Appendix I.3 – Attachment 1: Basis for Designating Negligible Hydromodification Impact Based on Cumulative Future Buildout

Appendix I.3 – Attachment 1: Basis for Designating Negligible Hydromodification Impact Based on Cumulative Future Buildout

1. BACKGROUND

Hydromodification impacts are typically most severe just downstream of development and tend to decrease if more undeveloped watershed area contributes to the channel in the downstream direction. Analyses were performed to evaluate thresholds for additional impervious cover, from existing conditions to buildout conditions, for the area tributary to a susceptible receiving water below which the cumulative hydromodification impact is considered negligible for that channel. This analysis assumes that the existing channel morphology is considered stable[1].

The following results are provided as a function of a susceptible channel's tributary area (A):

- If A $\geq$ 1 square mile, then the threshold of additional imperviousness is evaluated using the nomograph in Figure 1.

  Figure 1 is based on empirical flow duration equations (Hawley and Bledsoe, 2011), empirical channel geometry relationships (Coleman et al, 2005 and County of San Diego, 2011), and Erosion Potential analyses. The

[1] Geomorphic stability is a geomorphic term defined as a condition in which channel form is maintained over time within a natural range of variance. The components of channel form include: (1) bed and bank configuration (i.e. grain size, resistance to movement, and sequence of bed forms); (2) cross-sectional geometry (i.e. depth and width); (3) planimetric geometry (i.e. form of the channel when viewed from above); and (4) longitudinal geometry or slope (i.e. form of the channel when viewed in profile). True stability never exists in natural streams because they are frequently undergoing channel form adjustments in order to convey a range of discharges and sediment loads. However, fluvial systems can become relatively stable in the sense that, if disturbed, they will tend to return approximately to their previous state and perturbation is damped down (Knighton 1998). A large scale event, like a flood, forest fire or landslide, can cause dramatic changes in channel form, but the channel will often re-established its equilibrium form over time. However, a persistent alteration to the controls on channel form can cause the channel to begin an evolutionary change in morphology, leading to degradation and instability until it reaches a new equilibrium state. This evolution change can take up to several hundred years before the new equilibrium is reached. For the purpose of this analysis, stream stability is assumed as the ability of a fluvial system to return to an equilibrium channel form when disturbed by external forces. Instability, thus, occurs when the controls on channel form are perturbed to a point that the fluvial system must adjust to a new equilibrium channel form. Per the Southern California Channel Evolution Model (CEM) (SCCWRP, 2013), Type I and Type V channels on the top and bottom rows of Figure 17 in Appendix I.3 are considered to be in dynamic equilibrium.

results range from 0.88% to 2.27% additional imperviousness, depending on watershed size and mean annual precipitation (MAP).

- If A < 1 square mile, then the threshold of additional imperviousness is 0.86%. (See Section 6.2 below.)

The analyses used to establish these thresholds are described below.

2. HYDROLOGIC ANALYSIS

2.1 Identify the Typical Range of Rainfall Conditions

For the purposes of this analysis, the range of mean annual precipitation (MAP) in South Orange County is assumed to be 11.5 inches to 31.0 inches per year based on the isohyetal map (Figure 2). It is anticipated that future development will impact the most miles of susceptible channel in the San Juan Creek and San Mateo Creek Watersheds and some in the upper portions of the Aliso Creek Watershed.

2.2 Identify the Range of Watershed Areas

The range of typical watershed areas used in the sensitivity analysis were established based on an inventory of a subset of natural drainage channels that have significant urban development in their tributary areas. While areas in South Orange County drain to only two large rivers with watershed area over 100 square miles (San Juan Creek and San Mateo Creek) most of the susceptible channels downstream of development have watershed areas less than 30 square miles. Seven categories of watershed area (1-, 2-, 5, 10-, 20-, 50-, and 100-square miles) were used in this analysis.

2.3 Identify Length of Daily Flow Record

A 30-year length of daily flow record was assumed in this analysis. During preliminary runs it was found that the threshold of additional impervious cover was not sensitive to changes in the assumed length of daily flow record.

2.4 Calculate Necessary Peak Flow Inputs (Q_2, Q_5, Q_{10})

Empirical peak flow equations used to estimate the 2-, 5-, and 10-year recurrence interval flows (Hawley and Bledsoe, 2011). The general form of the equation is:

$$Q_i = e^{(Incpt)} * A^a * P^p * e^{(impmax*Impmax)}$$

Where:

Q_i	=	the instantaneous peak flow at return interval i years (cfs)
Incpt	=	the vertical axis intercept of the log-transformed linear regression model
A	=	total drainage area (mi^2)
P	=	average annual precipitation (in)
Impmax	=	the maximum spatial extent of the total impervious area during the gage record as a fraction of the total drainage area (mi^2/mi^2)

a, p, and impmax = regression parameters specific to each return period

Table 1 provides the regression parameters for each return period of interest.

