Flood Insurance Program. Specific areas of interest include various California Department of Parks and Recreation (State Parks) facilities located on the former Odello West property just west of State Route 1 and the treatment plant operated by the Carmel Area Wastewater District (CAWD). This memo summarizes the currently-effective BFE information and anticipated changes in BFE at each location to better characterize project impacts with respect to flood control.

<u>Floodplain Mapping Considerations</u>

As you may be aware, the currently-effective FEMA hydraulic modeling for the Carmel River uses three distinct flow paths to represent the complexities of the river valley west of approximately Rancho Cañada. These include the main channel, north overbank, and south overbank. The various flow paths are used to model scenarios that include all levees remaining intact through the flood event, failure of the south bank levees, and, finally, failure of the north bank levees. For the purposes of the CRFREE project, the main channel and south overbank flow paths are most pertinent, in no small part because the risk of flooding in the north overbank would be markedly reduced through implementation of the project.

Our staff reviewed the FEMA modeling files to confirm that they correspond appropriately to the base flood information shown on the Flood Insurance Rate Map and in the Flood Insurance Study for this reach of the river. That information is an appropriate base case against which to measure any impacts from the CRFREE project. Revised hydraulic modeling using the project geometry was then used to tabulate the predicted post-project BFE values. Predicted flood elevations do change slightly due to increases in the portion of the flood discharge that would be conveyed through the south overbank after the floodplain is restored and the new State Route 1 causeway. These changes are summarized briefly below.

State Parks Properties

State Parks owns and administers Carmel River State Beach, which occupies much of the valley bottom and lagoon area west of State Route 1. This area includes several structures remaining from past agricultural activities on the former Odello West property, including sheds and a barn. These facilities are shown on the attached Figure 1, which also depicts several of the cross-sections from the hydraulic model runs. The entire area with structures shown in Figure 1 is within the south overbank portion of the models. Table 1 below summarizes the predicted base flood elevations at the respective cross-sections for both the pre- and post-project conditions.

Table 1. Predicted base elevations in the vicinity of the State Park barn

	Base Flood Elevation (*ft, NAVD*)		
Cross-section	Pre-project	Post-project	Difference
35+45	18.6	18.4	-0.2
33+11	18.0	17.9	-0.1
30+59	17.2	17.3	0.1
28+40	16.9	17.1	0.2

The model output summarized in Table 1 shows that base flood elevations are predicted to both decrease and increase along the reach with the State Park structures. At the eastern end, nearer the highway, BFEs are shown to decrease slightly due to the fact that under existing conditions floodwaters have to flow over the roadway and do not then have a defined channel down to the lagoon. In the post-project condition the causeway and restored floodplain channel prevent roadway overflow and let water move more efficiently down to the lagoon even though the flow rates are higher. However, far enough to the west, the effect of the increased south overbank flow predominates and there is a slight increase in the post-project case, reaching a maximum of 0.2 feet (2.5 inches) at the western end of the barn structure.

The area immediately adjacent to the barn structure and other outbuildings is subject to shallow flooding under existing conditions, with flood depths generally on the order of one to two feet. Should mitigation be desired for the small increase in base flood elevation, it could readily be achieved through a modest increase in the elevation of the driveway and construction of a low berm or wall structure along the west and south perimeter of the pad area.

CAWD Treatment Plant

From a flood modeling perspective, the CAWD treatment plant is uniquely situated along the border between the main channel and south overbank flow paths. The main channel reach sets the BFEs for the north, east, and west perimeters of the plant. The south overbank reach defines the BFEs for the south side. Predicted base flood information is summarized in Table 2.

Table 2. Predicted base elevations in the vicinity of the CAWD treatment plant

	Base Flood Elevation (*ft, NAVD*)		
Cross-section	Pre-project	Post-project	Difference
Main Channel			
39+86	19.1	18.4	-0.7
35+31	17.9	17.2	-0.7
29+72	16.1	15.8	-0.3
23+75	15.5	15.5	0.0
South Overbank			
29+65	17.0	17.2	0.2
26+34	16.7	16.9	0.2
20+99	16.5	16.6	0.1
14+30	16.2	16.3	0.1

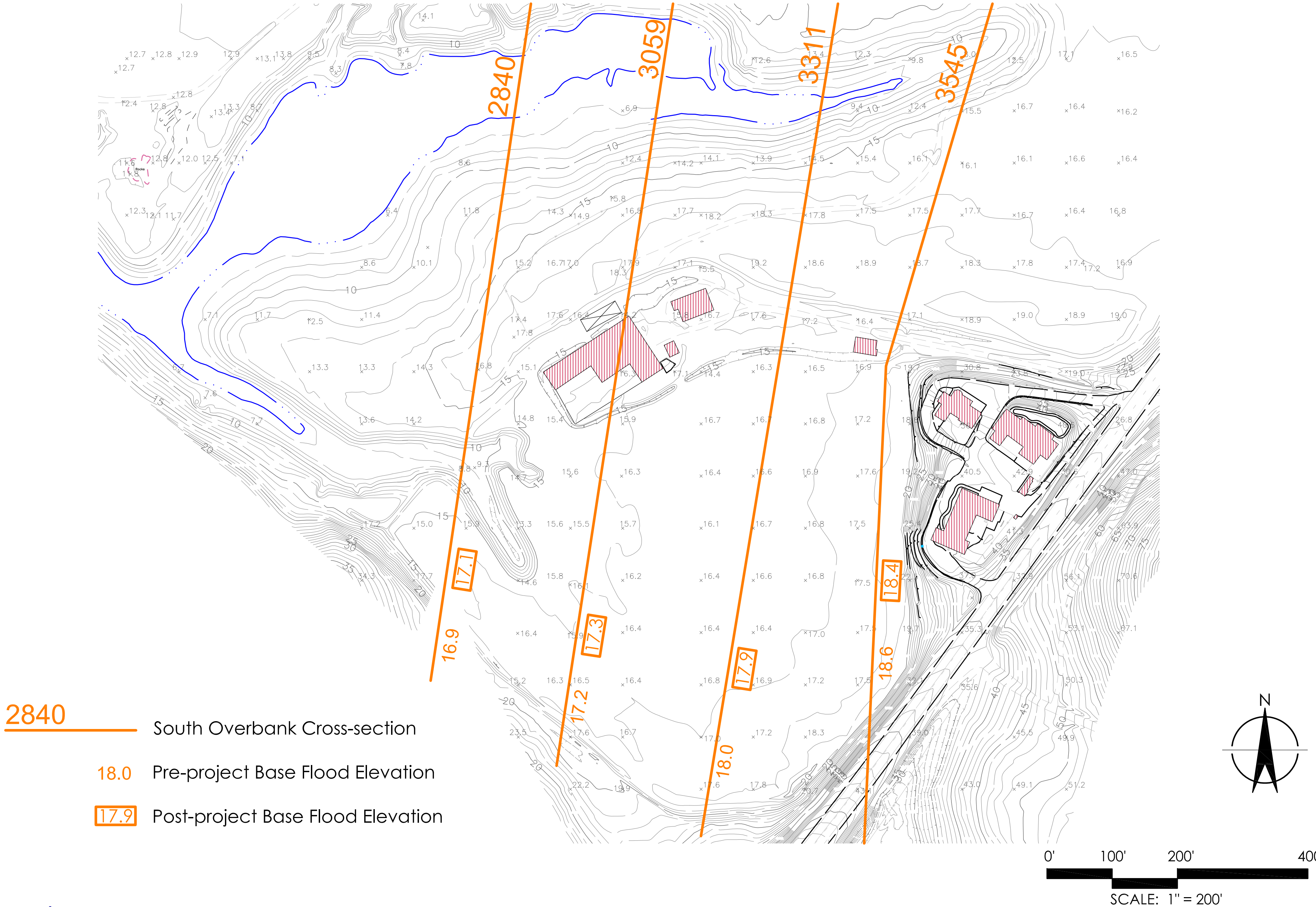
The values in Table 2 show that the reduction in the portion of the flood flow conveyed in the main channel generally leads to decreases in BFEs, especially along the north and east perimeter of the plant where the channel is much more confined. The increased discharge in the south overbank is predicted to lead to modestly higher BFEs along the south perimeter (maximum increase of 0.2 feet).

However, the residual flood risk to the plant is from the main channel, as the south perimeter is protected by high ground well in excess of the post-project BFE values. Therefore, the modeling predicts an overall reduction in the flood hazard at the CAWD facility as a result of the CRFREE project, and mitigation is not necessary.

Closing

Thank you for this opportunity to clarify the impacts of the proposed CRFREE project with respect to downstream flood elevations.

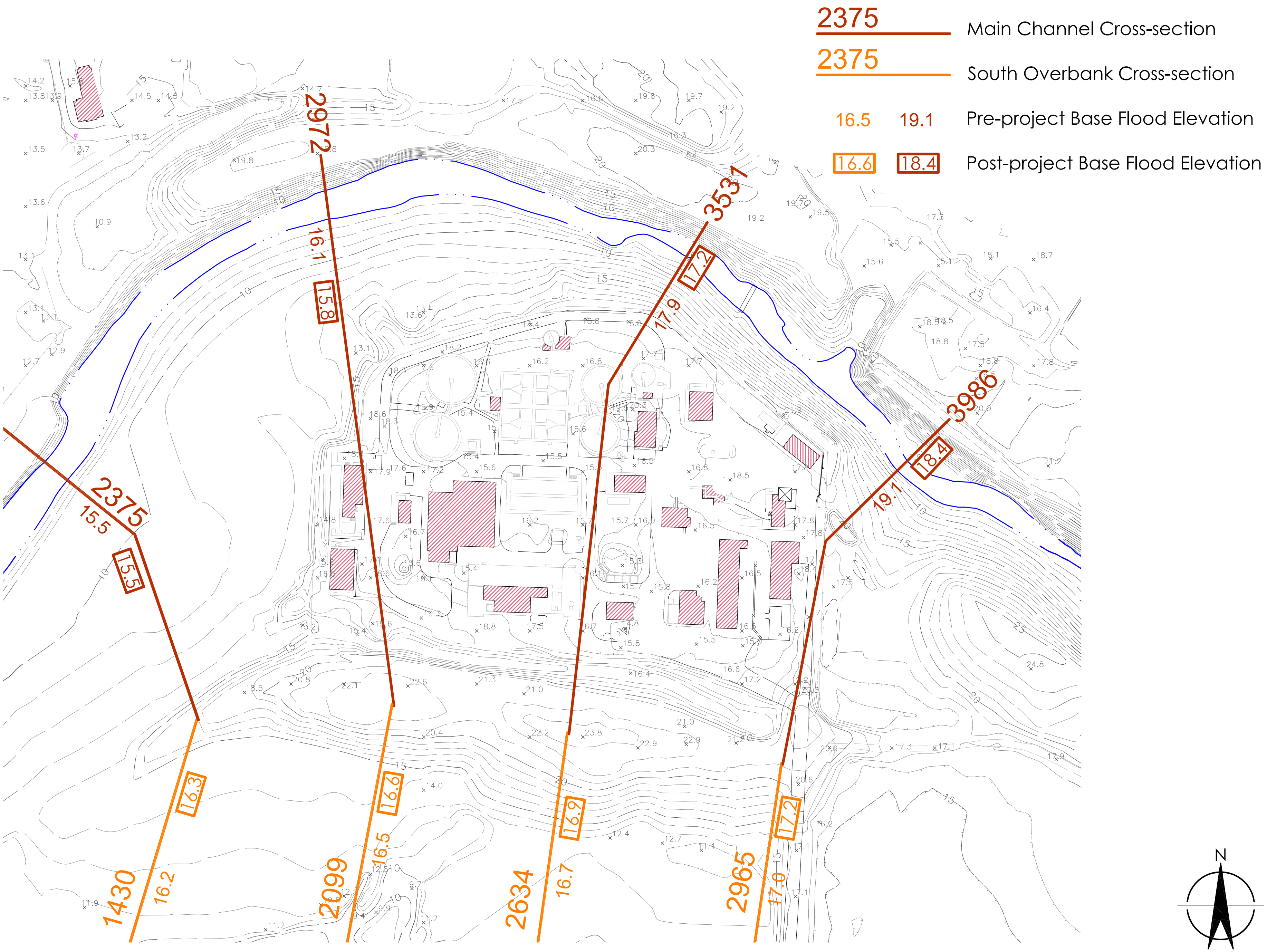
Do not hesitate to contact us if you have any questions or comments on the design assumptions and estimates summarized in this memo.



Balance Hydrologics, Inc.

Figure 1. Predicted pre- and post-project base flood (100-year) extents in the vicinity of the State Park barn area, Monterey County, California.

214044 100-year Flood Extents.dwg



Balance Hydrologics, Inc.

214044 100-year Flood Extents.dwg

Figure 2. Predicted pre- and post-project base flood (100-year) extents in the vicinity of the Carmel Area Wastewater District facility, Monterey County, California.

0' 100' 200' 400'

SCALE: 1" = 200'

Scour Calculation Summary Memo

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Memo

To: Cathy Avila, Avila and Associates Consulting Engineers

From: Eric Riedner and Edward Ballman

Date: August 29, 2008

Subject: Scour Calculation Summary for the Carmel River Causeway along Highway 1

General and local scour calculations were completed for a number of design alternatives being considered for the proposed Carmel River Causeway along Highway 1. Calculations were completed following recommended procedures contained within the Federal Highway Administration's Hydraulic Engineering Circular Number 18, *Evaluating Scour at Bridges* (HEC-18).

Calculations were focused on the 100-year recurrence interval flood event with key hydraulic parameters extracted from the HEC-RAS model summarized in our hydraulics memo dated August 27, 2008. Soil parameters were derived from grain size distributions of borings taken at the proposed causeway location by Kleinfelder that have been included as Appendix A.

<u>General Scour</u>

General scour will largely be limited to contraction scour at the bridge given the relatively uniform planform of the floodplain at the proposed causeway location. Contraction scour was calculated using the Laursen equation for live-bed scour as defined in HEC-18. A live-bed equation was selected to account for the bedload that is anticipated to be mobilized along the floodplain during the 100-year flood event.

Given the similarity in the contracted width and hydraulic parameters of the causeway alternatives being considered, a single calculation was completed to represent the anticipated contraction scour depth for all of the alternatives. This calculation predicts a contraction scour depth of approximately 3 feet. A summary of this calculation has been included as Appendix B.

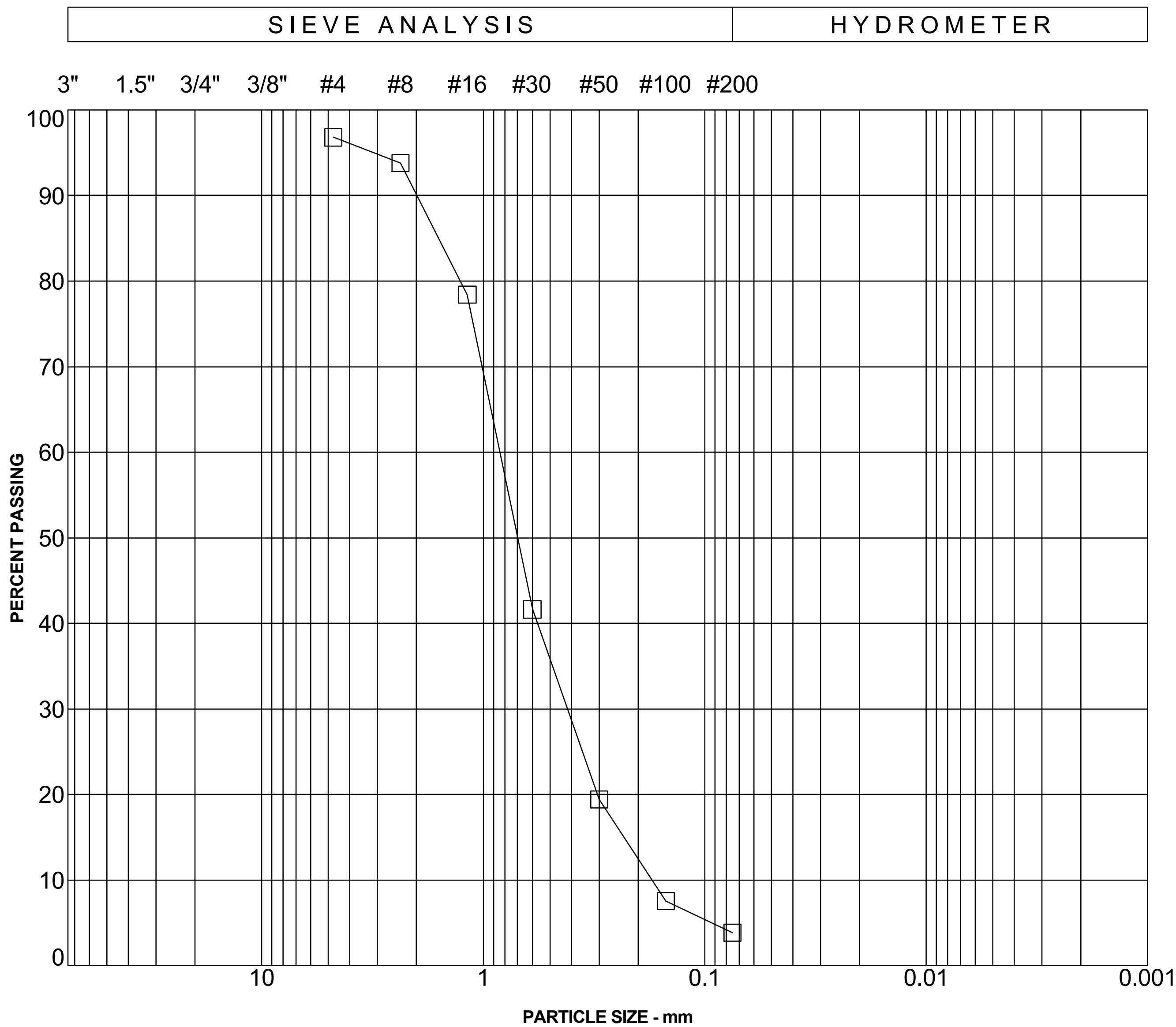
<u>Local Scour</u>

Local scour calculations were completed at the piers for each of the causeway alternatives being considered. Pier scour depths were estimated using the Colorado State University (CSU) equation as defined in HEC-18. Additional scour from debris caught on the piers, likely in the form of small floating debris rafts, was taken into account using the procedure recommended in Appendix D of HEC-18. Using these methods, local scour depths were estimated to vary between 6 and 11 feet deep over the range of causeway alternatives. A summary of these calculations has been included as Appendix C.

enclosures: Appendices A - C

Appendix A

Borings at Proposed Causeway Location



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	B-2	10.0	Olive Brown Poorly Graded Sand (SP)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO. 96313

PARTICLE SIZE ANALYSIS*

CARMEL CAUSEWAY
MONTEREY COUNTY, CALIFORNIA

PLATE

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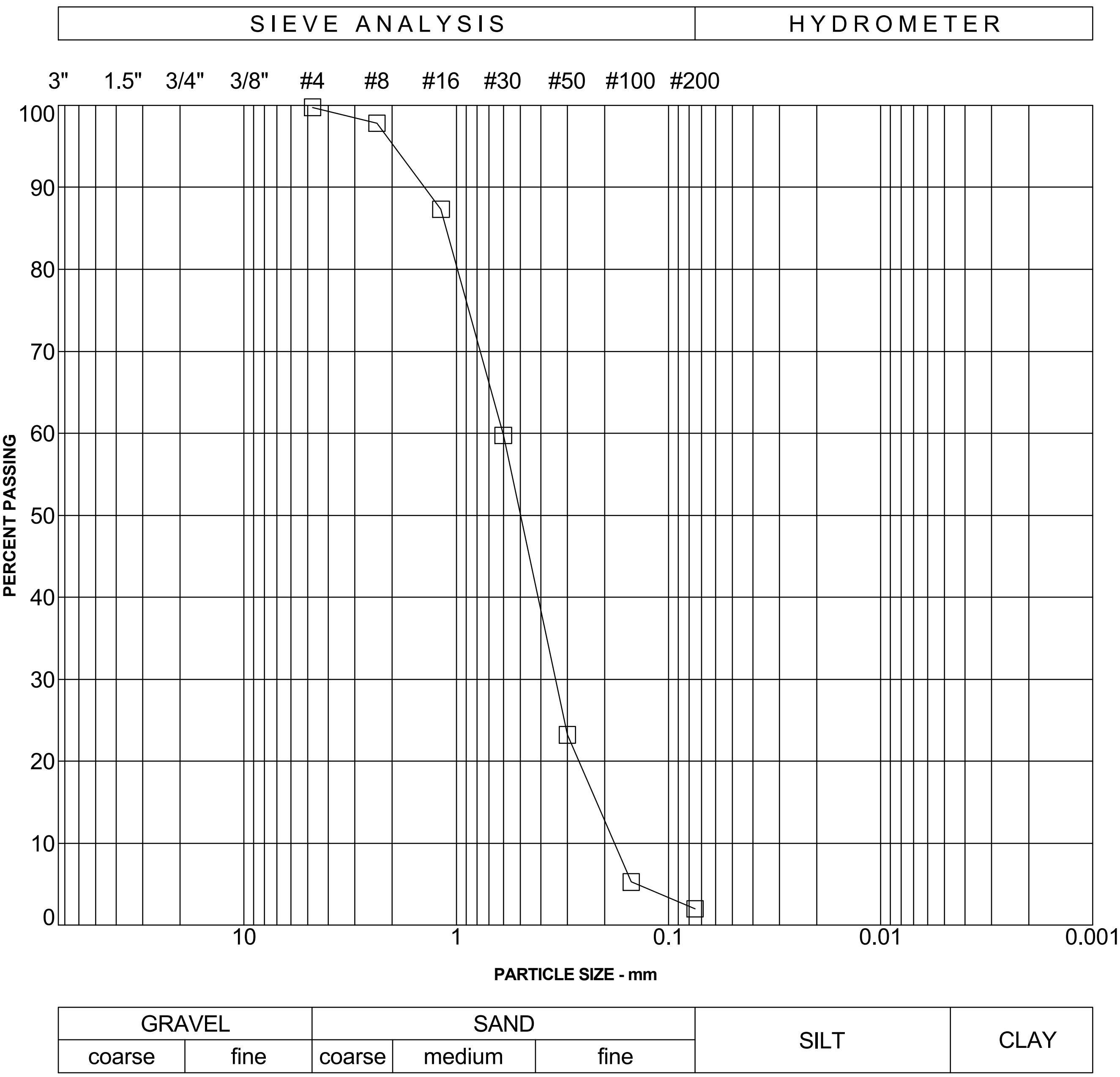

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	B-3	5.0	Dark Yellowish Brown Poorly Graded Sand (SP)

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PROJECT NO. 96313

PARTICLE SIZE ANALYSIS*

CARMEL CAUSEWAY
MONTEREY COUNTY, CALIFORNIA

PLATE

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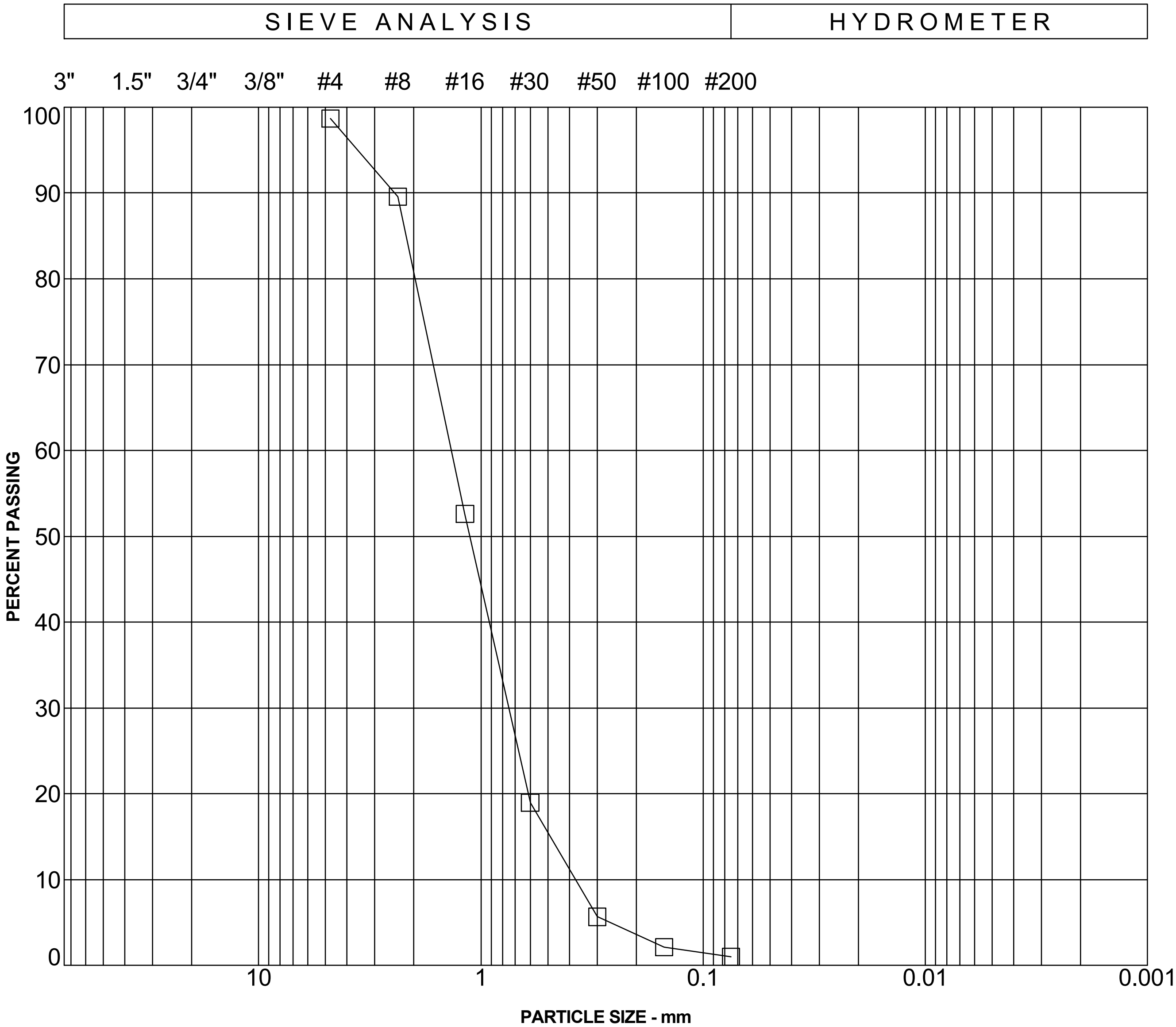

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	B-3	15.0	Olive Brown Poorly Graded Sand (SP)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

PARTICLE SIZE ANALYSIS*

PLATE

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO. 96313

CARMEL CAUSEWAY
MONTEREY COUNTY, CALIFORNIA

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Appendix B

Contraction Scour Calculations

Appendix B: Contraction Scour Calculations

Laursen live bed contraction scour equation as recommended in HEC-18:

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

$y_s = y_2 - y_o$

parameters:

Q_1 = flow in upstream channel =	14,000	cfs
Q_2 = flow in contracted channel =	14,000	cfs
W_1 = bottom width of upstream channel =	1,070	ft
W_2 = bottom width of contracted channel =	486	ft
y_1 = average depth in upstream channel =	4.5	ft
y_o = pre-scour depth in contracted channel =	4.3	ft
k_1 = bed material tranport mode exponent =	0.64	

solutions:

y_2 = average depth in contracted section =	7.5	ft
y_s = average scour depth =	**3.2**	ft

Appendix C

Pier Scour Calculations

Appendix C: Pier Scour Calculations

CSU equation as recommended in HEC-18:

$y_s = y_1\, 2.0\, K_1\, K_2\, K_3\, K_4\, (a_{proj} / y_1)^{0.65}\, Fr_1^{\,0.43}$

$a_{proj} = 2W + a\cos\theta$ or $a\cos\theta + L\sin\theta$ (whichever is greater)

parameters:

y_1 = flow depth upstream from piers =	4.9	ft
Fr_1 = Froude Number upstream from piers =	0.43	
K_1 = correction factor for pier nose shape =	1.0	
K_2 = correction factor for angle of attack =	1.0	
K_3 = correction factor for bed condition =	1.1	
K_4 = correction factor for armoring =	1.0	
θ = pier angle =	10	degrees

Bridge Type	Pier Width (a)	Pier Length (L)	Debris Width	a_{proj}	Scour Depth
	ft	*ft*	*ft*	*ft*	*ft*
slab - 15" piers	1.25	1.25	1.0	3.2	**5.7**
slab - 18" piers	1.5	1.5	1.0	3.5	**6.0**
slab - 24" piers	2.0	2.0	1.0	4.0	**6.5**
box girder - 5.5' x 8.25' piers	5.5	8.25	0.0	6.8	**9.3**
box girder - 7.0' x 10.5' piers	7	10.5	0.0	8.7	**10.9**

Large Woody Debris (drift) Potential Memo

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Memo

To: Cathy Avila, Avila and Associates Consulting Engineers

From: Mark W. Strudley, Ph.D., Shawn Chartrand, PG, CEG

Date: August 29, 2008

Subject: Large woody debris (drift) potential at the proposed Highway 1 causeway restoration, Carmel River, Monterey County, California

Large woody debris (drift) hazard potential

Drift, or debris that is floating on or through a stream, is a common problem for river crossings, especially in watersheds with mountainous, forested catchments or with large amounts of unstable man-made structures along channel margins or on the floodplain. Drift accumulation at bridge openings reduces flow conveyance, contributes to bed scour, and increases loading on bridge structures. An assessment of drift hazard at the proposed Highway 1 causeway on the Odello Property deserves special attention because of the proposed floodplain restoration activities that include opening portions of the south-bank levee system on the lower Carmel River. Below, we describe drift hazard in a 'source-to-sink' progression that highlights the processes and risks associated with each step in the delivery of drift to the Highway 1 causeway.

