

Exhibit H

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Analysis

Floodplain impacts are assessed using the FEMA effective steady-state HEC-RAS model of the Carmel River. In the HEC-RAS model, the applicable cross sections through the project site are located along reach FLDPLN2 and consist of cross sections 6207.832, 6557.074, 6635.903, 6798.585 and 6885.99. See Figure 2 for the cross-section locations shown on the FEMA floodplain relative to the project boundary. The steady-state flow data used in this analysis assumes failure of the right bank levee therefor the floodplain is represented as the “worst case” condition.

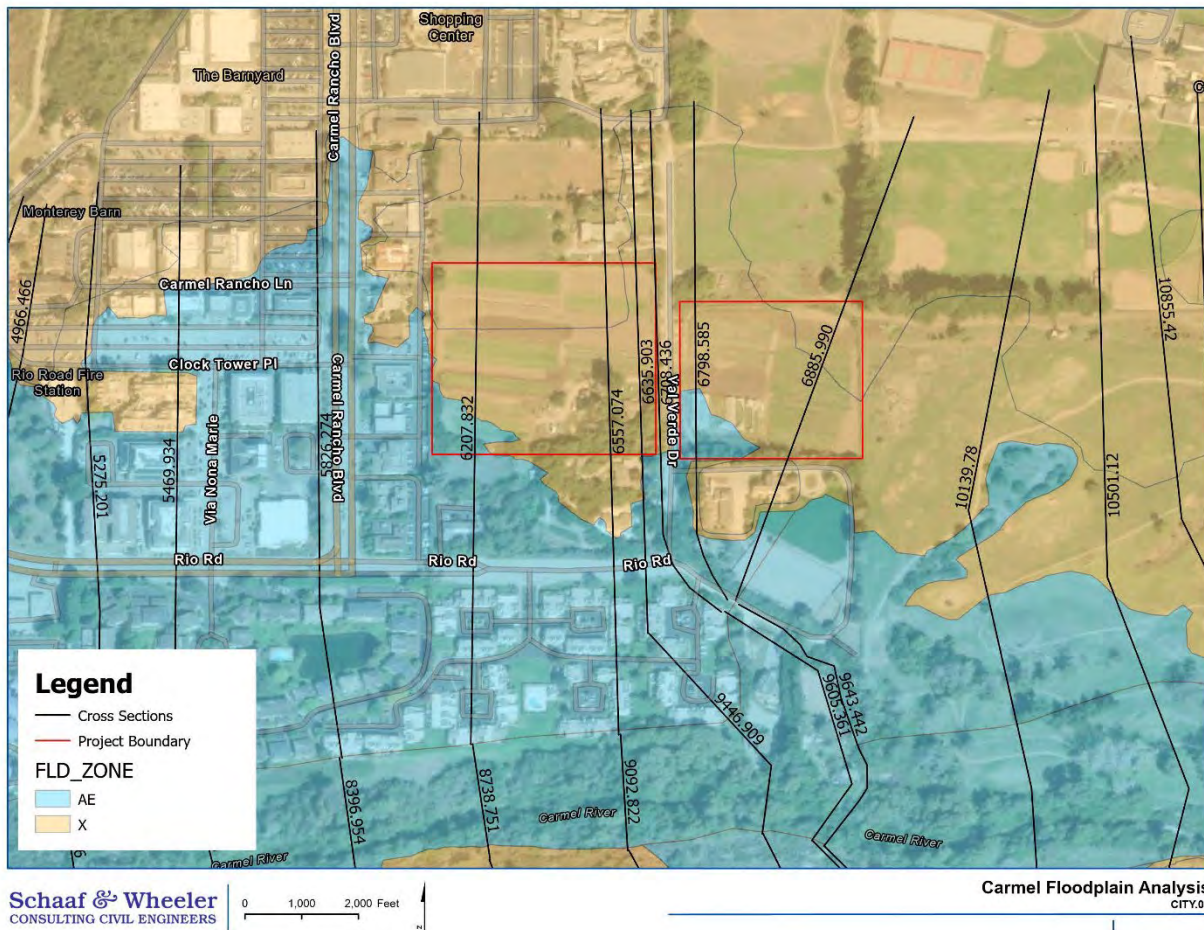


Figure 2. HEC-RAS Cross Section Data through Project Site on Effective FEMA Floodplain

The proposed project site outlines shown in the drawings provided by CBG Engineers are added to the floodplain as blockages. The blockage is represented in the model by obstructions which add 10 feet to the existing elevation of each station on the cross section located within the project boundary. This is a conservative assumption, as actual site blockages by structures occur outside of the floodplain boundary. The intent of this conservative assumption is to show that any development action on the site will not cause significant impacts to the floodplain. The table below shows the resulting change in WSEL when obstructing the entire project limits.

