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RECLAMATION

Technical Report No. ENV-2020-060

Sacramento River Gravel Augmentation Study

U.S. Department of the Interior **May 2020**

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Cover: Photograph of gravel on the bottom of the Sacramento River.

Sacramento River Gravel Augmentation Study

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Sedimentation and River Hydraulics Group
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PEER REVIEW DOCUMENTATION

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Client: California-Great Basin Region, Bay-Delta Office

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Contents

1. Introduction

The Technical Service Center (TSC) was requested by the Bay-Delta Office (BDO) of the Mid-Pacific Regional Office to conduct a study of the gravel augmentation along the Sacramento River from Keswick Dam to the confluence of Clear Creek, approximately 13 miles of river. The gravel augmentation has been ongoing since the early 1990s as part of the Spawning and Rearing Habitat Restoration Program, Section 3406 b(13) of the Central Valley Improvement Act (CVPIA). The Central Valley Project Improvement Act was formed under H.R. 429, Public Law 102-575 and includes 40 separate titles providing for water resource projects throughout the West. Title 34, the Central Valley Project Improvement Act, mandates changes in management of the Central Valley Project, particularly for the protection, restoration, and enhancement of fish and wildlife.

The goal of this study is to develop quantitative estimates of the gravel movement through the study reach and to assist the CVPIA program on making improvements to their gravel augmentation strategies.

2. Methods

2.1 Topobathymetric Data Collection and Surface Generation

The National Oceanic and Atmospheric Administration's Southwest Fisheries Science Center partnered with Quantum Spatial (QSI) to conduct an airborne topobathymetric LiDAR (Light Detection and Ranging) survey of the Upper Sacramento River near Redding, CA [*Quantum Spatial*, 2018]. The flights took place Sept. 10 - 17, 2017. Discharge in the study reach, as measured at the Keswick gage (USGS 11370500), was steady at about 255 m^3/s (9000 cfs) during the data collection.

Sub-aerial elevations collected with near-infrared LiDAR were combined with sub-aqueous elevations collected with green wavelength LiDAR to form a continuous bare Earth topobathymetric surface in the form of a digital elevation model (DEM) with 1 m x 1 m cells. Green LiDAR can only penetrate clear, relatively shallow water, so gaps in the LiDAR coverage were filled in by a boat-based sonar survey, described in the next section. An example of the LiDAR coverage is shown in Figure 1.

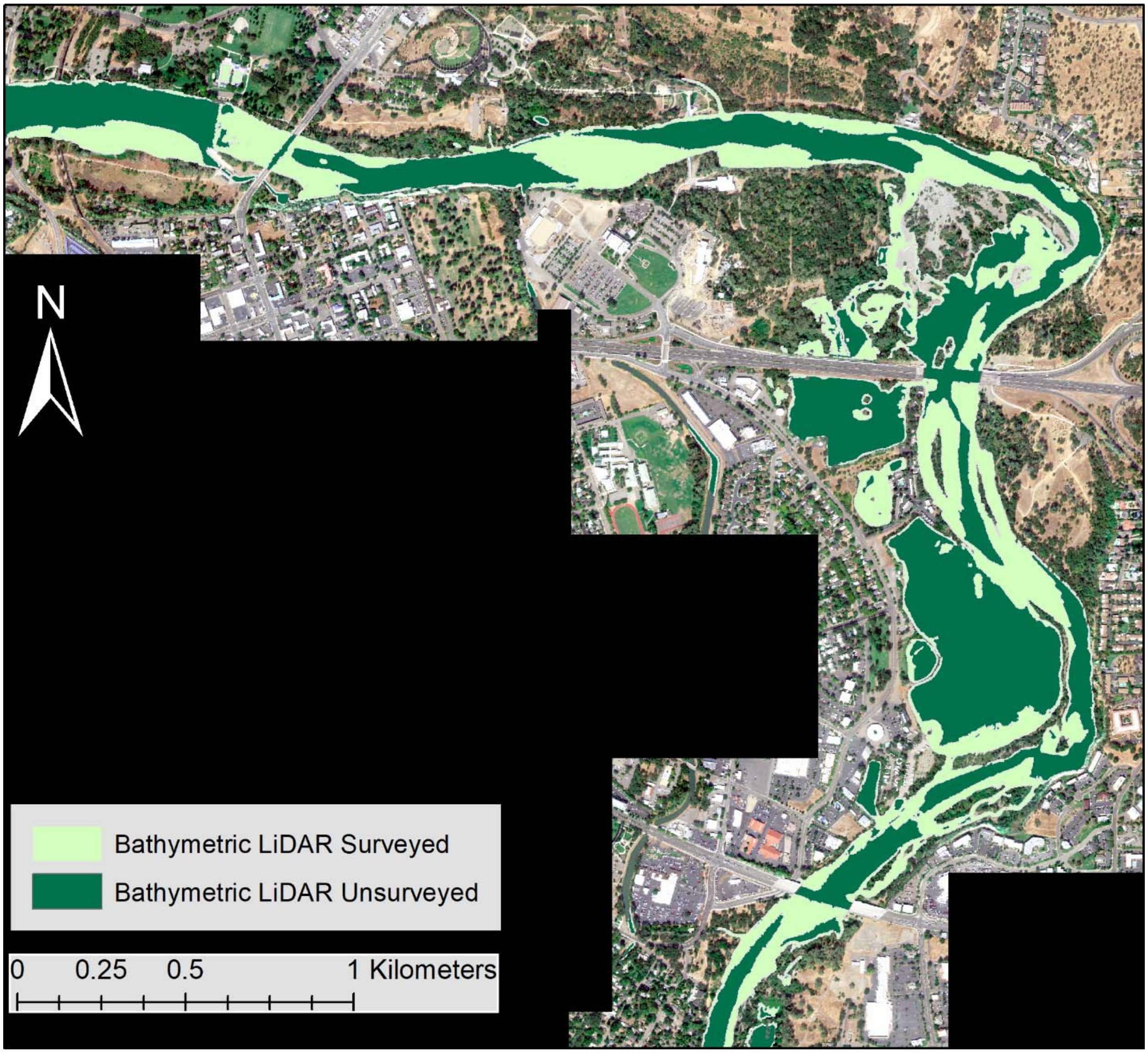


Figure 1. The bathymetric LiDAR was effective in the areas shown in light green. Dark green areas indicate water too deep or turbid for effective green LiDAR penetration.

2.2 Bathymetric Sonar Survey

To fill in the bathymetry gaps (shown in dark green in Figure 1), staff from the Bureau of Reclamation's Sedimentation and River Hydraulics group conducted a boat-based multi-beam sonar survey of the river on Sept. 23 - 29, 2018 using a Teledyne Odom MB1. Flow at the time of the survey at the Keswick gage averaged about 210 m^3/s (7415 cfs). The survey extended from Keswick to downstream of Clear Creek. Some side channels and other off-channel areas remain unsurveyed. More the 3 million bed elevation points were collected. An example of the sonar survey coverage shown in purple in Figure 2. TSC created a 1 m DEM from the sonar survey points and stitched it together with the Quantum Spatial DEM to form a single continuous surface that formed the basis of the hydraulic and sediment transport models discussed below. An example of the topobathymetric surface is shown in Figure 3.

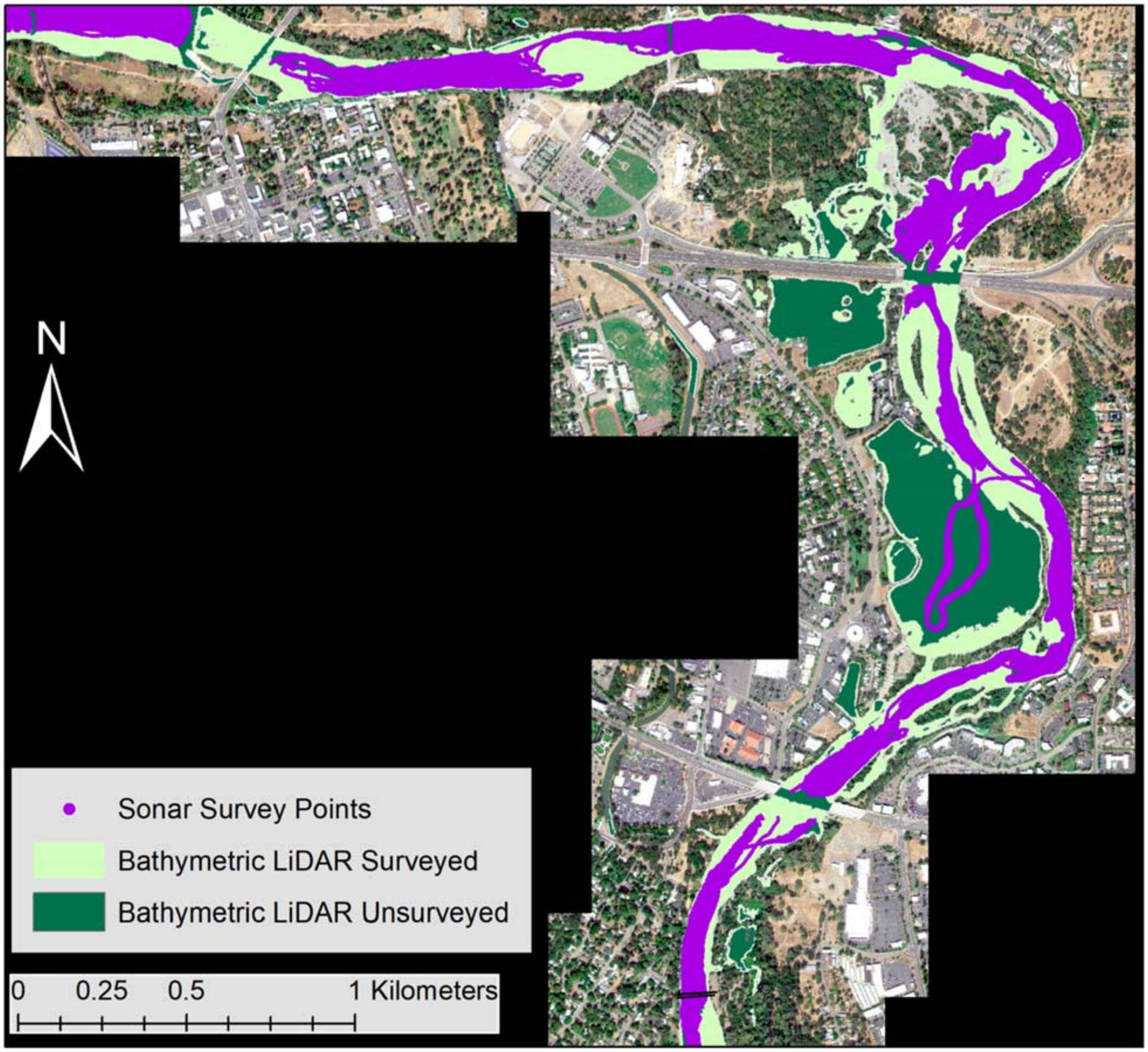


Figure 2. An example of the coverage of the boat-based sonar survey. Each purple dot represents a sonar sounding. (Scale 1:19000)

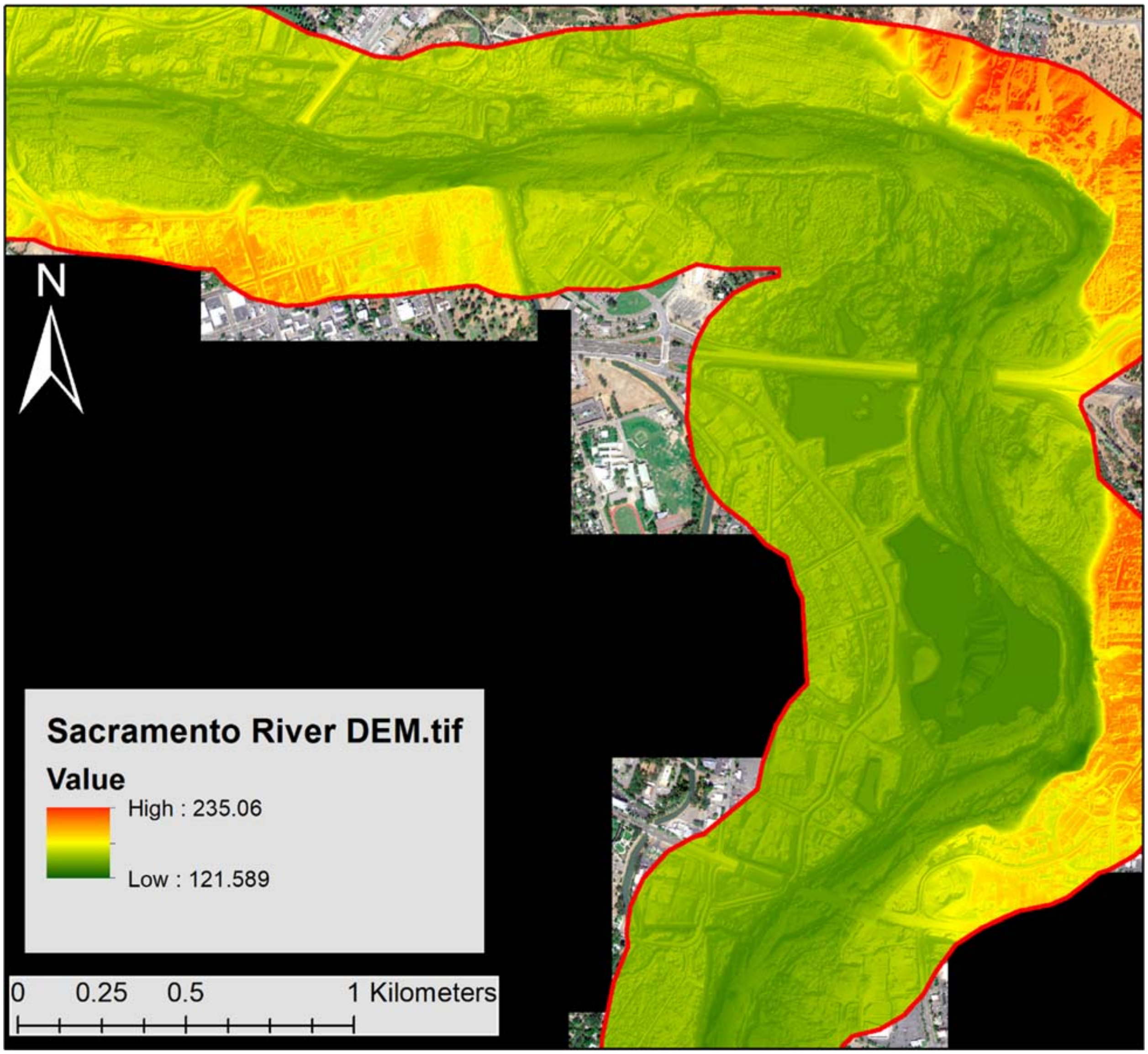


Figure 3. The final topobathymetric surface merging data from the LiDAR survey with the data from the boat-based sonar survey.

2.3 Hydraulic Model Development

We developed a two-dimensional (2D) hydraulic model of the study reach from Keswick Dam to the confluence with Clear Creek using the Bureau of Reclamation's Sedimentation and River Hydraulics 2D model, SRH-2D. SRH-2D is a two dimensional, depth-averaged hydraulic model [*Lai*, 2008]. The model solves the depth-integrated, dynamic wave approximation of the shallow water Navier-Stokes fluid flow equations with a finite-volume numerical method.

More information about SRH-2D can be found at http://www.usbr.gov/tsc/techreferences/computer%20software/models/srh2d/index.html.

Developing a hydraulic model consists of developing a model mesh, establishing the model boundary conditions, calibrating the model to observations of water surface elevation at a known discharge, and then analyzing the results.

2.3.1 Model Mesh

The first step in the development of a hydraulic model is the creation of the model mesh. The model mesh defines the model domain and discretizes the physical space represented by the model so that the differential equations describing the continuous physical process of fluid flow can be solved numerically between mesh elements. The mesh represents the underlying terrain by assigning elevations to the mesh nodes. The model mesh also defines the spatial resolution of the model. The hydraulic variables computed by the model (water depth and velocity, for example) are spatially averaged over the area represented by each of the mesh elements. Smaller mesh elements average over a smaller area and are therefore better able to represent the slow, shallow water that provides rearing habitat to juvenile salmonids. Smaller mesh elements also represent variations in bed roughness and elevation in more detail. The downside to using smaller mesh elements is that more elements are required to cover a given area and consequently, the model requires more computation time. The model mesh represents a trade-off between the model resolution and the time required for computation of a solution.

We developed a model mesh with just under 430,000 elements. 55% of the total number of mesh elements are quadrilateral channel elements and the remainder are triangular floodplain elements. The modal size of the channel elements is about 6 m long (in the streamwise direction) and 4 m wide. An example of the model mesh at Lake Redding is shown in Figure 4. The distributions of mesh element length and width are plotted in Figure 5.

Figure 4. The hydraulic model mesh at Lake Redding. The entire model mesh has 427,934 elements.

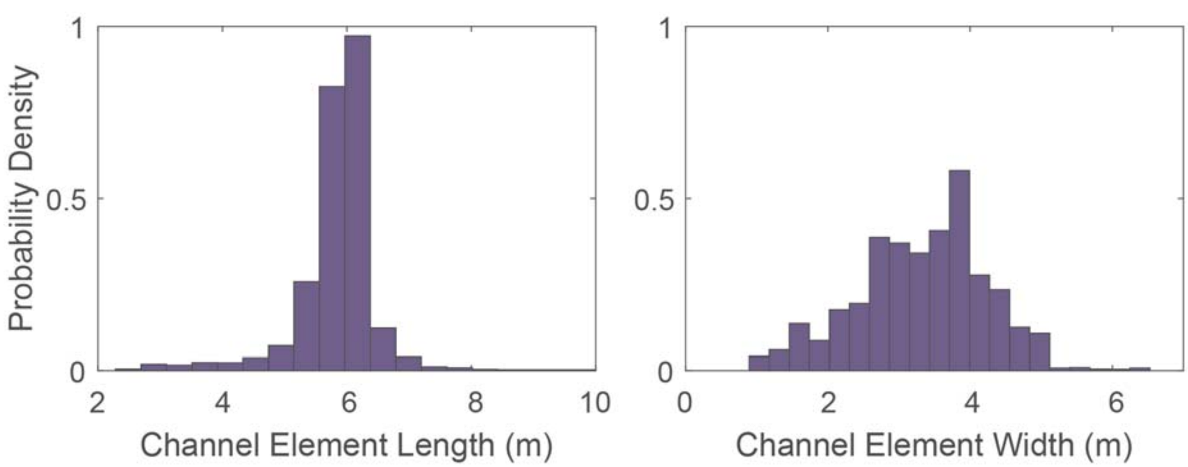


Figure 5. Histograms of channel element streamwise length (left) and cross stream width (right). The modal size of channel elements is about 6 m x 4 m.