Drift production and size

Drift accumulation at bridges begins with drift production upstream. The rate of drift production depends, in part, on the degree of channel stability; unstable banks, meander migration, and lateral shifts in channel dimension and position. These processes may all introduce new woody debris to channels as vegetation is undermined or entrained with flows (Diehl, 1997). Drift production also depends on the forces imparted to man-made structures in the channel or on the floodplain which can cause these structures to break loose and enter the waterway, representing a largely unquantifiable and presumably minimal component of drift that is not accounted for in the discussion below. Bank stability was previously discussed in our report of June 2008 which included sections describing historical changes to channel configuration (Riedner and others, 2008). In brief, episodes of bank instability in the past have been partly curtailed as bank stabilizing features have been set in place over the last few decades. Even if bank stability was more predictable in the Carmel River system, we have no systematic way of linking bank stability to the rate of woody debris production. Therefore, we present data that inventory woody debris currently in the channel system as a measure of, or proxy for, drift production.

Very few quantitative assessments of the prevalence of large woody debris (LWD) have been made on the Carmel River. Two recent studies (Smith, 2003; 2004) inventory the concentration of LWD at representative study reaches between San Clemente Dam and the lagoon. For the purposes of this memo and the debris inventories, large woody debris is defined as any piece of wood with a minimum diameter of 15 cm and minimum length of 1.5 m. Notably, in the 2002 survey (Smith, 2003), the highest concentration of LWD was found at two locations, downstream from Via Mallorca to the head of the

Carmel Lagoon and between deDampierre and the Saddle Club. For 2002, the average density of LWD in the sampled reaches was 19 pieces per river mile, but the LWD was not evenly distributed along the river. Based on visual inspection and inference of the 150 inventoried pieces of LWD (Smith, 2003), only about one third appeared to be modifying channel morphology, and only 9% of this subpopulation appeared to be inducing bank scour. Interestingly, the highest incidence of bank scour induced by LWD occurred in reaches downstream from Via Mallorca, but Smith (2003) notes that this may be more a function of the gravel-sand transition in the Carmel River rather than a characteristic of LWD size, transport, and deposition.

The LWD inventory conducted in 2003 (Smith, 2004) yielded 471 occurrences of large wood or large wood accumulations from a more intensive sampling which included additional reaches in the survey. The average density of LWD per river mile increased to 36.7 occurrences, again with an uneven density distribution. This survey, however, highlights changes in the upper watershed more than at the Odello Property project reach, because flow conditions limited sampling in lower reaches such that the 2002 survey data were used for the reach extending downstream from Via Mallorca. (Low peak discharges in the winter of 2002-2003 and the large size of LWD in this lower reach suggest that statistical errors may have been minimal using data from the prior year's survey.) The 2003 data illustrate some detail that the pilot study in 2002 omits that is pertinent to drift hazard at bridges on the Carmel River: rootballs and LWD accumulations from upstream reaches tend to break up as they move downstream, yielding a relatively greater proportion of individual pieces transported through the project reach than accumulations of debris. However, video footage from the MPWMD website of the Highway 1 collapse during the flood of 1995 (one of the largest floods on record) indicates that debris accumulations in downstream reaches are possible when large floods follow prolonged drought.

It is important to note that Smith (2003, 2004) only extended his surveys to within 5 km of San Clemente Dam because the great surplus of LWD in this upper 5 km would have required resources outside of the projects' budgets. This suggests that the upper watershed supplies a great quantity of LWD to the lower channel segments, which may be episodically sequestered and transported through upstream and downstream reaches. The episodicity of LWD storage in, and delivery from, upper reaches is likely a function of the episodic, but natural, disturbances that have historically occurred in the surrounding catchment, notably the Marble Cone Fire of 1977. Other such disturbances, such as large earthquakes and mass movements (landslides) will likely and analogously induce infrequent but important pulses of LWD through the Carmel River system.

Although it is clear that the lower Carmel River is not free of woody debris, production rates are not excessive, pieces are generally not very large, and large pieces and accumulations tend to get broken into smaller pieces or conglomerations downstream.

Transport of drift onto the restored floodplain

Drift hazard at the proposed causeway overcrossing along Highway 1 on the Odello Property depends first on the likelihood of drift being transported from the main stem to overbank areas during flood flows. Hydraulic steering, the process that orients logs lengthwise along flow lines in flowing channels, suggests that logs may only enter the restored floodplain if a) flow velocities comparable to those above the main stem's thalweg exist through levee openings, and/or b) obstructions or channel features (submerged bars, for example) in the main stem divert logs into levee openings, and/or c) avulsion occurs, focusing more flow from the main stem through levee openings. Diehl (1997) maintains that when the depth of the floodplain inundation exceeds roughly one-third of the channel depth, the zone of surface convergence in the channel becomes discontinuous or ceases to exist, and most surface flow follows the axis of the valley through which the river is flowing. Our report of June 2008 (Reidner and others, 2008), however, articulates a quantitative analysis suggesting that avulsion is unlikely. Furthermore, the lower Carmel River in the vicinity of the project site is not a dominantly meandering channel winding across a constricted valley. Instead, the lower Carmel is a largely straight river in a wide floodplain constrained by levees (mostly along the right [north] bank), and therefore larger flood flows are not likely to induce a drastic refocusing of flows through the floodplain. Thus it appears that irregularity in drift geometry (crooked logs [sycamores and oaks], roots, or debris accumulation) combined with variations in channel geometry or the presence of obstructions in the channel are the most likely means by which drift may get directed onto the floodplain along the Odello Property.

In addition to the above considerations for logs entering the floodplain, the depth of flow above the floodplain at the levee opening must be sufficient to overcome the buoyancy depth (the depth at which flotation occurs in a horizontal channel) of either a log without rootwad, or a log with a rootwad. The buoyancy depth (also called critical floating depth) varies as a function of the density of the log and the diameter of the log. For wood found in the debris inventories in the two downstream-most study reaches (Rancho Cañada to the head of the Lagoon, and Via Mallorca along Rancho Cañada reach; Smith, 2003, 2004), log diameter ranges between 15 and 100 cm (with few logs greater than 100 cm diameter), and we would posit that density is perhaps half as much as that of water, reflecting the concern of recently mobilized wood during flood flows (ρ_{log} = 500 kg/m^3). Using the relationship developed by Braudrick and others (1997), critical floating depth is half the log diameter (for the above density). (The critical floating depth to log diameter ratio approaches unity as the wood density increases, say for oaks versus ponderosa pines.) Thus, for small logs (15 cm diameter), a bare minimum of 7.5 cm of water depth is required. For the larger logs with diameter of 100 cm or more, at least 50 cm (1.65 ft) water depth is required. The depths required are probably much greater because of dynamic changes in wood orientation while floating, and the potential for mobile debris to get caught on obstructions or shallow areas during transport. Our hydraulic modeling under proposed restoration conditions (Reidner and others, 2008) for the '10-year flood' suggests water depths at levee openings on the order of 2 feet, while the '100-year flood' produces water depths of approximately 4 feet or greater in the simulations. Thus, if one or more of the above conditions (a, b, or c) are met during the 100-year flood, wood within the size ranges documented in the Carmel River in 2002 and 2003 (Smith, 2003, 2004) could potentially enter the restored floodplain, while for the 10-year flood, wood with diameter approaching the maximum recorded in the inventory would likely run aground along the channel margins at the levee openings. Rootwads would further prevent logs from entering the floodplain, as a much larger critical floating depth would be required to prevent protrusions of the rootwad from interacting with the bed or vegetation.

In summary, irregularity in drift geometry combined with variations in channel geometry or the presence of obstructions in the channel are the most likely means by which drift may get directed onto the floodplain along the Odello Property, although the potential is not high. The depths required for log transport are probably much greater than the minimums calculated (7.5 cm of water depth for 15 cm-diameter logs and 50 cm depth for 100 cm-diameter logs) because of dynamic changes in wood orientation while floating, and the potential for mobile debris to get caught on obstructions or shallow areas during transport. Rootwads would further prevent logs from entering the floodplain, as a much larger critical floating depth would be required to prevent protrusions of the rootwad from interacting with the bed or vegetation.

Transport of drift across the restored floodplain, and deposition potential

Our hydraulic modeling, combined with analysis conducted by Braudrick and Grant (2001) suggest that once drift has entered the restored floodplain through levee openings, only local deviations in topography may hinder transport. This is because the simulated flow depths under proposed restoration conditions for both the 10-year and 100-year floods are generally above the critical floating depth across the floodplain for logs up to the maximum size encountered in Smith's survey (2003, 2004). However, the presence of rootwads or other complicating geometries, possibly associated with rafts or other accumulations, might tend to reduce transport under less intense floods or in shallow areas. Drift transport is also sensitive to the mean channel width and to the radius of curvature of any channel bends carved across the floodplain in the flow path. In general, logs longer than the mean channel width tend to exhibit reduced transport capability while sharper bends are more difficult for long logs to maneuver through. Thus, graded restoration elements such as oxbow lakes, meander bends, crevasse-splay channels, or alternate or mid-channel bars may support topography that would reduce water depths locally, encouraging drift deposition. Braudrick and Grant (2001) have found that meander bends are more efficient than other topographic elements in suppressing transport distance, and that abrupt flow expansion and shallowing across the entire flow's cross section are more adept at causing drift to deposit. For large drift pieces, the effects of flow expansion and shallowing, and/or obstructions to flow, may not be effective because of 'momentum-maintained transport', a condition in which the size and velocity of drift (*i.e.*, momentum) overwhelms the effect of bed obstructions. This momentum effect is likely to only affect admission of debris onto the floodplain and not transport across it, since flow velocities will decrease drastically downstream of the levee openings.

In sum, drift transport across the floodplain is not likely to be a commonplace occurrence and may only be a concern during the largest of floods with sufficient water depths and velocities that can carry large pieces or accumulations over and through the various obstructions and topographic variations likely to exist on a restored floodplain. Furthermore, the topographic expression of the restored floodplain is likely to assume the form of sand-splay complexes, with dendritic (finger-like) and discontinuous channels that are likely to interact with floating debris. Other types of complexity in floodplain topography (meander bends, oxbow lakes, distributary channels, and/or bars) will reduce drift transport under less intense flooding conditions (*e.g.*, the 10-year flood).

Entrainment of logs from the restored floodplain

Once drift has entered the restored floodplain and comes to rest at a position upstream of the Highway 1 causeway, what is the potential for remobilization and transport to the causeway? First and foremost, remobilization will be easier for logs with lower residence times and/or for logs that are partially decomposed (lower density). Increased residence time will likely reduce mobility because of progressive burial, as long as decomposition doesn't reduce density significantly. Smith's (2003, 2004) assessment indicates that 95% of the inventoried logs in the Rancho Cañada-to-Lagoon segment are either well-embedded or are only mobile during peak flows, suggesting episodic transport and deposition of drift that is significant in terms of accumulation at the Highway 1 overcrossing. However, these logs were inventoried in the main stem channel and not on the floodplain, so Smith's information may not be pertinent in terms of assessing the mobility potential of logs on the lower floodplain.

A complication in assessing drift mobility during the rising limb of a flood is that piece stability is strongly controlled by orientation (Braudrick and Grant, 2001). Usually drift is deposited in an orientation parallel to flow lines, with any rootwads pointing upstream (Diehl, 1997). This will tend to make drift entrainment more difficult for subsequent flows across the floodplain (see below). However, changes over time in the location of eroded and deposited features on the floodplain may cause pieces deposited during one flood to be oriented obliquely to flow lines in a subsequent flood. Thus in what follows, we will try to present information in terms of piece orientation, as well as in terms of other pertinent factors that control drift mobilization on the Odello floodplain.

Other than orientation, piece diameter is very important in determining piece stability. For example, a log 15 m long (longer than most logs inventoried on the Carmel in 2002 and 2003) with a density of 500 kg/m^3 resting on a floodplain surface with a slope of 0.01 requires a *minimum* of 0.12 m (5 in.) of water depth to mobilize if the bole diameter is 50 cm (20 in.), and its counterpart with a 2-m wide rootwad requires a *minimum* of 0.25 m (10 in.) depth. These numbers are for pieces oriented at 90 to 45 degrees away from flow lines (lying across the flow direction at a right or oblique angle). However, if the same pieces are oriented parallel to flow, than a *minimum* of 0.25 m (10 in.) and 0.35 (14 in.) are required, respectively. Under the same conditions using the same logs, but with a 1 m diameter bole, the water depth required for transport becomes roughly 0.25 – 0.45 m (90 – 0 degrees orientation) for logs without rootwads and 0.4 – 0.6 m (90 – 0 degrees orientation) for logs with rootwads. Doubling piece diameter increases the entrainment depth by up to 93% for pieces without rootwads and 84% for pieces with rootwads (Braudrick and Grant, 2001).

The numbers in the paragraph above should be used as a guide only because the proposed graded floodplain slope on the Odello Property is roughly 0.003, whereas the water depths required in the paragraph above are for surfaces with a slope of 0.01. Braudrick and Grant (2001) suggest that increasing surface slope from 0.005 (roughly equivalent to the proposed grading plan on the Odello Property) to 0.01 decreases the ratio of water depth to log diameter by 4% for pieces without rootwads oriented parallel to flow, but decreases the ratio by 19-22% for pieces without rootwads oriented oblique and perpendicular to flow. Thus, for slopes equivalent to the restored floodplain, the numbers above apply for pieces parallel to flow, but if the pieces are oriented obliquely to flow, the water depth required will be much greater.

In sum, Smiths' inventories (2003, 2004) suggest that the infrequency of drift delivery to the lower Carmel River gives time for drift burial, and that drift will only episodically be remobilized. Because drift is commonly deposited parallel to flow lines, and because the floodplain-main stem planform geometry suggests consistency in flow path lines over time, drift remobilization will be more difficult because pieces will be oriented parallel to flood flows. And although larger floods (such as the modeled 100-year event) may be capable of entraining logs off of floodplain surfaces, the minimum water depths required on a low-sloping floodplain surface such as on the Odello Property are on par with, or greater than, those depths created by statistically more frequent flood flows (such as the modeled 10-year event). Log mobility and reorganization on the floodplain near the levee openings may be an ongoing occurrence, but transport across the entire Odello Property, through floodplain vegetation and over bed irregularities, will likely be restricted to only the largest floods.

Drift countermeasures and recommendations

Drift countermeasures at bridges include using solid, round-nosed piers aligned with the flood flow, and avoiding placement of hammerhead piers in the water column or pile caps above the channel bed surface (Diehl, 1997). Inclined upstream pier noses and drift deflectors can also minimize drift accumulation at bridges. Increasing freeboard and avoiding pier placement along the outside of meander bends may also help (Diehl, 1997).

Diehl has recently provided techniques to qualitatively assess drift accumulation potential at bridges, and we follow his procedure loosely in providing our own assessment of drift accumulation potential at the proposed Highway 1 overcrossing/causeway on the Lower Carmel River. (Diehl mentions in his report that Australian and New Zealand bridge design criteria suggest that bridge spans will not be completely blocked if spaced apart by 20 to 15 meters, respectively, but suggests using a more robust and general assessment procedure.) Single-pier drift accumulations are typically developed on a "master" log or logs that extend the full width of the accumulation, while span blockage develops when logs extend from pier to pier. Thus the maximum width of these types of accumulation is approximately equal to the maximum length of sturdy logs delivered to bridges. This 'design log' length does not represent the absolute maximum length of drift debris, but represents the length above which logs are insufficiently abundant or insufficiently strong to produce drift accumulations equal to their length. Design log length is defined at a given site by the smallest of three values:

- the width of the channel upstream from the site,
- the maximum length of sturdy logs, and
- in much of the United States, 9 m (30 ft) plus one quarter of the width of the channel upstream from the site.

The maximum sturdy-log length reaches about 45 m (about 150 ft) in parts of northern California and the Pacific Northwest (Diehl, 1997), but the LWD inventories provided by Smith (2003, 2004) indicate that logs catalogued between Rancho Cañada and the Lagoon are predominantly either in the 4.5-6 m length category (29%) or the >9 m length category (33%). None of these logs were found on the floodplain or at the channel center, but were found along the margins of both the low flow and bankfull channels (which is not in itself surprising because of the extensive levee system in place). Thus, a conservative (upper) estimate of design log length is 9 meters. Smith's assessment indicates that 95% of the inventoried logs in the Rancho Cañada-to-Lagoon segment are either well-embedded or are only mobile during peak flows, suggesting episodic transport and deposition of drift that is significant in terms of accumulation at the Highway 1 overcrossing.

In sum, the conservative estimate of design log length of 9 meters is roughly equivalent to the design centerline spacing of 36 feet for piers supporting the Highway 1 causeway for the most economical design option. Thus it is unlikely, although theoretically possible, that logs of this size may accumulate at the causeway during flood events. However, this would require logs of this size to be transported across the floodplain, which is even less likely, given the analysis presented above.

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CAWD Outfall Pipe Impact Assessment Memo

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MEMO

To: Nathaniel Milam, P.E. (Whitson Engineers)

From: Anna Nazarov, P.E., CFM, Edward D. Ballman, P.E.

Date: October 20, 2016

Subject: Changes to 10- and 100-Year Water Surface Elevations in the vicinity of the CAWD Pipe Due to the Carmel River Floodplain Restoration and Environmental Enhancement Project

In August of 2015, Balance Hydrologics reviewed the hydraulic modeling analyses prepared for the Carmel River Floodplain Restoration and Environmental Enhancement (CRFREE) Project and summarized anticipated changes in downstream flood elevations in a technical memo. The focus of that review was on Base Flood Elevations (BFEs), also known as 100-year flood elevations, since they are the primary regulatory standard for County ordinances and regulations under the National Flood Insurance Program. This memo builds off of that work to also consider the 10-year event and summarize the changes to existing conditions at the Carmel Area Wastewater District (CAWD) outfall pipe crossing located west of the proposed Causeway.

Existing Conditions

The CAWD outfall is a 24-inch pipe, located west of Highway 1, crossing the Carmel Lagoon via iron and concrete supports, downstream of the confluence of the North Arm and South Arm branches. The pipe is shown below in Figure 1.

Figure 1. CAWD Pipe at Lagoon Crossing (photo courtesy Denise Duffy & Assoc.)

In existing conditions, flood flows that break out onto the south overbank of the Carmel River pool behind the Highway 1 embankment and spill over a low point in the roadway into State Parks property and onward into the Lagoon. Additional floodwaters flow over the CAWD access road from the south bank of the river west of the highway. At this time, Balance does not have detailed information about pipe or support structure elevations at the pipe crossing so we cannot comment on how often the pipe is inundated under existing conditions.

Per the 2014 staff report to the CAWD Board of Directors, the 2004 excavation of the South Arm of the lagoon "created the perfect corrosive environment resulting over time in the deterioration of the iron and concrete supports that suspend it." The staff report indicates that CAWD engaged Kennedy Jenks Consultants to conduct a structural review and provide recommendations for the replacement of this portion of the outfall and that "the options have been narrowed down to the replacement of the existing 160 feet of pipe with a new portion of pipe, installed beneath the southern finger of the Lagoon, along the same alignment of the existing pipe."

Post-Project Conditions

The CRFREE project proposes to construct a causeway to convey floodwaters from the east to the west side of Highway 1. During the 10-year event, flows will increase from 800 cfs over the highway embankment to roughly 5,000 cfs through the causeway. During the 100-year event, flows will increase from 7,300 cfs over the highway embankment to roughly 13,000 cfs through the causeway. As mentioned above, these flows will join those entering the lagoon from the north, which pass over the CAWD access road, west of the highway. This will result in increases in water surface elevations (WSEs) and velocities at some locations on the south overbank.

The CAWD pipe is located between cross-section 4+71 and 6+85 of the south overbank reach in the currently-effective FEMA hydraulic model. Discharges, WSEs, and velocities at the two sections, as well as at the pipe crossing location between the two sections, are provided in Table 1 below. The values at the pipe crossing are taken as the simple average of those at the bounding cross-sections.

During the 10-year event, the WSE is predicted to increase approximately 0.13 feet (or approximately 1.5 inches) at the location of the pipe crossing. The velocity is predicted to increase approximately 0.53 feet per second. During the 100-year event the impacts are less pronounced, in large part because the lagoon is in a more backwatered condition in both pre- and post-project scenarios. As such, the WSE is predicted to increase 0.08 feet (or approximately 1 inch), while the velocity is predicted to increase 0.30 feet per second.

Table 1. 10- and 100-year model data for pre- and post-project scenarios

		At x-sec 471 (D/S of pipe)	At x-sec 685 (U/S of pipe)	At pipe crossing	Change from pre to post conditions
10-year event					
Pre-project flow	(cfs)	5,600	5,600	5,600	
Post-project flow	(cfs)	6,800	6,800	6,800	1,200 cfs
Pre-project WSE	(ft, NGVD)	8.68	9.04	8.86	
Post-project WSE	(ft, NGVD)	8.73	9.24	8.99	0.13 ft (1.5 in)
Pre-project velocity	(fps)	3.96	1.69	2.83	
Post-project velocity	(fps)	4.74	1.96	3.35	0.53 fps
100-year event					
Pre-project flow	(cfs)	16,900	16,900	16,900	
Post-project flow	(cfs)	18,500	18,500	18,500	1,600 cfs
Pre-project WSE	(ft, NGVD)	12.9	13.26	13.08	
Post-project WSE	(ft, NGVD)	12.95	13.38	13.17	0.08 ft (1.0 in)
Pre-project velocity	(fps)	4.52	2.36	3.44	
Post-project velocity	(fps)	4.92	2.55	3.74	0.30 fps

Implications and Considerations

Considering the lagoon substrate material, we anticipate that the threshold velocity for erosion and scour to be on the order of 4 feet per second (see Table 2-5 in U.S. Army Corps of Engineers' Hydraulic Design of Flood Control Channels). The 0.53 feet per second increase from 2.83 feet per second to 3.35 feet per second during the 10-year event does not increase the velocity to the threshold for erosion and scour. WSE changes are minimal (1 to 1.5 inches) since the entire pipe is predicted to be submerged in both the pre- and post-project conditions.

An additional concern might be the potential for increased large woody debris loading in the lagoon as a result of the CRFREE project. While the project does remove portions of the south bank levee upstream of Highway 1 (providing additional paths onto the floodplain from the main river channel), the flow depths through the levee openings and across the restored floodplain are not predicted to be large enough to support transport of large debris. Any logs that do make it into the floodplain would have to travel between 3,000 and 4,500 feet (depending on the levee opening) through a series of floodplain restoration features before arriving at the causeway and entering the lagoon. We thus anticipate that any large logs will drop out and be sequestered on the floodplain.

Closing

If you have any additional questions about this memo, please do not hesitate to contact us.

MEMO

To: Nathaniel Milam, P.E. (Whitson Engineers)

From: Anna Nazarov, P.E., CFM, Edward D. Ballman, P.E.

Date: February 8, 2017

Subject: Response to Concerns Raised in the South Arm Lagoon Velocity Analysis for the CRFREE Project, Monterey County

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We have reviewed the technical memorandum prepared by Schaaf & Wheeler Consulting Engineers (S&W) for Carmel Area Wastewater District (CAWD), dated November 28, 2016. The subject of the memorandum was "South Arm Lagoon Velocity Analysis" for the Carmel River Floodplain Restoration and Environmental Enhancement (CRFREE) Project.

The S&W memo describes two analyses: 1) an alternative flow analysis of the south overbank of the Carmel River (South Arm of the Carmel River Lagoon); and 2) an analysis of debris transport potential. These analyses are used as the basis for assertions regarding the Project's potential to increase risk to the CAWD outfall pipeline.

This response provides more detail as it pertains to the concerns raised in the memorandum.

<u>Flow Analysis</u>

The Schaaf & Wheeler memorandum, first and foremost, describes an analysis of 10- and 100-year velocities in the lagoon. The memo states that "the hydraulic conditions in the South Arm were analyzed using the HEC-RAS models developed by both Balance Hydrologics and Schaaf & Wheeler." We have not been given the opportunity to review the Schaaf & Wheeler model. We are therefore not able to verify which version of our modeling Schaaf & Wheeler used for their analysis, and cannot comment on the modeling details which were ultimately used by S&W.

The S&W memorandum states that "flows in the South Arm at Highway 1 are taken directly from the October 20, 2016 memo by Balance." It goes on to briefly describe the modeling. This description notes that the models were run with a mixed flow regime and with a tidal boundary set to critical depth based on the latest sand bar breach geometry. While it is true that this approach might better approximate the maximum potential lagoon velocity, the utilized flow regime and boundary condition are departures from the effective FEMA model that forms the basis of our analysis. As a consequence, results obtained using these parameters cannot be directly compared to results outlined in the October Balance memo; also, due to the interconnected nature of the main channel of the Carmel River, the floodplain, and the tidal boundary (i.e. the complex flow splits across levee notches upstream of the causeway and the lateral weir along the CAWD road), flow splits between the main river channel and south floodplain should be expected to change when using a different set of model parameters. Therefore, the flows which are presented in the October memo, and which S&W's memo states were utilized as the basis of their analysis, should be reevaluated by S&W to account for the different flow regime and boundary condition outlined in the S&W report.

The S&W memo also states that they "optimized" and "un-optimized" the lateral structures in their analysis to "bracket" the flow analysis. Results are listed for both "optimized" and "un-optimized" model runs. The memorandum states that, "*Unoptimized assumes the main channel flows do not reach the South arm*" and "*Optimized allows the lateral structures to freely pass flows across the south bank.*" In other words, it seems that the un-optimized model run ignores the significant amount of flow that can (and does) break out of the main channel west of Highway 1, across the CAWD access road. This occurred during the flood events of 1995 and 1998 (estimated to be 25-year and 30-year events), and occurred as recently as this past January when a nearly 10,000-cfs flow was the peak recorded at USGS Gage 11143250 at the Via Mallorca bridge. It is therefore our opinion that the "un-optimized" model run does not correctly model the flow splits in the study area; the "lateral weir structure" which HEC-RAS uses to balance flows between the main river channel and the south overbank should be "optimized."

The S&W Memo describes pre- and post-project flows during the 10- and 100-year design events in their Table 1. Said table presents the flows in the South Arm at Highway 1 and shows a dramatic increase in both the 10- and 100-year discharges. Please note that, while these flows are described in our October memorandum, the discharges in the South Arm at the CAWD pipeline are significantly different than those presented in the S&W memo in Table 1. Table 1R, below, provides the existing and proposed 10- and 100-year discharges at the CAWD pipeline, as stated in our October memorandum. Note that the post-project flows are estimated to be similar to the pre-project flows at the CAWD pipeline. We feel that this comparison is a better and more reasonable comparison of pre- to post-project conditions than those presented in the S&W memo in Table 1 (which gives the flows at the highway), due to the flows which are anticipated to break out of the main river channel, downstream of the highway, during flood events.

Table 1R. South Arm Flows at CAWD Pipeline.

	10-Year Discharge cfs	100-Year Discharge cfs
Existing	5,600	16,900
Proposed	6,800	18,500

We observe that the "optimized" analysis provided by S&W, which utilizes the revised model parameters, indicates increases in flow velocity similar to those given in our October memorandum. Namely, the increase in velocity due to the project is minimal and the maximum predicted 10-year velocity of 3.7 feet per second is below the anticipated threshold velocity of 4 feet per second for erosion and scour.[1]

Debris Loading

The memorandum notes that previous project documentation did not provide a detailed analysis or degree of uncertainty for debris capture upstream of Highway 1. However, Balance produced a memorandum titled "Large woody debris (drift) potential at the proposed Highway 1 causeway restoration" in August of 2008 and updated said memorandum in May of 2015. The following information is provided to build on that analysis.