Table 1. 100-year Water Surface Elevation Comparison at Cross Sections

Parameter	XS 6207.832	XS 6557.074	XS 6635.903	XS 6798.585	XS 6885.99
Existing WSEL (ft)	32.22	32.31	34.35	35.03	35.17
Proposed WSEL (ft)	32.20	32.20	34.39	35.02	35.14
Increase in WSEL (ft)	-0.02	-0.11	0.04	-0.01	-0.03

There is a negative impact to the WSEL for all cross sections on the project site except for XS 6635.903 where the WSEL increases by 0.04 feet. The negative impact to the WSEL at most of the cross sections is likely due to “shadowing” which is the effect where structures or other obstructions create areas of reduced or altered flood flow. There is no significant change in WSEL in the upstream and downstream cross sections from the proposed development as a result of shadowing in the floodplain.

Conclusion

Based on this analysis, there is not a significant impact to the Carmel River floodplain due to blockages on the project site represented conservatively as an obstruction to the entire project boundary in the proposed condition. The largest increase to the WSEL is below 0.1 therefore the project is considered to have negligible impact.

Per Monterey County Municipal Code Section 16.16.050 K:

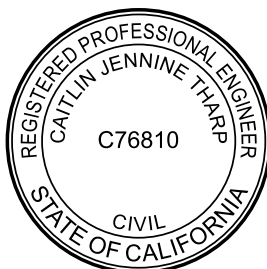
- **The proposed development is set back approximately 1,000 feet from the top of bank of the Carmel River, which exceeds the 200-foot minimum requirement.**
- **The proposed development will not significantly reduce the capacity of existing rivers or watercourses or otherwise adversely affect any other properties by increasing stream velocities or depths or diverting the flow.**

Any questions can be directed to Caitlin Tharp, PE, at ctharp@swsv.com or by cell phone at (415) 823-4964.

Sincerely,
Schaaf & Wheeler



Caitlin Tharp, PE
CFM, QSD/QSP, CPSWQ
Vice President
RCE #76810



MEMORANDUM

DATE: April 25, 2025
TO: Pamela Nieting
City Ventures
FROM: Cindy Steele, PE
Caitlin Tharp, PE
SUBJECT: 26500 Val Verde Drive Hydromodification Study

Job #: CITY.06.25



Introduction

City Ventures contracted Schaaf & Wheeler to prepare hydromodification calculations for the proposed development at 26500 Val Verde Drive in Monterey County near Carmel by the Sea. Each of the two proposed drainage management areas are modeled for the 2-, 5-, 10-, 25-, 50-, and 100-year 24-hour pre- and post- project storm events. The proposed storage basins are modeled and modifications to size and outlet structures are recommended. The 12.5-acre site is shown in Figure 1.