2.2.2 Boundary Conditions

The model boundary conditions define how water enters and exits the model domain. The inlet boundary condition is the amount of water flowing into the model from the upstream end of the domain and from any number of tributaries. The outlet boundary condition is the water surface elevation (WSE) at the model boundary for the simulated discharge. The Upper Sacramento River model has an inlet at the Keswick Dam spillway and an outlet boundary just downstream of the confluence with Clear Creek on the south side of Redding (Figure 6). No tributaries were included in the model. Inlet flows were chosen to cover a range from the flow during the sonar survey up to the 100-year flood. The outlet boundary conditions are specified by a rating curve developed from an U.S. Army Corps of Engineers (USACE) HEC-RAS model at cross sections 282.909 and 283.33 (USACE, 2002). The water surface specified by the rating curve was lower than the bed elevation at the model outlet at a flow of 210 m^3/s, so we shifted the curve upwards by 1.8 m to yield a realistic water depth of about 1 m at 210 m^3/s. The boundary conditions modeled are summarized in Table 1.

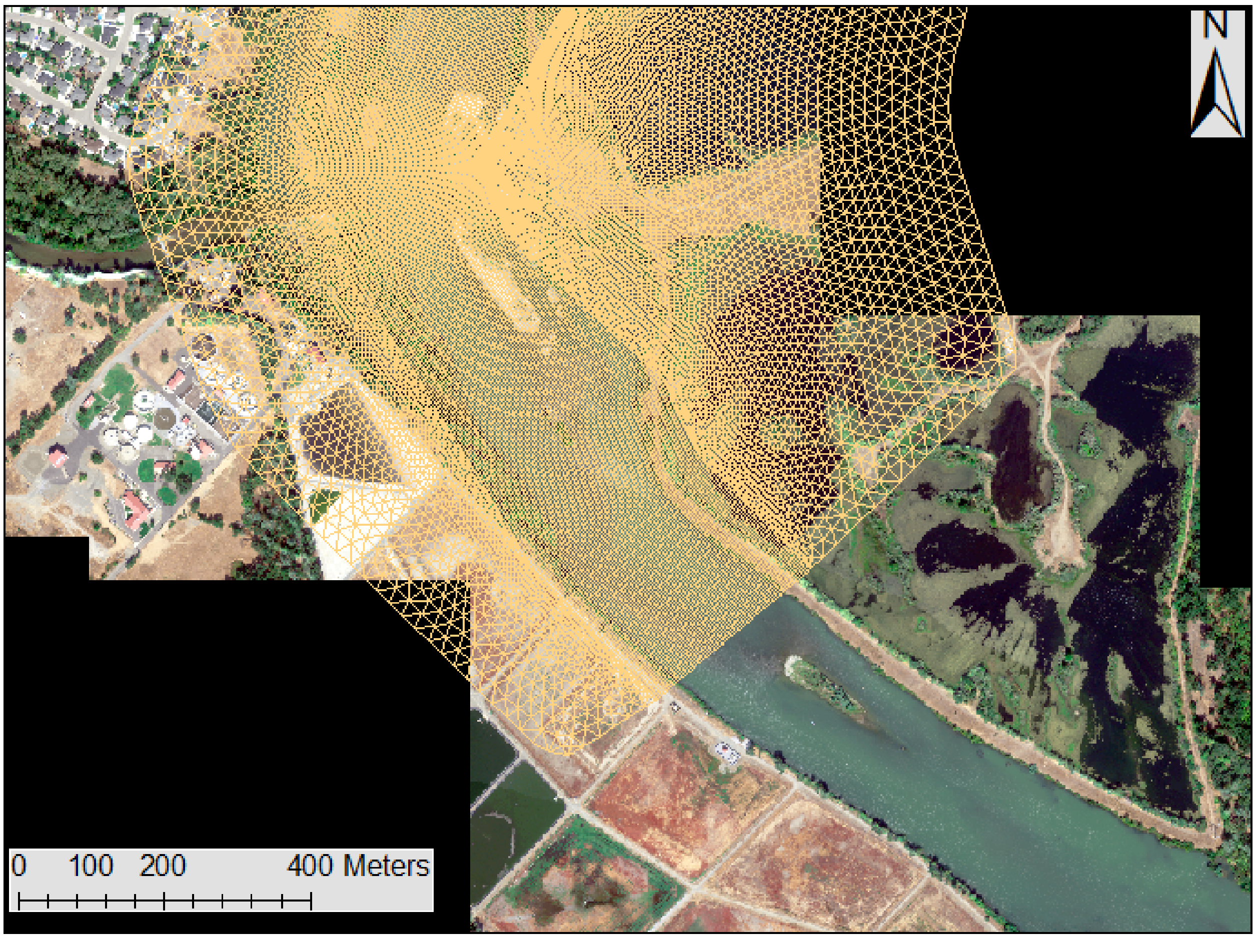


Figure 6. The model mesh at the downstream end of the model, near Clear Creek, visible at upper left.

Table 1. Outlet water surface elevation boundary conditions for a range of inlet flows at Keswick Dam

Q (m^3/s)	**Q (cfs)**	**WSE (m)**
100	3531	127.30
170	6003	127.8
210	7416	128.00
500	17,657	128.89
750	26,485	129.43
1000	35,313	129.83
1250	44,142	130.18
1500	52,970	130.49
2000	70,627	131.03
2240	79,102	131.27

2.2.3 Model Calibration

We surveyed nineteen water surface elevations (WSEs) with a Real-time Kinematic Global Positioning System (RTK GPS) at locations where the river could be accessed from shore in Sept. 2018 during the sonar survey. These locations were at Middle Creek, Lake Redding Park, Sundial Park, East Turtle Bay, Park Marina, Bonnyview, and Cascade Park. The locations are shown as blue triangles in Figure 7.

We also surveyed the elevations of presumably stable surfaces (roads, parking lots, etc.) to compare to the LiDAR survey. The distribution of these elevation residuals (GPS survey elevation – LiDAR elevation) and the reported accuracy of the topobathymetric LiDAR survey informed our judgement of what level of hydraulic model calibration was sufficient. It is pointless to attempt to calibrate a model beyond the level of error in the underlying topographic surface and water surface elevation measurements. There is also error associated with the measured flow rate.

We ran the model with an inlet flow of 210 m^3/s (the approximate flow at the time of the survey) and compared the model water surface to the observations by calculating the WSE residuals (observed WSE – modeled WSE) and then adjusted the model bed roughness in the form of Manning's n to match the surveyed water surface elevations as closely as possible. Our calibration goal was a distribution of WSE residuals that was symmetric around 0 with most residuals in the range of +/- 10 cm.

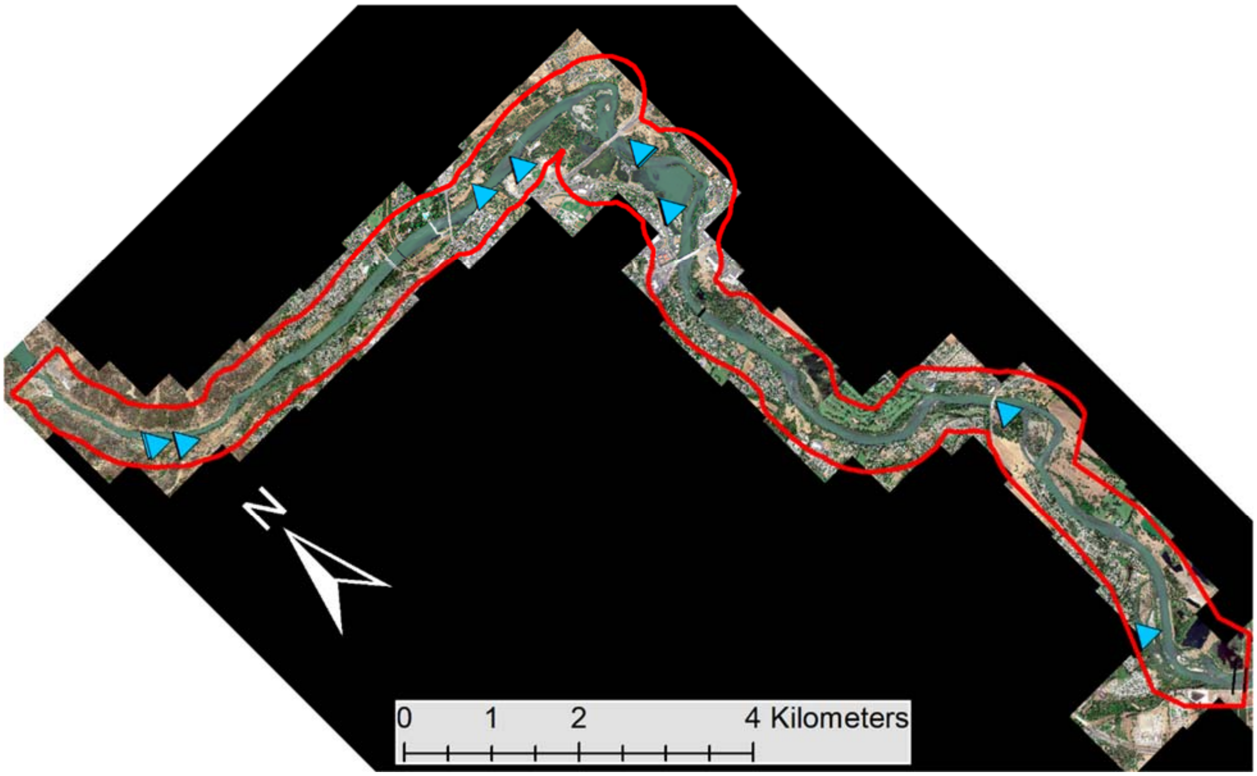


Figure 7. The hydraulic model domain covers Keswick to downstream of Clear Creek. The mesh boundary is shown in red. The blue triangles mark locations where water surface elevations were surveyed for model calibration, discussed below.

2.4 Hydraulic Model Analysis

After calibrating the model, we ran the range of flows listed in Table 1. An example of the depth and velocity estimated by the model at 1000 m^3/s is shown in Figure 8. We used the hydraulic results to estimate the amount of available salmonid habitat at a range of flows and to compute indicators of sediment transport. Those analyses are described below.

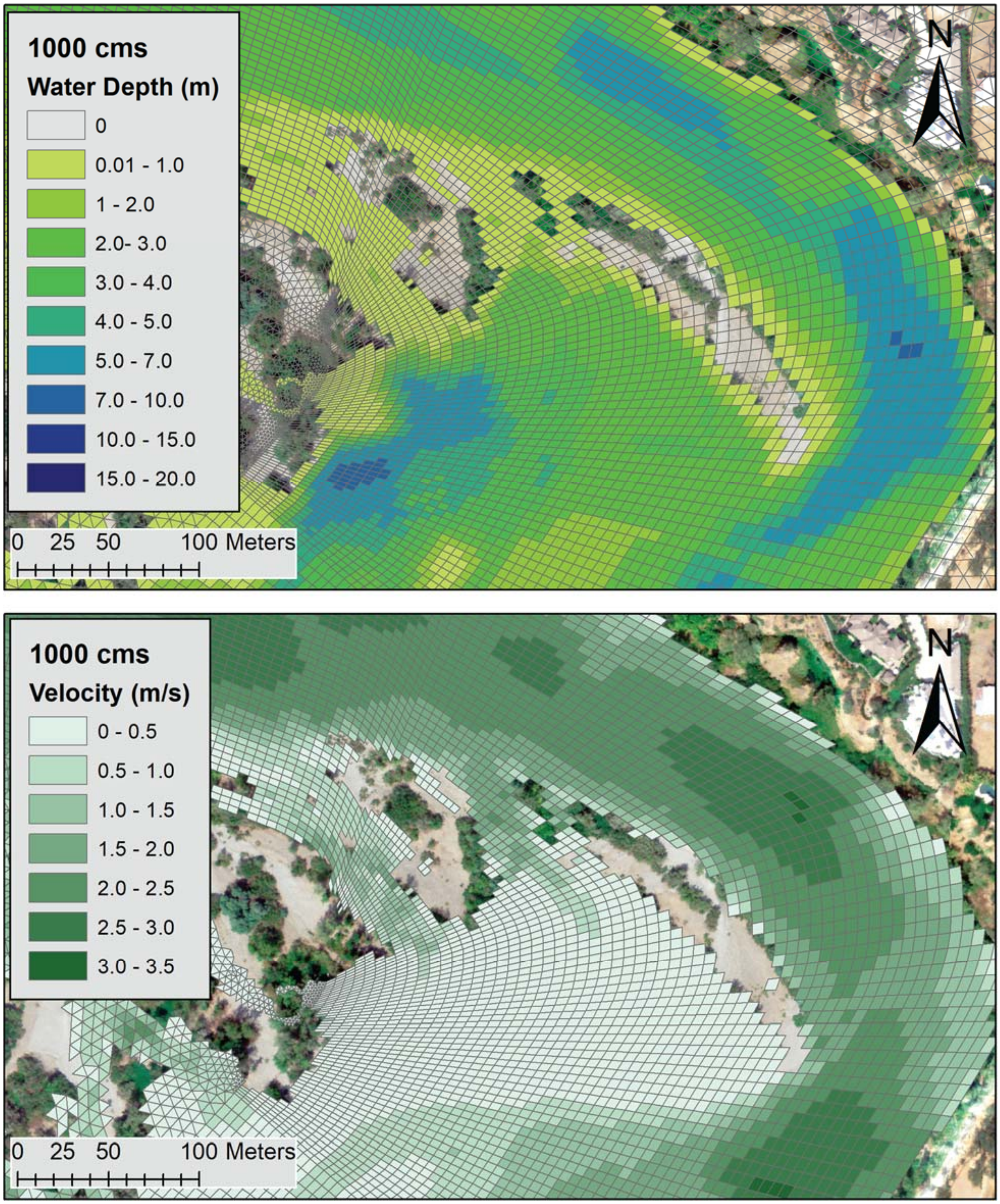


Figure 8. Examples of the model depth (top) and velocity (bottom) at Turtle Bay at 1000 m^3/s (~35,300 cfs).

2.4.1 Salmonid Habitat Suitability

A habitat suitability index (HSI) is a measure of how "good" habitat conditions are for a particular species. In the context of a hydraulic model, the HSI is function of hydraulic variables such as water depth and water velocity. We computed salmonid spawning and rearing HSI for each model cell and integrated the HSI over the model domain to estimate the amount of habitat in the study reach.

The suitability of a model cell for spawning is a function of the water depth and velocity. We used HSI curves supplied by the Bay-Delta Office. Those curves are shown in Figure 9.

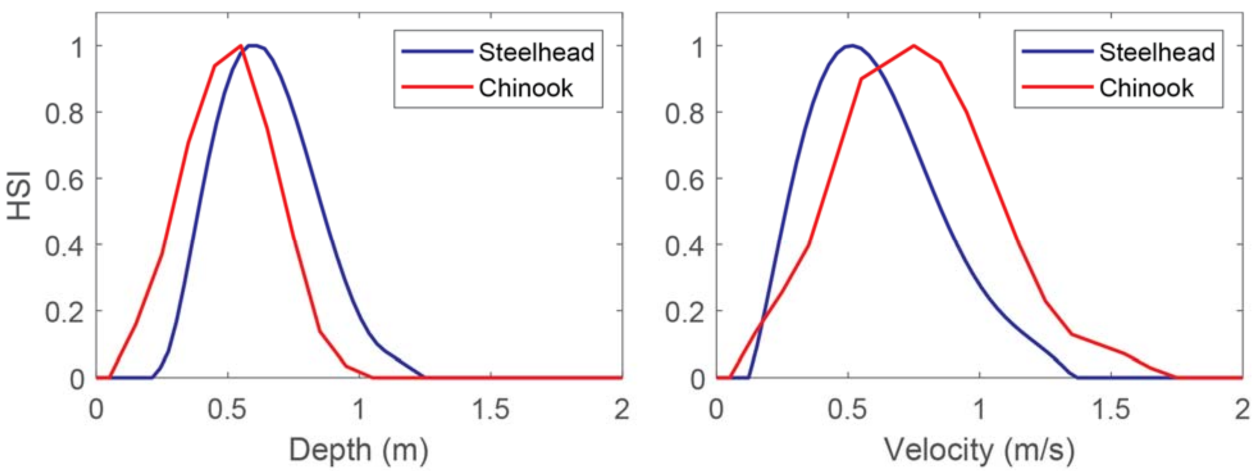


Figure 9. Spawning Habitat Suitability Index (HSI) curves for depth (left) and velocity (right).

The spawning HSI is the geometric mean of the depth and velocity indices (Equation 1). An example of spatially distributed HSI is shown in Figure 10.

Equation 1

$$HSI_i = \sqrt{HSI_{D_i} HSI_{V_i}}$$

The spawning Weighted Useable Area (WUA) for model cell i is the product of cell area and the suitability index, $A_i HSI_i$. The total WUA in a stream reach is the sum of the model cells in that reach.

Equation 2

$$WUA = \sum_{i=1}^{N} A_i HSI_i$$

where the sum is over the model cells in the reach. We divided the model domain into 4 reaches, shown in Figure 11. The reach polygons over which habitat was calculated.These polygons define the domain of the summation in Equation 2.

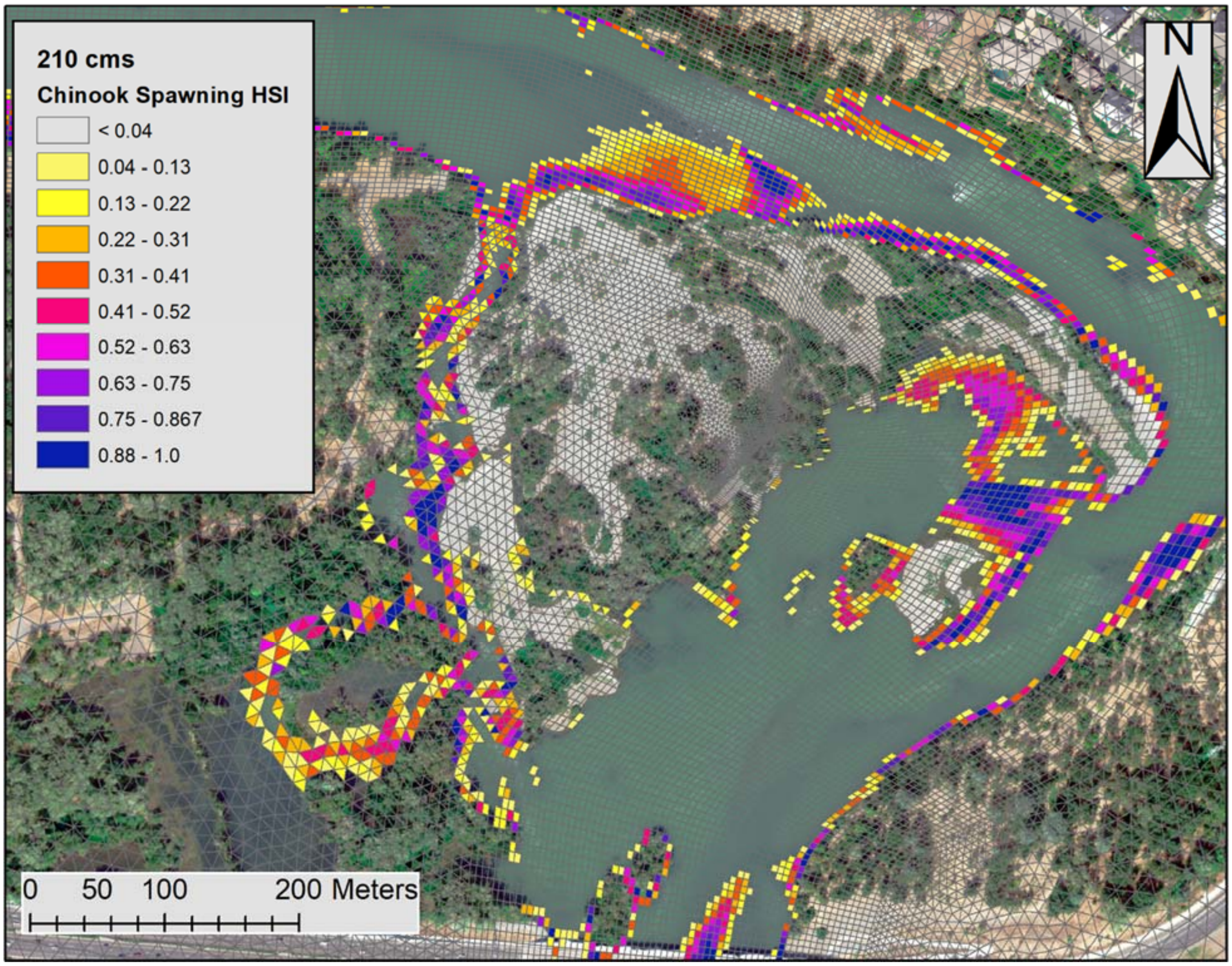


Figure 10. An example of Chinook spawning HSI in the vicinity of Turtle Bay at 210 cubic meters per second (cms).