Per the project plans, the restored floodplain upstream of the future causeway (Highway 1) will be activated solely by flows passing through five proposed levee notches (i.e., sections of levee that are proposed to be lowered).[2] Table 2 below summarizes the proposed elevation and length of each of the notches using a) distance from the causeway to the center of the levee opening, b) the average finished grade at the opening[3], and c) the approximate length of the opening[4] as measured parallel to the flow in the main channel of the Carmel River. We have attached the overall site plan (Sheet S-1) for the CRFREE project for reference. The notches can be seen along the northeastern portion of the grading limit. They are unlabeled on the overall site plan, but are numbered in the table below such that Notch No. 1 is the most downstream and Notch No. 5 is the most northeastern point of the grading limit.

Table 2. Geometric characteristics of proposed levee notches.

	Distance U/S of Causeway	Elevation of Opening	Length of Opening
	feet	feet, NAVD88	feet
Notch No. 1	2,500	23.5	190
Notch No. 2	3,000	25.5	250
Notch No. 3	3,600	27.5	200
Notch No. 4	4,200	30.5	230
Notch No. 5	4,850	33.5	380

The S&W memorandum makes reference to a flow depth of 5 to 7 feet and uses this to infer that debris up to 10 feet in diameter could enter the South Arm via the proposed causeway. The

[1] Given the infrequency of the event, it is not expected that conditions associated with the 100-year flood would be a primary driver in the geomorphic evolution of the lower river/lagoon system. As such, a focus on 10-year velocities is much more representative of the potential geomorphic changes in the lagoon.
[2] The most upstream levee opening is not technically a notch as it utilizes the existing grade and doesn't lower a levee section. It is counted here as a notch since it allows flows from the main channel to enter the floodplain.
[3] As an example, the upstream side of Notch No. 1 is at elevation 23.6 feet while the downstream side is at 23.4 feet, resulting in an average finished grade of 23.5 feet.
[4] The notches have sloped sides resulting in a trapezoidal flow area. This, of course, means that that flow length changes based on the depth of flow. For simplicity, the length here is the base of the trapezoid when dry.

referenced depth does represent the predicted flow depth below the causeway for the 100-year flood. However, that is essentially the single deepest predicted depth within the entire restoration plan area (precisely because all the flow will pass under the causeway). The channel depth under the causeway is not representative of the depths available to transport large woody debris across the 2,500 to 4,850 feet of floodplain, from the various notches to the causeway. We have attached a grading sheet (Sheet G-3) and the associated cross-sectional views (Sheet X-1) from our plan set to show the wide and shallow floodplain in more detail.

A much better metric for characterizing limiting debris transport flow depths is the predicted depth where flow spreads out onto the floodplain from each notch opening. Review of the detailed hydraulic modeling shows that these depths are in the range of only 1.5 feet for the 10-year event and 3.5 feet for the 100-year event. Therefore, the assertion that debris as large as 10 feet in diameter could be transported (even if it existed within the watershed) is not appropriate. Considering that natural stream wood will almost certainly include branches or rootwad bearing members, the conclusion of limited transport potential would apply, even if the notches represented completely clear flow paths.

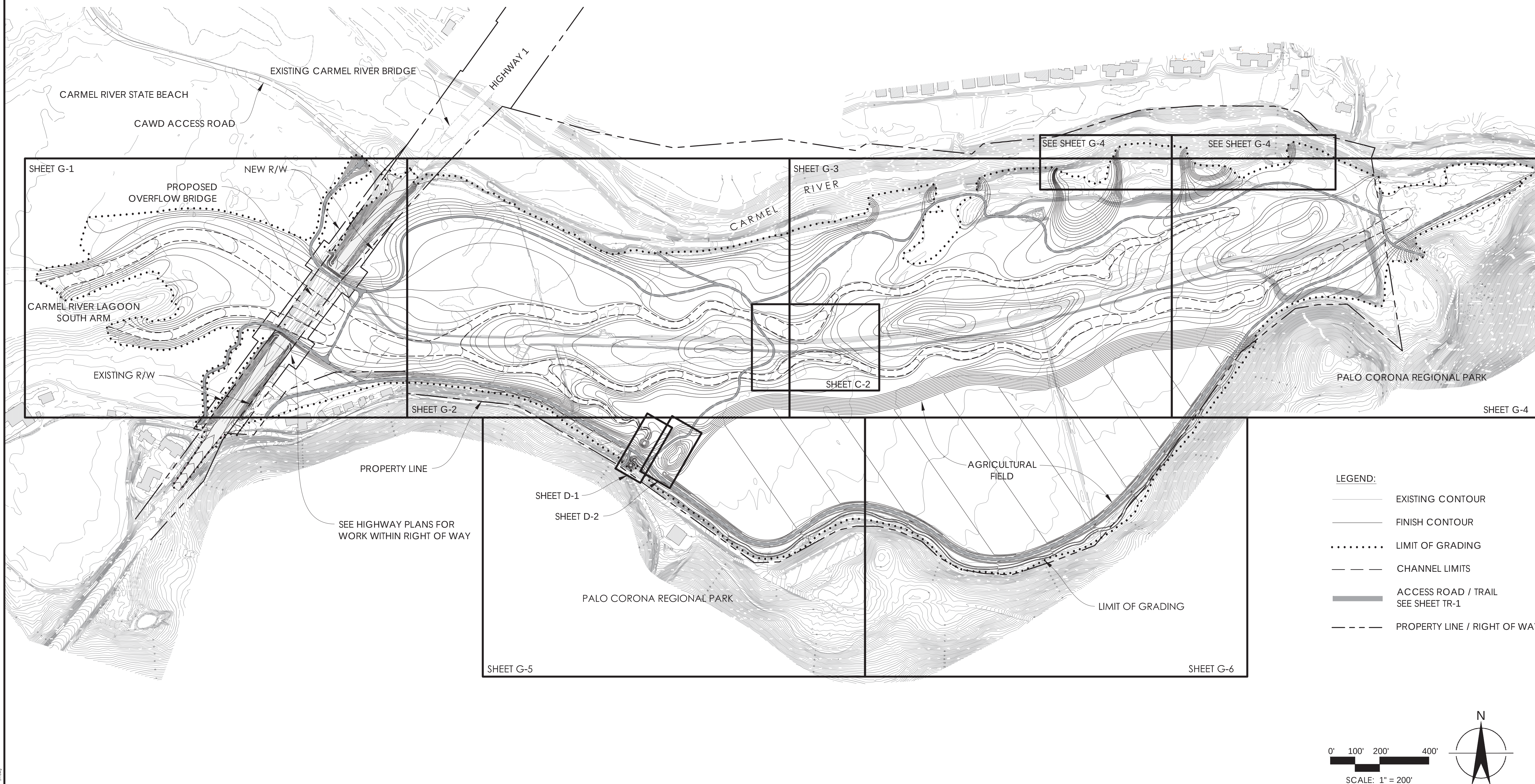
With respect to the latter point, the S&W memorandum did not consider the long-range management plan for the portions of the floodplain adjacent to the notches. The design now identifies these areas as "maintained flow conveyance areas" (MFCAs) wherein more active management would be used to provide a restored environment that directly considers both flow conveyance and the need to control the potential for large stream wood diversion onto the floodplain from the main channel. In fact, the Restoration and Management Plan explicitly notes on page 20 that the MFCAs can include woody species that are appropriately-spaced and trimmed. This achieves both objectives of maintaining flood conveyance and screening the passage of stream wood. An excellent (though less well-maintained) analog is the existing willow thicket located between the main channel and the CAWD access road, through which large scale debris transport is not reasonably possible.

It is correct that there are concerns about a temporary increase in debris load due to the removal of San Clemente Dam upstream of the site. However, this was also anticipated in the restoration design. During the period of potentially increased debris transport, "levee plugs" have been factored into the design to reduce the frequency of the floodplain being engaged during the early post-construction years. A levee plug is essentially fill placed in the notch openings and compacted to a stable condition that will limit the flow depths while the overall floodplain vegetation plan is established, including that in the MFCAs.

Closing

In summary, it is our continued opinion that 1) the Project will not result in significant increases in flow velocity at the CAWD outfall pipeline, nor exceed the 4 feet per second threshold for channel scour at the CAWD outfall pipeline; and 2) the Project will not result in significantly increased transport of large stream wood to the South Arm of the Lagoon.

If you have any additional questions or need additional technical details and references in this regard, please do not hesitate to contact us.



DRAFT 60% SET - NOVEMBER 18, 2016 - NOT FOR CONSTRUCTION

OVERALL SITE PLAN

BIG SUR LAND TRUST

RESOURCE MANAGEMENT AGENCY

CARMEL RIVER FLOODPLAIN RESTORATION AND ENVIRONMENTAL ENHANCEMENT PROJECT

PROJECT No. 7200

COUNTY PLAN SET

REVISIONS:

BY:	DATE:	DESCRIPTION:

DATE: 11/18/2016

SCALE: 1" = 200'

DRAWN BY: DJ

JOB #: BH # 214044

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Balance Hydrologics, Inc.
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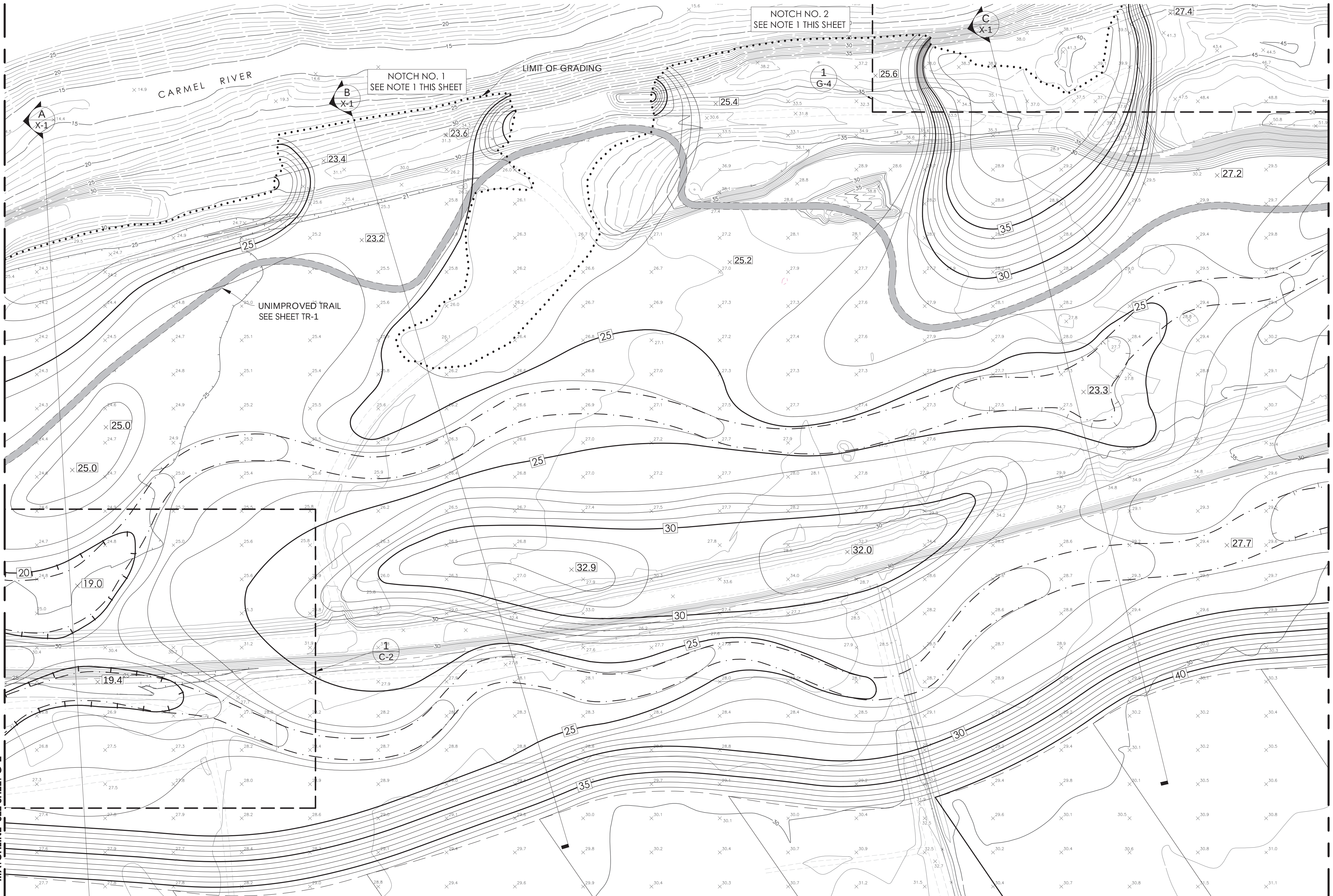
SHEET S-1 OF

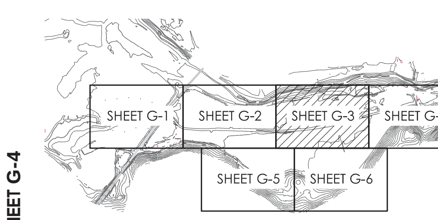


NOTES:

1. THE GRADING PLAN CALLS FOR THE EXCAVATION OF FOUR NOTCHES IN THE BERM ALONG THE SOUTH SIDE OF THE CARMEL RIVER. NOTCHES 1 AND 2 ARE SHOWN ON THIS SHEET. NOTCHES 3 AND 4 ARE SHOWN ON SHEET G-4. AFTER GRADES SHOWN ON THIS SHEET HAVE BEEN ACHIEVED, FURTHER EARTH WORK IN THE NOTCH AREAS IS REQUIRED. SEE SHEET G-7.
2. SEE SHEET C-1 FOR DETAILS ON LOG PLACEMENT.

LEGEND:

30	EXISTING INDEX CONTOUR
	EXISTING INTERMEDIATE CONTOUR
×21.5	EXISTING POINT ELEVATION
15	FINISH INDEX CONTOUR
	FINISH INTERMEDIATE CONTOUR
×23.3	FINISH POINT ELEVATION
	BREAKLINE
	LIMIT OF GRADING
	CHANNEL LIMITS
	TRAIL SEE SHEET TR-1



SHEET INDEX

N

0' 30' 60' 120'

SCALE: 1" = 60'

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GRADING PLAN

CARMEL RIVER FLOODPLAIN RESTORATION AND ENVIRONMENTAL ENHANCEMENT PROJECT

PROJECT No. 7200

COUNTY PLAN SET

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SHEET

G-3

OF

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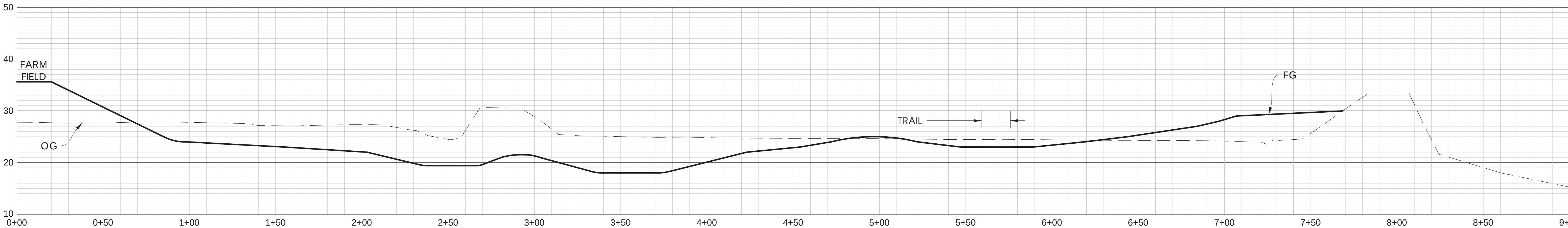

G-3 (A) SECTION
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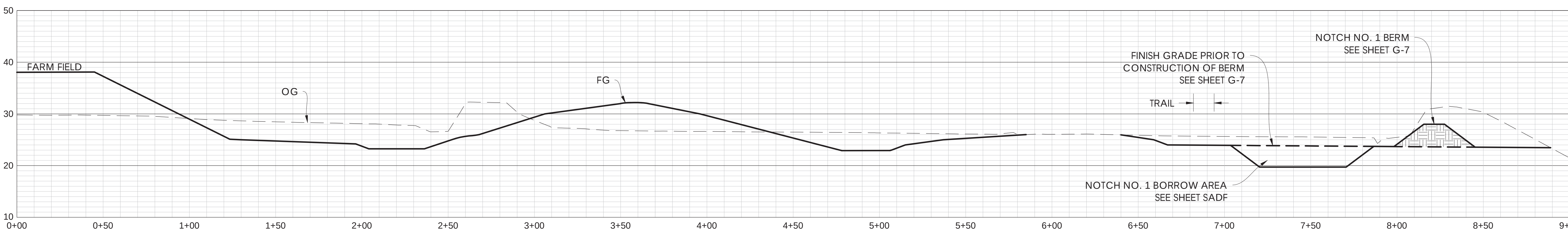
1" = 10' VERTICAL



G-3 (B) SECTION
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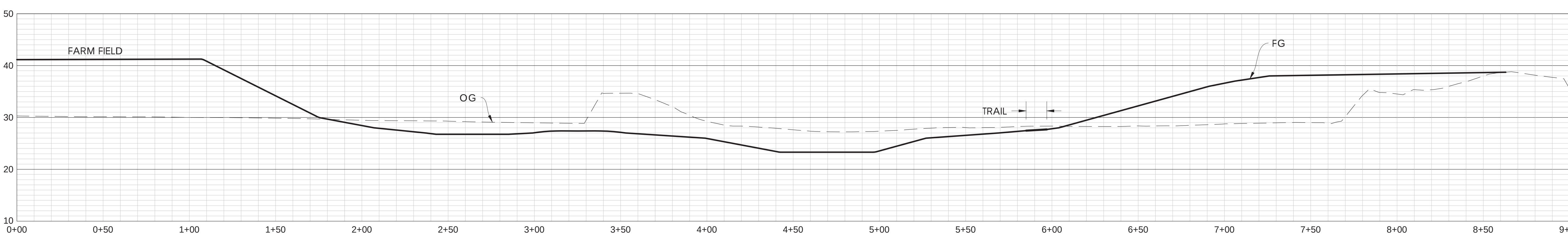

G-3 (C) SECTION
SCALE: 1" = 30' HORIZONTAL
1" = 10' VERTICAL

SECTIONS

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CARMEL RIVER FLOODPLAIN RESTORATION
AND ENVIRONMENTAL ENHANCEMENT PROJECT
PROJECT No. 7200
COUNTY PLAN SET

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BY:	DATE:	DESCRIPTION:

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SHEET
X-1
OF

State Parks Barn Complex Impact Assessment Memo

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MEMO

To: Nathaniel Milam, P.E. (Whitson Engineers)
From: Anna Nazarov, P.E., CFM, Edward D. Ballman, P.E.
Date: September 15, 2016

Subject: Potential Impacts to the State Parks Barn Complex Due to the Carmel River Floodplain Restoration and Environmental Enhancement Project

In August of 2015, Balance Hydrologics reviewed the hydraulic modeling analyses prepared for the Carmel River Floodplain Restoration and Environmental Enhancement (CRFREE) Project and summarized anticipated changes in downstream flood elevations in a technical memo. The focus of that review was on Base Flood Elevations (BFEs), also known as 100-year flood elevations, since they are the primary regulatory standard for County ordinances and regulations under the National Flood Insurance Program. This memo builds off of that work to summarize the potential impacts at the State Park Barn Complex located west of the proposed Causeway.

Existing Conditions

The State Parks Barn Complex is located west of Highway 1. These buildings are known as the Barn, the Blacksmith Shop, the Creamery, and the Former Residence, and can be seen on Figure 1. All four are currently mapped in a FEMA 100-year floodplain since the existing berm along the north edge of the property does not provide full flood protection and does not meet FEMA's levee accreditation standards. As a result, the structures are subject to a FEMA mapped risk, which represents the worst case scenario (i.e., it's mapped as if the berm does not exist).

Unlike the FEMA mapping methodology, the actual flood risk (and the question of actual project impact) is determined by several factors: the ability of the existing berm to remain intact during the design 100-year flood event; flooding potential from the east (flow-through flooding), and flooding potential from the west (backwater flooding). Balance has not performed the detailed seepage, stability, and scour analyses needed to determine if the berm is able to provide flood protection during the 100-year event. For the analysis of the two types of risk that follows, it is assumed that the berm would not fail during the 100-year event.

1. **Flow-Through Flooding Risk.** While the top of berm is, in general, above or just at the existing 100-year BFE, a low point exists near Highway 1 (see Figure 1). Under existing conditions, the modeled BFE at this low point is 19.8 feet compared to a low ground elevation of just under 18.0 feet based on best available topographic data. Actual flood elevations at the Barn Complex would be expected to be lower than this modeled elevation because the 100-year peak flow would likely not crest long enough and the low point would not be wide enough to equalize the water surface elevations on the inboard

and outboard sides of the berm. Instead, this flooding risk represents a high velocity, “flow-through” risk to the structures.
2. **Backwater Flooding Risk.** The Barn Complex is also subject to shallow backwater flooding from the west up to an elevation of 16.9 feet.[1] Unlike the high velocity flow that could be expected through the low point in the berm, this represents a stillwater elevation.

Post-Project Conditions

While some water surface elevations are expected to decrease with the construction of the Causeway, all four properties would still be mapped in the FEMA 100-year floodplain unless substantial work is completed to covert the berm to a FEMA-certified levee with the appropriate freeboard and structural integrity. The pre- and post-project mapped BFEs are shown on the cross-sections in Figure 1.

Again, if we assume that the existing berm could withstand the 100-year event, we can assess the residual risk to the structures:

1. **Flow-Through Flooding Risk.** The aforementioned flow-through risk decreases for all structures as the post-project BFE at the low point in the berm upstream of the Barn Complex is decreased by 0.5 feet[2].
2. **Backwater Flooding Risk.** Due to the larger volume of flow that will be routed under the Causeway and out to the South Arm of the Carmel Lagoon, the backwater flooding elevation increases by 0.2 feet (from elevation 16.9 to 17.1 feet) immediately downstream of the berm protecting the Barn Complex.

Improvement Options

Several options to mitigate the two types of flooding risk are available, including:

1. **Flow-Through Flooding Risk.** Although the flow-through flooding risk decreases with the construction of the Causeway, it is not fully eliminated. One option is to eliminate the flow-through risk altogether by improving the existing berm to provide additional freeboard and closing off the low point that allows flow-through flooding.
2. **Backwater Flooding Risk.** Backwater flooding risk will remain unless a) the existing berm is improved as above and the protection is extended around the west and south boundaries of the Barn Complex with a berm or a low flood wall or b) the affected buildings and/or the entire site is elevated.

Improving the exiting berm is a relatively inexpensive option that would involve adding additional fill to the existing berm to provide freeboard and protect from the high velocity

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[1] This backwater flooding originates in the South Arm of the Carmel Lagoon and would impact the site from the southwest coming around the end of the berm, since it does not fully enclose the area on the west and south.
[2] Counterintuitively, 100-year elevations on the downstream side of the proposed Causeway are expected to decrease when compared to existing conditions. In existing conditions, water must pond on the upstream side of Highway 1 before spilling over the top of the roadway. The Causeway will allow for more natural water conveyance, reducing flood elevations when compared to the presently-mapped “waterfall” condition over Highway 1.

flooding risk. This option would eliminate the flow-through flooding, but would not affect the backwater flood elevation of 17.1 feet just downstream of the berm. Extending the berm around the west and south is expected to be more expensive due to the length of required new berm/floodwall. This option has the potential to eliminate the backwater flooding risk, but would have some limited riparian habitat impact along the west side of the Barn building. This impact would require separate mitigation and compensatory habitat planting, which would contribute to the cost. Elevating the structures is likely the costliest, but would protect the elevated buildings from both mapped and actual flooding risk if completed in accordance with Monterey County's floodplain ordinance requirements.

Whitson Engineers' survey crew visited the property on September 12, 2016 to assess the existing first floor elevations (FFEs) for the four buildings. They are shown in the following table along with the lowest adjacent grade (LAG), the modeled pre and post-project 100-year BFEs, and the recommended target elevation that each building's first floor should be elevated to in order to comply with Monterey County's floodplain ordinance requirement of one foot above the BFE.

Building	Existing FFE (ft)	LAG (ft)	Pre-Project BFE (ft)	Post-Project BFE (ft)	Target FFE (ft)
Barn	17.7 (concrete floor, southern bay)	17.1 (next to concrete floor) 14.3 (next to NE corner)	17.2	17.3	18.3
Blacksmith Shop	18.5	16.1	17.5	17.5	18.5
Creamery	16.7 (western addition, slab on grade) 17.3 (eastern addition, slab on grade) 17.5 (raised wood floor)	15.5	17.6	17.6	18.6
Former Residence	18.7	16.0	18.5	18.3	19.3

As the table shows, the post-project BFE (the mapped flooding risk) is expected to increase by 0.1 feet in the vicinity of the barn. No change in mapped flooding risk is expected at the Blacksmith Shop and the Creamery. The Former Residence should expect to see a 0.2-foot reduction in BFE (see footnote 2 above).

Regardless of whether changes in the mapped risk are expected, the existing FFEs of the Barn, the Blacksmith Shop, and the Former Residence buildings are higher than both the pre- and post-project BFEs[3]. These FFEs are also higher than the pre- and post- project backwater elevations of 16.9 and 17.1 feet, respectively. Without improvements to the berm, the buildings are still subject to flow-through flooding, but the risk is reduced in the post-project scenario as the flood elevation at the low point in the berm decreases from 19.8 to 19.3 feet. Therefore, the CRFREE

[3] While the Blacksmith Shop does appear to have the required one foot of freeboard, the survey crew noted that it is presently elevated on blocks. While this provides the required freeboard, it does not provide adequate protection from velocity flows through the site and would not be considered properly anchored based on the floodplain ordinance requirements.

project does not expose these structures to a significantly increased risk of flooding with respect to FFEs during the 100-year event.

While the Creamery first floors (slab on grade and raised wood floor portions) are below the pre-project BFE, the CRFREE project does not increase the BFE at this location. The western addition of this building is subject to backwater flooding in pre-project conditions (flooding elevation of 16.9 feet compared to an FFE of 16.7 feet). Without further improvements, this risk is increased by 0.2 feet as the post-project backwater elevation increases to 17.1 feet. Similarly to the other buildings, the Creamery is subject to flow-through flooding if no berm improvements are made, but the risk is reduced in the post-project scenario.

Closing

If you have any additional questions about this memo, please do not hesitate to contact us.