Table 1. Regression parameters for the 2-, 5-, and 10-year peak flows

Return Period (yrs)	Incpt (-)	a (mi^2)	p (in)	Impmax (-)
2	-0.644	0.667	1.29	8.61
5	2.137	0.838	0.773	3.23
10	2.90	0.868	0.767	0

Table 2 presents the resulting flowrates for each combination of tributary area and mean annual precipitation analyzed (14 total) assuming no imperviousness.

Table 2. Peak Flow (Q_2, Q_5, Q_{10}) Results

Tributary Area	Mean Annual Precipitation	Q2	Q5	Q10
A	MAP			
sq mi	in/yr	cfs	cfs	cfs
1	11.5	12.3	56.0	118.3
2	11.5	19.5	100.1	215.9
5	11.5	35.9	215.6	478.3
10	11.5	57.0	385.5	873.0
20	11.5	90.4	689.1	1,593.3
50	11.5	166.7	1,485.1	3,529.5
100	11.5	264.6	2,654.6	6,441.8
1	31.0	44.1	120.5	253.1
2	31.0	70.0	215.4	462.0
5	31.0	128.9	464.1	1,023.4
10	31.0	204.7	829.7	1,867.8
20	31.0	325.1	1,483.1	3,409.0
50	31.0	598.9	3,196.3	7,551.5
100	31.0	951.0	5,713.6	13,782.5

2.5 Calculate Inputs for Long-Term Cumulative Durations (Q_{max}, Q_{min}, day1, day2, N_B, $H_{B\text{-}log}$)

In order to represent the mean daily flows with cumulative duration curves, logarithmic histogram bins were created to represent flow frequencies without any discontinuities following the Hawley and Bledsoe (2011) methodology. The bin size of the logarithmically-spaced histogram bins ($H_{B\text{-}log}$) is represented as follows:

$$H_{B\text{-}log} = \{\ln(Q_{max})-\ln(Q_{min})\}/(N_B-1)$$

Where:

Q_{max} = the maximum flow of record (cfs)

Q_{min} = the minimum flow of record (cfs)

N_B = the number of bins

The minimum flow (Q_{min}) was set equal to 0.01 cfs, which represents the lowest non-zero mean daily flow reported at any gage used in the Hawley and Bledsoe (2011) analysis. The number of bins (N_B) was set at 25 to provide a balance between using small enough bin sizes for adequate resolution and ensuring that the flow-record data would be capable of populating each of the bins. The maximum flow of record (Q_{max}) is equivalent to the maximum mean 24-hour flow and is estimated using the following equation:

$$Q_{max}=e^{(-2.24)}*A^{0.979}*P^{1.79}*Yr^{0.341}$$

Where:

A	=	total drainage area (mi^2)
P	=	average annual precipitation (in)
Yr	=	the length of the mean daily flow record (30 years)

Q_{max} is also the scaling factor for the duration density function (DDF), or conditional probability density function, used to predict the cumulative durations of the binned geomorphically-effective flows. A power function is used to represent the duration in days, with the following form:

$$days = day1*Q^{day2}$$

The parameter 'day1' represents the magnitude of the power function calibrated in 'days' and 'cfs' and is estimated using the following relationship:

$$day1 = e^{(-12.9)}*A^{0.676}*P^{3.71}*Yr^{1.85}*e^{(13.8*Impav)}$$

Where:

A	=	total drainage area (mi^2)
P	=	average annual precipitation (in)
Yr	=	the length of the mean daily flow record (30 years)
Impav	=	the average spatial extent of the total impervious area expressed as a fraction of the total drainage area (mi^2/mi^2)

The parameter 'day2' represents the shape of the power function and is calibrated in 'days' and 'cfs' through the following relationship:

$$day2 = -1.60+0.166*\ln(Q_{10})-0.138*\ln(day1)+0.129*\ln(Yr)+0.720*Impav$$

Where:

Q_{10}	=	the instantaneous 10-year peak flow
Yr	=	the length of the mean daily flow record (30 years)
Impav	=	the average spatial extent of the total impervious area expressed as a fraction of the total drainage area (mi^2/mi^2)