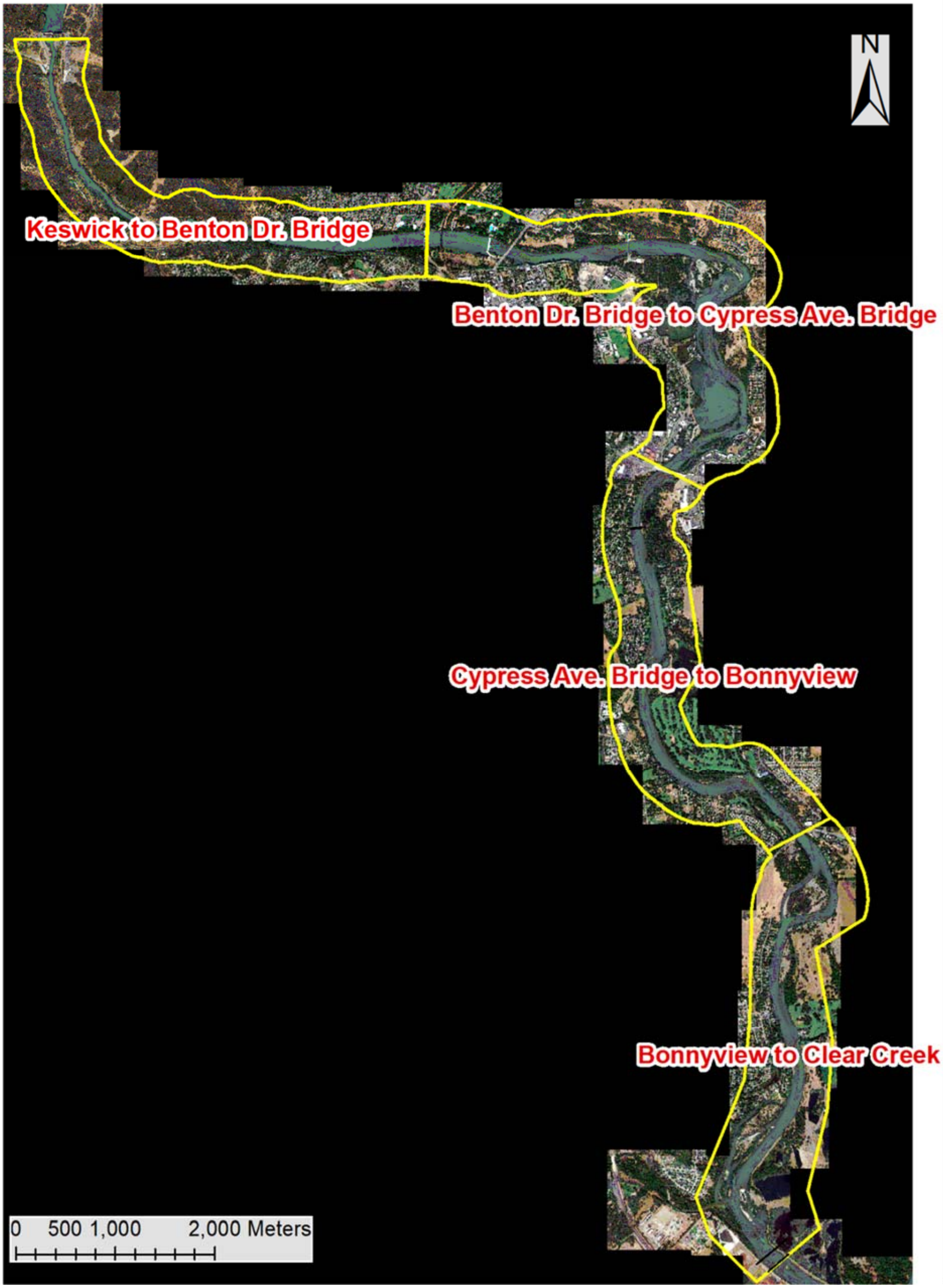


Figure 11. The reach polygons over which habitat was calculated.

We calculated the salmonid rearing HSI in a binary fashion (a model cell is or is not considered suitable habitat) based on criteria listed in Table 2. We did not model flows as low as 92-170 m^3/s and no map of escape cover was readily available, so we computed habitat based upon depth and velocity and summed the suitable area over the reach polygons shown in Figure 11. The reach polygons over which habitat was calculated. Rearing habitat criteria are not species specific. An example of the rearing HSI results are shown in Figure 12.

Table 2. Hydraulic conditions that constitute rearing habitat. Only the depth and velocity criteria were used in our analysis

Flow (m^3/s)	Depth Range (m)	Velocity Range (m/s)	Distance to Cover (m)
92 – 170	0.15 – 1.5	0.03 – 0.6	< 0.6

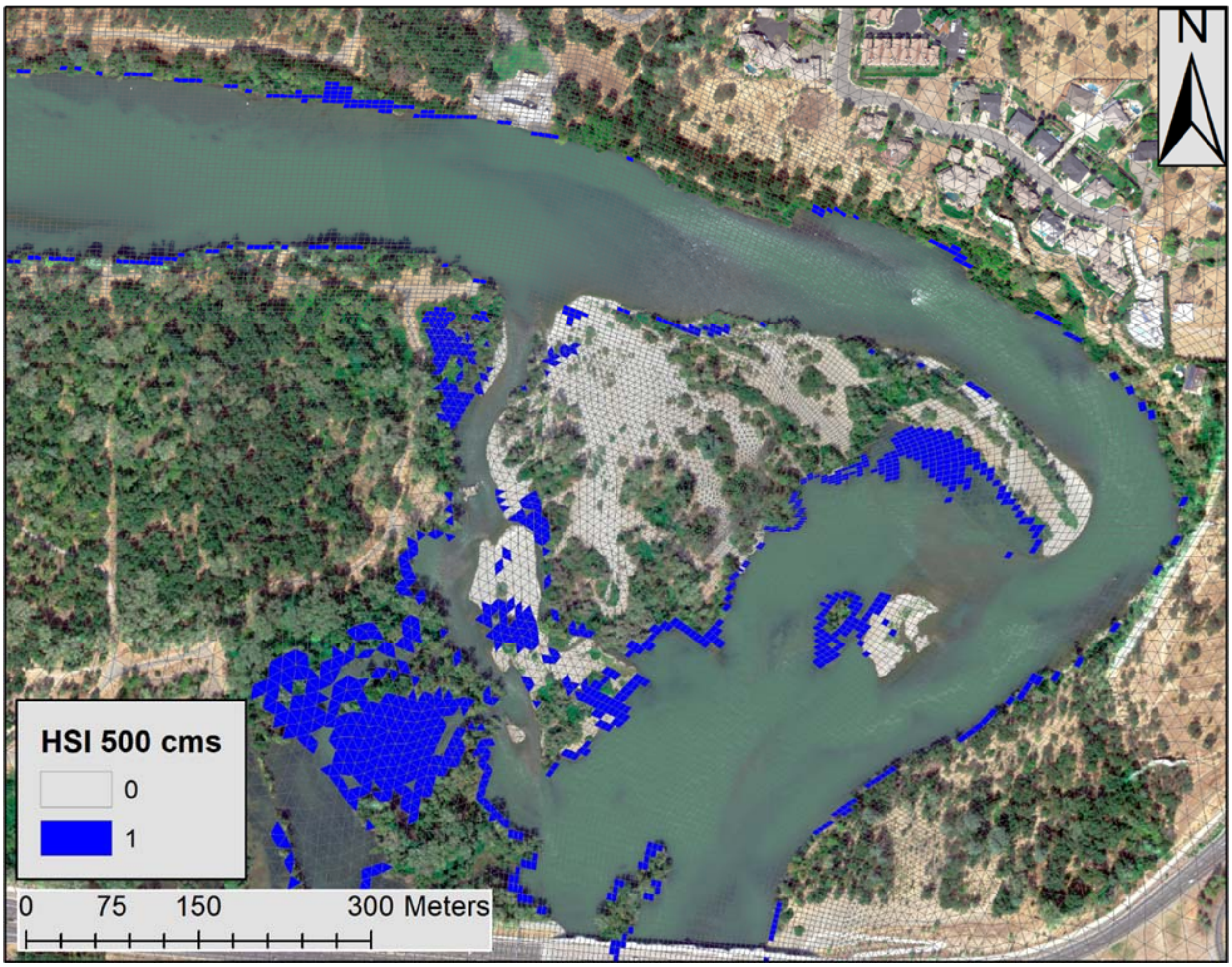


Figure 12. An example of rearing HSI (0=not suitable, 1=suitable) in the vicinity of Turtle Bay.

2.4.2 Indicators of Sediment Transport

2.4.2.1 Shields Stress

The Shields stress, θ, a measure of the river's ability to mobilize sediment of a given grain size D, is give by

Equation 3

$$\theta = \frac{\tau}{(\rho_s - \rho)gD}$$

where τ is the basal shear stress computed by the hydraulic model, ρ_s is the density of the sediment (2650 kg/m^3), ρ is the density of water, and g is gravitational acceleration. As the Shields stress exceeds a threshold, or reference stress θ_r, the grain is more likely to move. We computed Shields stress for grain sizes ranging from 20 mm to 100 mm for each model cell and summed the area exceeding a threshold Shields stress of $\theta_r = 0.04$ over the reach polygons shown in Figure 11 to yield an estimate of the fraction of wetted area in each reach capable of mobilizing sediment. An example of the spatial distribution of Shields stress is shown in Figure 13.

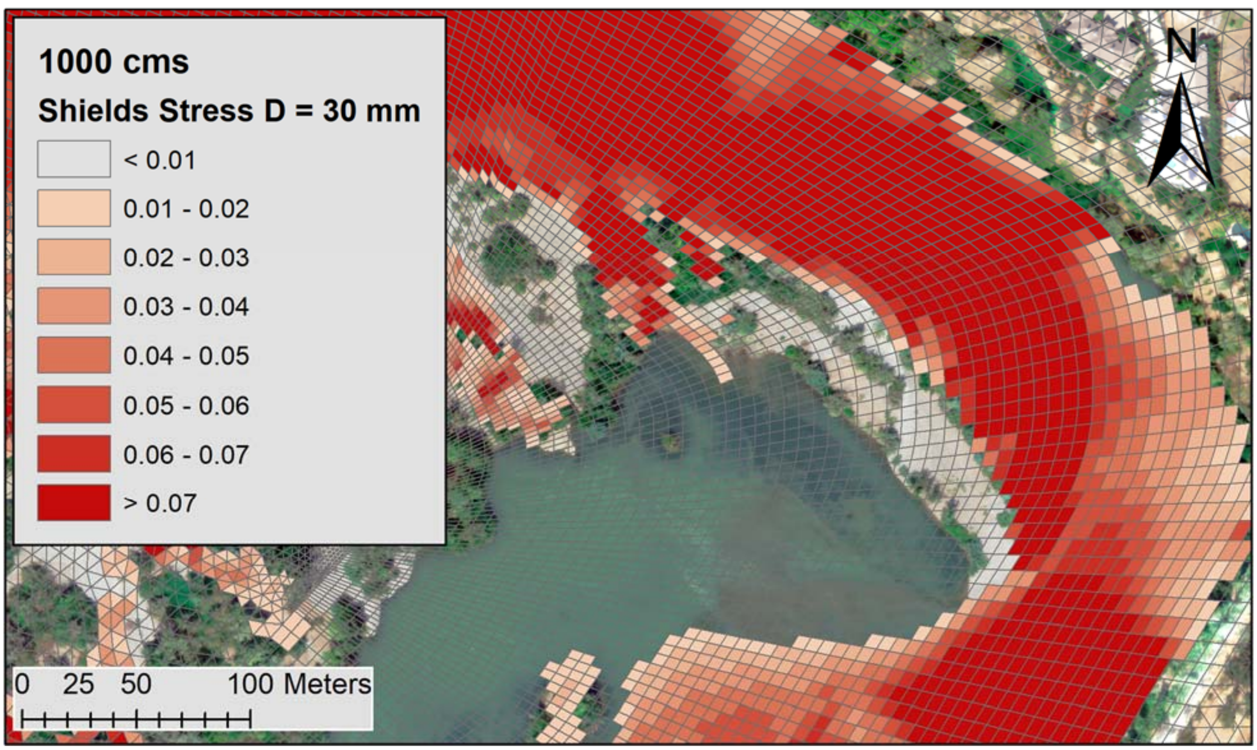


Figure 13. The Shields stress at Turtle Bay.

2.4.2.2 Sediment Transport Capacity

Sediment transport capacity is the amount of sediment the river is capable of transporting at a given flow if the river is not supply limited. There are a variety of transport capacity formulations, but they are generally based on excess shear stress, the amount of shear stress above a threshold. Unit transport capacity q_s is typically

$$q_s \sim (\tau - \tau_r)^m \text{ or } q_s \sim \left(\frac{\tau}{\tau_r}\right)^m$$

Where τ = basal shear stress, τ_r = reference or critical shear stress, and $m > 1$ for $\tau > \tau_r$.

We computed transport capacity using three different models, the Wilcock-Crowe equation [*Wilcock and Crowe*, 2003], the Gaeuman Trinity River modification of the Wilcock equation [*Gaeuman et al.*, 2009], and the Parker equation [*Bakke et al.*, 1999; *Parker*, 1990]. Each model was evaluated with a range of reference Shields stresses, shown in Table 3, using the sediment size distribution (Table 4) of augmented gravel in the Sacramento River as given the contract specifications. The assumed porosity of the gravel mixture was 0.35.

Table 3. Transport capacity equations evaluated

Transport Equation	Reference Shields Stress (θ_r)	Citation
Wilcock and Crowe	0.02, 0.03, 0.04	[*Wilcock and Crowe*,2003]
Gaeuman Trinity Model	0.02 0.03, 0.04	[*Gaeuman et al.*, 2009]
Parker	0.03, 0.04, 0.05	[*Bakke et al.*, 1999; *Parker*, 1990]

Table 4. The sediment size distribution used to compute sediment transport capacity and in the Market Street mobile bed model, discussed below

Size Class Lower Bound (mm)	Size Class Upper Bound (mm)	Fraction In Size Class (-)
0.1	6.35	0.05
6.35	12.7	0.15
12.7	19.05	0.15
19.05	25.4	0.15
25.4	50.8	0.35
50.8	127	0.15

For each model cell, we computed transport capacity vectors for each sediment size class ($\overrightarrow{q_{s_i}}$) and the total capacity vector ($\overrightarrow{q_s}$, the sum of the size class capacity vectors). An example of the cell-based total sediment transport capacity (q_s, the magnitude of $\overrightarrow{q_s}$) is shown in Figure 14.

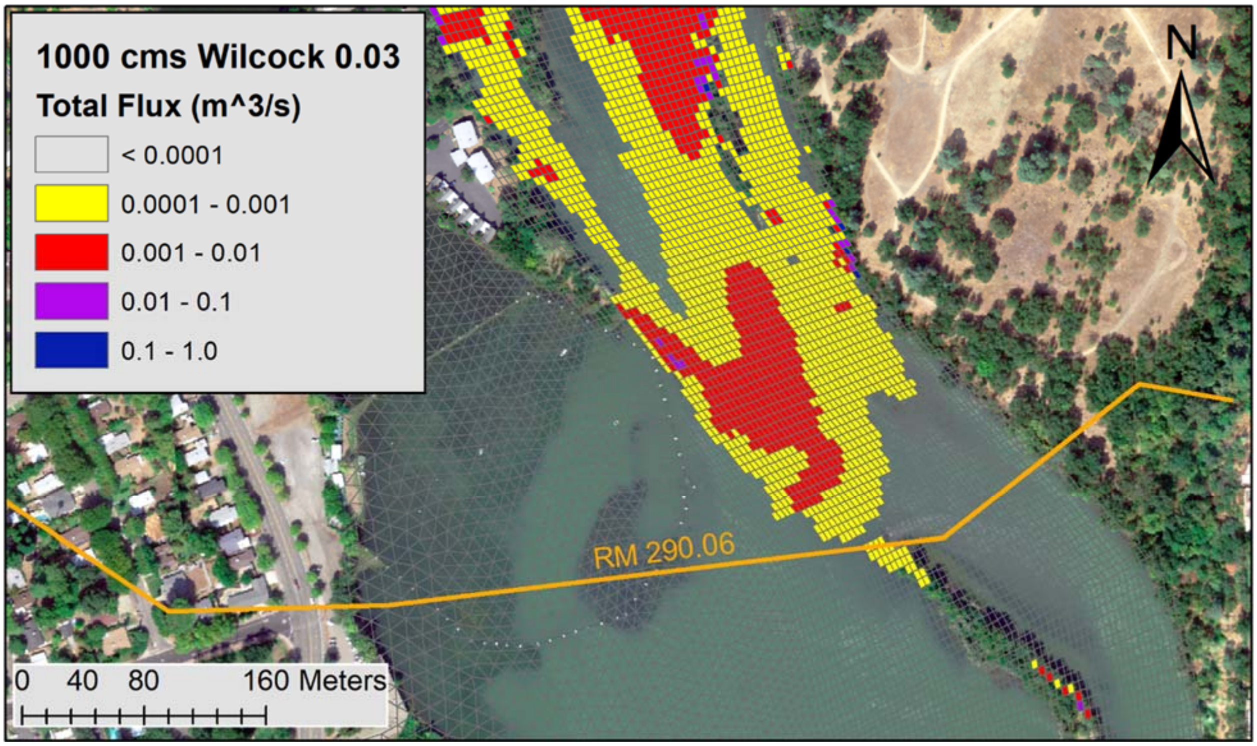


Figure 14. Transport capacity example at 1000 m^3/s downstream of Park Marina. Note the dramatic decline in capacity as the river widens. The yellow line is an Army Corps of Engineer's HEC-RAS model cross section.

These cell-based capacity vectors are similar to a specific transport capacity, except that they are integrated across the cell width and therefore have units of volumetric flux (L^3/T, e.g. m^3/s). To yield the total transport capacity across the river (Q_s), the total capacity vectors of cells intersecting 45 USACE HEC-RAS cross sections were integrated normal to the cross sections. The cross sections are shown in Figure 15. Where a cross section consisted of more than one segment, flux was computed normal to each segment. An example of the cross section normal vector is shown in Figure 16.