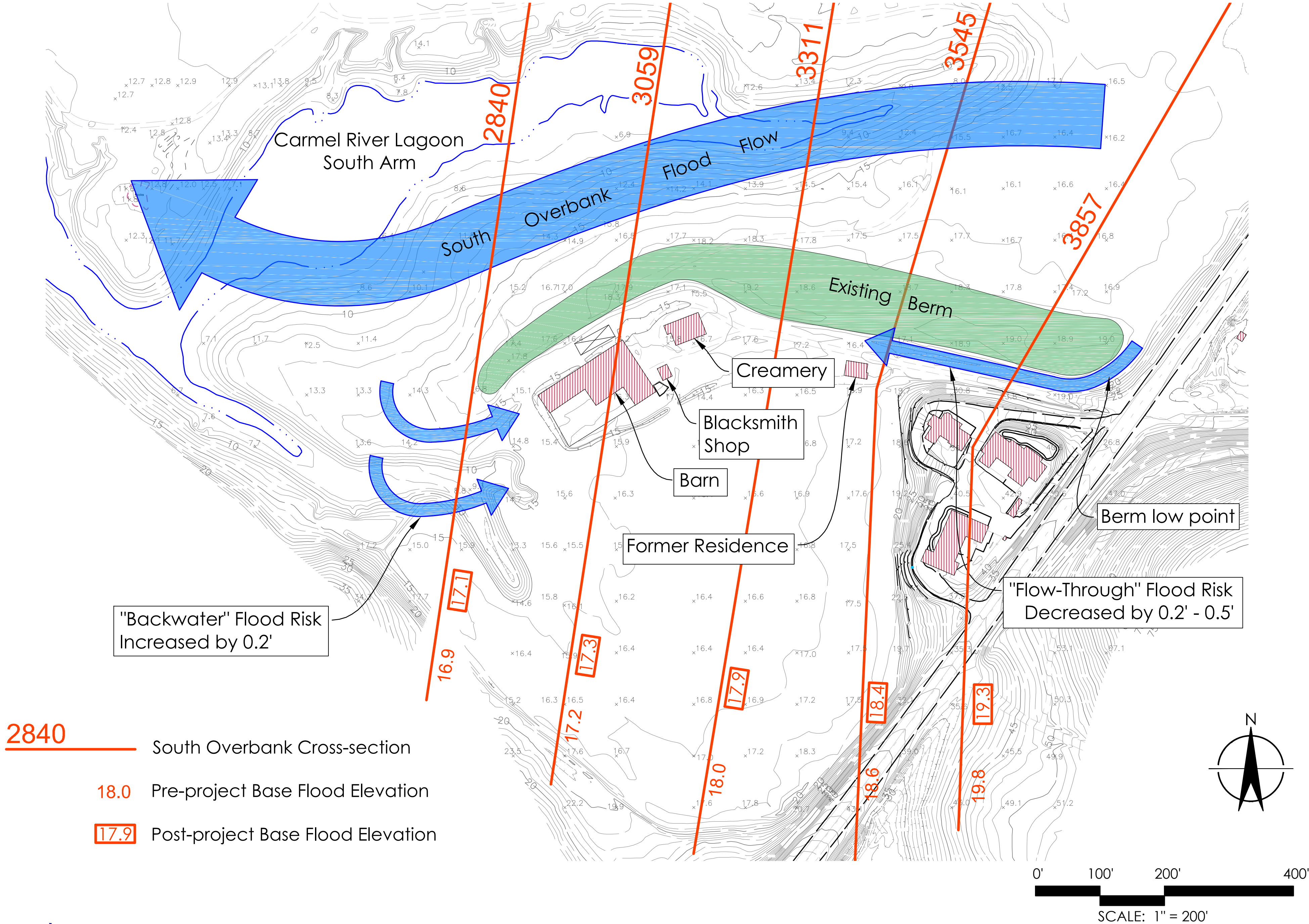

Figure 1. **100-year flood risk assessment at the State Parks Barn Complex, Monterey County, California.**

Balance Hydrologics, Inc.

214044 Flooding Risk Assessment.dwg

Supplementary 2D Model Results Memo

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MEMO

To: Josh Harwayne, Denise Duffy & Associates

From: Anne Senter, PhD and Ed Ballman, Principal Engineer

Date: June 12, 2018

Subject: Supplementary 2D Model Results for CRFREE Project Existing Conditions, Proposed Conditions, and a Reduced Conveyance Alternative

INTRODUCTION

Balance Hydrologics, Inc. has been the geomorphic, hydrologic, and hydraulic technical design partner for the CRFREE Project (Project) for over a decade. The Project is located at the Odello Property along the southern bank of the Carmel River Valley immediately upstream from Highway 1. An initial design alternatives report (Balance Hydrologics, 2007) was followed by supplemental technical analyses (Balance Hydrologics, 2008), which led to a preferred design basis report (Balance Hydrologics, 2015) and a number of additional memoranda (see **Project Documents** section directly after the **Reference** section for a complete list) addressing specific questions in support of the preferred Project design. In addition, we were technical partners in the CSA50 stormwater management and flood control analyses and report (Balance Hydrologics, 2014) that analyzed flood conditions for the properties on the north overbank of the Carmel River local to the Odello Property.

In 2016, HEC-RAS released a 2D model platform that has since been utilized to realize the benefits of higher resolution modeling at this latter stage of the Project, including investigation of predicted differences between existing and proposed Project conditions to downstream Carmel Area Wastewater District (CAWD) facilities (Balance Hydrologics, 2017). This new model platform allows additional ability to inspect hydraulic results at relatively fine-grained features, model the river/overbank system without manually splitting the flow regimes between the main channel and complex overbank flow paths. It is important to note that the technical design bases for the proposed Project as set forth in the design basis report (Balance Hydrologics, 2015) have not changed with this upgrade in the modeling platform.

This memo summarizes HEC-RAS 2D model results at specific locations pertinent to overall understanding of the Project as well as to regulatory impacts analysis, and provides a graphical overview of the entire Project footprint that helps to enhance understanding of differences between existing conditions, proposed conditions, and a reduced alternative.

The reduced alternative is under consideration as part of the CEQA regulatory process. A limited conceptual design description and subsequent discussion focuses on hydraulics, geomorphology, and engineering aspects of the concept in comparison to the proposed project and existing conditions.

HEC-RAS 2D MODELING PARAMETERS

Modeling parameters used for existing and proposed conditions were discussed (Balance Hydrologics, 2017) and presented to the Project team in a PowerPoint slide presentation (Balance Hydrologics, 2018; **Attachment A**). Briefly, the existing condition topographic digital elevation model was built using LiDAR data and cross-section data as well as supplementary survey data from Whitson Engineers. The proposed Project and the reduced alternative digital elevation model were modified in CAD according to design plans and conceptual design plans, respectively. Model extents included CSA-50, the Odello Floodplain, State Parks land at the south end of the lagoon, all CAWD facilities, and the lagoonal system and interface with the Pacific Ocean. Downstream tailwater conditions were set to normal depth with slope of 0.005. Roughness coefficients ranged from 0.035 in the mainstem channel to 0.10 for the riparian corridor, and were mapped accordingly depending on the model run. Model mesh cell sizes averaged 20-feet by 20-feet, which allowed for a similar scale of resolution for results.

Models were run for existing conditions, proposed conditions, and reduced alternative conditions for 10-year and 100-year flood flows. The peak flow of the storm event of February 21, 2017 was gaged at 9,200 cfs at the USGS gage 11143250 Carmel River near Carmel, California. The FEMA 10-year peak flood is estimated at 9,500 cfs; gaged flow was used to validate the 10-year design flow. The 100-year peak flood consistent with past modeling, was taken as 22,700 cfs as per the FEMA Flood Insurance Study.

REDUCED ALTERNATIVE DESIGN

As part of the CEQA process, an alternative with reduced floodplain grading and causeway length was chosen for exploration. Basic reduced alternative design elements include (a) lowering the elevation of the existing notch so that the floodplain is engaged at approximately the 2-5-year flood flow (same as proposed condition), (b) grading a 30-foot wide channel with 8:1 side slopes through the floodplain without extensive grading otherwise, (c) reducing the length of the causeway to 180 feet, and (d) continuing the one-channel grading concept into the lagoon. Lowering the existing notch to the same elevation as in the proposed project was chosen as the most efficient action by which to engage the floodplain at the 2-5-year flood flow. Because flows through one notch would be limited compared flows through multiple notches, the floodplain channel was reduced to one thread instead of the two distributary channels in the proposed project and sized appropriately for smaller flood flow conditions to approximately one-half of the proposed project channel capacity. Likewise, with more limited flows onto the floodplain, the size of the causeway was reduced by one-half.

Figure 1 presents a plan view and **Figure 2** presents three cross-sectional views of the grading plan. Note that the proposed design grading limits are presented on the plan view along with the reduced alternative grading limits. On the cross-sectional views, existing, proposed, and reduced alternative grades are presented for comparison.

HEC-RAS 2D MODEL RESULTS

Predicted results from the existing, proposed, and reduced alternative 2D model runs are reported at a set of specific locations as identified in **Figure 3**.

Positive differences between 10-year model results presented in **Table 1** and 100-year model results in **Table 2** indicate areas where new design elements compared to existing conditions, either under the proposed design or reduced alternative, increase flow conditions at certain locations. Negative differences indicate a reduction in flow conditions. Other information is explained via notes on the figures and tables.

Table 1 indicates that the four notches[1] in the proposed design are predicted to become engaged as expected prior to the 10-year flood flow (XS-2 through 5). Likewise, expansion of the existing notch in the reduced alternative allows for increased engagement at the 10-year flow compared to existing conditions (XS-2). Flood conditions improve substantially at the downstream Hwy 1 bridge (XS-6), CAWD access road (XS-7), and CAWD plant (XS-8) under the proposed design, and improve at a reduced level under the reduced alternative. The proposed design allows for more engagement of flows with the Odello floodplain, as designed to achieve Project goals, whereas the reduced alternative does not. Flows at the CAWD outfall pipe (XS-14) are greater in the proposed design, with higher water surface elevations and velocities as expected with increased flows through the floodplain.

Table 2 indicates that the four notches in the proposed design (XS-2 through XS-5) are predicted to be engaged as expected in the 100-year flood flow. Likewise, a lowering of the elevation of the existing notch (XS-2) in the reduced alternative allows for limited increased engagement at the 100-year flow compared to existing conditions. Flood conditions improve substantially at the downstream Hwy 1 bridge (XS-6), CAWD access road (XS-7), and CAWD plant (XS-8) under the proposed design, and improve at a reduced level under the reduced alternative. The proposed design allows for more engagement of flows with the Odello floodplain, as designed to achieve Project goals, whereas the reduced alternative does not. The area around the Red Houses (XS-9) remains ponded under reduced alternative conditions. Conditions upstream of the causeway (XS-10) and downstream of the causeway (XS-12) vary between proposed and reduced alternatives. The larger causeway opening and the gentler slope of the distributary channels in the proposed condition prevent excessive backwatering upstream, with maximum flow velocities downstream of the causeway under 6 ft/s. The narrower causeway in the reduced alternative slows upstream velocities (at XS-10) and the higher slope through the causeway increased velocities (at XS-12). Hwy 1 (at XS-11) still overtops in the reduced alternative, albeit at a rate of just 200 cfs. A modification to a slightly longer causeway opening in the reduced alternative could eliminate overtopping. The State Park Barn complex (XS-13) remains slightly ponded under proposed conditions as well as in the reduced alternative. There is a reduction in water surface elevation in the reduced alternative because smaller flows are allowed on the floodplain and through the causeway. The substantially larger flows with additional notches and longer causeway create a slightly higher backwater condition at the State Parks Barn complex in the proposed project condition as flows work through the south and western arms of the lagoon. Lower flows through

[1] "Notch 2" includes the two upstream proposed notches; 2d model results were more easily reported using one cross-section to report combined flow, water surface elevation, and velocity results.

the lagoon in the reduced alternative result in lower water surface elevations from backwatering at the State Parks Barn complex. Overland flow at the State Parks Barn complex under the reduced alternative is directly related to overtopping at Hwy 1, whereas less overland flow in the proposed project condition is from the higher flows through the larger causeway creating a backwater effect at the berm/roadway into the State Parks area. Flows at the CAWD outfall pipe (XS-14) are greater in the proposed design, with higher water surface elevations and velocities as expected with increased flows through the floodplain and into the lagoon.

Figure 4 through **Figure 9** provide a snapshot of each set of model results using flow depth as the visual background informative. Flow depth is intuitively easy to understand by allowing for identification of the primary flow pathways for each model run. Each table reports associated flow conditions as predicted at each cross-section.

GEOMORPHIC DISCUSSION OF REDUCED ALTERNATIVE

The conceptual design basis for the reduced alternative was to explore the possibility of a project that attains some of the overarching project goals while potentially reducing project impacts, grading, and costs. Interpretation of modeling predictions show that in the reduced alternative, benefits do accrue when compared to existing conditions, but to a lesser extent than in proposed project conditions except for two specific cases: the State Park Barn complex (XS-13) and the CAWD outfall pipe (XS-14). In all other areas within or downstream of the Odello floodplain (XS-2 through XS-12), the reduced alternative provides fewer benefits than the proposed project. Differences between existing, proposed, and reduced alternative results are conveyed in the 10-year results tabulation of **Table 1** and illustrations of **Figures 4-6**, and the 100-year results tabulation of **Table 2** and illustrations of **Figures 7-9**.

State Parks Barn complex

Under the reduced alternative, the State Parks Barn complex (**Figure 8**, XS-13) is predicted to experience lower water surface elevations (-0.4 feet) associated with backwatering effects in the 100-year flood event (see table embedded in **Figure 9**, and **Table 2**). This condition is directly related to the reduction in flow onto the floodplain at the upstream extent of the project, where one notch is engaged at high flows rather than multiple notches in the proposed project. Less flow onto the upper end of the floodplain mainfests through the entire floodplain, the narrower causeway (which also backs up flows), and into the lagoon as less total flood flow volume. The reduction in causeway length from project project conditions (1/2 of proposed length), will likely result in some limited overtopping of Hwy 1 under the 100-year flood flow. Flows that overtop Hwy 1 would spill down the State Parks roadway as overland flow and connect with backwatered flows from the lagoon. A modest increase in a still-reduced causeway length may eliminate flood flow overtopping of Hwy 1.

Conversely, under the proposed project, the State Parks Barn complex (**Figure 8**, XS-13) is predicted to experience slightly higher water surface elevations (+0.1 feet) associated with backwatering effects (see table embedded in **Figure 8**, and **Table 2**). This condition is directly related to the increase in flow onto the floodplain at the upstream extent of the project, where multiple notches are engaged at high flows rather than the single existing notch (**Figure 7**) and a single lower elevation notch in the reduced alternative (**Figure 9**). More flow onto the upper end of the floodplain mainfests through the entire floodplain, through the wider causeway (less backed up flow; see differences between **Figure 8** and **Figure 9** upstream of the causeway near XS-10), and into the lagoon as more total flood flow volume. The wider causeway eliminates overtopping of Hwy 1 under the 100-year flood flow, and grading eliminates backwatering at the Red Houses (XS-9). In the proposed project, backwatered flows spill over the berm west of Hwy 1 and down the State Parks roadway as overland flow and may connect with backwatered flows from the lagoon.

In both the proposed project and reduced alternative scenarios, the backwater impact and the overland flow impact at the State Park Barn complex could be eliminated by select placement of a levee and an increase in berm elevation, respectively.

CAWD outfall

Flow, water surface elevation, and velocities under the reduced alternative at the CAWD outfall (XS-14) are predicted to be lower in the 10-year flood event (**Table 1**) and in the 100-year flood event (**Table 2**) compared to the proposed project. In both cases, conditions are directly related to the differences between increases in flow onto the floodplain at the upstream extent of the project, where multiple notches are engaged at high flows in the proposed project (**Figure 8**) compared to the single existing notch (**Figure 7**) or a single lower elevation notch in the reduced alternative (**Figure 9**). Predicted values for flow (10,600 cfs), water surface elevation (13.4 feet), and velocity (9.3 ft/s) are nominally smaller for the reduced alternative than the proposed project flow (11,300 cfs), water surface elevation (13.6 feet), and velocity (9.5 ft/s). In both scenarios, water surface elevations are high enough to inundate the outfall pipe, and velocities are fast enough to promote scour.

Reduced benefits under the reduced alternative

Limited connectivity in the reduced alternative from the upstream extent of the project to the lagoon at the downstream extent provides provides fewer benefits than under the proposed project. The reduced alternative compared to the proposed project would yield less floodplain inundation (less grading), less channel complexity (no logs with rootwads, no sediment sequestration elements, fewer notches and MFCAs), less channel and causeway capacity (smaller channel, narrower causeway), less floodplain restoration area (less grading), less topographic diversity (no islands, less restored vegetation) and less flood control for CSA 50, the CAWD access road, and the CAWD facilites (fewer flood flows moving onto the floodplain).

The risk of channel erosion and scour potential increases for a number of interconnected reasons associated with a less stable geomorphic configuration of the floodplain channel in the reduced alternative. Limited floodplain grading would translate into a steeper slope at some point in the flowline as the channel descends to the lagoon. In the reduced alternative, the slope of the floodplain increases as it approaches and passes under the causeway to the lagoon (XS-12, **Table 2**, **Figure 9**). The combination of a steeper slope and a narrower causeway promotes backwatering upstream of the causeway and then faster velocities downstream. As velocities increase, the risk of headcutting channel scour and erosion (from downstream to upstream) increases.

The risk of channel avulsion (change in the direction of the main Carmel River flow path from its current course onto the floodplain) increases with the limitation of one notch through which flows will enter the floodplain during flood events (XS-2, **Table 2**, **Figure 9**). As flows concentrate through the notch, scouring and erosive forces could widen or deepen the notch and consequently the channel, which could potentially lead to gouging of rills and crevasses throughout the channel extent to the lagoon.

If any of these “worst-case” erosion/scour/avulsion scenarios were to occur, the risk of the Carmel River main channel avulsing onto the floodplain would increase substantially. The narrower causeway is limited to conveying flows of about 3500 cfs, so an avulsive shift of the channel onto the floodplain under a reduced alternative design would also create significant flooding problems at Hwy 1.

Reduced benefits in comparison to the proposed project include the potential for sediment transport into the lagoon to increase with the elimination of sediment sequestration elements and with any increase in erosion. Less floodplain grading would yield a higher elevation ground surface which is then a further distance from the local groundwater source. Less available groundwater for riparian plantings could lead to less vigorous vegetation establishment. Floodplain and channel habitat complexity and enhancements would decrease with the elimination of streamwood log placements, islands, sediment sequestration elements, and fewer MFCA areas.

Important flood control benefits would be reduced under the reduced alternative compared to the proposed project, though improved compared to the existing condition, as illustrated by comparison of the flood conveyancy capacity of the existing one notch conditions (4,100 cfs), the one deeper one notch in the reduced alternative (8,000 cfs), and multiple notches of the proposed project (13,000 cfs). The reduced conveyance between the proposed project and reduced alternative would translate into fewer flood control benefits for CSA50 (from a qualititatively ranked perspective – not currently quantified), the CAWD access road (XS-7, **Table 1** and **Table 2**, **Figure 8** and **Figure 9**) and CAWD facilities (XS-8, **Table 1** and **Table 2**, **Figure 8** and **Figure 9**).

Overall, the reduced alternative provides more floodplain engagement and more flood control than under existing conditions, but substantially fewer benefits than the proposed project.

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Balance Hydrologics, 2007, Design Alternatives Analysis for Floodplain Restoration at the Odello Property, Lower Carmel River Valley, County of Monterey, California; consulting report prepared for the Big Sur land Trust, 36 p.

Balance Hydrologics, 2008, Supplemental Analyses for Floodplain Restoration at the Odello Property, Lower Carmel River Valley, County of Monterey, California; consulting report prepared for the Big Sur Land Trust, 61 p.

Balance Hydrologics, 2014, County Service Area 50 Final Lower Carmel River Stormwater Management and Flood Control Report; consulting report prepared for Monterey County Resource Management Agency, 378 p.

Balance Hydrologics, 2015, Carmel River Floodplain Restoration and Environmental Enhancement Project 35% Design Basis Report; consulting report prepared for Monterey County Resource Management Agency, 73 p.

Balance Hydrologics, 2017, Parameters for supplemental modeling for the CRFREE Project, County of Monterey; consulting letter prepared for Monterey County Resource Management Agency, 3 p.

Balance Hydrologics, 2018, Carmel River FREE – 2-dimensional hydraulic modeling, review of preliminary results; PowerPoint presentation prepared for CRFREE team, 23 slides.

PROJECT DOCUMENTS

The documents listed in this section have been written to convey increasingly advanced technical analyses, observations, conclusions, and designs as the Project has matured over the past 10 years or so. The new HEC-RAS 2D model results presented in this memo provide greater detail and thus a more precise understanding of predicted results for existing, proposed, and a limited reduced conveyance alternative. Model results represented in this memorandum hereby supersede previous model results. The proposed CRFREE Project design plans have not changed as a result of new model results.

1. Balance Hydrologics, 2007, Preliminary hydraulic analyses of proposed design alternatives along the lower Carmel River; consulting report prepared for the Big Sur Land Trust, 8 p.
2. Balance Hydrologics, 2007, Design Alternatives Analysis for Floodplain Restoration at the Odello Property, Lower Carmel River Valley, County of Monterey, California; consulting report prepared for the Big Sur land Trust, 36 p.
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4. Balance Hydrologics, 2008, Hydraulic modeling summary of the Carmel River causeway along Highway 1, consulting memo prepared for Avila and Associates Consulting Engineers, 31 p.
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7. Balance Hydrologics, 2009, Carmel River floodplain restoration design alternatives assessment, consulting memo prepared for Big Sur Land Trust, 7 p.
8. Balance Hydrologics, 2014, County Service Area 50 Final Lower Carmel River Stormwater Management and Flood Control Report; consulting report prepared for Monterey County Resource Management Agency, 378 p.
9. Balance Hydrologics, 2015 (updated from 2008 memo), Large woody debris (drift) potential at the proposed Highway 1 causeway restoration, Carmel River, Monterey County, California; consulting memo prepared for Avila and Associates Consulting Engineers, 8 p.
10. Balance Hydrologics, 2015, Carmel River Floodplain Restoration and Environmental Enhancement Project 35% Design Basis Report; consulting report prepared for Monterey County Resource Management Agency, 73 p.
11. Balance Hydrologics, 2015, Anticipated changes in downstream base flood elevations due to the Carmel River Floodplain Restoration and Environmental Enhancement Project; consulting memo prepared for Denise Duffy & Associates, 5 p.
12. Balance Hydrologics, 2016, Flood benefits and impacts assessment, Lower Carmel River notch expansion project, Monterey County; consulting memo prepared for Monterey County Resource Management Agency, 10 p.

13. Balance Hydrologics, 2016, Potential impacts to the State Parks Barn Complex due to the Carmel River Floodplain Restoration and Environmental Enhancement project; consulting memo prepared for Whitson Engineers, 5 p.
14. Balance Hydrologics, 2016, Changes to 10- and 100-year water surface elevations in the vicinity of the CAWD pipe due to the Carmel River Floodplain Restoration and Environmental Enhancement project; consulting memo prepared for Whitson Engineers, 3 p.
15. Balance Hydrologics, 2016, Steelhead behavior as it relates to the Carmel River Floodplain Restoration and Environmental Enhancement project; consulting memo prepared for CRFREE team, 2 p.
16. Balance Hydrologics, 2017, Response to concerns raised in the South Arm Lagoon velocity analysis for the CRFREE Project, Monterey County; consulting memo prepared for Whitson Engineers, 7 p.
17. Balance Hydrologics, 2017, Carmel River Floodplain Restoration and Environmental Enhancement project design considerations and analysis related to fish stranding; consulting memo prepared for CRFREE team, 5 p.
18. Balance Hydrologics, 2017, Parameters for supplemental modeling for the CRFREE Project, County of Monterey; consulting letter prepared for Monterey County Resource Management Agency, 3 p.
19. Balance Hydrologics, 2018, Carmel River FREE – 2-dimensional hydraulic modeling, review of preliminary results; PowerPoint presentation prepared for CRFREE team, 23 slides.

10-year event

Cross-Section	Results locations	Change from Existing to Proposed (+/-)			Change from Existing to Reduced (+/-)		
		Q (cfs)	WSE (ft)	Vel (ft/s)	Q (cfs)	WSE (ft)	Vel (ft/s)
1	Mainstem, upstream of Project	no change	-0.3	+0.5	no change	-0.1	+0.4
2	Existing notch	+400	+1.6	+1.9	+700	+1.9	+2.2
3	New notch[1]	+900	+2.1	+2.2	no change[1]	no change[1]	no change[1]
4	New notch[1]	+700	+2.2	+2.4	no change[1]	no change[1]	no change[1]
5	New notch[1]	+1500	+2.9	+3.1	no change[1]	no change[1]	no change[1]
6	Mainstem at Hwy 1 Bridge	-3500	-1.6	-2.4	-700	-0.1	-1.4
7	CAWD access road	-2200	-0.9	-3.4	-500	-0.1	-1.2
8	Mainstem at CAWD Plant	-1200	-0.9	-0.9	-200	-0.1	+0.2
9	Red Houses	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]
10	Upstream of causeway	+3500	+3.8	+3.5	+700	+1.9	+4.3
11	Overtopping Hwy 1	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]
12	Downstream of causeway	+3500	+3.8	+2.2	+700	+1.9	+4.3
13	State Parks Barn complex	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]	no change[2]
14	CAWD outfall pipe	+1200	+0.4	+1.3	+200	+0.1	+0.3

Note: [1] *New notches do not exist in existing conditions or reduced alternative conditions.*
[2] *The Red Houses are above the 100-year FEMA base flow elevation (BFE) in proposed conditions.* .

Balance Hydrologics, Inc.

Table 1. **Model result differences for the 10-year flood event between existing conditions, proposed conditions, and reduced alternative conditions.**

100-year event

Cross-Section	Results locations	Change from Existing to Proposed (+/-)			Change from Existing to Reduced (+/-)		
		Q (cfs)	WSE (ft)	Vel (ft/s)	Q (cfs)	WSE (ft)	Vel (ft/s)
1	Mainstem, upstream of Project	no change	-0.6	1.4	no change	-0.5	+1.2
2	Existing notch	+2400	-0.1	-2.1	+3900	-0.3	-1.8
3	New notch[1]	+2800	+4.9	+3.4	no change[1]	no change[1]	no change[1]
4	New notch[1]	+1300	+4.8	+3.2	no change[1]	no change[1]	no change[1]
5	New notch[1]	+2400	+5.3	+4.1	no change[1]	no change[1]	no change[1]
6	Mainstem at Hwy 1 Bridge	-8100	-2.3	-4.0	-3100	-0.7	-2.5
7	CAWD access road	-5600	-1.0	-1.0	-2100	-0.2	no change
8	Mainstem at CAWD Plant	-2100	-1.2	-1.5	-800	-0.3	-0.2
9	Red Houses	not backwatered [2]	-4.2[2]	-1.1[2]	no change	-0.6	+1.5
10	Upstream of causeway	+8700	-7.5	+3.8	+3500	-0.8	+0.8
11	Overtopping Hwy 1	-4300	-	-	-4100	-0.6	-4.3
12	Downstream of causeway	+8700	-0.3	+5.7	+3500	+1.4	+7.8
13	State Parks Barn complex	backwatered	+0.1	-[3]	backwatered	-0.4	-[3]
14	CAWD outfall pipe	+1100	+0.3	+0.7	+400	+0.1	+0.5

Note: [1] *New notches do not exist in existing conditions or reduced alternative conditions.*
[2] *The Red Houses are above the 100-year FEMA base flow elevation (BFE) in proposed conditions.*
[3] *Backwatered locations may feature eddying velocities up to 1.5 ft/s.*

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Table 2. **Model result differences for the 100-year flood event between existing conditions, proposed conditions, and reduced alternative conditions.**

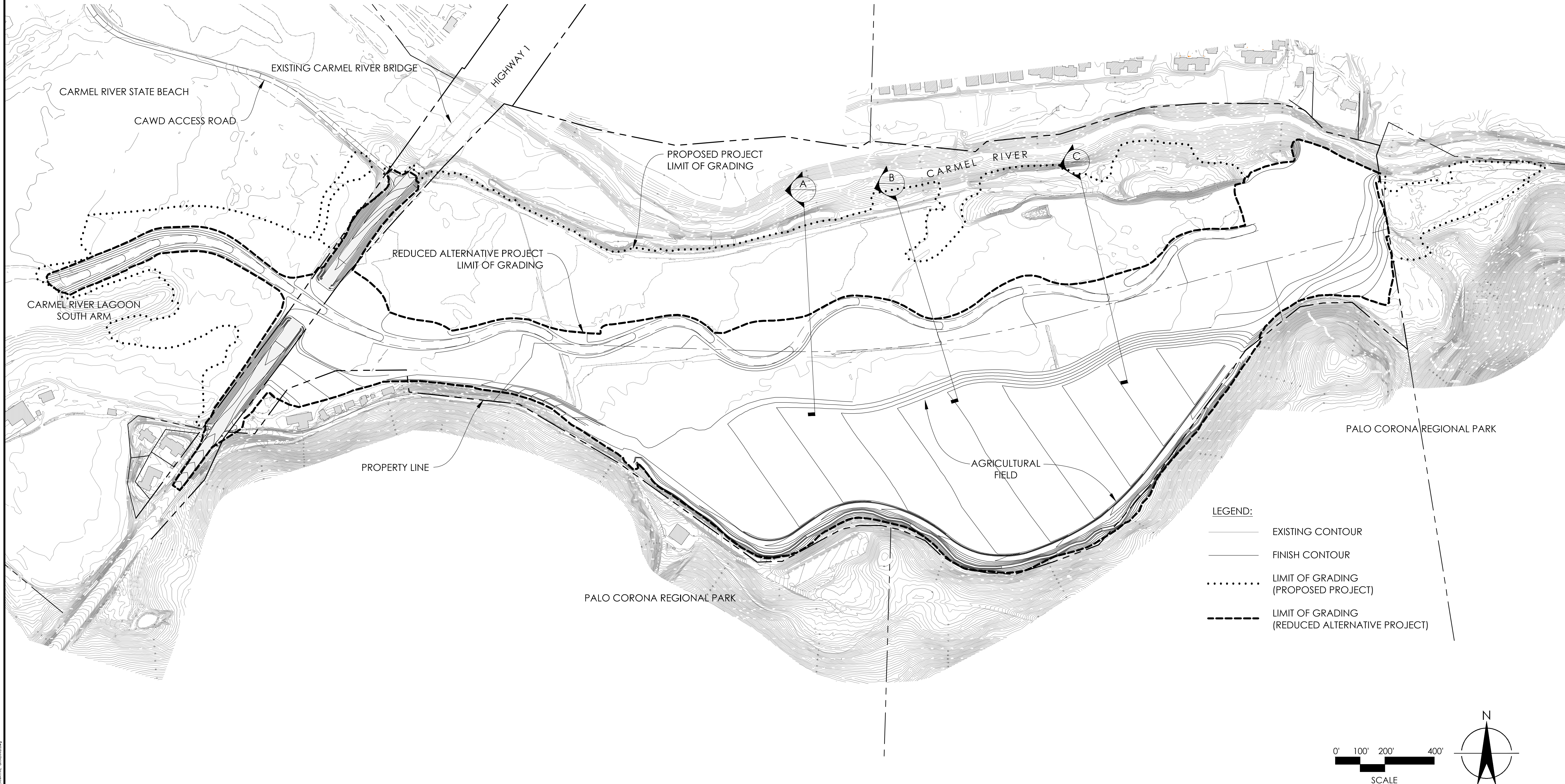


FIGURE 1. PLAN VIEW OF THE REDUCED ALTERNATIVE GRADING LIMITS

BIG SUR
LAND TRUST

CARMEL RIVER FLOODPLAIN RESTORATION AND ENVIRONMENTAL ENHANCEMENT PROJECT

PROJECT No. 7200
COUNTY PLAN SET

REVISIONS:		
BY:	DATE:	DESCRIPTION:

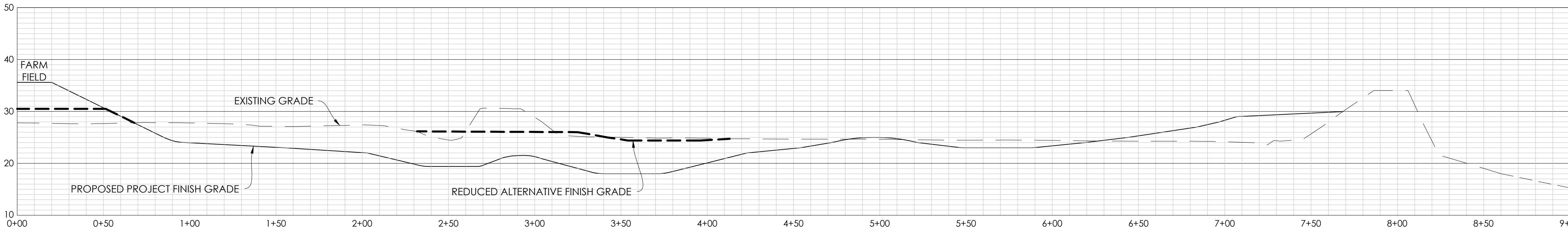
DATE: 05/30/2018
SCALE: 1" = 200'
DRAWN BY: DJ
JOB #: BH # 214044

NOT FOR CONSTRUCTION

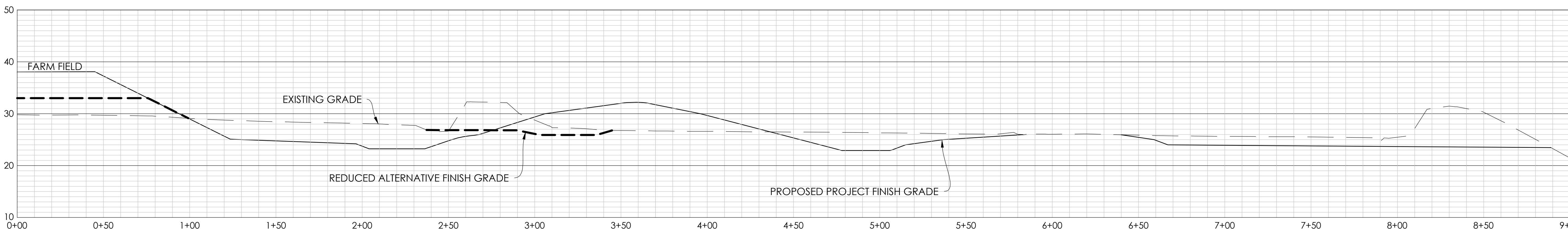
Balance Hydrologics, Inc.
800 Bancroft Way · Suite 101 · Berkeley, CA 94710
tel (510) 704-1000 · fax (510) 704-1001 · www.balancehydro.com

SHEET

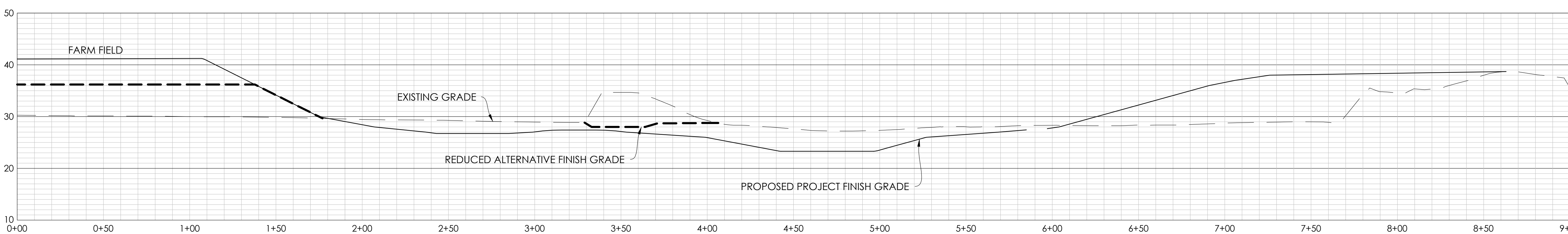
OF



A SECTION
SCALE: 1" = 30' HORIZONTAL
1" = 10' VERTICAL



B SECTION
SCALE: 1" = 30' HORIZONTAL
1" = 10' VERTICAL



C SECTION
SCALE: 1" = 30' HORIZONTAL
1" = 10' VERTICAL

FIGURE 2. CROSS-SECTIONAL VIEWS OF THE REDUCED ALTERNATIVE

SECTIONS

BIG SUR LAND TRUST

RESOURCE MANAGEMENT AGENCY
PLANNING · BUILDING SERVICES · PUBLIC WORKS · ENVIRONMENTAL SERVICES

CARMEL RIVER FLOODPLAIN RESTORATION
AND ENVIRONMENTAL ENHANCEMENT PROJECT
PROJECT No. 7200
COUNTY PLAN SET

REVISIONS:		
BY:	DATE:	DESCRIPTION:

DATE: 05/30/2018
SCALE: AS NOTED
DRAWN BY: DJ
JOB #: BH # 214044

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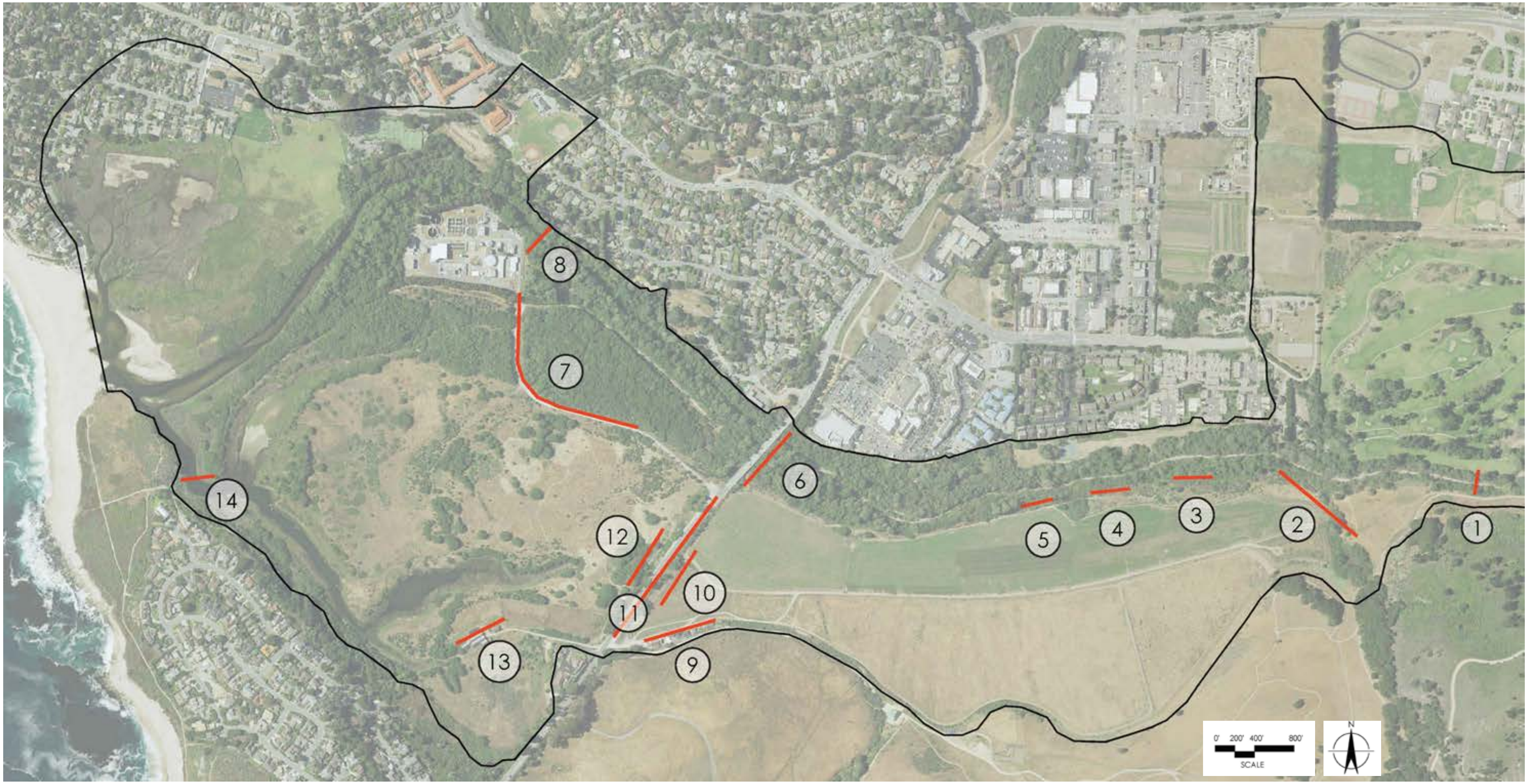
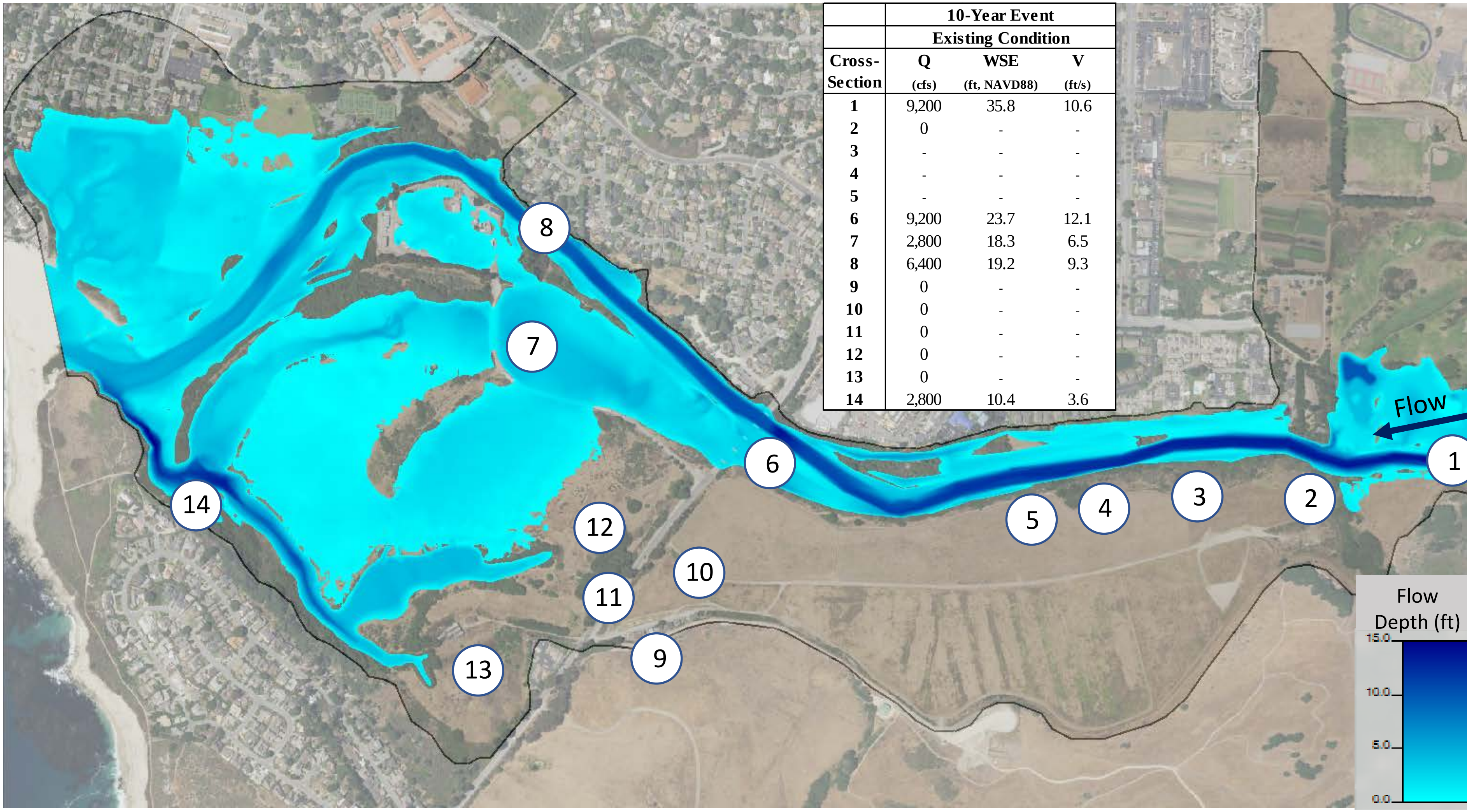
OF



Note: The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.

Balance Hydrologics, Inc.

Figure 3. **Cross-section locations where depth, velocity, water surface elevation, and flow results are presented (if applicable) from each HEC-RAS 2D model run.**

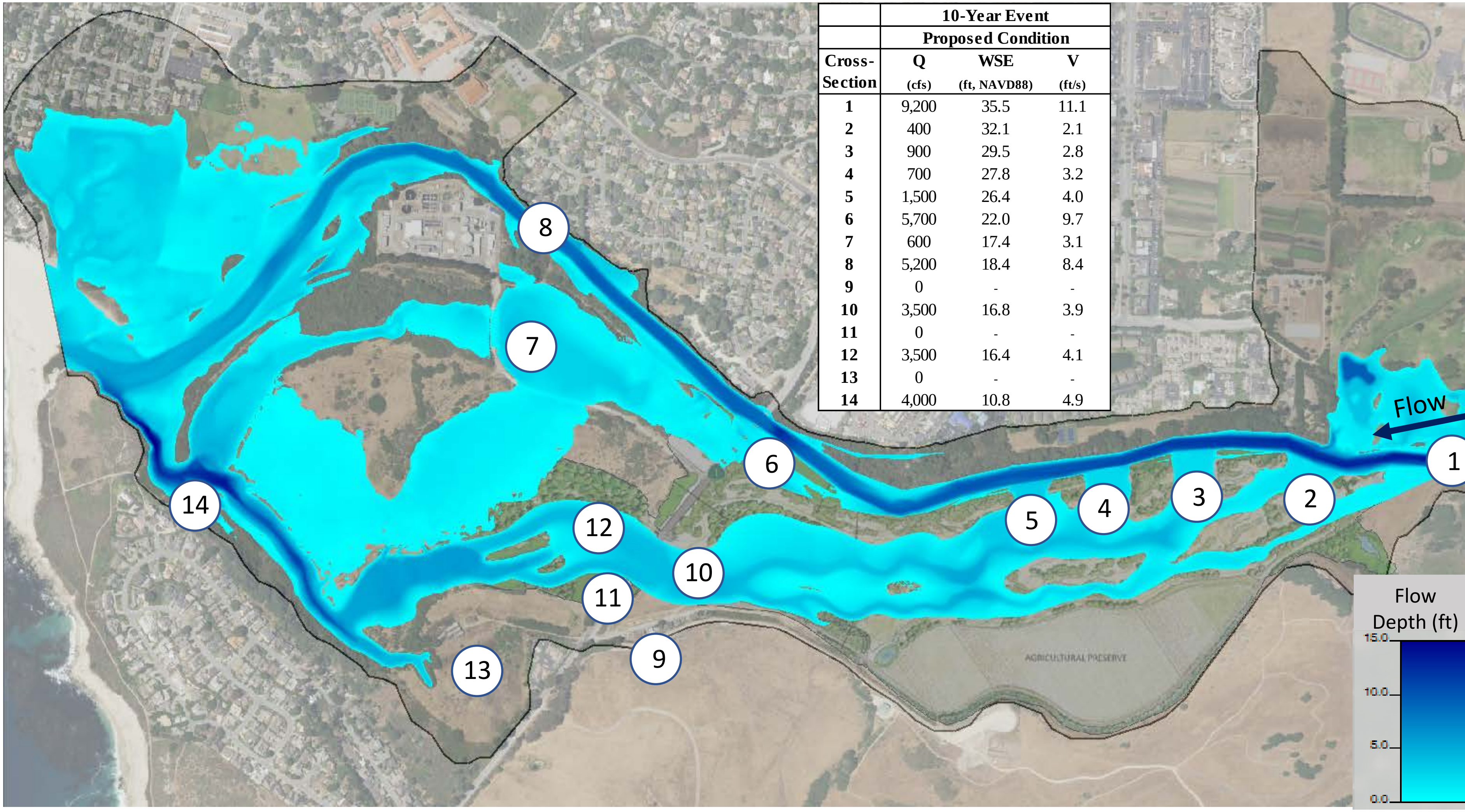


	10-Year Event		
	Existing Condition		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	9,200	35.8	10.6
2	0	-	-
3	-	-	-
4	-	-	-
5	-	-	-
6	9,200	23.7	12.1
7	2,800	18.3	6.5
8	6,400	19.2	9.3
9	0	-	-
10	0	-	-
11	0	-	-
12	0	-	-
13	0	-	-
14	2,800	10.4	3.6

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 4. **2D results at the 10-yr modeled flow of the existing site conditions.**

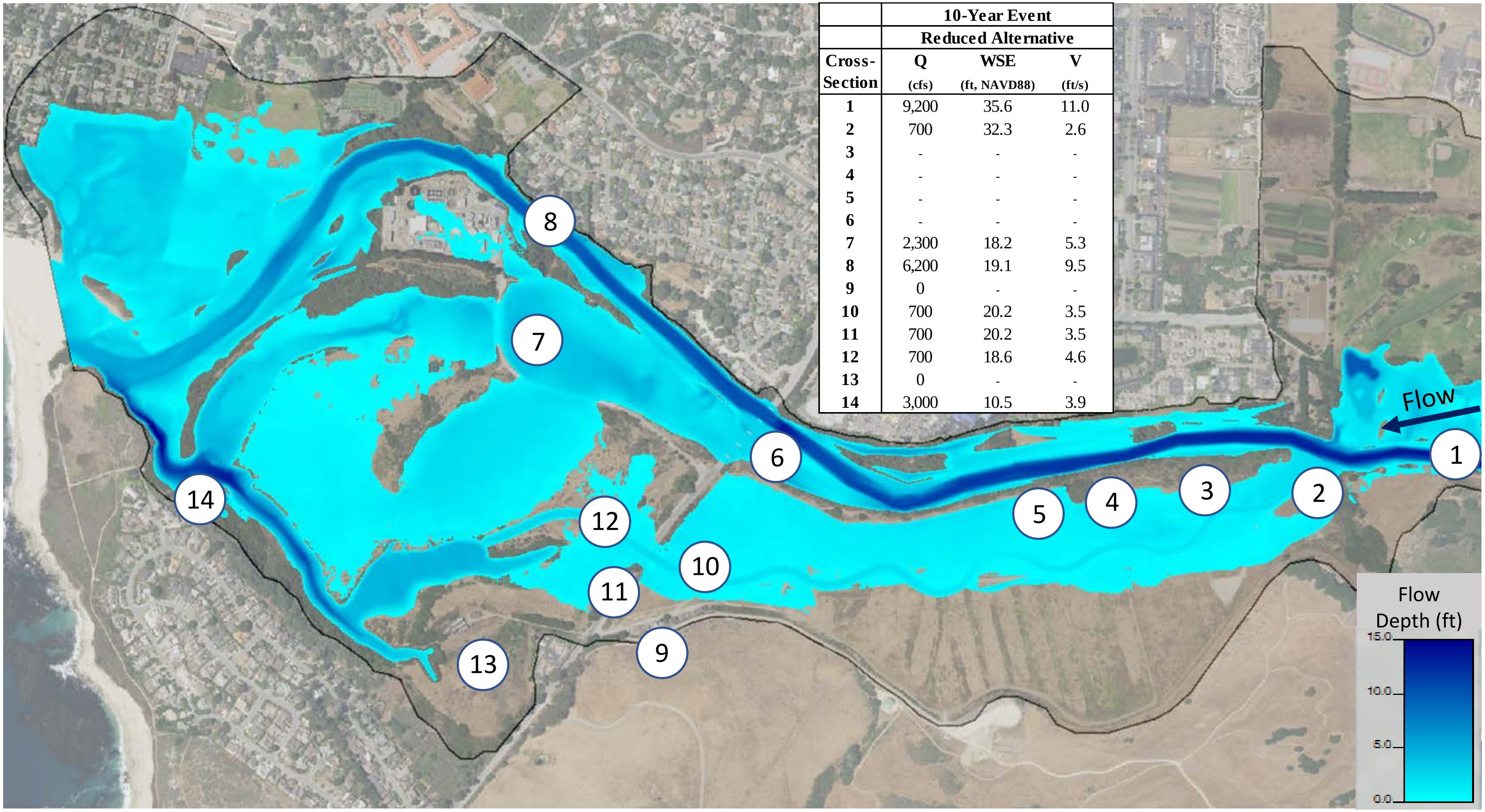


	10-Year Event		
	Proposed Condition		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	9,200	35.5	11.1
2	400	32.1	2.1
3	900	29.5	2.8
4	700	27.8	3.2
5	1,500	26.4	4.0
6	5,700	22.0	9.7
7	600	17.4	3.1
8	5,200	18.4	8.4
9	0	-	-
10	3,500	16.8	3.9
11	0	-	-
12	3,500	16.4	4.1
13	0	-	-
14	4,000	10.8	4.9

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 5. **2D results at the 10-yr modeled flow of the proposed design conditions.**

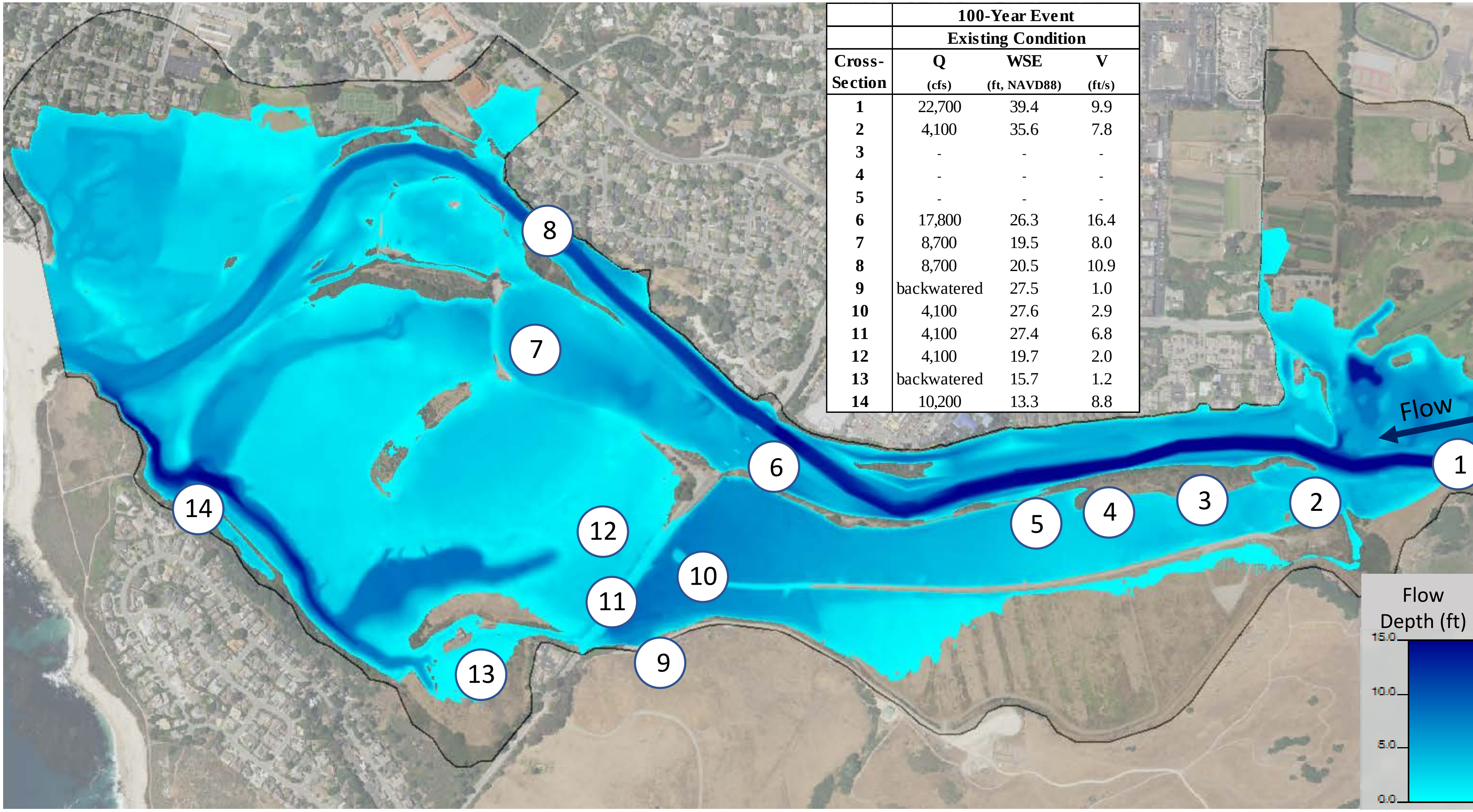


	10-Year Event		
	Reduced Alternative		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	9,200	35.6	11.0
2	700	32.3	2.6
3	-	-	-
4	-	-	-
5	-	-	-
6	-	-	-
7	2,300	18.2	5.3
8	6,200	19.1	9.5
9	0	-	-
10	700	20.2	3.5
11	700	20.2	3.5
12	700	18.6	4.6
13	0	-	-
14	3,000	10.5	3.9

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 6. **2D results at the 10-yr modeled flow of the reduced alternative conditions.**