2.6 Calculate Long-Term Cumulative Durations for Each Flow Bin (B, $B_{lwr-log}$, $B_{upr-log}$, Q, days)

Using the bin size estimated above (H_{B-log}), the lower and upper bounds of each logarithmically-spaced bin (B) can be calculated as follows:

$$B_{lwr-log} = e^{\{\ln(Qmin)+(B-2)*HB-log\}}$$

$$B_{upr-log} = e^{\{\ln(Qmin)+(B-1)*HB-log\}}$$

The average flow within each of the bins was used in the power function to calculate the cumulative duration for the histogram.

$$Q = (B_{lwr-log}+B_{upr-log})/2$$

$$days= day1*Q^{day2}$$

3. HYDRAULIC ANALYSIS

3.1 Identify a Range of Typical Receiving Channel Geometry Dimensions

An empirical relationship developed by Coleman et al (2005), modified by Stein (County of San Diego, 2011) was used to express channel dimensions (width, depth, and, to a lesser extent, gradient) as a function of dominant discharge (Q_{bf}, in cfs). The Stein and Coleman relationship was used because it: (1) produced more consistent and conservative results than the Hey-Thorne (1986) relationship; (2) resulted in critical discharges within the range of values suggested for implementation in Hydromodification Management Plans (HMPs) throughout California ($0.1Q_2$ to $0.5Q_2$); (3) was general in that it did not require an assumption of grain size (i.e., D50); and (4) is applicable to the most sensitive sand bedded channels, which the Parker (2007) relationship is not. The geometry relationships are as follows:

$$\text{Width (ft)} = 0.6012 * Q_{bf}^{0.6875}$$

$$\text{Depth (ft)} = 0.3854 * Q_{bf}^{0.3652}$$

Q_{bf}, assumed to be approximately the 5-year peak discharge (Q_5), was estimated using the empirical equation from Hawley and Bledsoe (2011) provided in Section 2.4 of this Appendix. This equation calculates Q_5 (cfs) as a function of watershed area (sq. mi.), mean annual precipitation (MAP, in/yr), and percent impervious cover (%) based on empirical observations of USGS gages.

Manning's equation was used to iteratively find the slope for each channel dimension, such that the wetted cross sectional area at bankfull conveys the Q_5. Manning's equation is expressed as:

$$Q = \frac{1.49 A R^{0.67} S^{0.5}}{n}$$

Where:

Q	=	Flowrate (cfs)
A	=	Cross Section Flow Area (ft^2)
R	=	Hydraulic Radius (ft) = A / P
P	=	Wetted Perimeter (ft)
S	=	Energy Gradient Assumed Equal to Longitudinal Slope (ft/ft)
n	=	Manning Roughness (unitless)

The hydraulic analysis assumed a Manning Roughness value (n) of 0.035 for the main channel, corresponding to a non-vegetated, straight channel with no riffles and pools. This reflects the small, ephemeral receiving channels which are prevalent in Southern California. A relatively low 'n' value was used at the request of the San Diego Regional Water Board in the development of the San Diego HMP. A Manning's roughness of 0.07 was used for the over bank floodplain with an assumed side slope of 10 to 1 (Horizontal:Vertical). The overbank parameters were not as sensitive of parameters as longitudinal slope and channel geometry for the purpose of this analysis, therefore a range was not evaluated.

The receiving channel geometry dimensions used for hydraulic analysis of each model scenario are presented in Table 3.

Table 3. Receiving Channel Geometry Dimensions

Tributary Area	Mean Annual Precipitation	Longitudinal Slope	Bankfull Width	Bankfull Depth
A	*MAP*	*S*	*W*	*D*
sq mi	in/yr	%	ft	ft
1	11.5	0.50	9.6	1.7
2	11.5	0.34	14.3	2.1
5	11.5	0.20	24.2	2.7
10	11.5	0.14	36.0	3.4
20	11.5	0.09	53.7	4.2
50	11.5	0.06	91.1	5.5
100	11.5	0.04	135.8	6.9
1	31.0	0.30	16.2	2.2
2	31.0	0.20	24.2	2.7
5	31.0	0.12	41.0	3.6
10	31.0	0.08	61.1	4.5
20	31.0	0.06	91.0	5.5
50	31.0	0.03	154.3	7.3
100	31.0	0.02	230.1	9.1

3.2 Calculate Effective Shear Stress and Velocity for Each Flow Bin

The flow velocity was calculated after iterating for the slope to achieve $Q=Q_5$ as:

$$V = Q/A$$

Where:

V	=	Flow Velocity (ft/s)
Q	=	Flowrate (cfs)
A	=	Cross Section Flow Area (ft^2)

Average boundary shear stress was calculated as:

$$\tau = \gamma R S$$

Where:

τ	=	Effective Shear Stress (lb/ft^2)
γ	=	Unit Weight of Water (62.4 lb/ft^3)
R	=	Hydraulic Radius (ft)
S	=	Longitudinal slope (ft/ft)

4. WORK ANALYSIS

4.1 Identify Critical Flowrate (10%Q_2)

The regional default critical flowrate of 10% Q_2, per the South Orange County HMP, was used for this analysis. Flow rates below this value were assumed to perform no work on the channel.

4.2 Calculate Work for Each Flow Bin

The simplified effective work equation used is one cited in previous HMPs (SCVURPPP, 2005; FSURMP, 2009; City of Vallejo, 2013; VCSQMP, 2013), Watershed Management Area Analysis (WMAA) (County of San Diego, 2015), and stormwater permits (LARWQCB, 2010 and LARWQCB, 2012) in California. The effective work equation is expressed as:

$$W = (\tau - \tau_c)^{1.5} V$$

Where:

W	=	Work [dimensionless];
τ	=	Effective Shear Stress [lb/ft^2];
τ_c	=	Critical Shear Stress [lb/ft^2];
V	=	Flow Velocity [ft/s]

If the effective shear stress for a given flow bin is less than the critical shear stress, then the effective work is equal to zero.

5. CUMULATIVE WORK ANALYSIS

Cumulative work is a measure of the long-term total work or sediment transport capacity performed at a creek location. It incorporates the distribution of both discharge magnitude and duration for the full range of flowrates simulated. To calculate cumulative work, first the work and duration associated with each flow bin is multiplied. Then the cumulative work for all flow bins is summed to obtain total work. This analysis can be expressed as:

$$W_t = \sum_{i=1}^{n} W_i \, \Delta t_i$$

Where:

W_t	=	Total Work [unitless]
W_i	=	Work per flow bin [unitless]
Δt	=	Duration per flow bin [days]
n	=	number of flow bins

6. EROSION POTENTIAL ANALYSIS

Ep is calculated by simply dividing the total work of the post-development condition by that of the pre-development condition, which in this case is the existing condition. Ep is expressed as:

$$E_p = W_{t,post} \,/\, W_{t,pre}$$

Where:

E_p	=	Erosion Potential [unitless]
$W_{t,post}$	=	Total Work associated with the post-development condition [unitless]
$W_{t,pre}$	=	Total Work associated with the pre-development condition [unitless]

6.1 Iterate % Impervious Cover to Meet the Erosion Potential Management Objective (Ep < 1.1)

The Erosion Potential (Ep) management objective written in the South Orange County HMP (County of Orange, 2015) states that:

> *Hydromodification management measures will be selected and designed to maintain the Ep ratio within 10 percent of the target value in the receiving waters. The target Ep will be adjusted to account for changes in bed sediment supply.*

It is assumed that an Ep value less than 10% of the target value meets this Ep Management Standard. Additional basis for the use of a 10% allowance on Ep is supported by the statement in the South Orange County HMP that, "studies have demonstrated that achieving an optimum capacity supply ratio within 10 percent of the unity should ensure the dynamic stability of a stream while allowing the river to recover some of the morphological detail that cannot be designed a-priori (USACE, 2001)".

An iterative process was used to determine the percentage of additional impervious cover that meets the Ep management objective (Ep < 1.1), assuming negligible change in bed sediment supply. The percent imperviousness is an input for the DDF power function coefficient and exponent (day1 and day2, respectively) and modifies the duration of flows within each of the logarithmically-spaced flow bins. The new durations (days) for each flow bin are multiplied by the work per flow bin (W_i) and summed across all bins to arrive at a new value for total work associated with the post-project condition ($W_{t,post}$) and Erosion Potential (Ep). Percent impervious cover is subsequently adjusted until an E_p of 1.1 is converged upon.

In preliminary runs it was evaluated that the threshold additional impervious cover was not highly sensitive to the baseline pre-development (or existing condition) imperviousness. For example, an increase in imperviousness from 0% to 1% resulted in the same Ep as an increase from 10% to 11%. The resulting thresholds of additional imperviousness from existing (at the time of the HMP effective date) to buildout conditions are provided below in Table 4 and in Figure 1. The results provided use an existing imperviousness of 0%. Given that the resulting threshold imperviousness values are low, it is anticipated that changes in bed sediment supply associated with this level of buildout development would be negligible (i.e., well less than 10% reduction).