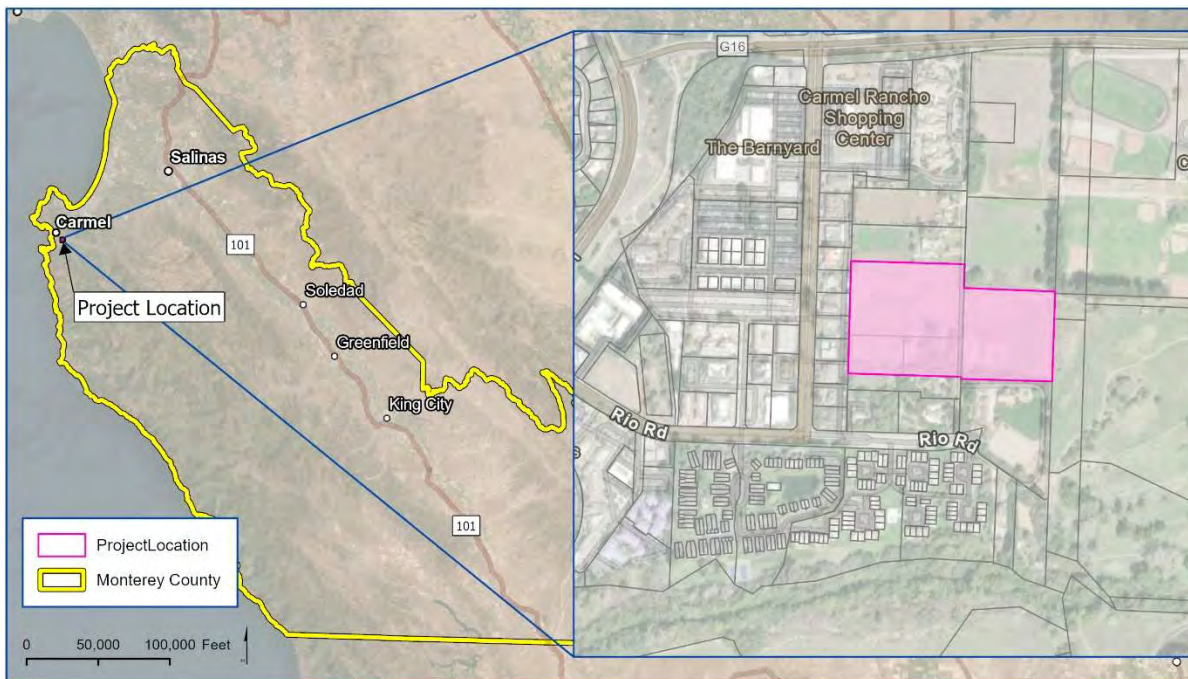


Figure 1: Project Location

Located southeast of the Carmel by the Sea city limits, the site is currently used as agricultural land with one single family residence. The proposed project includes 59 single-family homes and 15 multifamily units on 60 lots. The site is divided into two drainage management areas, each draining to bioretention basins on the south side of the project. Using the grading on the

Tentative Map plans prepared by CBG, the bioretention basins were modeled by Schaaf & Wheeler to mitigate for hydromodification of the 2-year through 100-year design storm events.

Analysis

Schaaf & Wheeler analyzed the peak flow from the drainage management area in HEC-HMS using the SCS Unit Hydrograph method and Frequency storm with 50 percent Intensity position based on NOAA Atlas 14 precipitation values. The SCS Unit Hydrograph method utilizes curve numbers based on land use, underlying soil type, and antecedent moisture condition to determine the portion of precipitation lost to infiltration. Excess precipitation is routed to the subbasin outlet according to lag time in minutes which is calculated based on the length, slope, and roughness of the longest flow path. The subbasin parameters and calculated runoff are listed in the following tables.

Table 1. Subbasin Parameters

Subbasin	Area (sq mi)	Curve Number	% Impervious	Lag Time
DMA 1, Existing	0.0117	67	0	11
DMA 2, Existing	0.0078	67	0	10
DMA 1, Proposed	0.0117	67	56	9
DMA 2, Proposed	0.0078	67	55	6

Table 2. Existing and Post-Project Runoff

Storm Event (24-hour)	DMA 1: Existing Runoff (cfs)	DMA 1: Post-Project Runoff (cfs)	DMA 2: Existing Runoff (cfs)	DMA 2: Post-Project Runoff (cfs)
2-year	0.3	5.7	0.2	4.4
5-year	1.4	7.7	1.0	6.1
10-year	2.7	9.7	1.9	7.6
25-year	4.9	12.7	3.4	10.0
50-year	7.0	15.3	4.8	12.1
100-year	9.4	18.3	6.5	14.4

The capacity of the bioretention basins to mitigate for hydromodification was checked by routing the subbasin runoff through reservoirs in HEC-HMS. The reservoirs are defined by elevation-area curves and outlet parameters. Allowances were made for native infiltration and soil layer infiltration through underdrain by modeling diversions between the subbasins and reservoir elements in HEC-HMS. A native infiltration rate of 2.0 in/hr was used with a factor of safety of 2 based on the project site geotechnical report from Quantum Geotechnical, Inc. If the original design by CBG was insufficient to mitigate for hydromodification, basin storage was augmented by increasing the reservoir top elevation. The final basin parameters that mitigated for hydromodification and the mitigated runoff are listed in the following tables.