The total transport capacity Q_{s_n} across cross section n is

Equation 4

$$Q_{s_n} = \sum_{j=1}^{N_{seg}} \sum_{k=1}^{N_{cells}} \overrightarrow{q_{s_k}} \cdot \overrightarrow{n_j}$$

Where $\overrightarrow{q_{s_k}}$ is the total capacity vector for cell k, $\overrightarrow{n_j}$ is the normal vector to the j^{th} segment of cross section n, the inner sum is over the N_{cells} intersected by the j^{th} segment, and the outer sum is over the number of segments N_{seg} in cross section n. It is worth repeating here that $\overrightarrow{q_{s_k}}$ is integrated across the cell width (defined as the distance across the cell in a direction normal to the flow vector) and therefore has units of L^3/T.

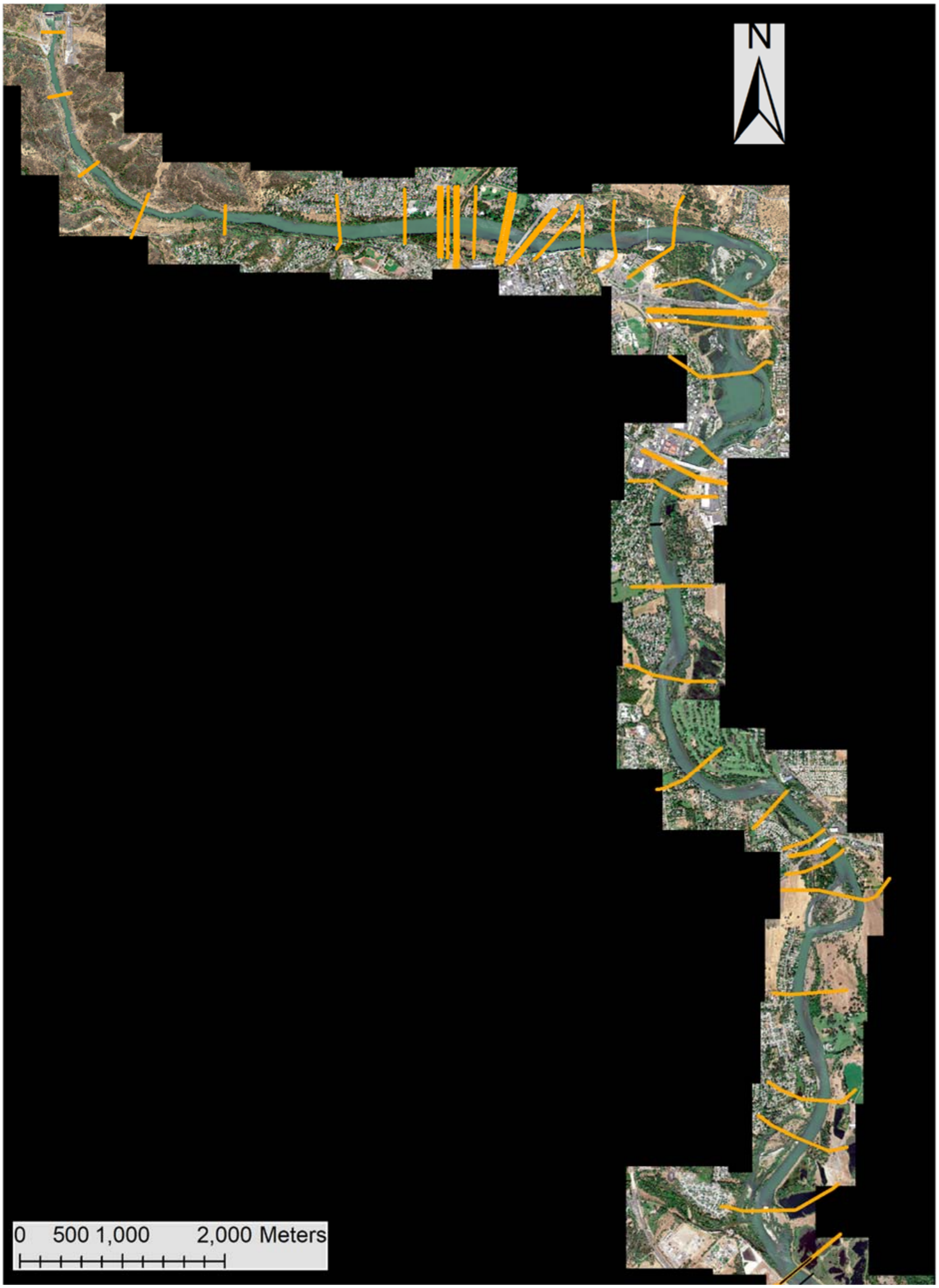


Figure 15. The USACE cross sections used to integrate sediment transport capacity.

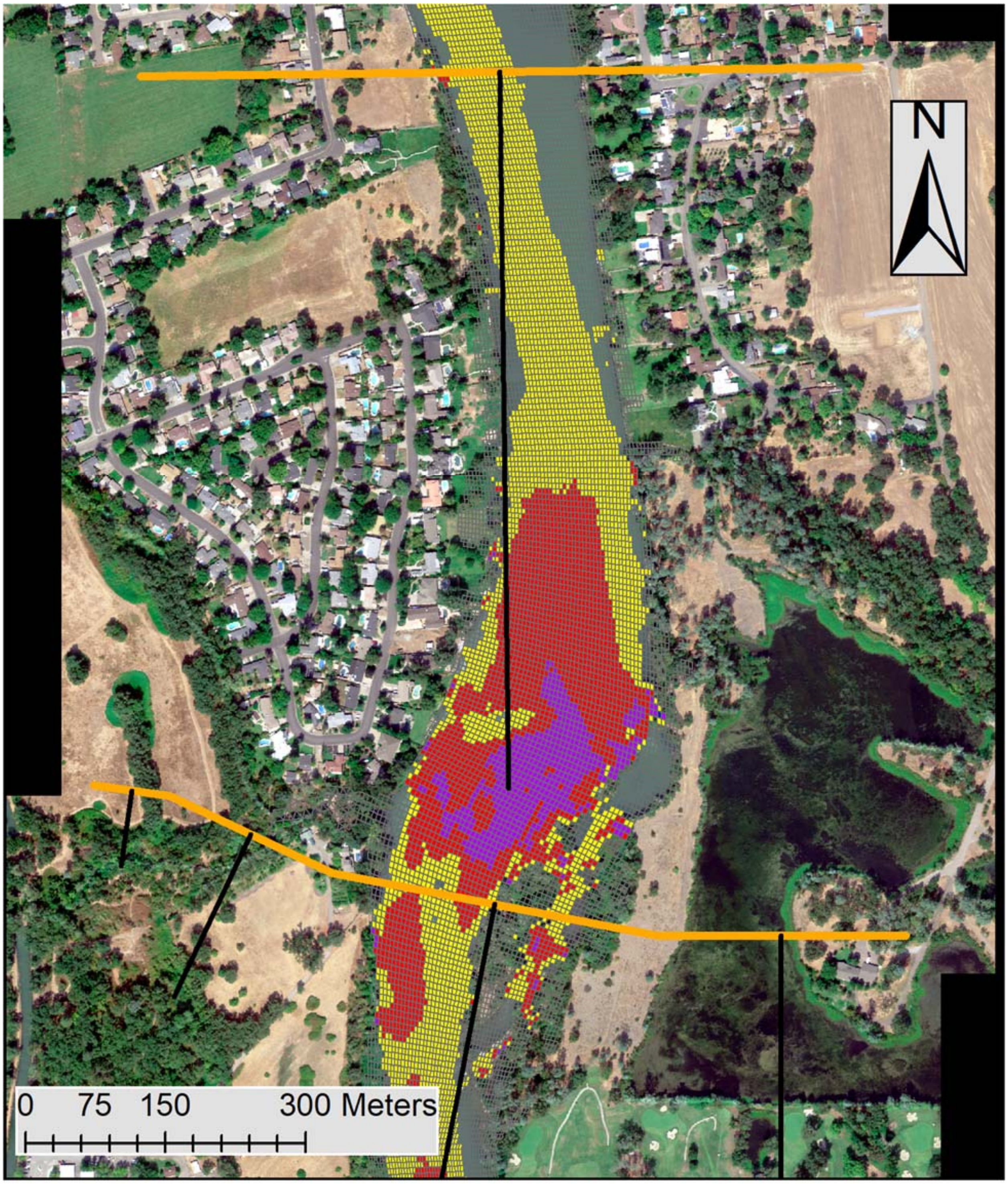


Figure 16. An example of the cross section normal vectors used in computing total transport capacity. The northern cross section is a single segment with a single normal vector. The southern cross section has four segments and four normal vectors.

2.4.2.3 Annual Sediment Load

We estimated the average annual sediment load (if sediment supply were not an issue) from the cross sectional transport capacity estimates and an average annual hydrograph derived from 10 years of data from the Keswick gage. The annual sediment load $Q_{s_{annual}}$ in m^3 is,

Equation 5

$$Q_{s_{annual}} = Q_s(Q_w) \times t(Q_w)$$

where $Q_s(Q_w)$ is the sediment rating curve in m^3/s, the sediment flux as a function of discharge derived from the hydraulic model and transport capacity estimates, and $t(Q_w)$ is a flow duration curve, the total amount of time in seconds during an average year that the river has a particular discharge.

To develop the flow duration curve, we used the Keswick gage record from Oct 1, 2009 to Sept 30, 2019, shown in Figure 17. The gage record was resampled to the middle of each 15-minute interval to better capture rapidly changing flow and to fill any gaps in the record. The hydrograph was sorted into 1 m^3/s bins and the time in each bin summed to give the number of seconds at each flow over the 10-year record. Dividing this total by 10 years yielded the average annual flow duration curve shown in Figure 18.

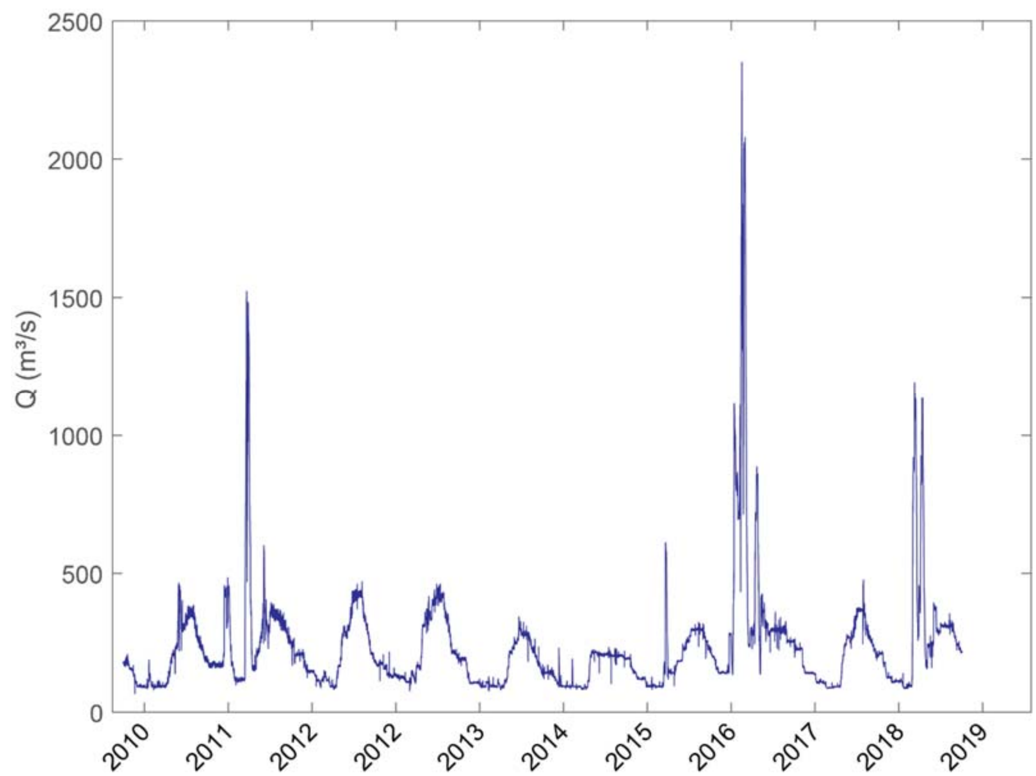


Figure 17. The Keswick gage record (USGS 11370500) from Oct. 2009 through Sept. 2019.

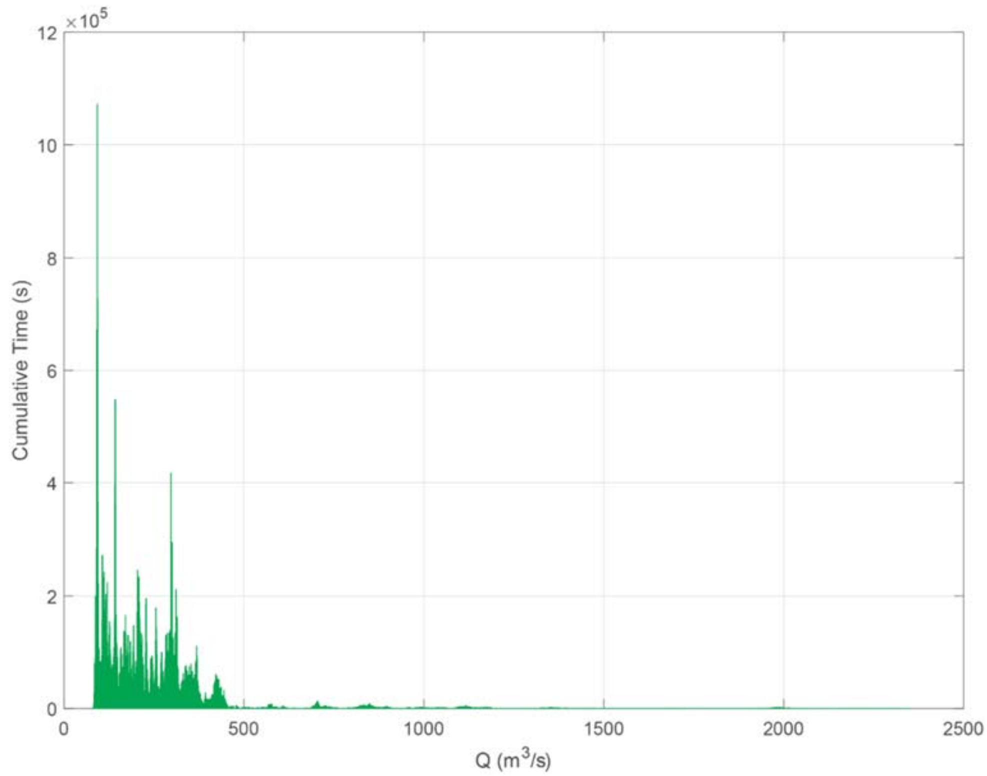


Figure 18. The average flow duration curve derived from the Keswick gage record.

2.5 Market Street Mobile Bed Model

We developed a combined hydraulic and sediment transport model (a mobile bed model) to simulate the 2016 gravel augmentation at the Market Street site. This model was used to predict erosion from the Market St. augmentation pile and deposition downstream at a range of flows and in response to synthetic hydrograph derived from the 2016-2017 flow record following gravel augmentation.

2.5.1 Model Setup

The mobile bed model domain extends from about 1800 m upstream of the Anderson-Cottonwood Irrigation District (ACID) diversion dam to about 700 m downstream of the Cypress Avenue bridge (Figure 19). The mesh for the mobile bed model was derived from the hydraulic model mesh and has 42,210 elements. The mesh elements are larger than those in the hydraulic model, with a modal size of about 13 m long by 6 m wide (Figure 20). This mesh coarsening was necessary to reduce the number of mesh elements and computation time and to improve model stability.

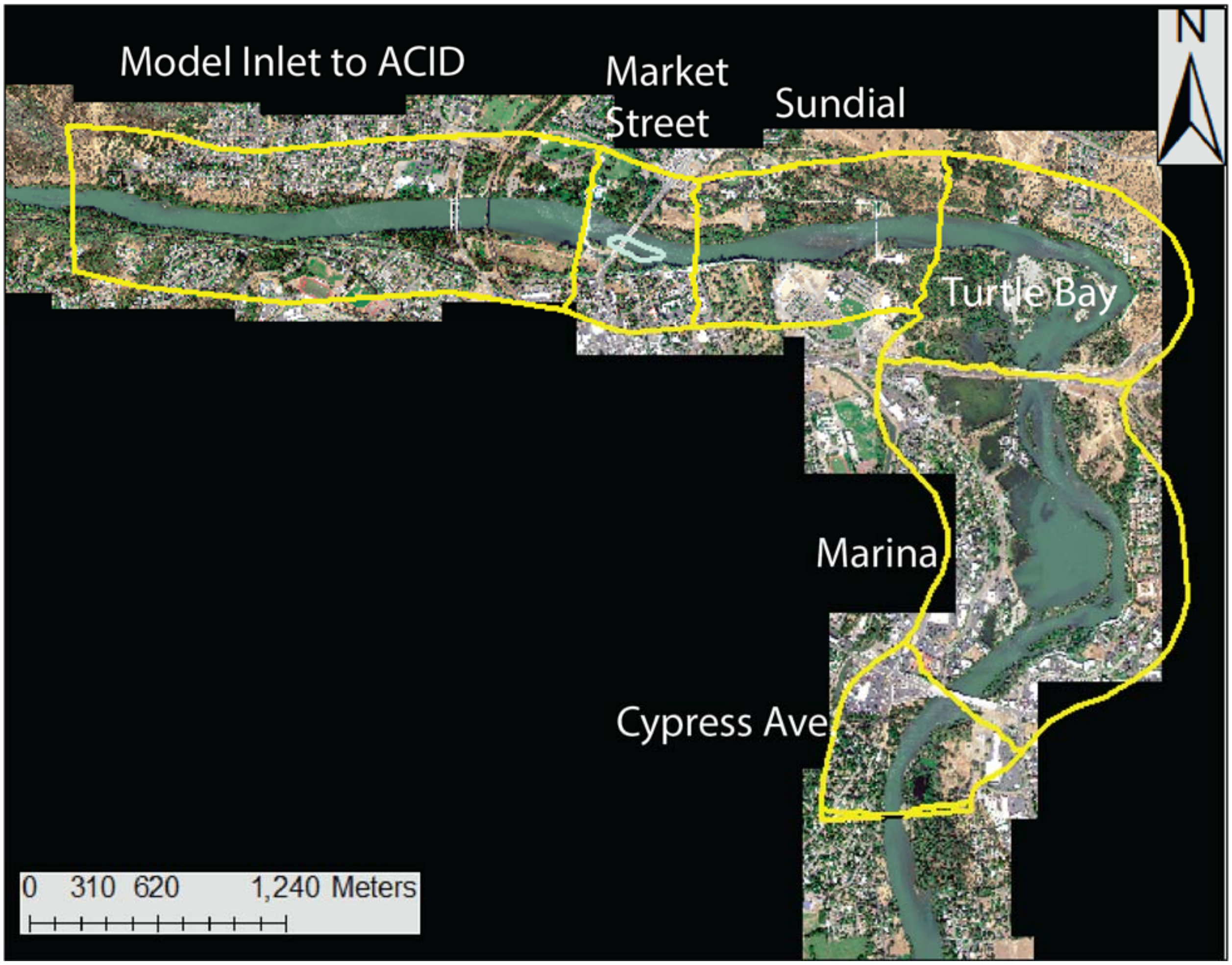


Figure 19. The Market Street mobile bed model domain and the reach polygons used to evaluate conservation of sediment mass and bed erosion or deposition (see below). The model inlet is approximately 1.8 km upstream of ACID. The outlet is approximately 700 m downstream of the Cypress Avenue bridge. The Market Street gravel augmentation is outlined in light blue.