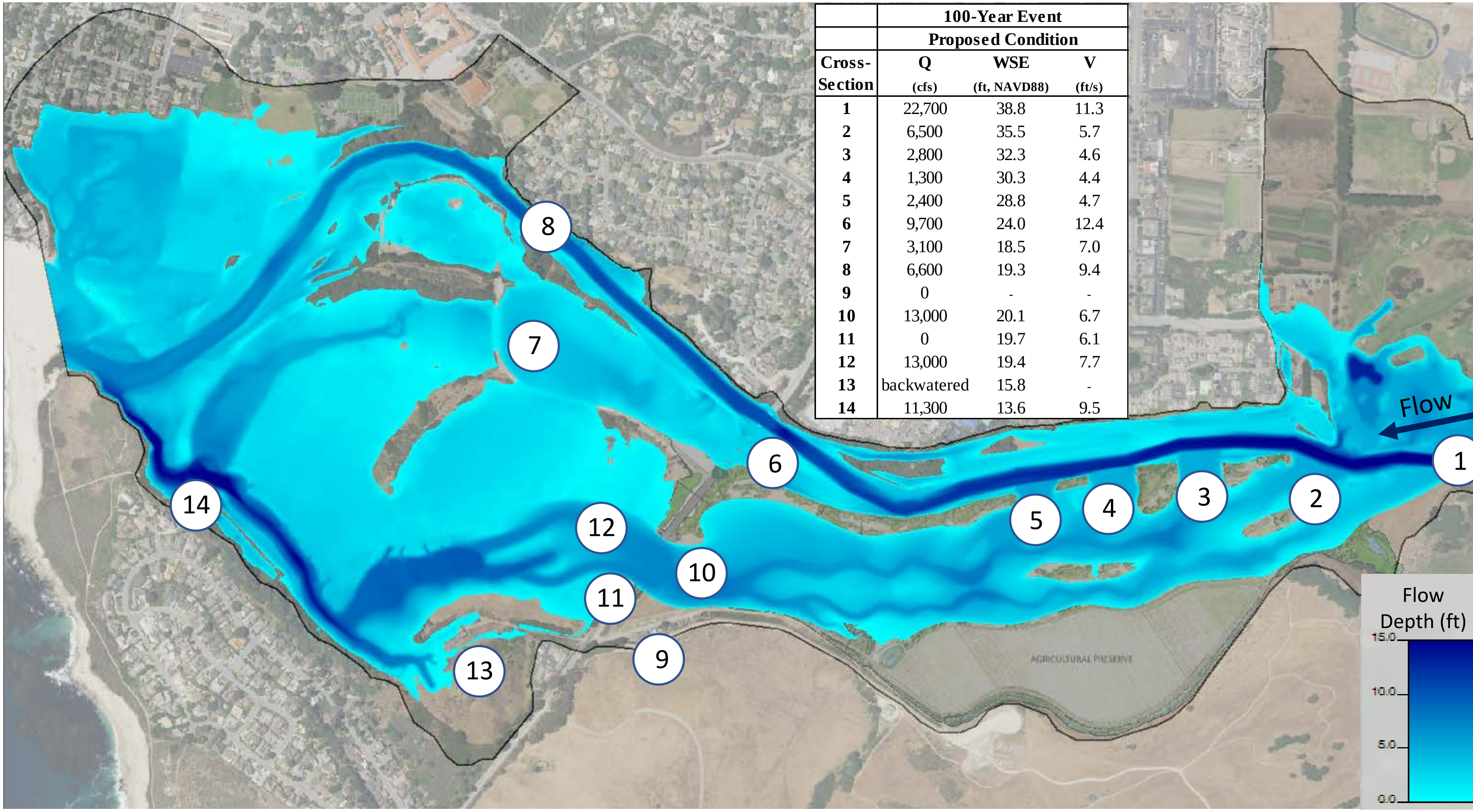


	100-Year Event		
	Existing Condition		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	22,700	39.4	9.9
2	4,100	35.6	7.8
3	-	-	-
4	-	-	-
5	-	-	-
6	17,800	26.3	16.4
7	8,700	19.5	8.0
8	8,700	20.5	10.9
9	backwatered	27.5	1.0
10	4,100	27.6	2.9
11	4,100	27.4	6.8
12	4,100	19.7	2.0
13	backwatered	15.7	1.2
14	10,200	13.3	8.8

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 7. **2D results at the 100-yr modeled flow of the existing site conditions.**

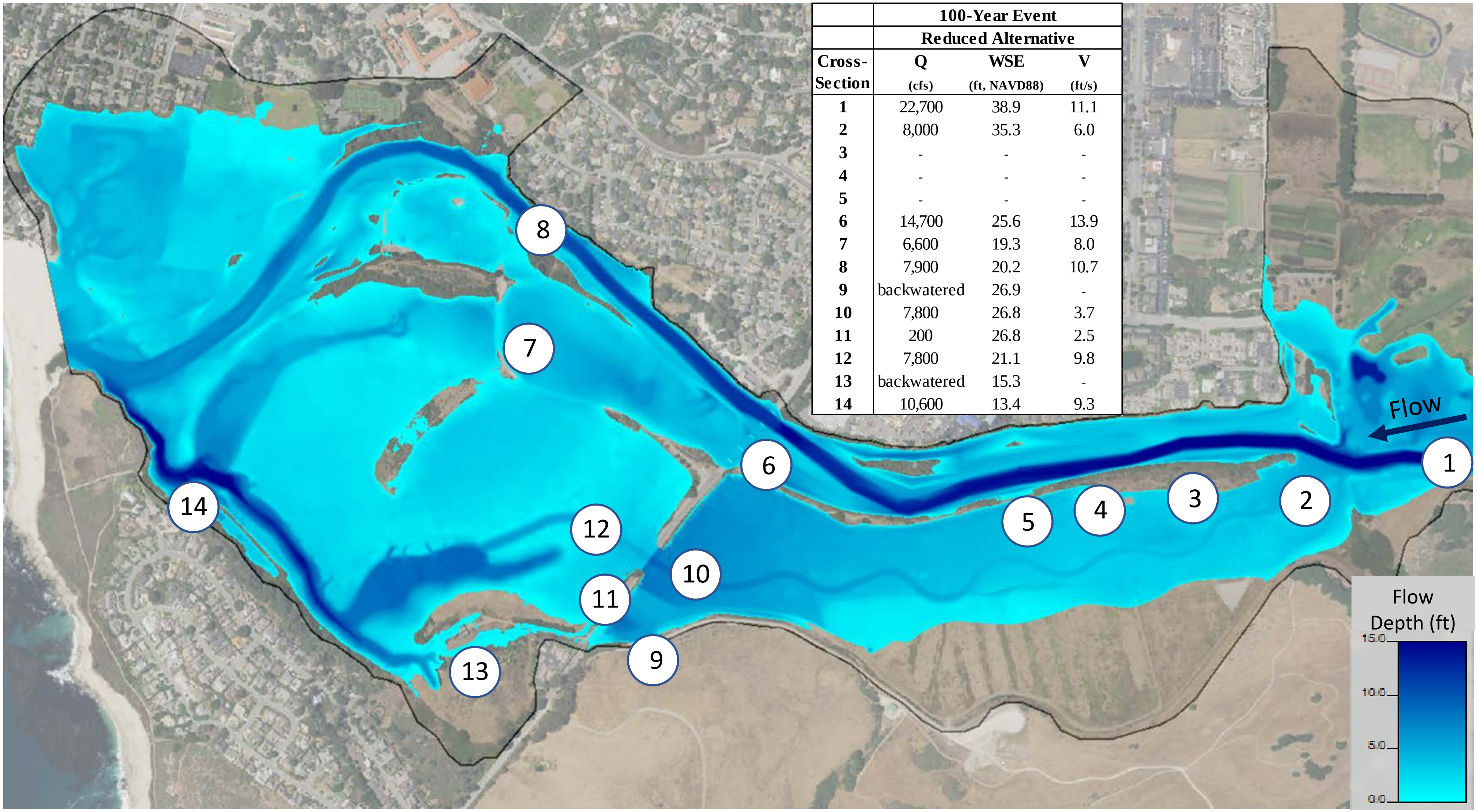


	100-Year Event		
	Proposed Condition		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	22,700	38.8	11.3
2	6,500	35.5	5.7
3	2,800	32.3	4.6
4	1,300	30.3	4.4
5	2,400	28.8	4.7
6	9,700	24.0	12.4
7	3,100	18.5	7.0
8	6,600	19.3	9.4
9	0	-	-
10	13,000	20.1	6.7
11	0	19.7	6.1
12	13,000	19.4	7.7
13	backwatered	15.8	-
14	11,300	13.6	9.5

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 8. **2D results at the 100-yr modeled flow of the proposed design conditions.**



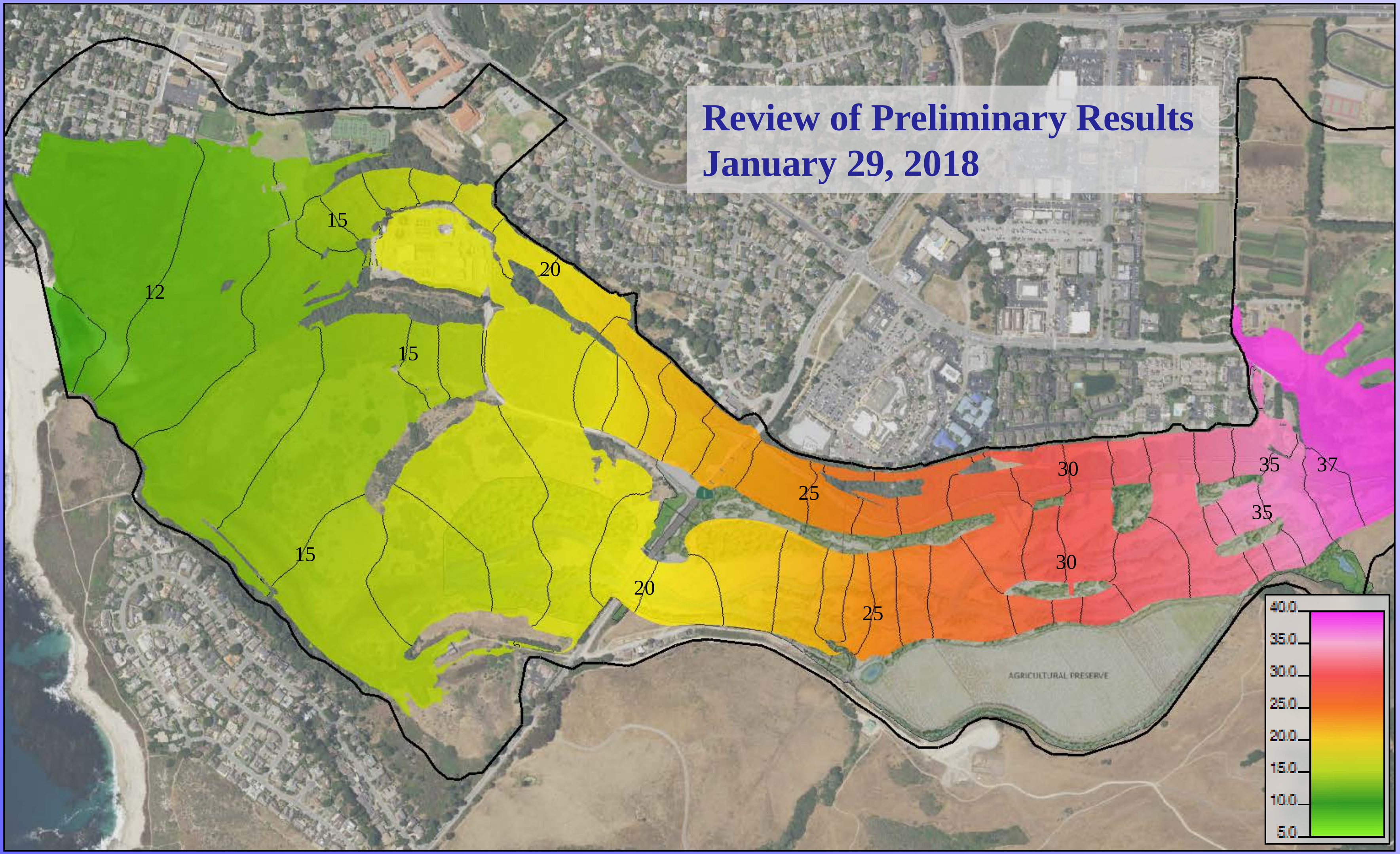
	100-Year Event		
	Reduced Alternative		
Cross-Section	Q (cfs)	WSE (ft, NAVD88)	V (ft/s)
1	22,700	38.9	11.1
2	8,000	35.3	6.0
3	-	-	-
4	-	-	-
5	-	-	-
6	14,700	25.6	13.9
7	6,600	19.3	8.0
8	7,900	20.2	10.7
9	backwatered	26.9	-
10	7,800	26.8	3.7
11	200	26.8	2.5
12	7,800	21.1	9.8
13	backwatered	15.3	-
14	10,600	13.4	9.3

Notes: *The notches at cross sections 3, 4, and 5 exist only in the proposed project design conditions.*
Backwatered locations may feature eddying velocities up to 1.5 ft/s.
Q = 0 indicates that no flow occurs at these locations under the specified condition.
Dashes indicate no 2d model results, either because of no flow or not applicable under the specified condition.
Cross-section 11 reports information on flow over SR 1. Flow under the causeway, if present, is reported upstream at section 10 and downstream at section 12.

Balance Hydrologics, Inc.

Figure 9. **2D results at the 100-yr modeled flow of the reduced alternative conditions.**

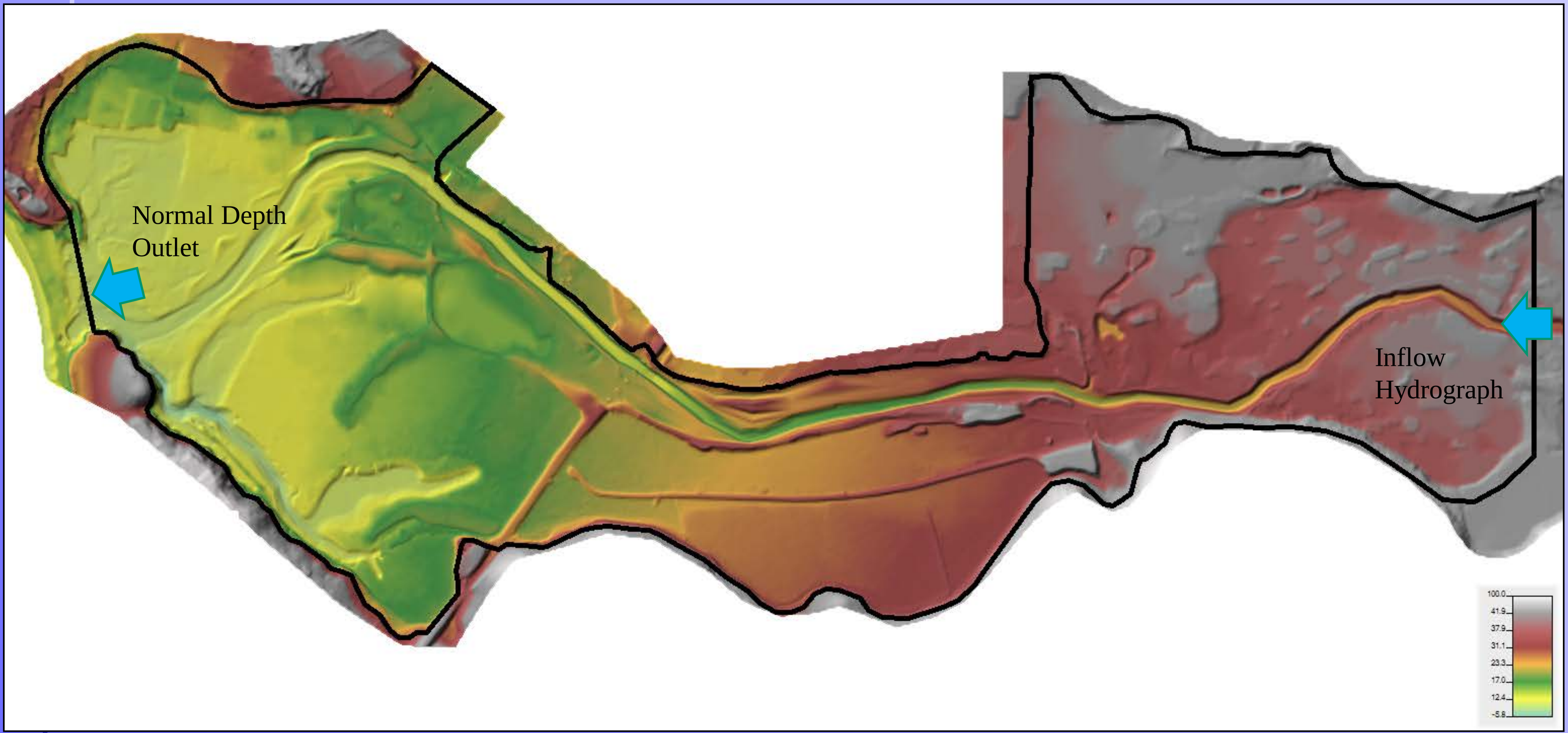
Carmel River FREE – 2 Dimensional Hydraulic Modeling



Presentation Overview

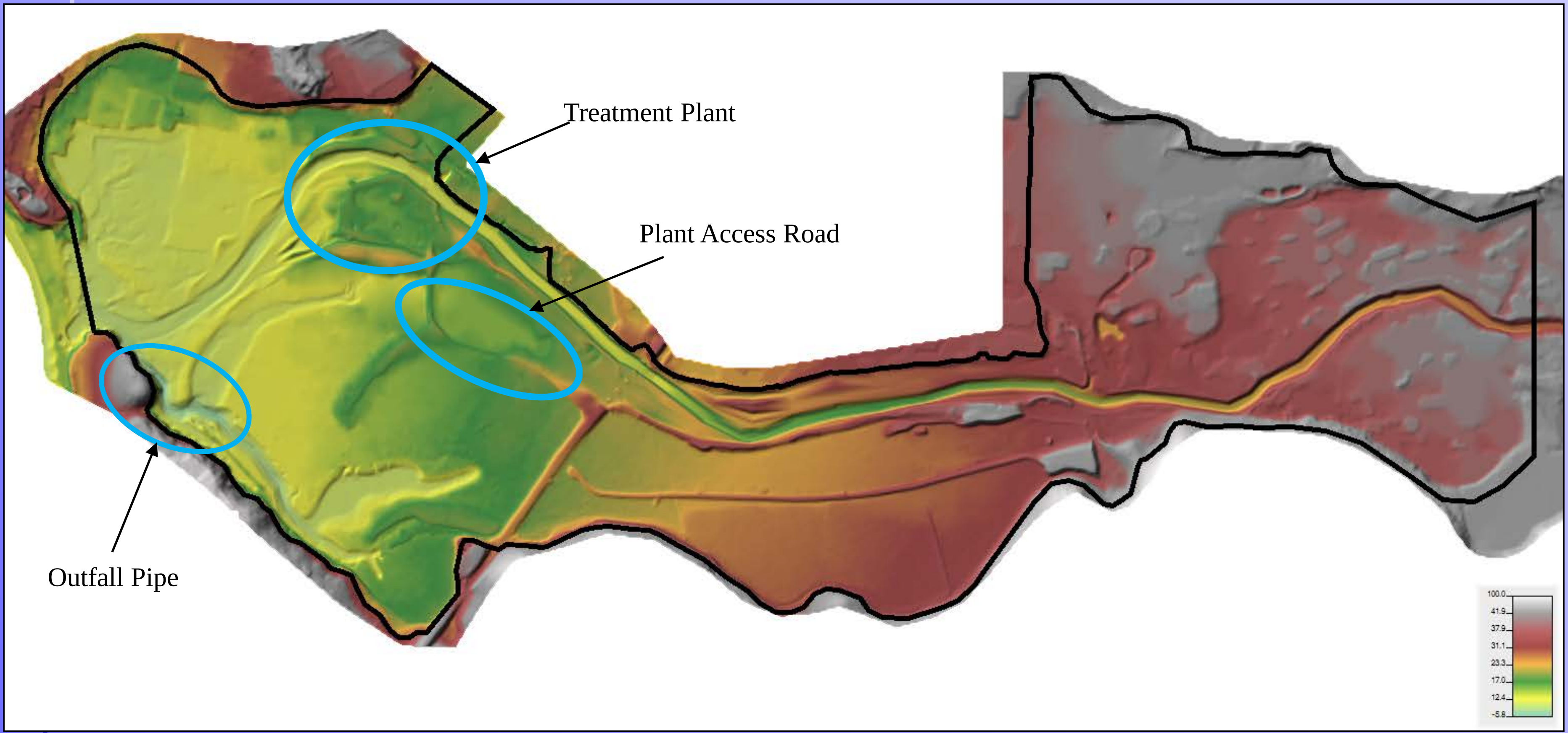
1. **Goals and Extents of Additional Hydraulic Modeling**
2. **Model Parameters and Input Hydrographs**
3. **Review of Modeling Results**
   - **A. Water Surface Elevations**
   - **B. Flood Flow Partioning**
   - **C. Flow Velocities (Scour)**
   - **D. Flow Depths (Streamwood Transport)**
4. **Discussion and Questions**

Topography and Model Extents



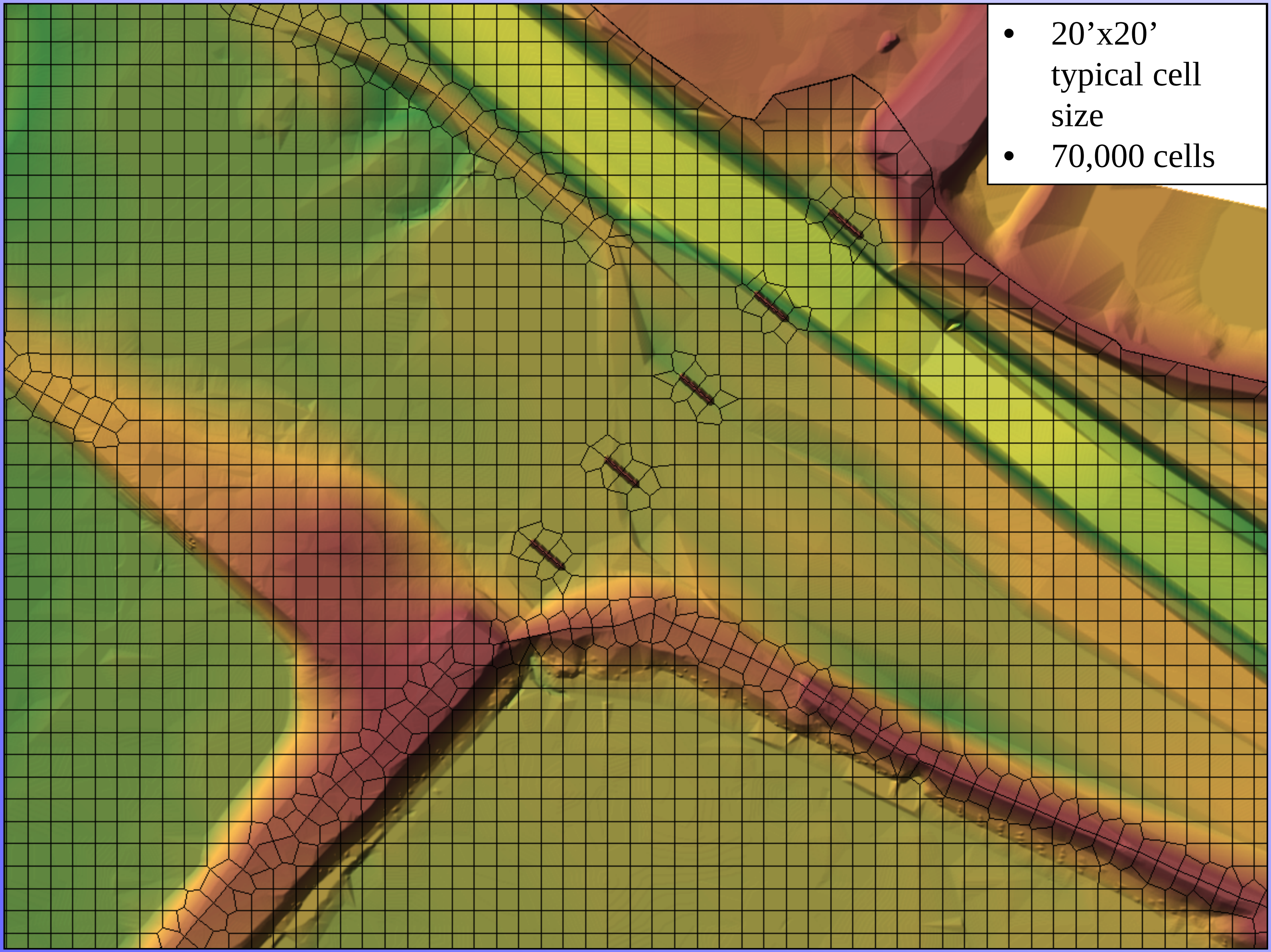
Data sources:
Whitson Engineers, FEMA LiDAR data,
FEMA cross-sections

Focus on Potential Impacts to CAWD Facilities



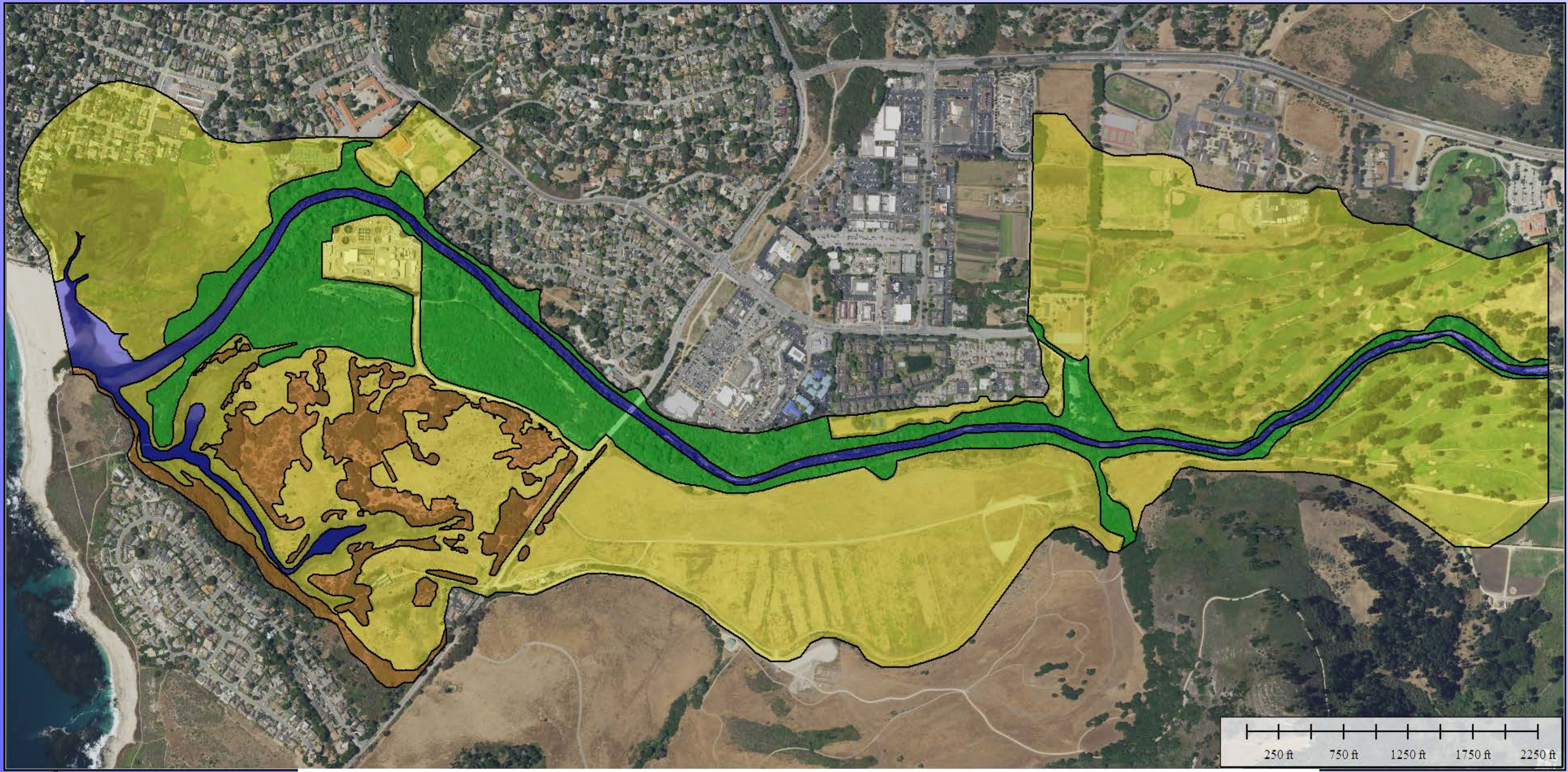
Main focus here is potential for impacts west of Highway 1, though the modeling has wide-ranging applications for planning and design refinements.