Table 3. Reservoir 1 Elevation-Area

Elevation	Area (sf)
25.6	4210
29.5	4220
29.6	8240
32.0	10197
33.0	11000

Table 4. Reservoir 2 Elevation-Area

Elevation	Area (sf)
27.8	2490
31.7	2500
31.8	4993
36.0	9116

Table 5. Reservoir Outlet Structures

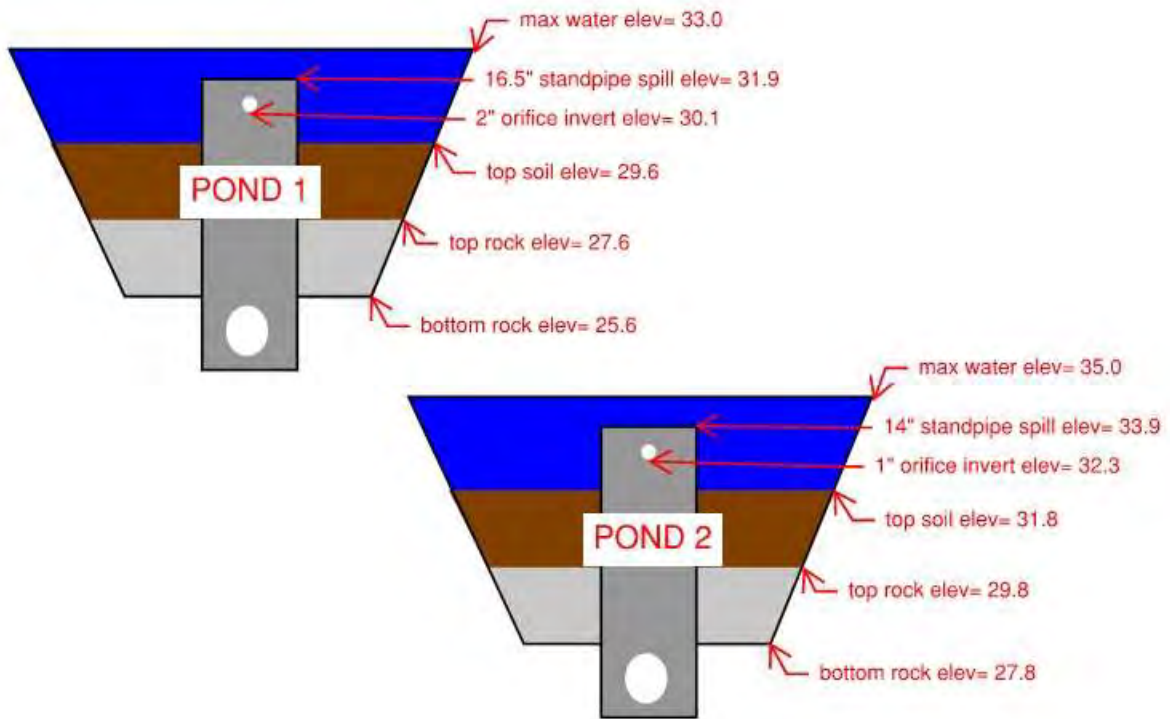
Reservoir	Native Infiltration Diversion	Soil Underdrain Diversion	Orifice 1 Invert Elevation	Orifice 1 Diameter	Standpipe Spillway Elevation	Standpipe Diameter	Max Water Elevation
DMA 1	0.2 cfs	0.1 cfs	30.1	2"	31.9	16.5"	33.0
DMA 2	0.1 cfs	0.1 cfs	32.3	1"	33.9	14.0"	35.0

Table 6. Existing and Mitigated Runoff

Storm Event (24-hour)	DMA 1: Existing Runoff (cfs)	DMA 1: Mitigated Runoff (cfs)	DMA 2: Existing Runoff (cfs)	DMA 2: Mitigated Runoff (cfs)
2-year	0.3	0.2	0.2	0.2
5-year	1.4	1.0	1.0	0.6
10-year	2.7	2.4	1.9	1.2
25-year	4.9	4.9	3.4	3.3
50-year	7.0	6.5	4.8	4.6
100-year	9.4	7.9	6.5	5.6

Conclusion

Based on this analysis, the post-project runoff can be mitigated to match existing peak flow runoff conditions from the 2-, 5-, 10-, 25-, 50-, and 100-year, 24-hour design storms according to the basin schematics below.



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