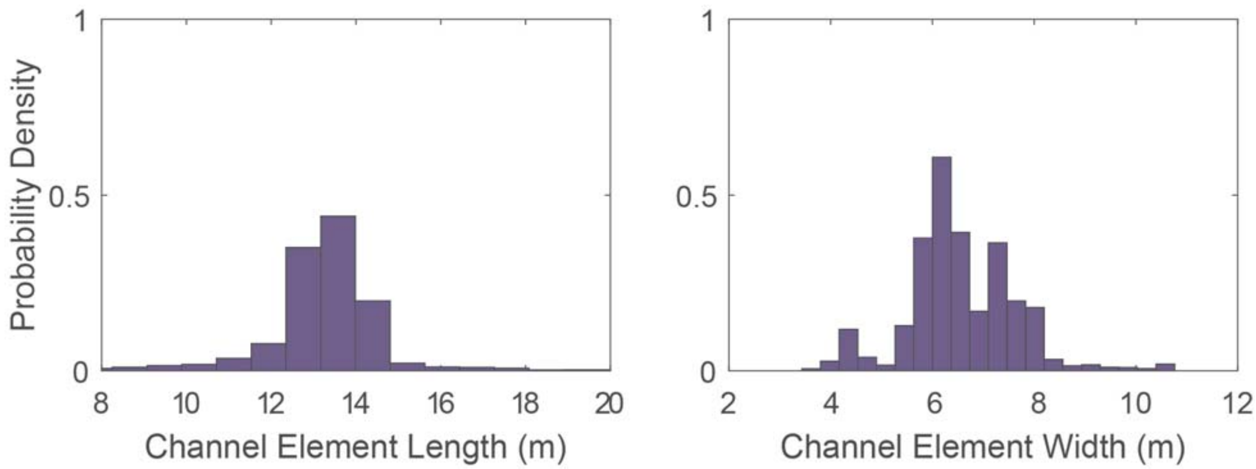


Figure 20. Histograms of channel element streamwise length (left) and cross stream width (right). The modal size of channel elements is about 13 m x 6 m.

Figure 21. The Market Street gravel augmentation footprint from https://www.sacramentoriver.org/channels/index.php?id=map&loadmap=1&link=PkWQi0ki4ahIIXPSYaMA. Metadata associated with the project indicates a project footprint of 3.7 acres.

To simulate the Market Street gravel augmentation, we treated the river bed outside the project area (Figure 21) as armored and immobile. The Market Street augmentation pile was represented by increasing the river bed elevation by the appropriate amount and allowing it to be mobile with the size gradation and bulk density determined by the specifications of the gravel augmentation (Table 4).

The Market Street project added a mass M of 12,000 tons of gravel to the river. This mass was converted to bulk volume V_b and, ultimately, a pile height h using

$$V_b = \frac{MK}{\rho(1-\phi)} = 6320\ m^3$$

$$h = V/A = 0.42\ m$$

where $K = 907.2\ \ kg/ton$, the rock density $\rho = 2650\ \ kg/m^3$, the gravel pile porosity $\phi = 0.35$ and the site area $A = 15{,}006\ m^2$. The initial topography of the Market St. model (vertically exaggerated 10x to make the pile visible) is shown in Figure 22.

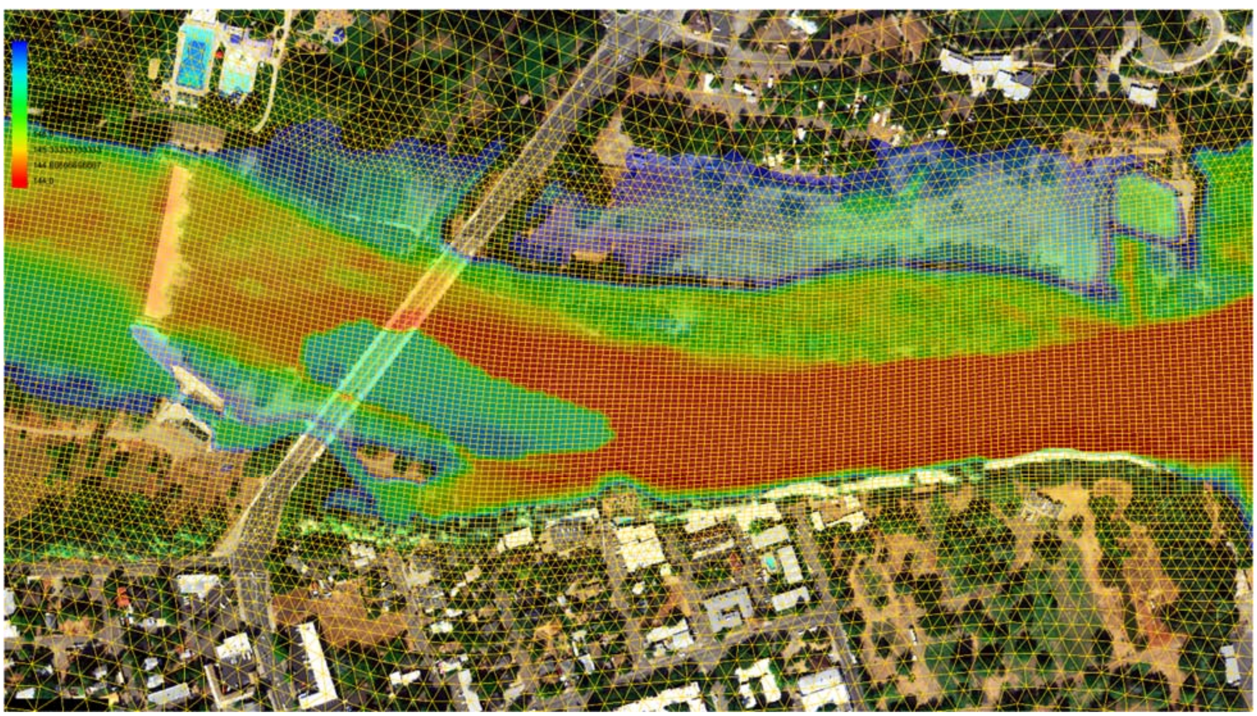


Figure 22. The model topography adjusted upward by 0.42 m to mimic the 3.7 acre site footprint. The polygon in Figure 21 was enlarged to reach 3.7 acres. The elevation of the augmentation pile is exaggerated 10x in this image to enhance visibility.

We ran the mobile bed model with 10 steady flows (Table 5) for 400 hours each and evaluated the amount of erosion and deposition in each reach, how the volume of sediment changed over the course of a model run, and how erosion or deposition varied with model discharge. We also evaluated the extent to which the model conserved sediment mass. Those analysis methods are described below.

Table 5. Market Street mobile bed model boundary conditions for steady discharge runs–outlet boundary conditions were extracted or extrapolated from the hydraulic model

Q (m^3/s)	**Outlet Boundary Condition (m)**
100	138.3
250	139.3
350	139.7
500	140.1
750	140.7
1000	141.1
1250	141.5
1500	141.9
1750	142.3
2000	142.7

We also ran the mobile bed model with a synthetic hydrograph based on the 19 months following the 2016 Market Street gravel augmentation (March 15th, 2016 to Sept. 30th, 2017). The gage record was resampled to a daily average and periods where flow was less than 350 m^3/s were removed from the synthetic hydrograph to reduce model execution time. A runup time of 1 day from a 250 m^3/s static initial condition was added to the beginning of the hydrograph. The real Keswick gage hydrograph and the synthetic hydrograph used in the model are plotted in Figure 23. The synthetic hydrograph is 2496 hours long. Model output was written every 12 hours.

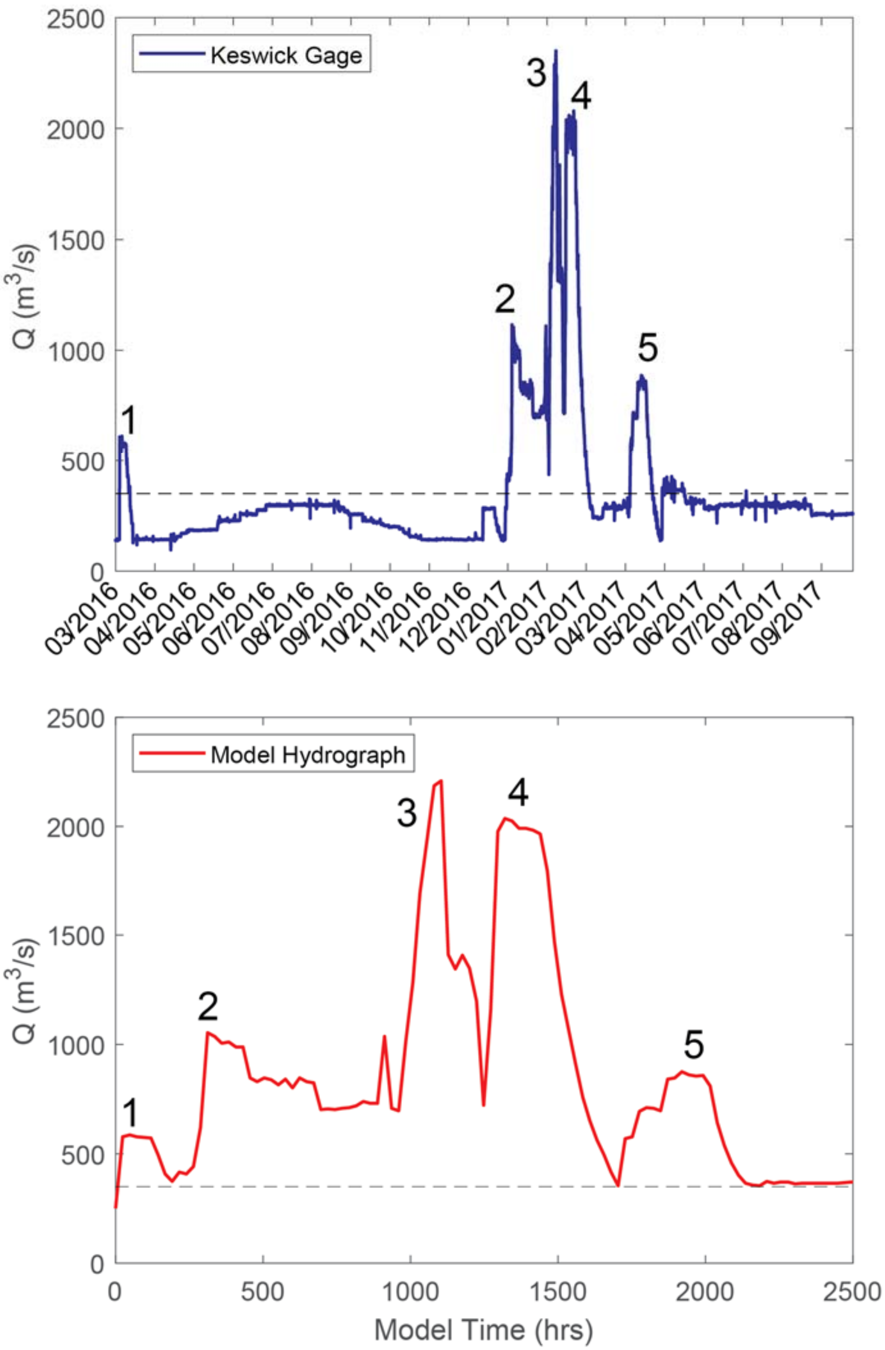


Figure 23. The Keswick gage record following the 2016 Market Street gravel augmentation (top) and the synthetic hydrograph derived from the gage record (bottom). Corresponding peaks in the two hydrographs are numbered.

2.5.2 Model Analysis

To compute the amount of sediment eroded from or deposited in the polygons shown in Figure 19, we integrated the bed elevation change and amount of sediment in transport reported by SRH-2D over each of the polygons.

The material volume of sediment ΔV_i eroded from or deposited in a model cell as a function of bed elevation change Δh_i is

Equation 6

$$\Delta V_i = \Delta h_i A_i (1 - \phi)$$

where A_i is the area of the cell and ϕ is the gravel porosity. The total material volume change over polygon p is

Equation 7

$$\Delta V_p = \sum_{i=1}^{N_p} \Delta V_i$$

where N_p is the number of model cells in the polygon.

A full accounting of the model sediment budget must also include the sediment in a cell that is in transport (as opposed to contributing to bed elevation change). SRH-2D reports the amount of sediment in transport in each cell in parts per million, C_{ppm}. We converted this to a volumetric concentration $C_{i_{volumetric}}$ using

Equation 8

$$C_{i_{volumetric}} = \frac{C_{i_{ppm}}}{10^6 \gamma + C_{i_{ppm}}(1 - \gamma)}$$

where γ is specific gravity (2.65). To convert this to a volume of suspended sediment, $V_{i_{susp}}$, we multiplied by the volume of water in the cell $h_i A_i$.

Equation 9

$$V_{i_{susp}} = h_i A_i C_{i_{volumetric}}$$

The total sediment in transport in polygon p is

Equation 10

$$V_{p_{susp}} = \sum_{i=1}^{N_p} V_{i_{susp}}$$

This volume is typically small relative to the volume of sediment exchanged with the model stream bed, but becomes important when integrated over a large area.

The sediment mass balance in polygon p is a version of the Exner equation [*Paola and Voller*, 2005]

Equation 11

$$V_{ml_p} - V_{ml_{p-1}} = \Delta V_p + V_{p_{susp}}$$

where V_{ml_p} is the total volume of sediment SRH-2D reported crossing the monitoring line at the downstream end of polygon p, Vlmp-1 is the total volume of sediment SRH-2D reported crossing the monitoring line at the upstream end of polygon p, and the right hand side of the equation is the net amount of sediment in a polygon. Equation 11 was used to check that the model conserved sediment mass.

3. Results

3.1 Hydraulic Model Calibration

Ultimately, model calibration at a flow of 210 m^3/s required only two different values of Manning's n, a relatively high value of 0.05 in the bedrock reach immediately downstream of Keswick, and 0.035 from about 1.3 km upstream of ACID to the model outlet near Clear Creek. The roughness polygons and the water surface elevation residuals (Observed WSE – Modeled WSE) are shown in Figure 24. About 84% of the residuals are in an interval of +/- 10 cm. Approximately 16% of the residuals exceed 10 cm, indicating that the model water surface is slightly low in some places. The median residual is about -4 mm and the distribution is roughly symmetric around the median.

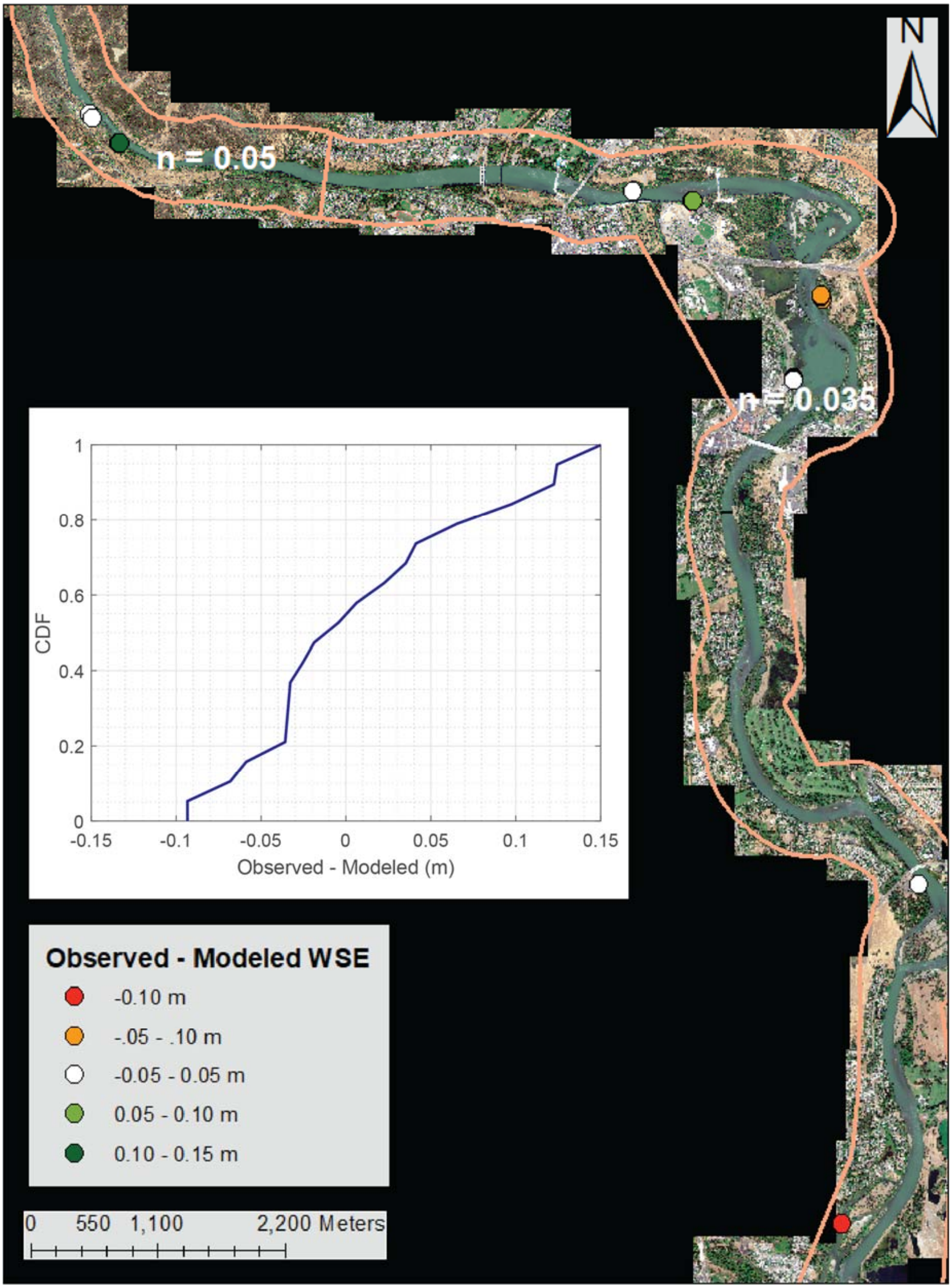


Figure 24. The water surface elevation residuals (observed WSE – modeled WSE) after model calibration and the probability distribution of residuals.