Model Mesh and Resolution



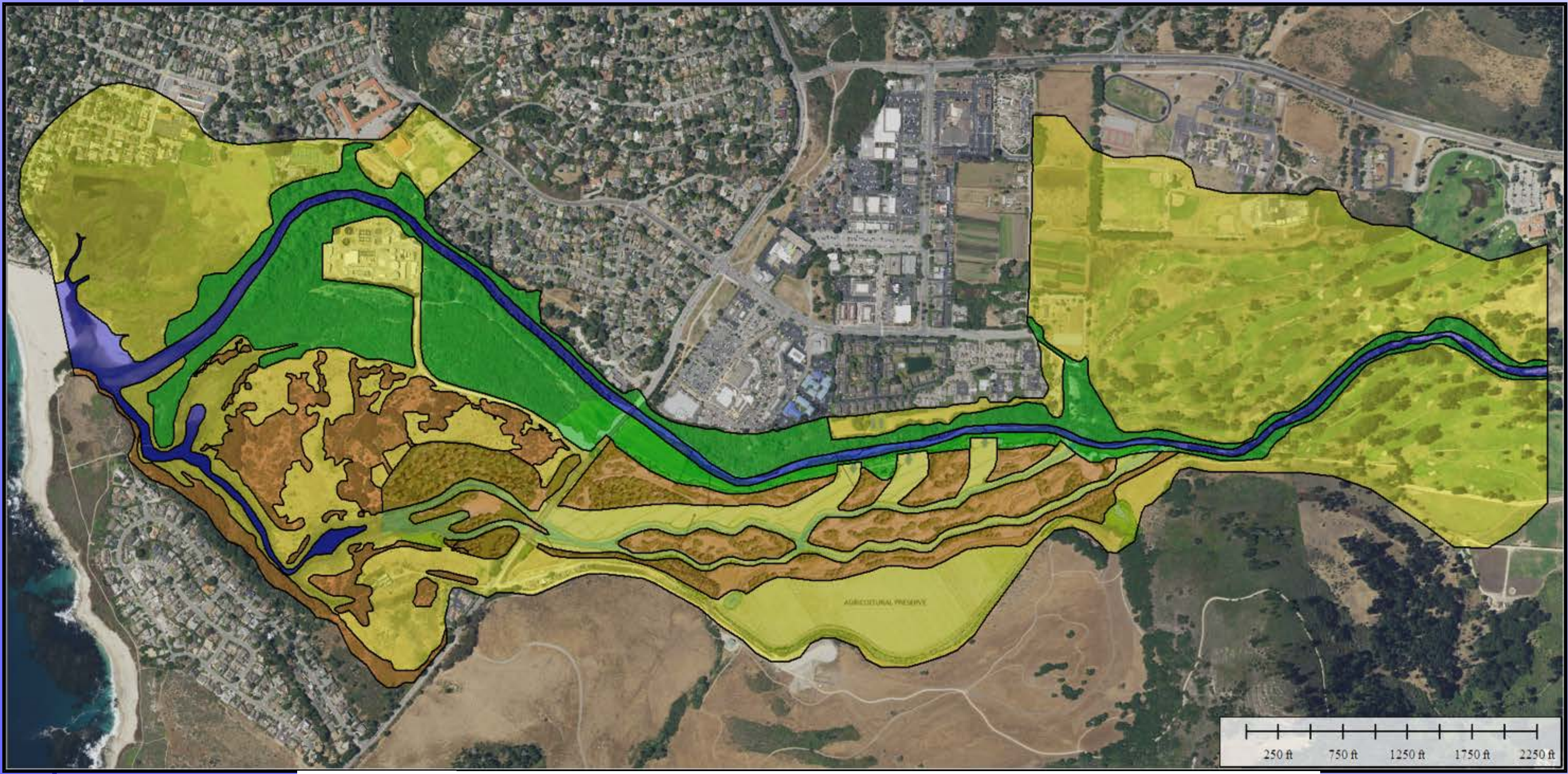
- 20'x20' typical cell size
- 70,000 cells

Pre-project Hydraulic Roughness Values



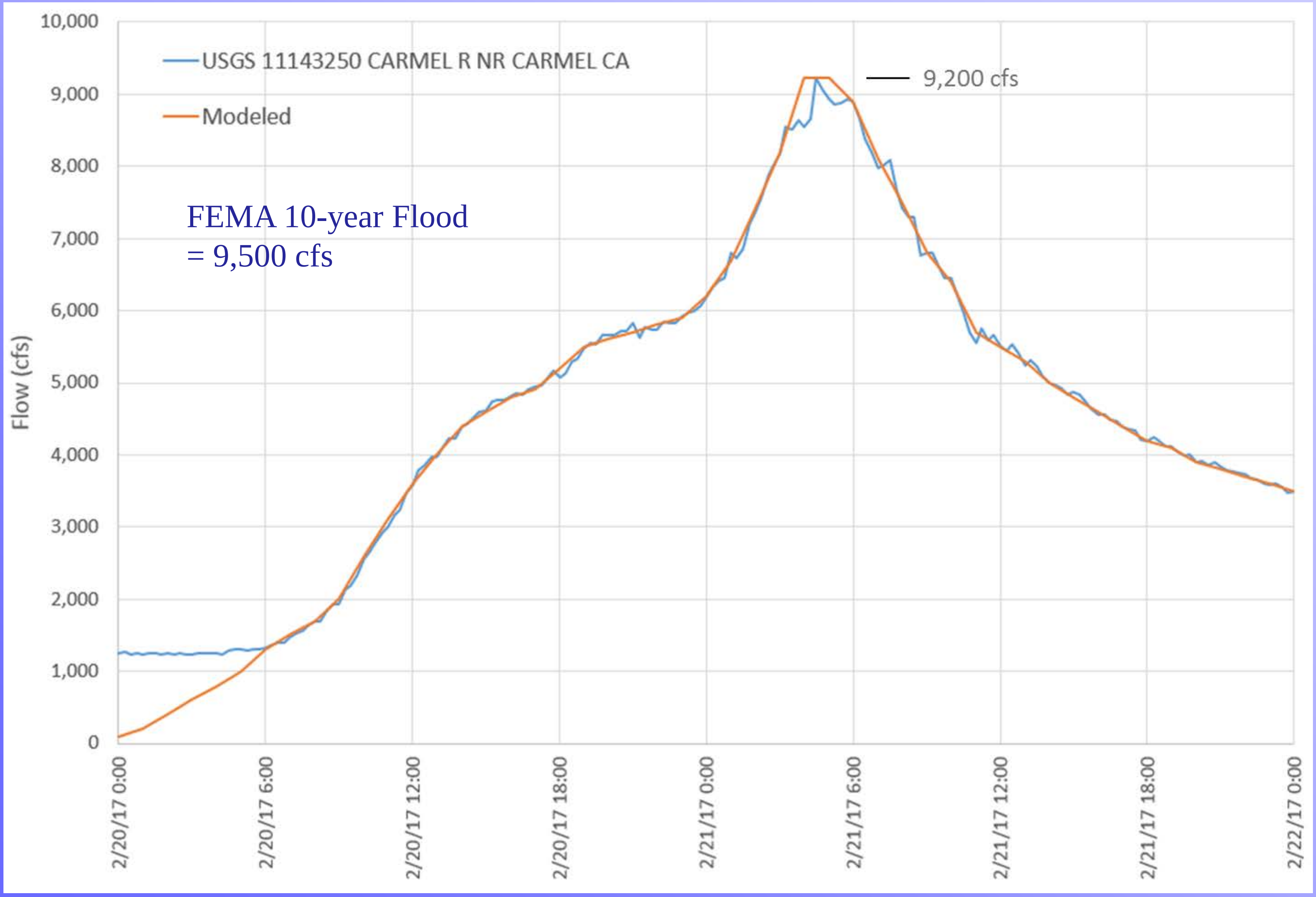
CHANNEL n=0.035	FLOODPLAIN GRASS n=0.04
RIPARIAN n=0.10	FLOODPLAIN MIXED n=0.06

Post-project Hydraulic Roughness Values

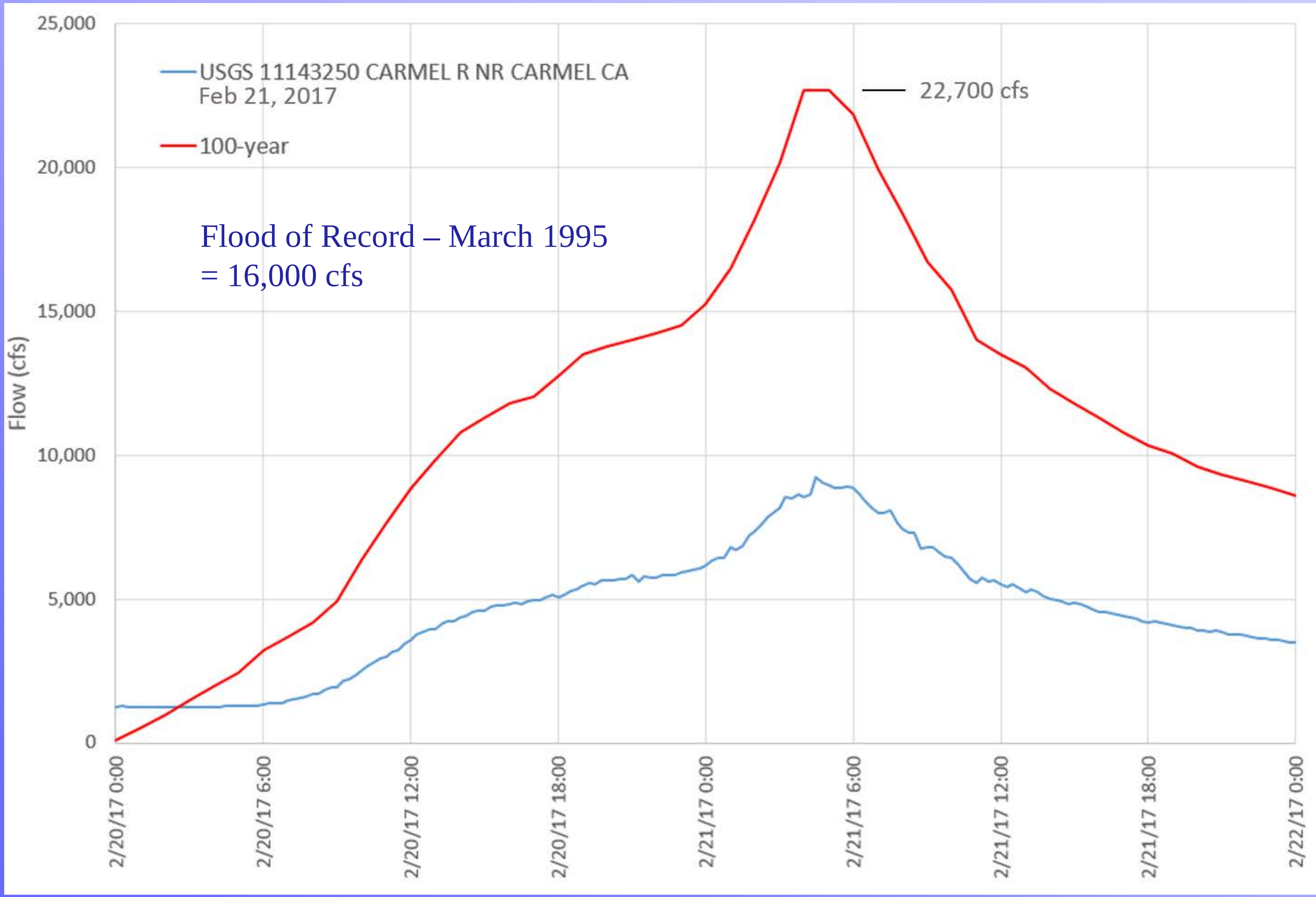


CHANNEL n=0.035	FLOODPLAIN GRASS n=0.04
RIPARIAN n=0.10	FLOODPLAIN MIXED n=0.06

Design Flows – Storm of February 21, 2017

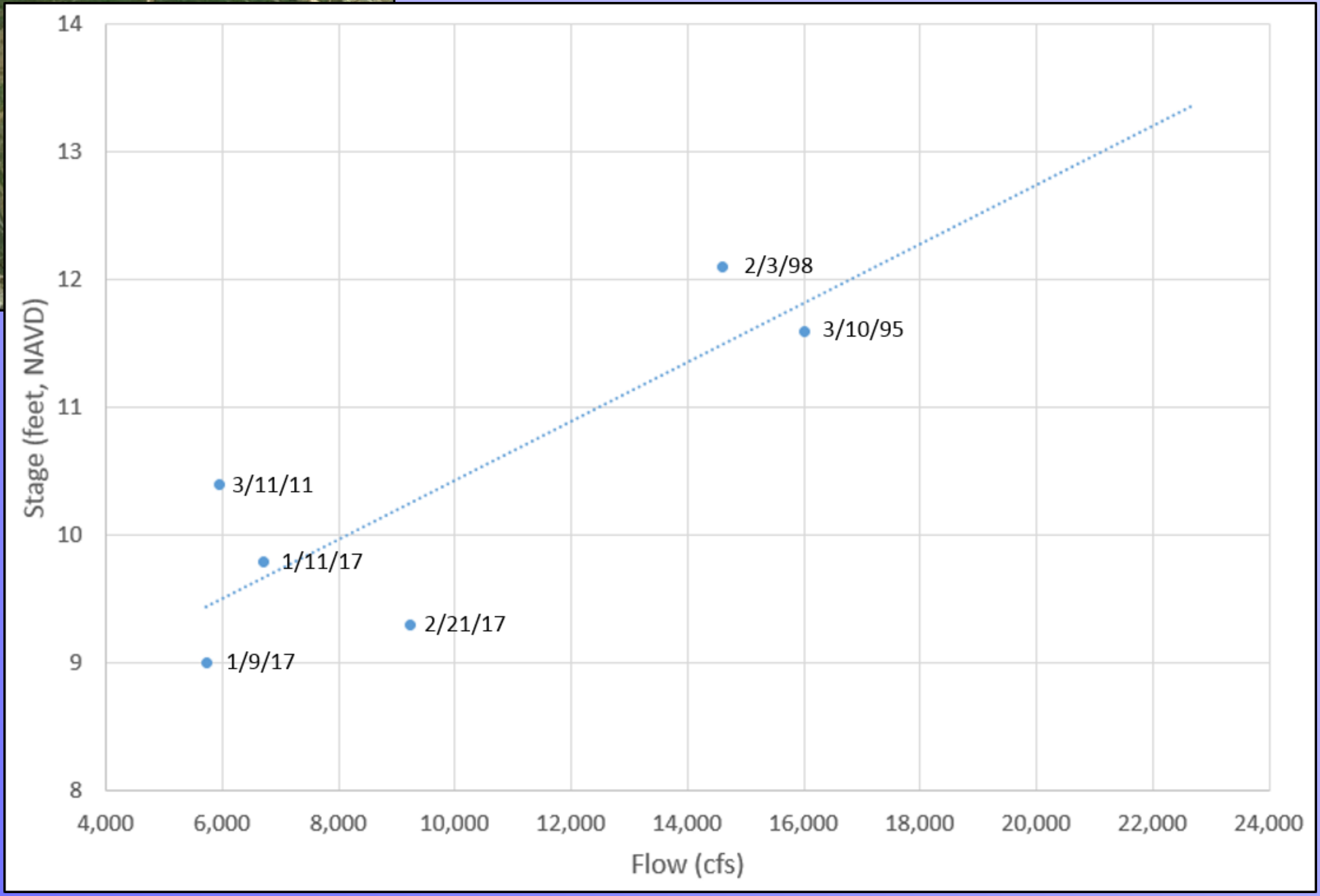


Design Flows – 100-year Flood Event

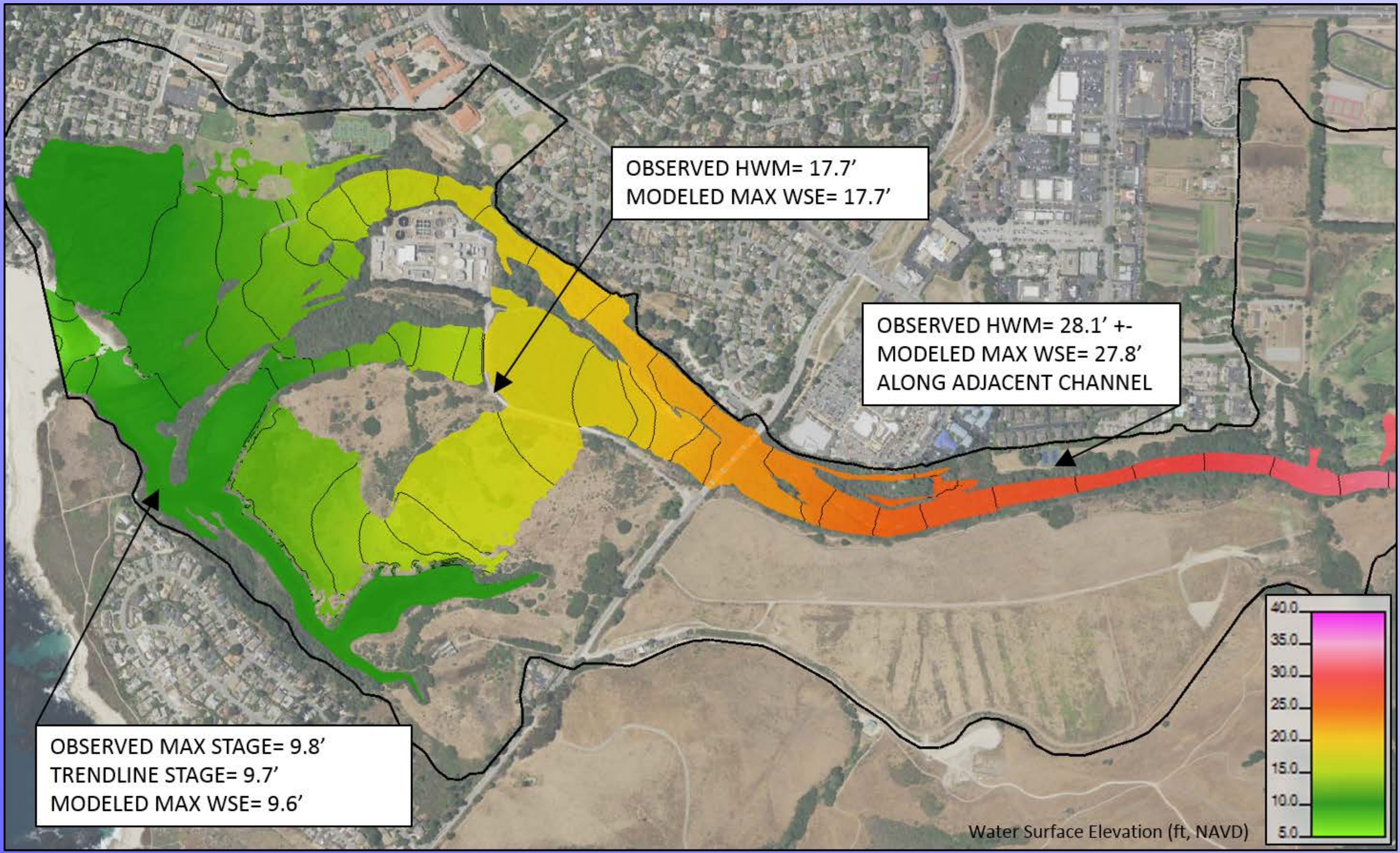


Downstream Boundary Conditions (Tailwater)

Normal depth used to define downstream boundary condition with slope = 0.005

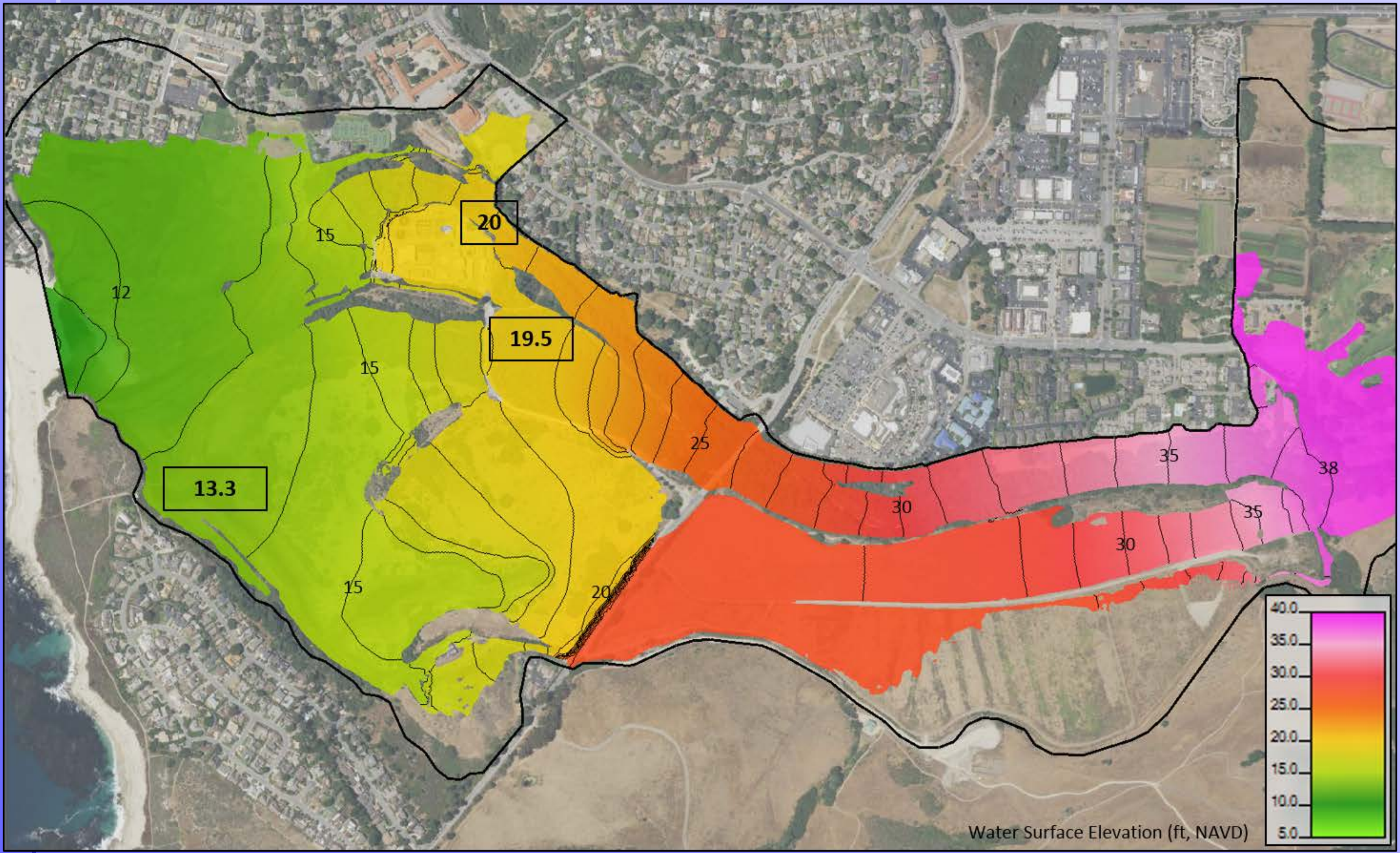


Model Validation – January 11, 2017 Event

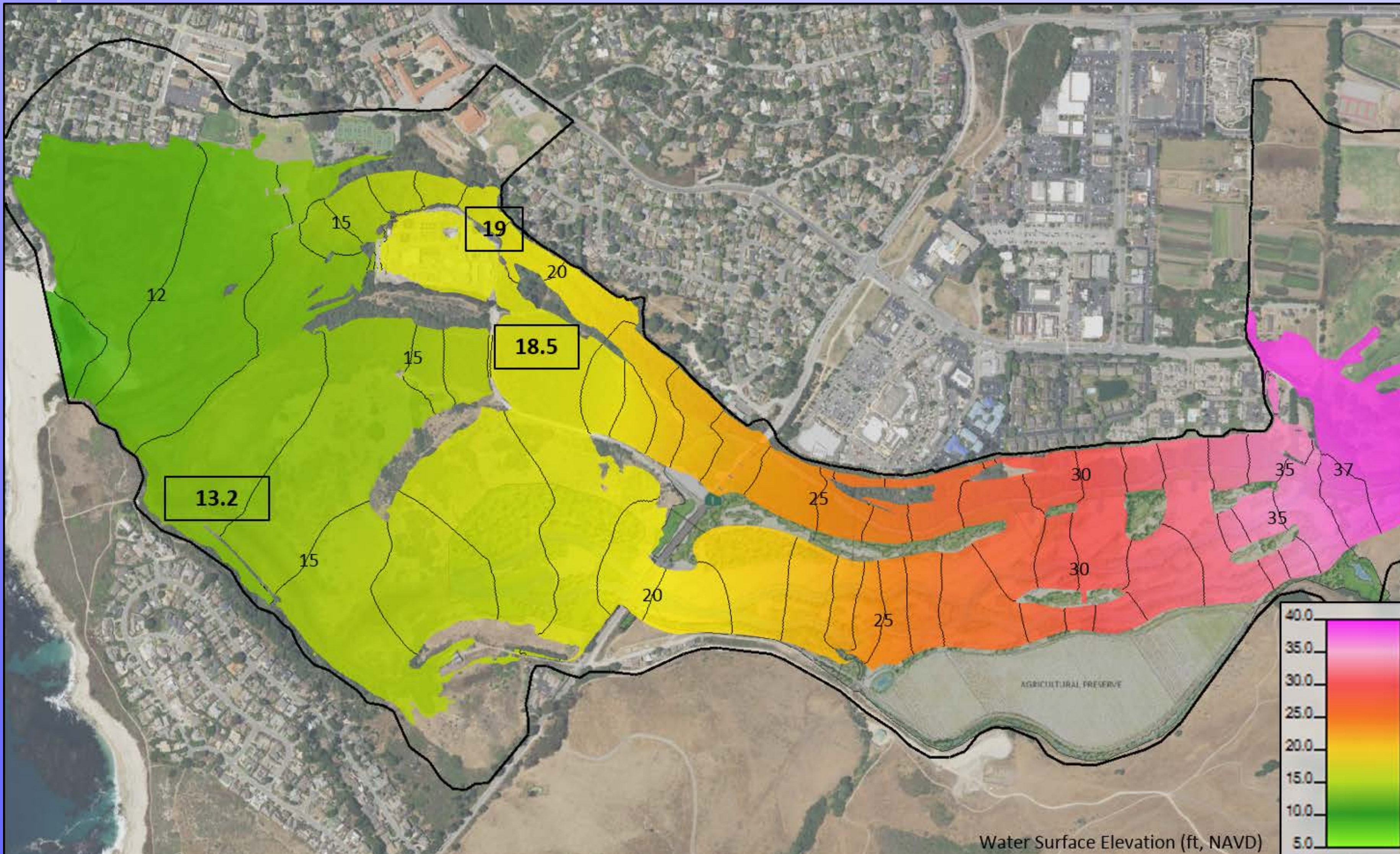


Dynamic View of the Results (100-year flood)

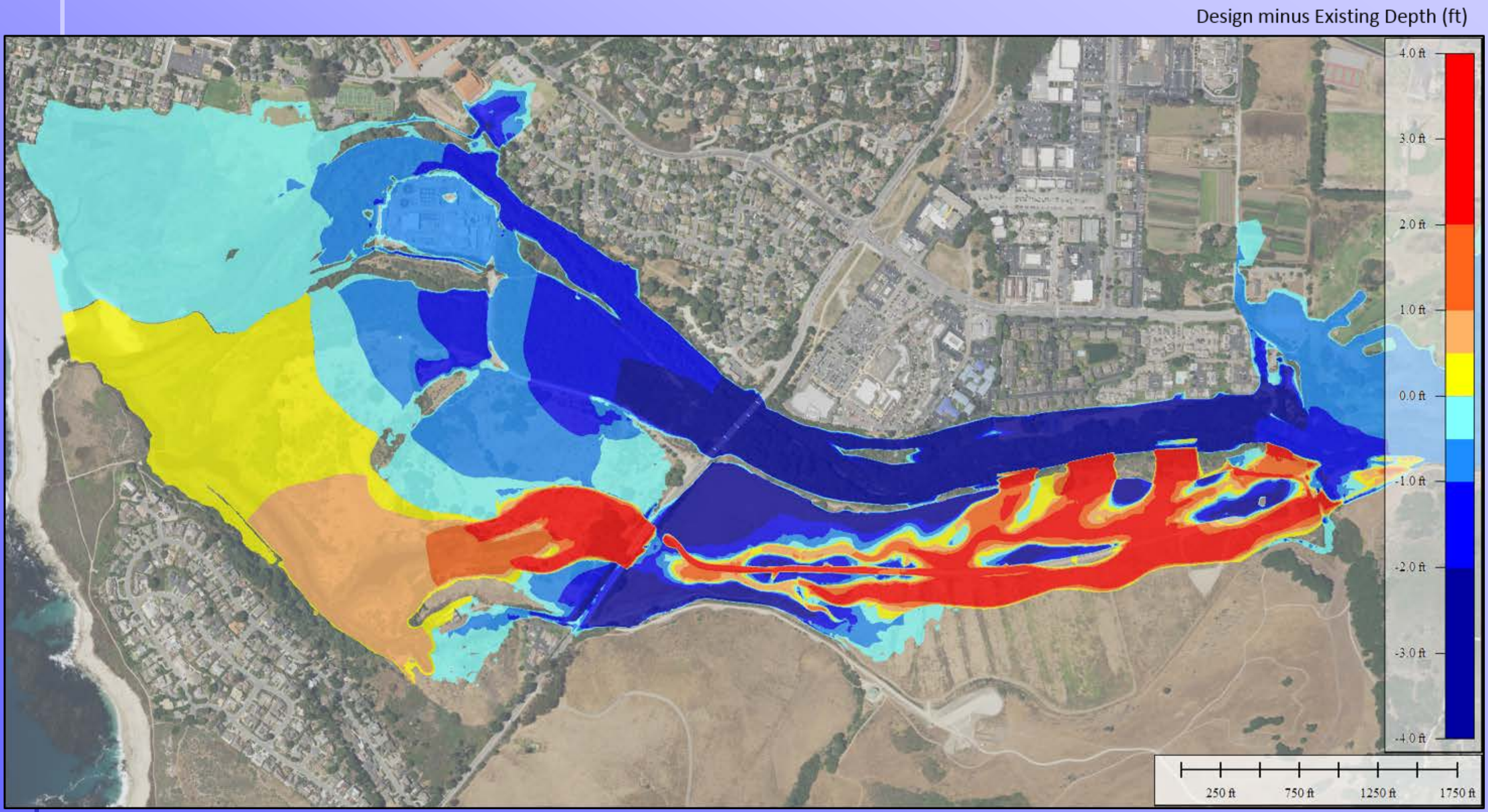
Water Surface Elevations – Pre-project 100-year Flood