Table 4: Threshold Additional Imperviousness Results

Tributary Area	Mean Annual Precipitation	Threshold Additional Imperviousness	Pre-Development Total Cumulative Work	Post-Development Total Cumulative Work	Erosion Potential
A	*MAP*	*Impav*	*Wt, pre*	*Wt, post*	*Ep*
sq mi	in/yr	%	--	--	--
1	11.5	0.88	1.08	1.18	1.10
2	11.5	0.94	1.02	1.12	1.10
5	11.5	1.05	1.02	1.12	1.10
10	11.5	1.15	1.08	1.19	1.10
20	11.5	1.27	1.20	1.32	1.10
50	11.5	1.48	1.50	1.65	1.10
100	11.5	1.69	1.87	2.06	1.10
1	31.0	1.03	5.47	6.02	1.10
2	31.0	1.12	4.34	4.78	1.10
5	31.0	1.27	3.43	3.77	1.10
10	31.0	1.41	3.03	3.34	1.10
20	31.0	1.60	2.81	3.09	1.10
50	31.0	1.92	2.64	2.91	1.10
100	31.0	2.27	2.67	2.94	1.10

6.2 Thresholds of Additional Imperviousness Based on Hawley and Bledsoe (2013)

For susceptible channels with a tributary area less than one square mile, the threshold of additional imperviousness below which hydromodification impact is considered negligible for that channel is 0.86%. This result is based on equating two of the channel enlargement equations listed in Hawley and Bledsoe (2013) and solving for an Ep of 1.1. The two enlargement functions are:

$$Ar = 1.18 * Ep^{0.998}$$

and

$$Ar = 1.18 * e^{(11.0 * Imp)}$$

Where:

Ar = Enlargement expressed as the relative magnitude

Ep = Sediment-transport capacity load ratio between 25-yr post-developed and pre-developed DDF simulations

Imp = Total impervious area as a fraction of total drainage area

The following equation expresses Imp as a function of Ep:

$$Imp = (0.998/11) * \ln(Ep)$$

Assuming Ep is equal to 1.1, the resulting Imp is 0.86%.

7. REFERENCES

Coleman, D., MacRae, C., and Stein, E.D. 2005. Effect of Increases in Peak Flows and Impervious on the Morphology of Southern California Streams, Technical Report 450. Southern California Coastal Water Research Project.

City of Vallejo. 2013. Hydromodification Management Plan. April.

County of Orange. 2015. South Orange County Hydromodification Management Plan (HMP). OC Watersheds. April 1.

County of San Diego. 2011. Final Hydromodification Management Plan. March.

County of San Diego. 2015. San Diego County Regional Watershed Management Area Analysis. October.

Fairfield-Suisun Urban Runoff Management Program. 2009. Hydromodification Management Plan for the Fairfield-Suisun Urban Runoff Management Program. February 4.

Hawley, R.J., and Bledsoe, B.P. 2011. "How do flow peaks and durations change in suburbanizing semi-arid watersheds? A southern California Study," Journal of Hydrology, Elsevier, Vol 405, pp 69-82.

Hawley, R.J., and Bledsoe, B.P. 2013. "Channel enlargement in semiarid suburbanizing watersheds: A southern California case study," Journal of Hydrology, Elsevier, Vol 496, pp 17-30.

Hey, R.D., and Thorne, C.R. 1986. "Stable Channels with Mobile Gravel Beds," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol 112, No. 8, pp 671-689.

Los Angeles Regional Water Quality Control Board. 2010. Ventura County Municipal Separate Storm Sewer System (MS4) Permit. Order No. R4-2010-0108.

Los Angeles Regional Water Quality Control Board (LARWQCB). 2012. Los Angeles County MS4 Permit. Order No. R4-2012-0175.

Parker, G.P., P.R. Wilcock, C. Paola, W.E. Dietrich, J.C. Pitlick. 2007. "Physical basis for quasi-universal relationships describing bankfull hydraulic geometry of single-thread gravel bed rivers". Journal of Geophysical Research – Earth surface. vol. 112.

Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP). 2005. Hydromodification Management Plan. April 21.

US Army Corps of Engineers (USACE). 2001. Channel Restoration Design for Meandering Rivers. Coastal and Hydraulics Laboratory. ERDC/CHL CR-01-1. September.

Ventura Countywide Stormwater Quality Management Program (VCSQMP). 2013. Ventura County Hydromodification Control Plan. September.