3.2 Hydraulic Model Analysis

3.2.1 Habitat Suitability

The total wetted area, the amount of Steelhead and Chinook spawning habitat, and the amount of rearing habitat in each reach is presented in Figure 25 and Figure 26. Habitat is quantified as Weighted Useable Area (WUA) as described by Equation 2 and plotted for the range of modeled flows. The amount of habitat is between about 20,000 m^2 and 100,000 m^2 in all reaches and the habitat curves are relatively flat, indicating the amount of habitat does not change dramatically with increasing flow. For comparison, the amount of wetted area is between 400,000 m^2 and 2,000,000 m^2. Roughly speaking, less than 10% of the river is suitable habitat at any flow and suitable habitat is primarily located along the channel margins. The values plotted in Figure 25 and Figure 26 are summarized in Table 6.
Figure 27 and Figure 28 show Chinook spawning and generic rearing HSI near the downstream end of the Cypress Avenue to Bonnyview reach.

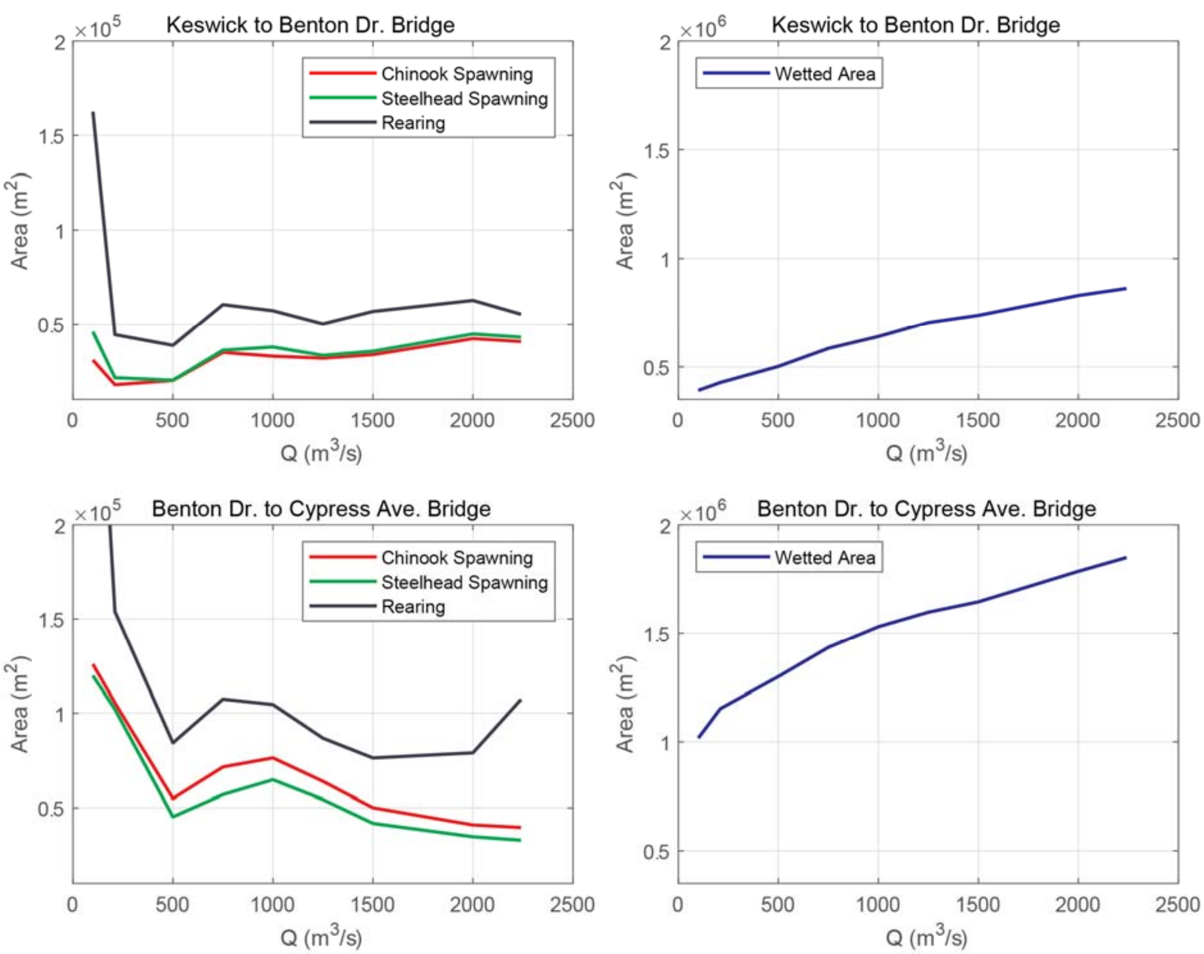


Figure 25. Habitat (WUA, left) and wetted area (right) as a function of flow for Keswick to Benton Dr. (top) and Benton Dr. to Cypress Ave. (bottom). Note that the y-axis scales are different for habitat and wetted area.

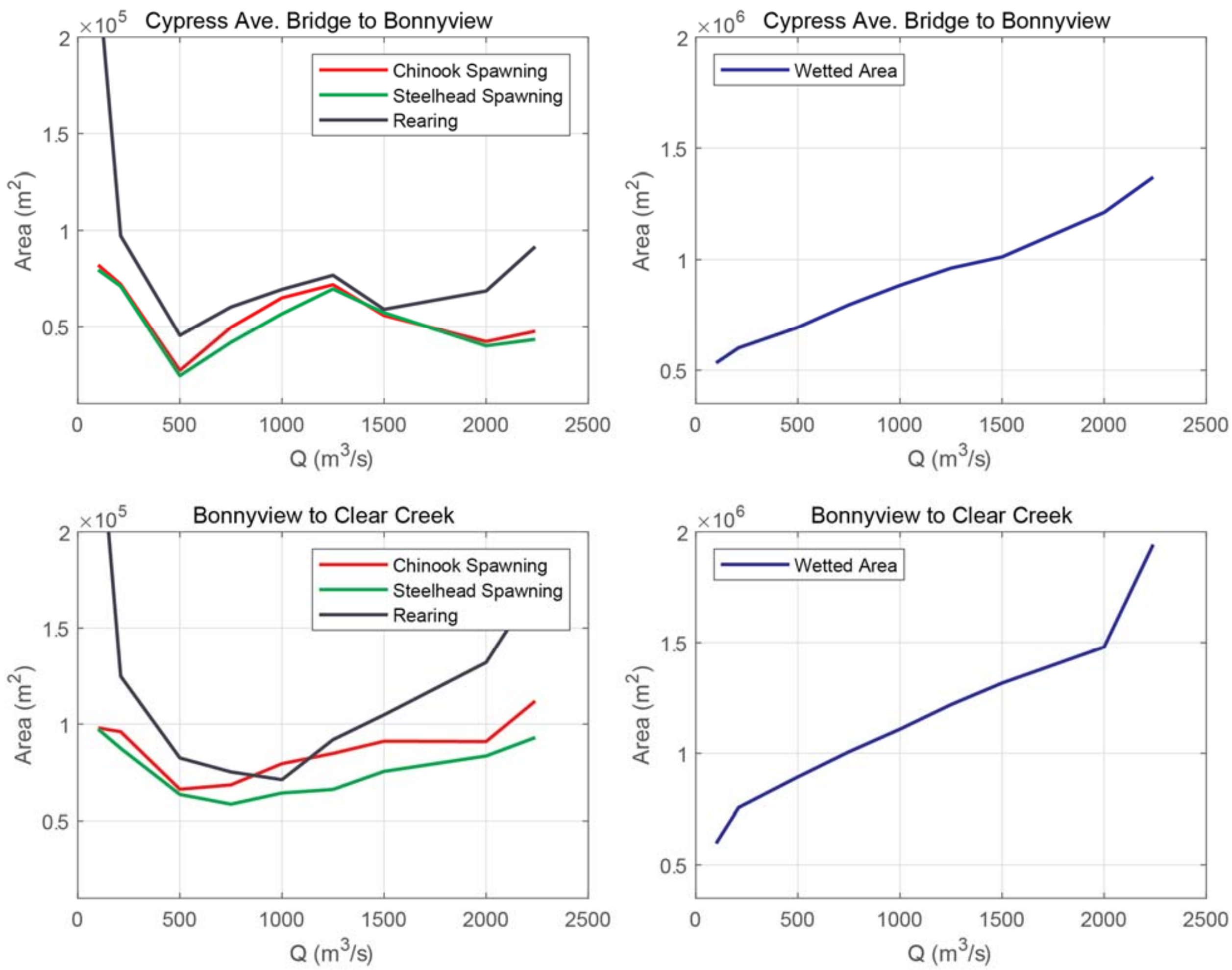


Figure 26. Habitat (WUA, left) and wetted area (right) as a function of flow for Cypress Avenue to Bonnyview (top) and Bonnyview to Clear Creek (bottom). Note that the y-axis scales are different for habitat and wetted area.

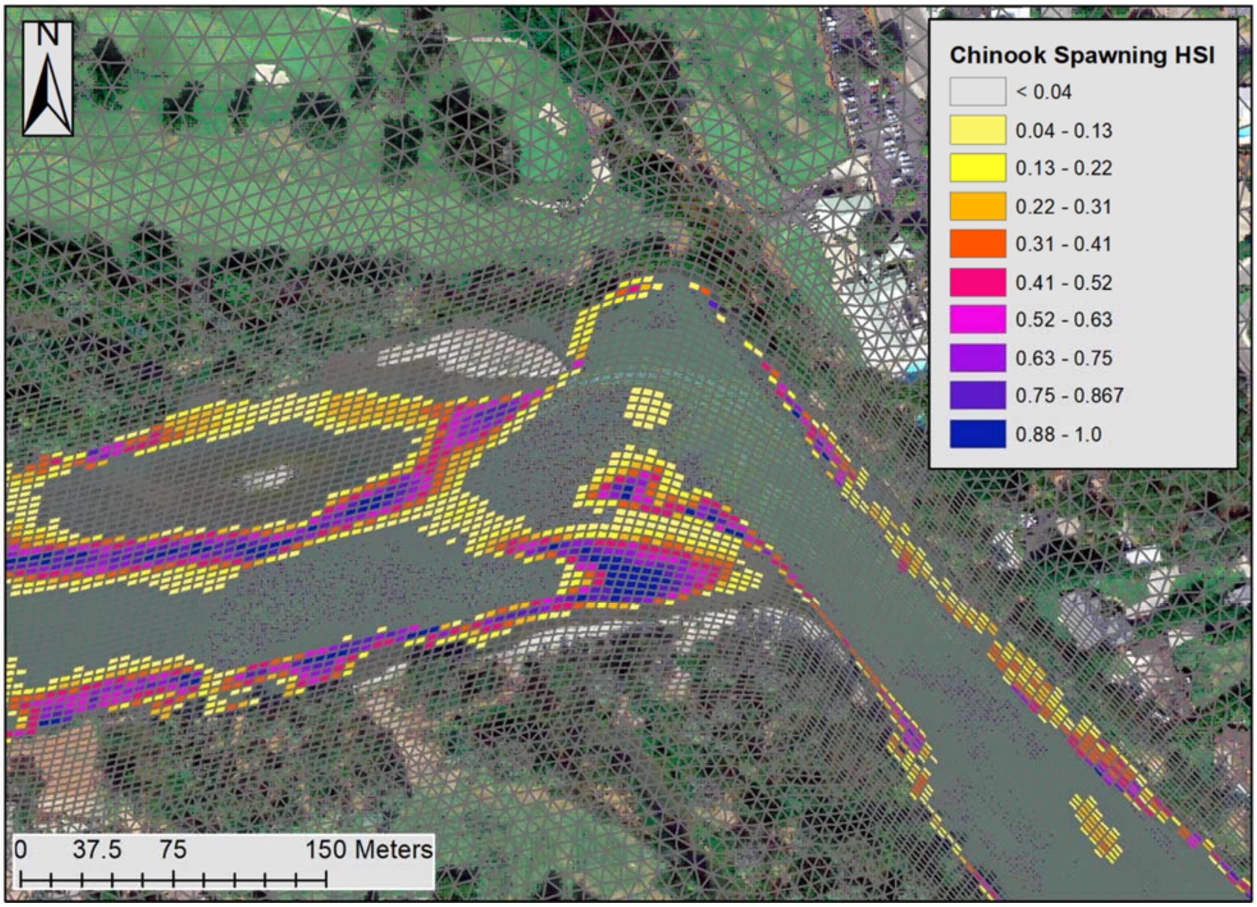


Figure 27. Chinook spawning HSI at a flow 100 m^3/s at the downstream end of the Cypress Avenue to Bonnyview reach.

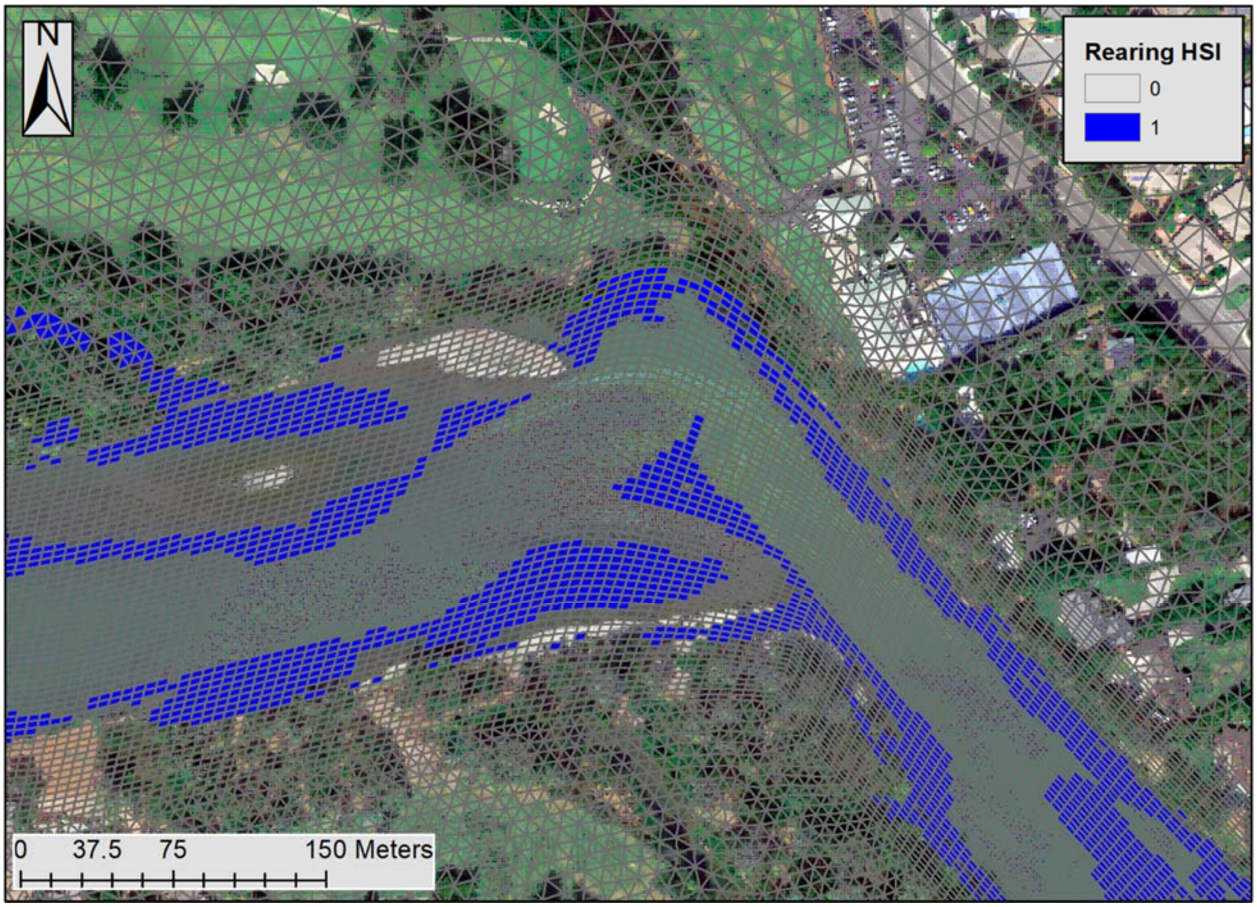


Figure 28. Rearing HSI at a flow 100 m^3/s at the downstream end of the Cypress Avenue to Bonnyview reach.

Table 6. The habitat data plotted in Figure 25 and Figure 26

Q (m³/s)	Keswick to Benton Dr. Bridge (m²)					Benton Dr. Bridge to Cypress Ave. Bridge (m²)			
	Wetted Area	Chinook Spawning	Steelhead Spawning	Rearing		Wetted Area	Chinook Spawning	Steelhead Spawning	Rearing
100	391,533	30,857	45,904	162,516		1,017,726	125,892	119,801	354,695
170	415,962	18,826	24,390	111,389		1,116,599	116,850	111,862	319,114
210	427,662	17,785	21,482	44,401		1,152,644	105,461	102,116	153,961
500	502,112	20,033	20,188	38,708		1,299,768	54,945	45,307	84,334
750	585,266	34,961	36,110	60,451		1,434,099	71,608	57,007	107,436
1,000	640,262	32,943	37,833	57,199		1,530,263	76,409	64,854	104,604
1,250	705,184	31,891	33,354	50,235		1,597,224	64,002	54,426	86,817
1,500	738,483	33,763	35,572	56,845		1,645,145	50,101	41,812	76,353
2,000	830,334	42,272	44,709	62,685		1,785,868	41,010	34,759	79,071
2,240	862,699	40,741	43,043	55,371		1,849,261	39,681	32,931	107,310

Q (m³/s)	Cypress Ave. Bridge to Bonnyview (m²)					Bonnyview to Clear Creek (m²)			
	Wetted Area	Chinook Spawning	Steelhead Spawning	Rearing		Wetted Area	Chinook Spawning	Steelhead Spawning	Rearing
100	533,647	82,094	79,540	230,723		596,868	98,000	97,351	263,989
170	585,171	75,791	73,276	196,259		712,665	93,726	85,357	235,754
210	601,714	72,062	71,093	97,242		756,439	96,066	87,450	124,887
500	694,027	27,223	24,463	45,483		893,720	66,197	63,518	82,433
750	796,064	49,624	41,831	60,216		1,006,814	68,463	58,462	75,190
1,000	884,273	65,005	56,652	69,416		1,108,757	79,467	64,258	71,169
1,250	962,095	71,860	69,640	76,783		1,219,596	84,790	66,102	91,923
1,500	1,012,470	55,829	57,257	59,038		1,316,795	91,125	75,458	104,816
2,000	1,213,077	42,246	40,046	68,606		1,479,910	90,945	83,477	132,075
2,240	1,371,791	47,584	43,340	91,572		1,942,235	111,917	92,994	171,391

3.2.2 Shields Stress

The fraction of wetted area that exceeds a reference Shields stress of $\theta_r = 0.04$ for of range of grain sizes and flows is plotted for each reach in Figure 29. This is an indication of how mobile an unarmored natural bed or augmented gravel pile is likely to be at a given flow. At flows of 500 m^3/s and above, the river is likely capable of mobilizing 20 mm gravel over more than 50% of the wetted area. The Benton Drive to Cypress Ave. reach, which includes Turtle Bay and the Marina is an exception, with just under 50% of the area capable of mobilizing 20 mm gravel. The fraction of wetted area capable of mobilizing 40 mm gravel is substantially less, but still more than 30% in most cases. Gravel 60 mm and larger is much less mobile, except at the highest flows and in the Keswick to Benton Dr. reach, which includes the deep and fast bedrock reach downstream of the dam.