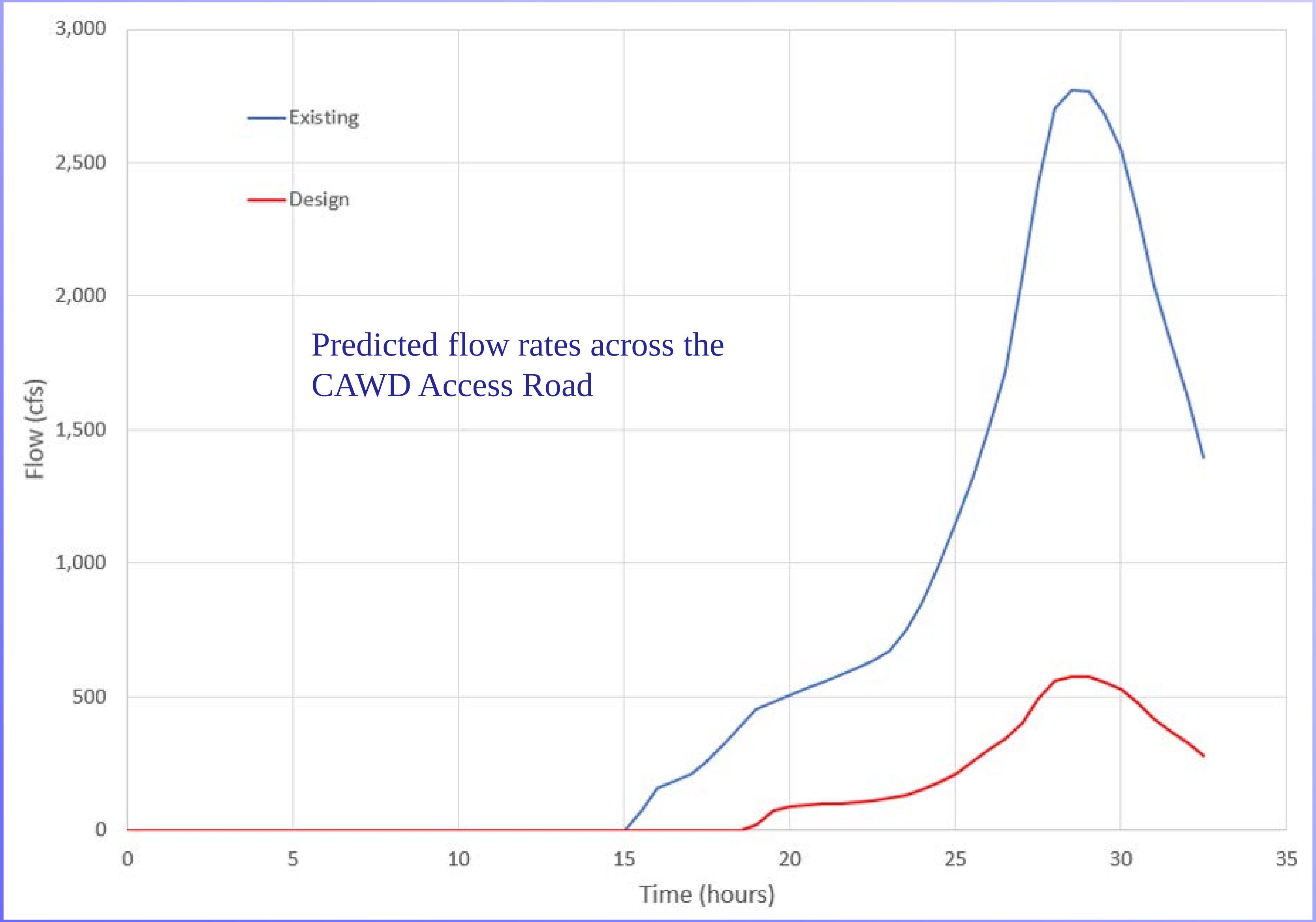
Water Surface Elevations – Post-project 100-year Flood



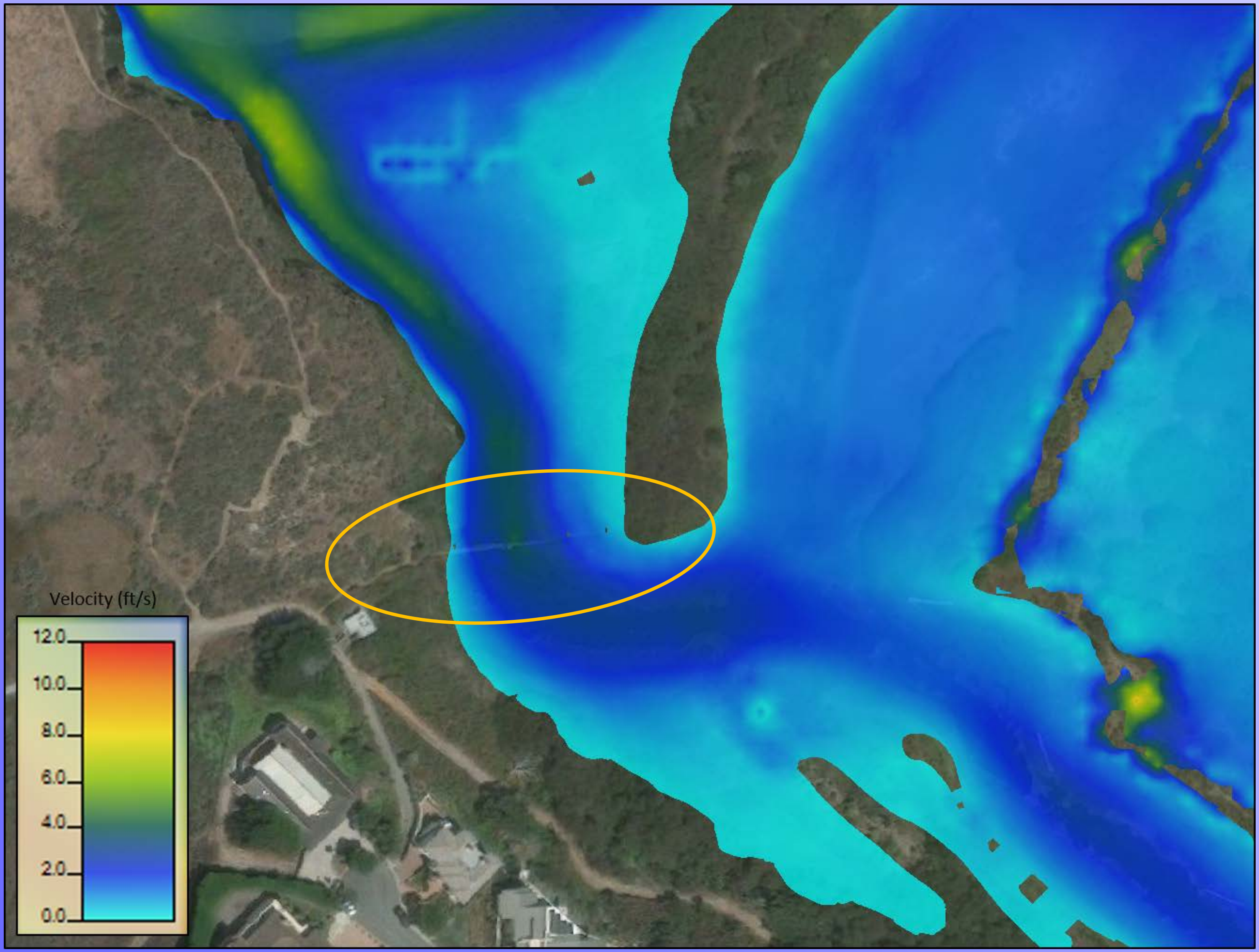
Water Surface Elevations –100-year Flood Depth Comparison



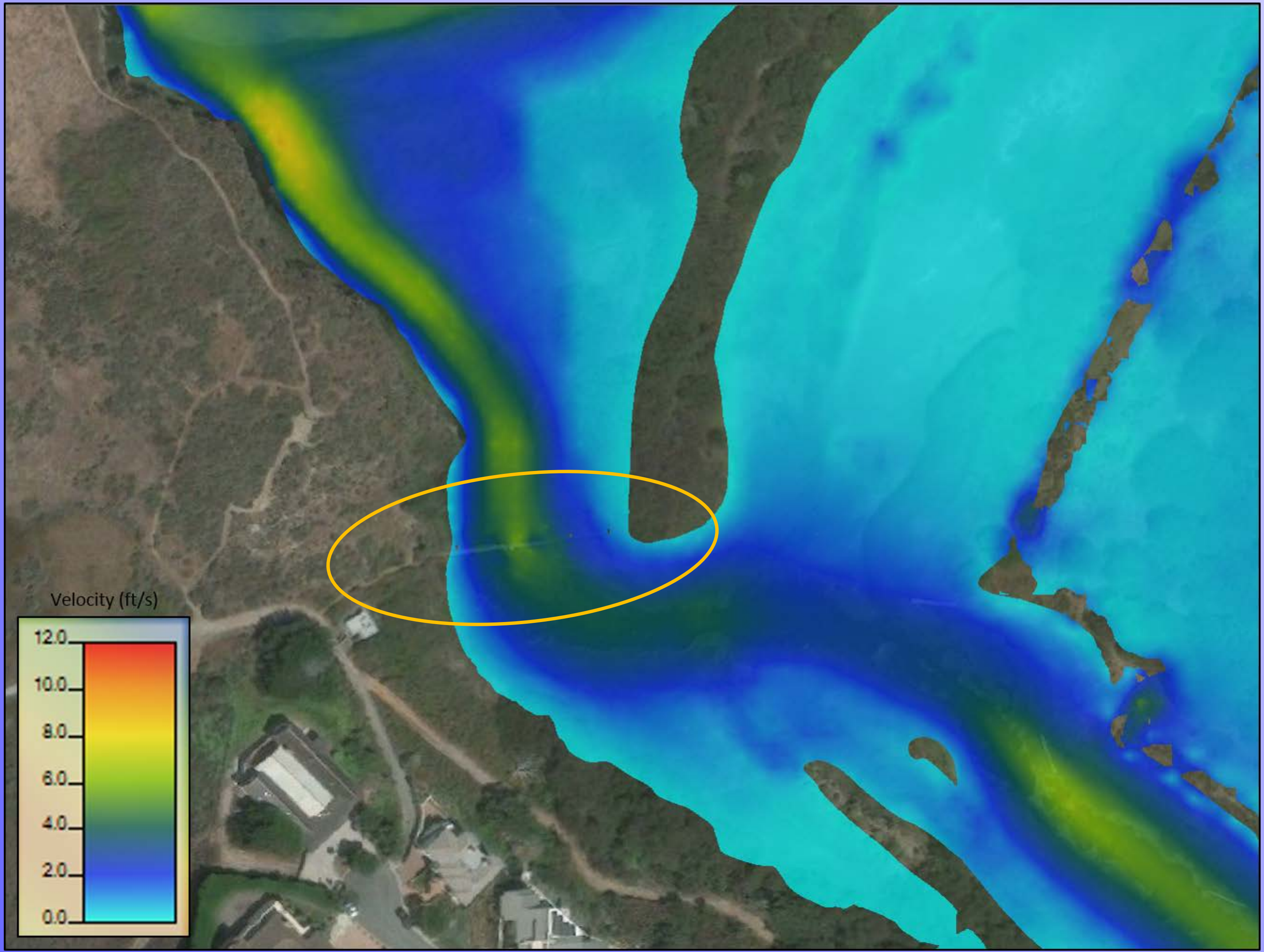
Flow Partioning – Comparison of 10-year Flows



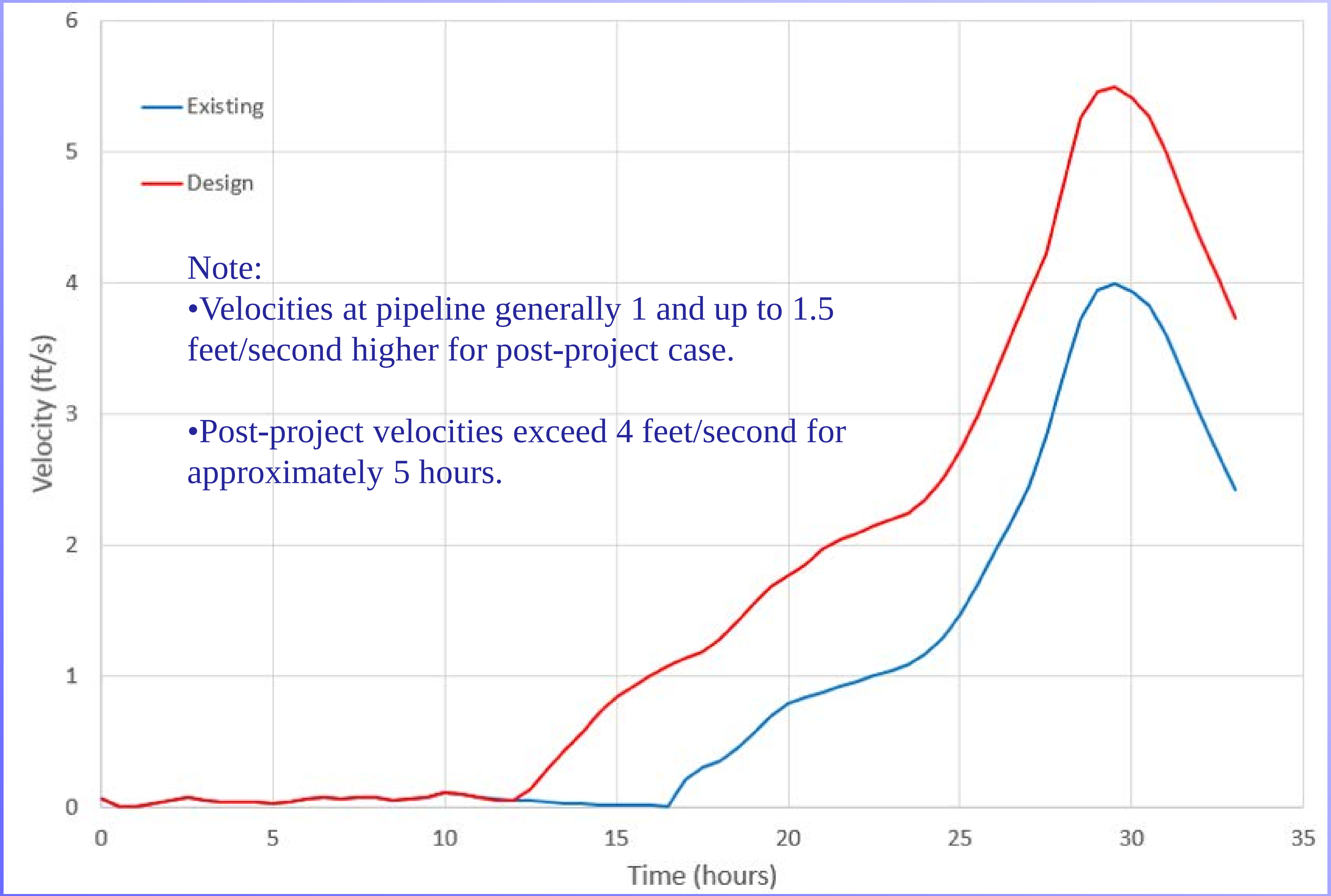
Flow Velocities – Pre-project 10-year at CAWD Outfall Pipe



Flow Velocities – Post-project 10-year at CAWD Outfall Pipe



Flow Velocities – Comparison of 10-year Values at Outfall

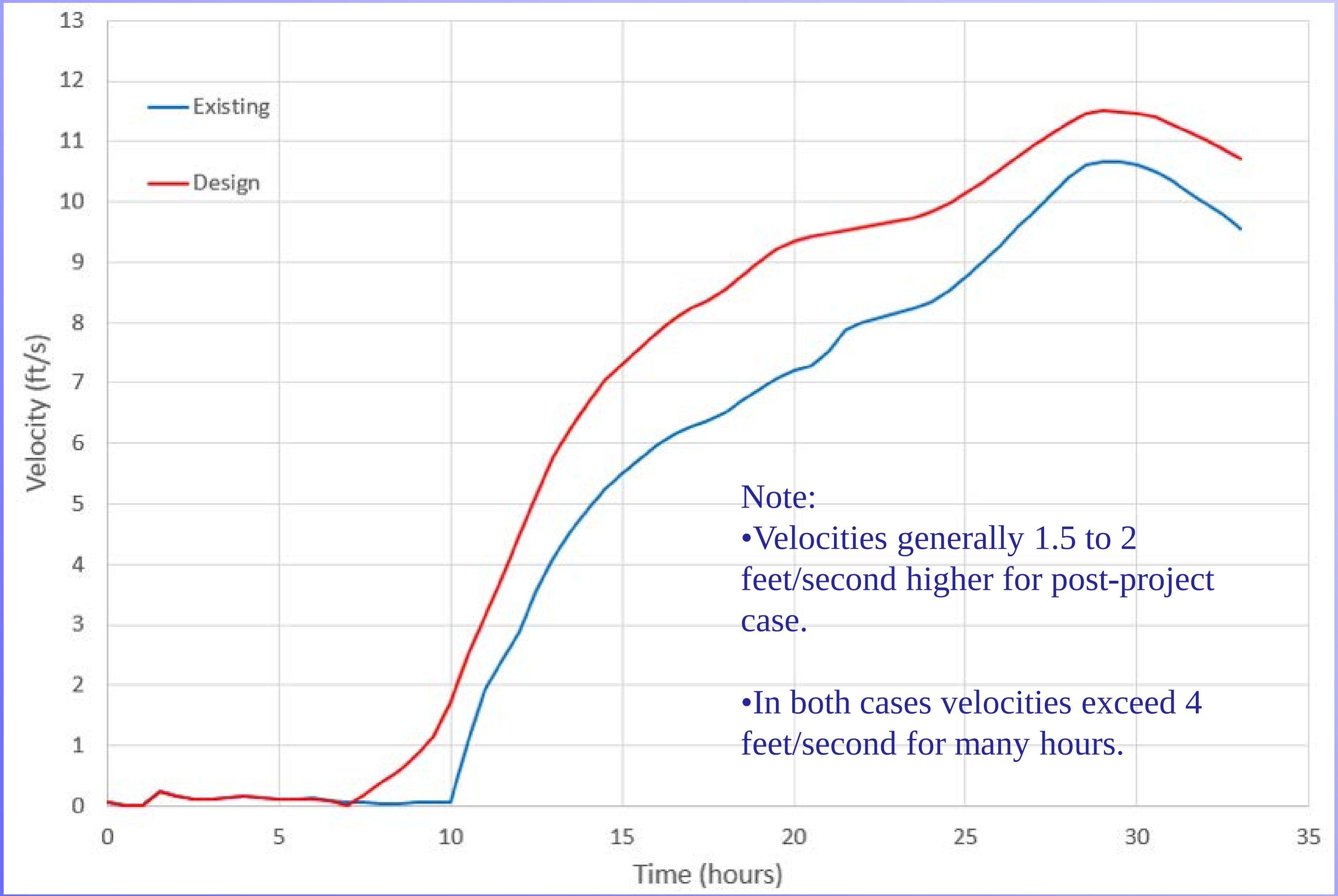


Note:

•Velocities at pipeline generally 1 and up to 1.5 feet/second higher for post-project case.

•Post-project velocities exceed 4 feet/second for approximately 5 hours.

Flow Velocities – Comparison of 100-year Values at Outfall

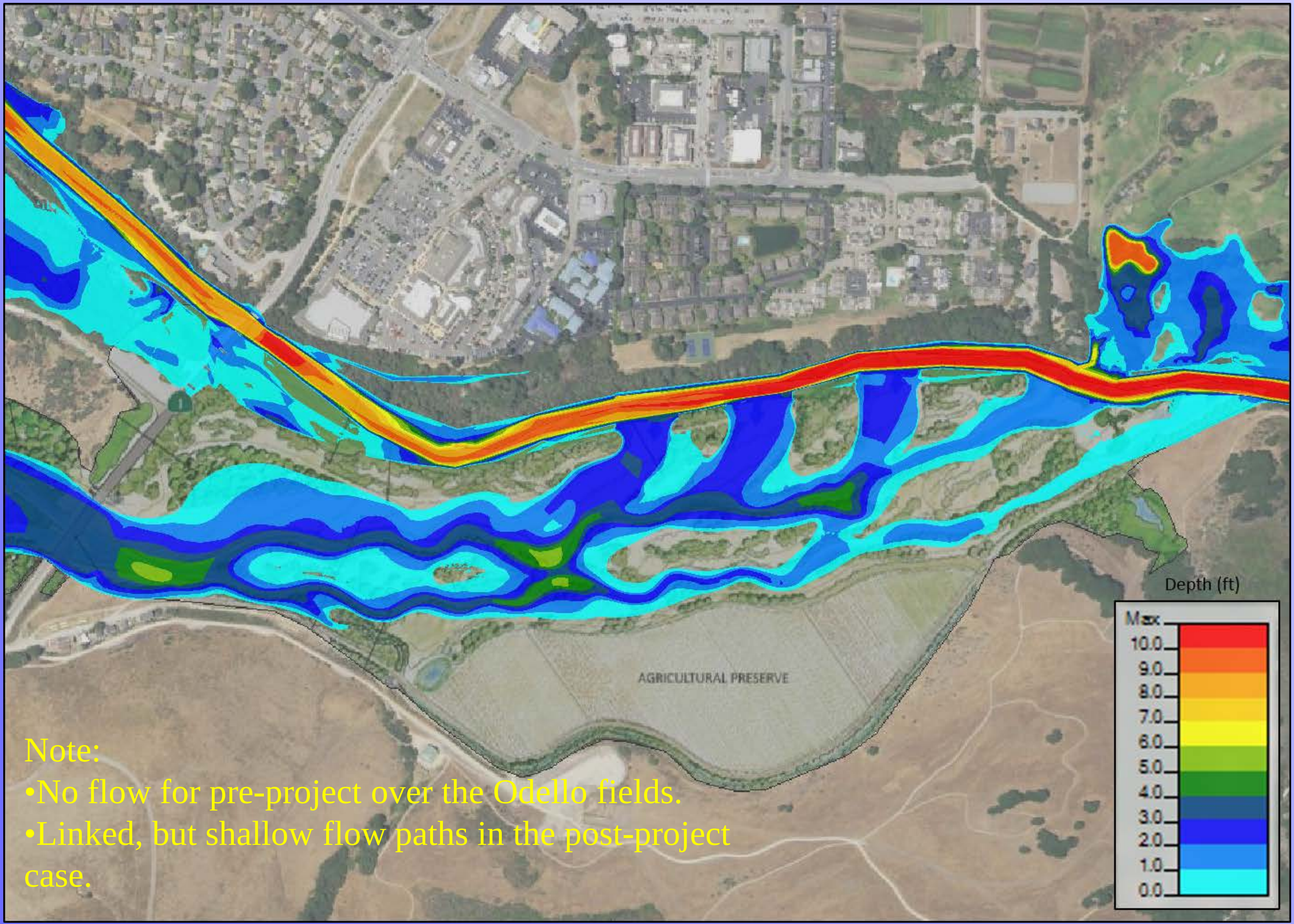


Note:

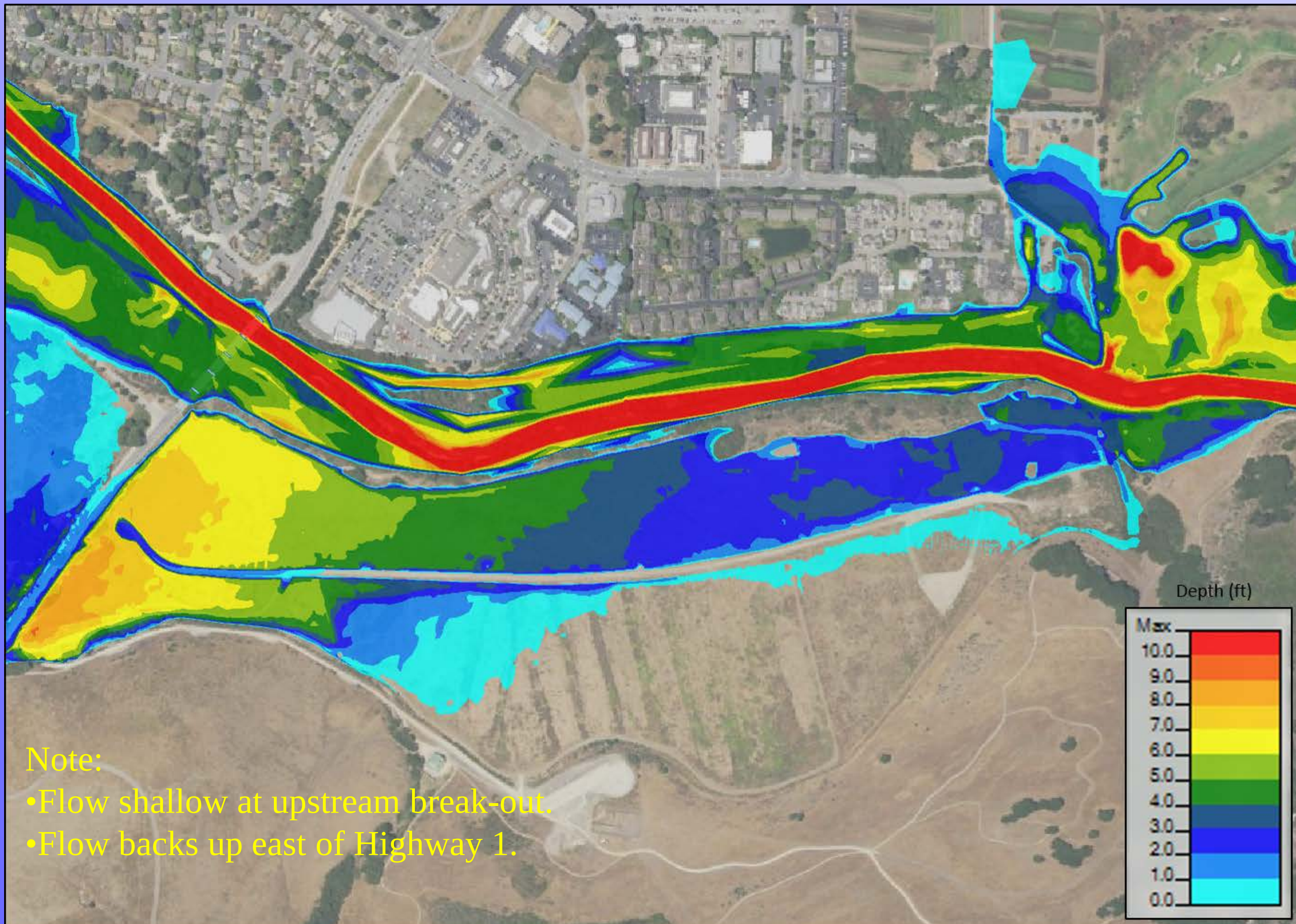
•Velocities generally 1.5 to 2 feet/second higher for post-project case.

•In both cases velocities exceed 4 feet/second for many hours.

Flow Depth – Post-project 10-year Upstream of Highway 1



Flow Depth – Pre-project 100-year Upstream of Highway 1



Flow Depth – Post-project 100-year Upstream of Highway 1