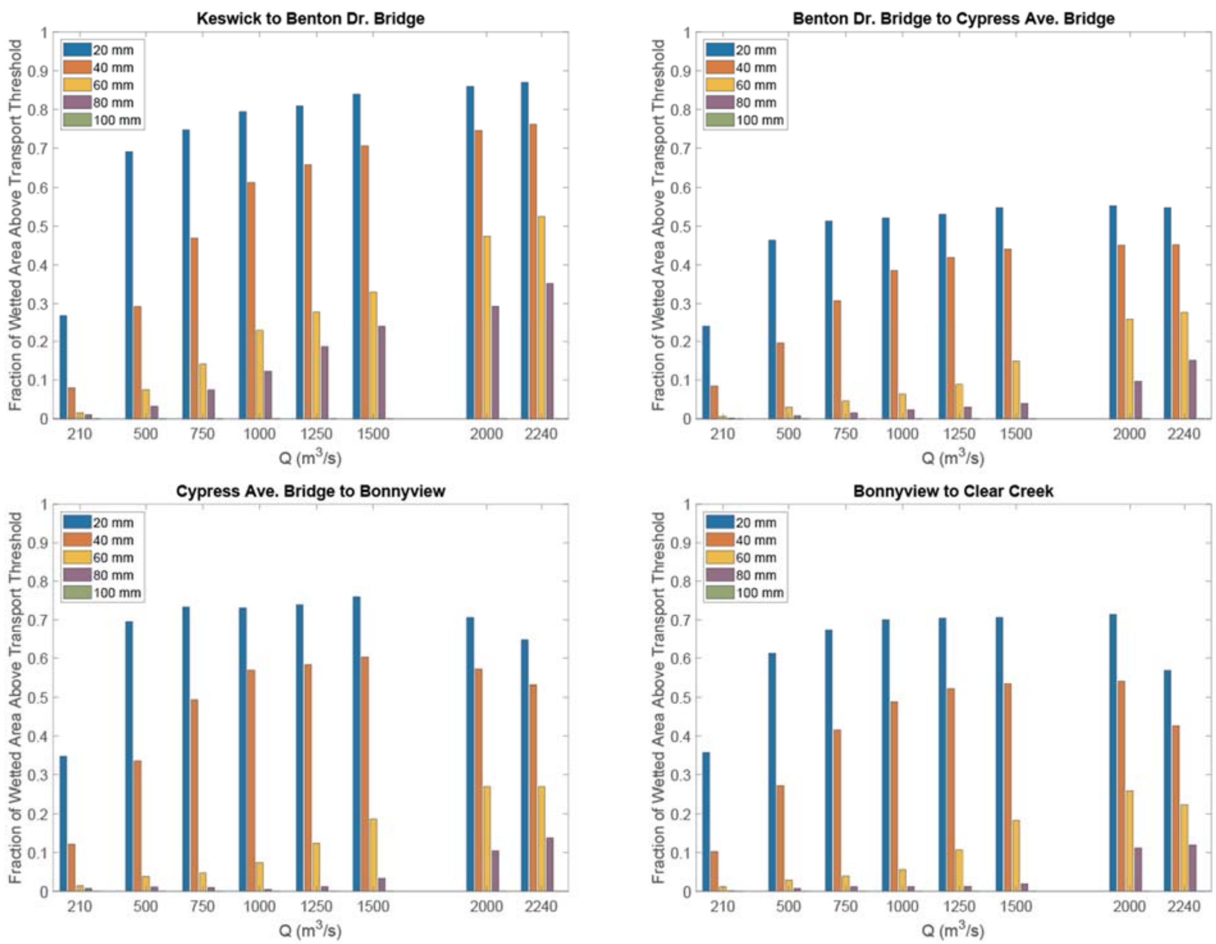


Figure 29. The fraction of each reach that exceeds a threshold Shields stress of $\theta_r = 0.04$ for a range of grain sizes and flows. Finer gravel is mobile over much of the river at flows of 500 m^3/s and above. The least mobile reach is Benton Dr. to Cypress Avenue which includes Turtle Bay and Park Marina.

3.2.3 Sediment Transport Capacity

Transport capacity rating curves for four selected cross sections are plotted in Figure 30. Only results from the middle of the reference Shields stress range listed in Table 3 are included. The Parker model predicts the highest sediment transport capacity. The Gaeuman and Wilcock models predict less sediment transport and are quite similar to one another, which is to be expected because the Gaeuman model is a modification of the Wilcock model developed based on observations of bedload transport in the Trinity River [*Gaeuman et al.*, 2009].

With the exception of the ACID cross section, the transport capacity predicted by the Gaeuman and Wilcock models does not increase much with increasing flows and is less than 1 m^3/s. At the ACID cross section, these models predict transport rates of 1-2 m^3/s at the highest flows. The Parker model predicts increasing transport capacity with increasing flow at 2 cross sections, reaching about 3 m^3/s at the Cypress Ave. Bridge and more than 6 m^3/s at ACID. Curiously, the Parker model predicts a peak transport capacity of about 3 m^3/s at Clear Creek at 500 m^3/s and around 1.5 m^3/s at flows above 750 m^3/s. Even more curiously all models predict transport capacity of about 0.1 m^3/s at the cross section downstream of Keswick Dam, even at the highest flows.

This seems counterintuitive because the river is narrow, deep, and fast in the bedrock reach downstream of the dam and may indicate shortcomings in the transport capacity models in deep water. There are a few limitations of sediment transport models that may be underestimating the sediment transport capacity in this reach. The first is that models require a separation of total roughness versus grain roughness. Typically, only the grain roughness is used to compute bed load transport and in the reach immediately downstream of Keswick, there are many bedrock outcrops that increase the total roughness, but the grain roughness will be the same in all the reaches. The bed rock outcrops essentially extract energy from the flow that then is not used to transport sediment. The exact process is difficult to parameterize and creates uncertainty in the results. Another uncertainty in the model is caused by the low width to depth ratios in this reach. The width in this section of river is narrow, typically 45 m wide, whereas further downstream the width is typically over 120 m. At high discharges, like 2000 cms, the depth can be near 15 m and the width to depth ratio of 4. The flow patterns in low width to depth ratios can become highly three-dimensional and the assumptions used to develop the sediment transport formulas begin to break down. Despite these uncertainties, the model is still predicting some transport in these reaches, and as shown in the following section, the transport rates are more than that being supplied from gravel augmentation.

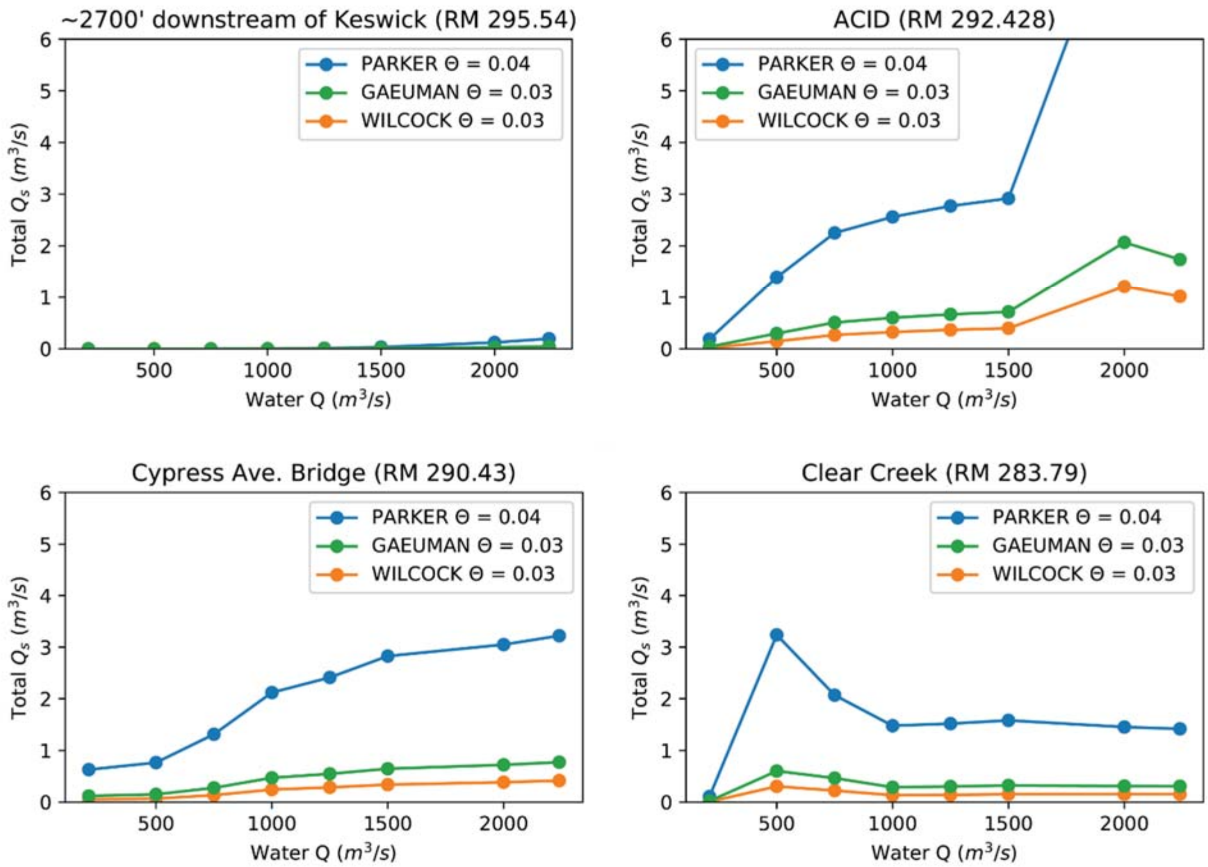


Figure 30. Sediment transport capacity rating curves at selected cross sections.

Figure 31 shows the transport capacity averaged over all the USACE cross sections in each reach. The patterns are similar to those shown in Figure 30. The Wilcock and Gaeuman models predict transport capacity below 1 m^3/s and increasing only slightly with increasing flow. The Parker model predicts higher rates of transport and a steeper, mostly monotonic increase with increasing flow. The odd peak at 500 m^3/s at the Clear Creek cross section disappears when transport capacity is averaged over the entire Bonnyview to Clear Creek reach. An exception is the Cypress Avenue to Bonnyview reach, where the Parker model predicts a transport rate at 210 m^3/s that is only exceeded by flows of 2000 m^3/s and higher.

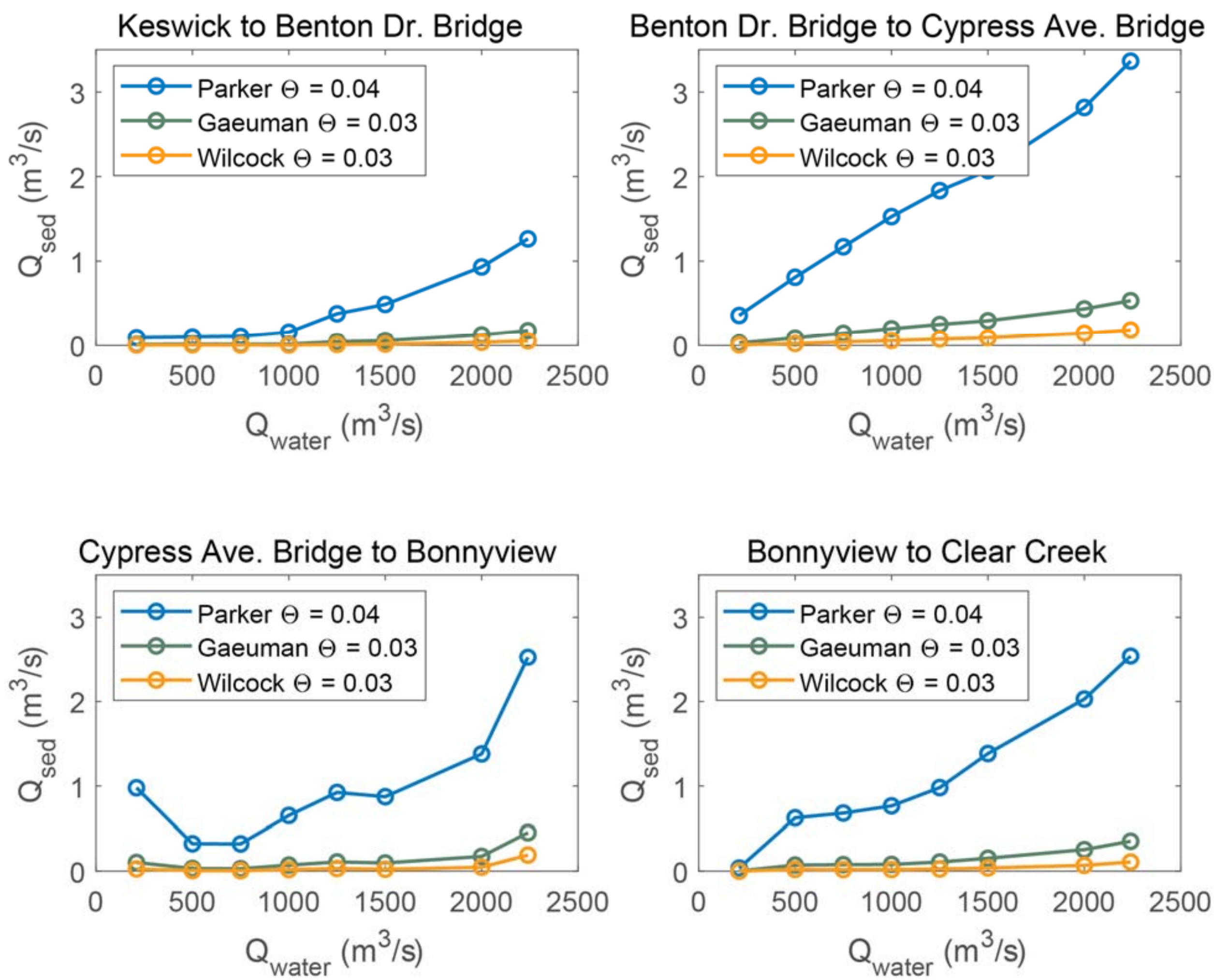


Figure 31. Reach averaged sediment transport capacity rating curves. The capacity at each flow is the average over all the cross sections in the reach.

3.2.4 Total Average Annual Load

Cross section integrated transport rates of less than 1 to a few m^3/s might seem low, especially considering that river is more than 100 m wide in most places. However, when integrated over the course of an annual average hydrograph, the total average annual load is quite large. Figure 32 shows the annual load computed as described by Equation 5 for each of the USACE HEC-RAS cross sections. There is a lot of scatter among the cross sections and the Parker model differs by as much as an order of magnitude from the Wilcock model, but downstream of Market Street, the model predictions appear to vary around an annual load of about 10^5 m^3. The distributions of annual load for each model shown in Figure 33 and the summary statistics in Table 7 confirm that the Wilcock and Gaeuman models predict a median value of annual sediment load on the order of $5 \times 10^4 – 10^5$ m^3. We recommend that the median be used to represent the transport capacity of the reach because it is not affected by extremely high transport rates at the structures such as bridges. The Parker models predicts a median value of about 3.7×10^5 m^3. Modal values are within a similar range. For comparison, the lowest end of this range is approximately eight times the ~6300 m^3 of gravel added to the river by the 2016 Market St. gravel augmentation. A very conservative transport capacity estimate of 10^4 m^3/year is still 1.6 times the volume of the Market Street augmentation. To more accurately quantify the gravel transport rates, some comparison with observed bed load transport rates would be necessary.

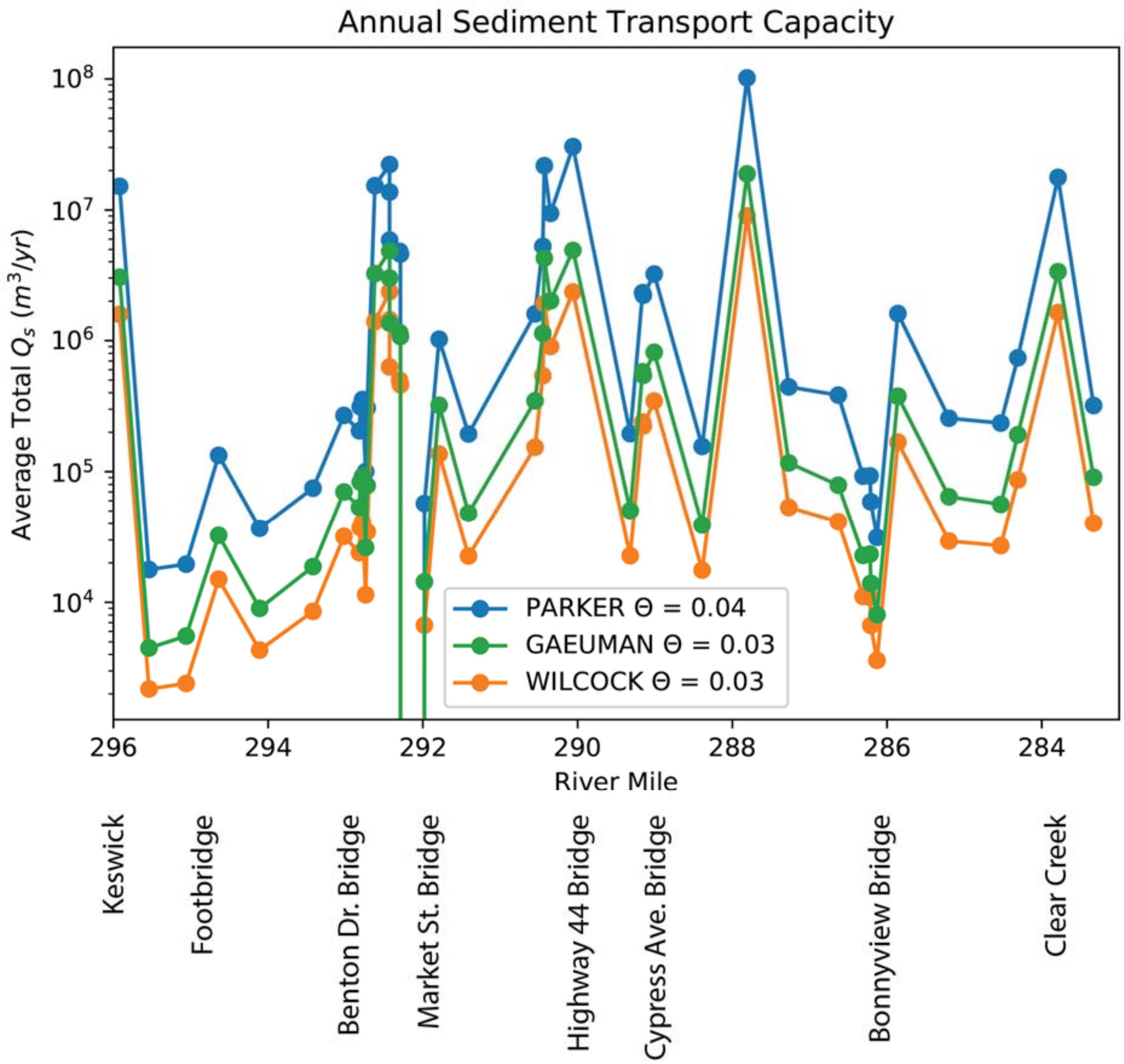


Figure 32. Average annual sediment transport capacity by river mile. 10^5 m^3 is approximately 190,000 tons.

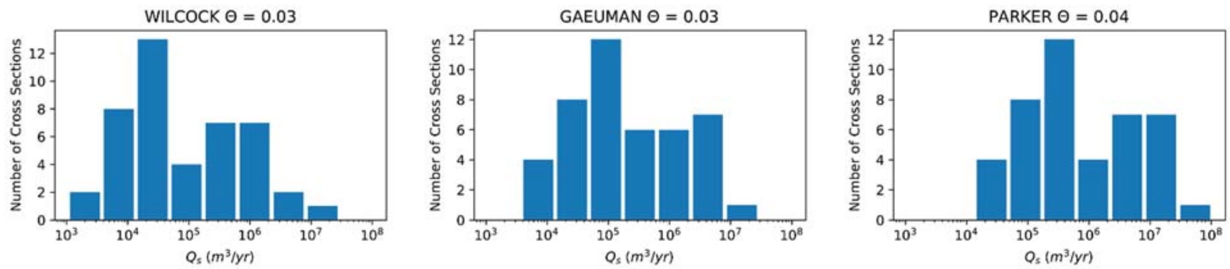


Figure 33. The distributions of annual transport capacity.

Table 7. Statistics of the transport capacity distributions shown in Figure 33

Model	**Mean $Q_{s_{annual}}$ (m^3/yr)**	**Median $Q_{s_{annual}}$ (m^3/yr)**	**Std. Dev. $Q_{s_{annual}}$ (m^3/yr)**
Parker $\Theta_r = 0.04$	6,488,887	369,904	16,251,353
Gaeuman $\Theta_r = 0.03$	1,285,655	91,4788	3,018,681
Wilcock $\Theta_r = 0.03$	605,225	41,319	1,444,966

3.3 Market Street Mobile Bed Model

We decided to use the Wilcock transport formula in the Market Street mobile bed model based on the results of the transport capacity calculations (Figure 31). The Wilcock model predicts the lowest transport rates, so this represents a conservative choice, meaning that the other transport formulas would predict the sediment to be transported faster and further in the river channel.

3.3.1 Steady Flows

We ran the mobile bed model at 10 steady flows ranging from 100 – 2000 m^3/s (Table 5) for 400 hours and analyzed how sediment volumes in each reach changed according to Equation 7 through Equation 11. The bed elevation change due to erosion and deposition at 1000 m^3/s is shown in Figure 34. Qualitatively, the sediment eroded from the Market Street gravel augmentation pile is mostly deposited in Turtle Bay and the Marina reach, with some localized deposition along the right bank immediately downstream of the augmentation pile. This is quantified in Figure 35 and Figure 36, which show the total sediment volume change in each reach as a function of flow. Figure 38 summarizes those results, showing what fraction of the sediment eroded from Market Street is in each reach at the end of the simulation. The sediment totals in Figure 38 include sediment still in motion at the end of the model run (Equation 10).

An important factor in interpreting these results is that our initial conditions surface is the Market Street augmentation imposed upon the surface from the Sept. 2018 survey. This is after the 2016 augmentation and after the 2017 high flows, so the initial surface will include the deposition downstream of the site due to the augmentation. We would need a survey before and after a specific augmentation to test the validity of these predictions. This fact also brings up the point that the specific downstream deposition patterns will depend upon the history of prior augmentations, and the same augmentation can result in different responses downstream.

The lowest flows, those up to 350 m^3/s, are equally capable of eroding about 1400 m^3 of sediment from the Market Street gravel augmentation pile. About 200 m^3 of this sediment is deposited in next reach downstream, the Sundial reach (Figure 35). A little more than 300 m^3 is deposited in both the Turtle Bay reach and the Marina reach (Figure 36).

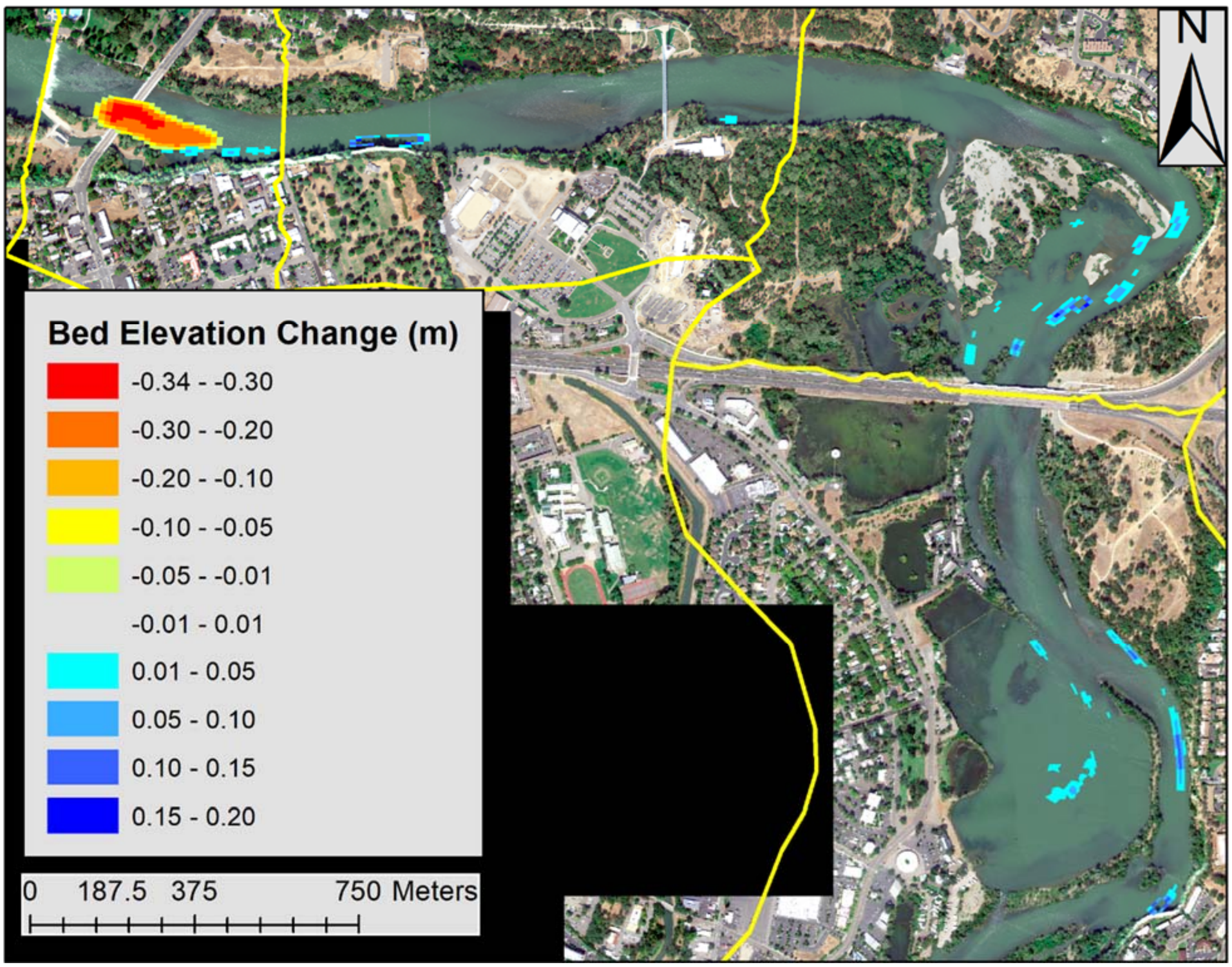
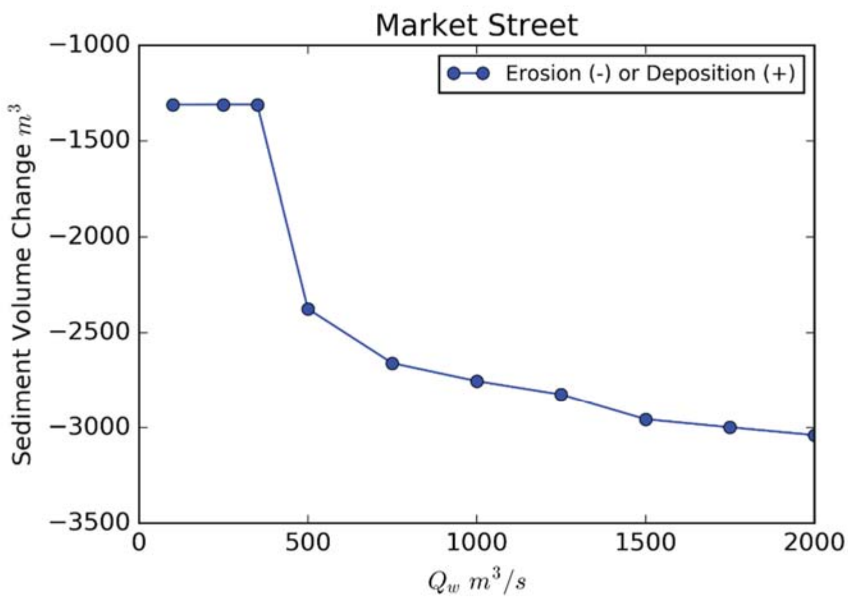


Figure 34. The change in bed elevation resulting from a flow of 1000 m^3/s after 400 hours. The Market Street gravel augmentation pile (at upper left) was substantially eroded and re-deposited downstream. The original pile was 0.42 m higher than the underlying topography. Areas where bed elevation change was less than 1 cm are unshaded.

At flows of 500 m^3/s and above, more than 2400 m^3 of sediment were eroded from Market Street and deposited downstream. At flows of 500 and 750 m^3/s, between 600 and 700 m^3 of this sediment is deposited in the Sundial reach (Figure 35). Approximately 900-1000 m^3 is deposited in Turtle Bay and the Marina.

At flows of 1000 m^3/s and above, the amount of sediment eroded from Market Street increased from about 2700 m^3 to about 3000 m^3 at 2000 m^3/s. As flow increases, the amount of sediment deposited in Sundial decreases to near zero. Sediment deposition in Turtle Bay decreases to a bit more than 250 m^3 at a flow of 1750 m^3/s and then ticks up at 2000 m^3/s. Deposition in the Marina decreases from a peak of almost 650 m^3 at 1000 m^3/s to slightly more than 400 m^3 at 2000 m^3/s.

Figure 37 shows that almost no sediment is deposited in the Cypress Ave reach or the tiny Model Outlet polygon. This raises a question. Approximately 3000 m^3 of sediment was eroded from Market Street at 2000 m^3/s, about 750 m^3 of which was deposited in Turtle Bay and the Marina. Where are the remaining 2250 m^3 of sediment? As shown in Figure 38, a small fraction of mass leaves the model domain altogether, a small fraction is lost as unconserved mass, and the rest is still in motion, largely in the Marina reach at the end of 400 hours.



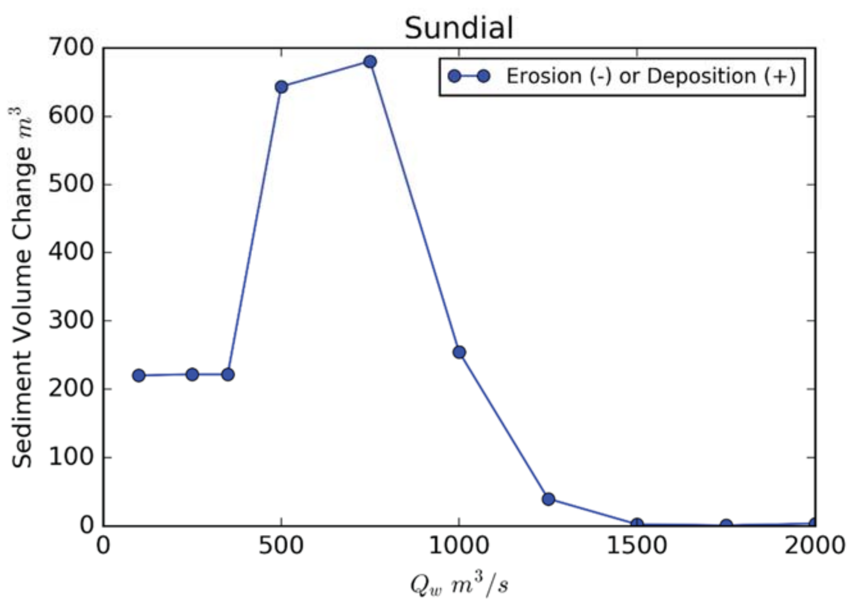


Figure 35. Total erosion or deposition after 400 model hours as a function of model flow in the Market Street reach (top) and Sundial reach (bottom).

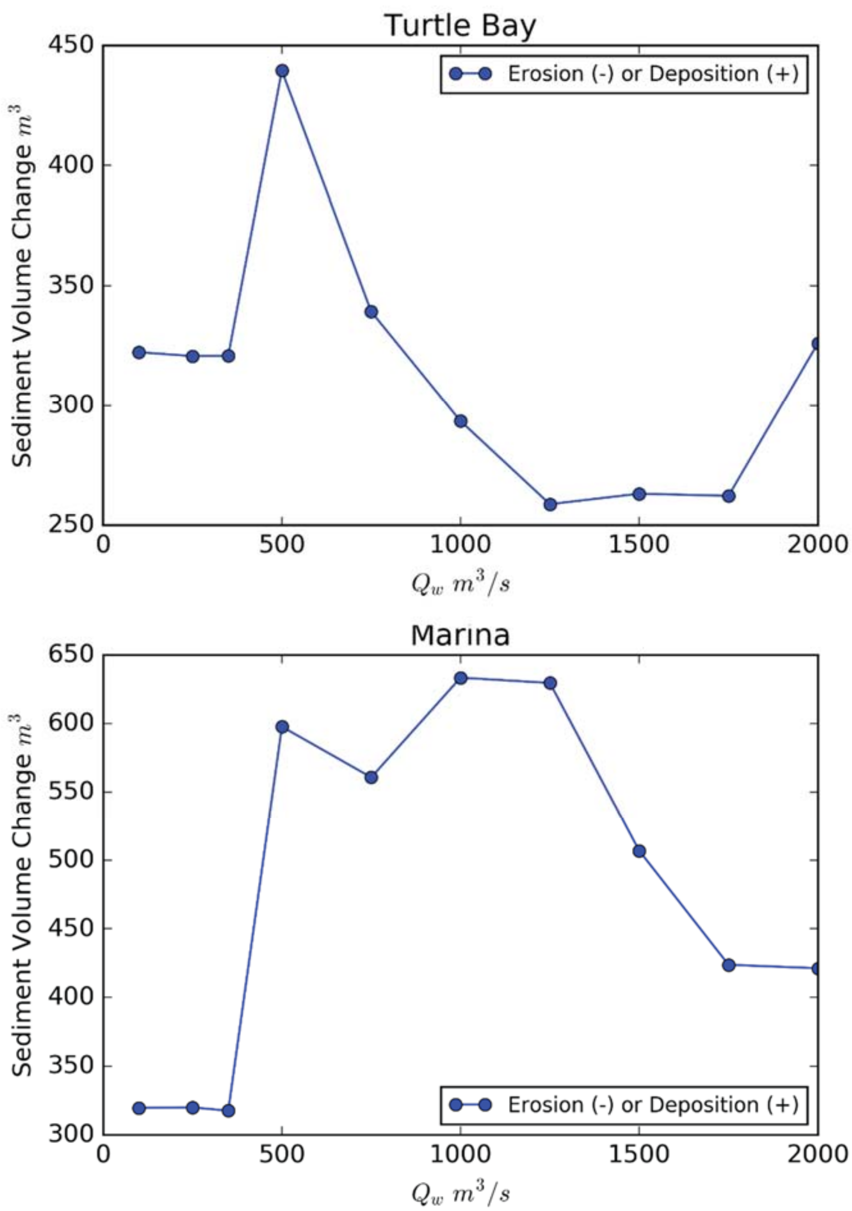


Figure 36. Total erosion or deposition after 400 model hours as a function of model flow in the Turtle Bay reach (top) and Marina reach (bottom).

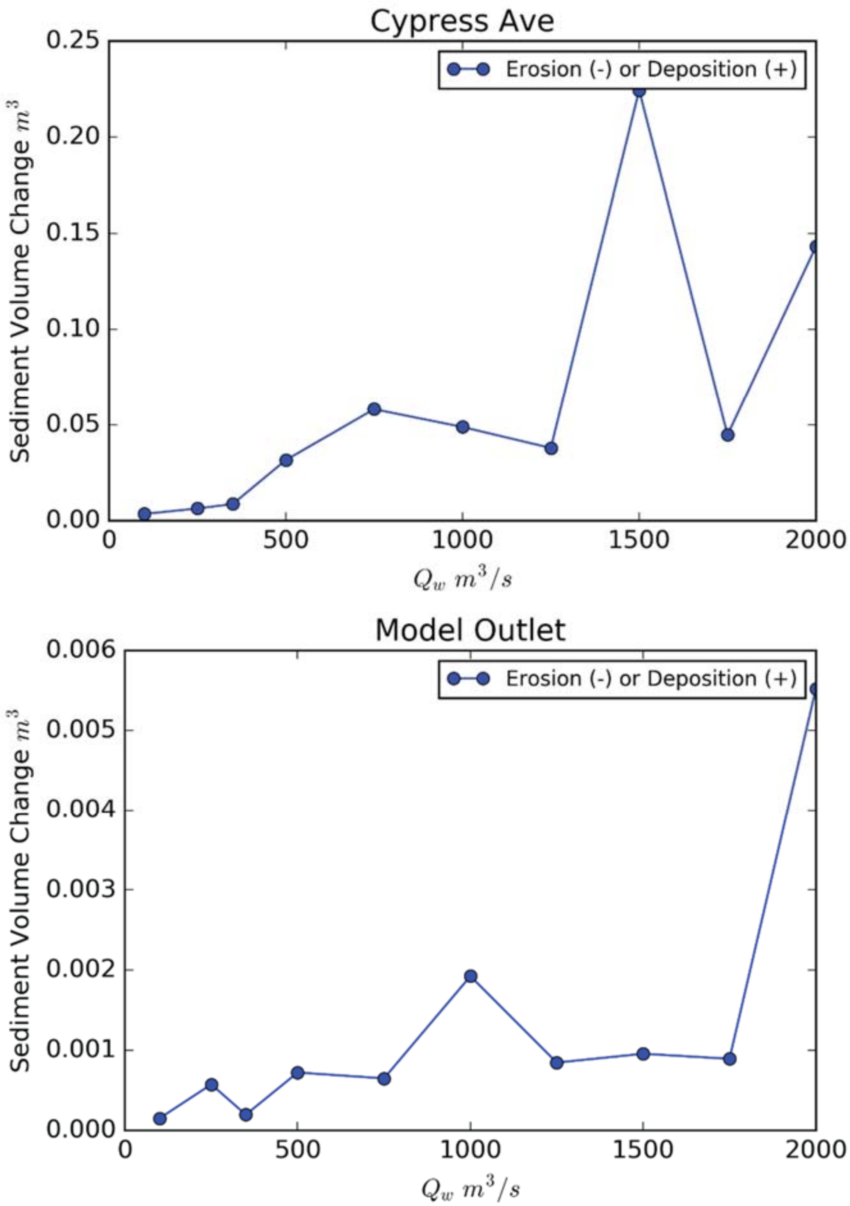


Figure 37. The Cypress Ave. reach (top) and Model Outlet (bottom) receive almost no sediment transported from upstream after 400 model hours at any flow.

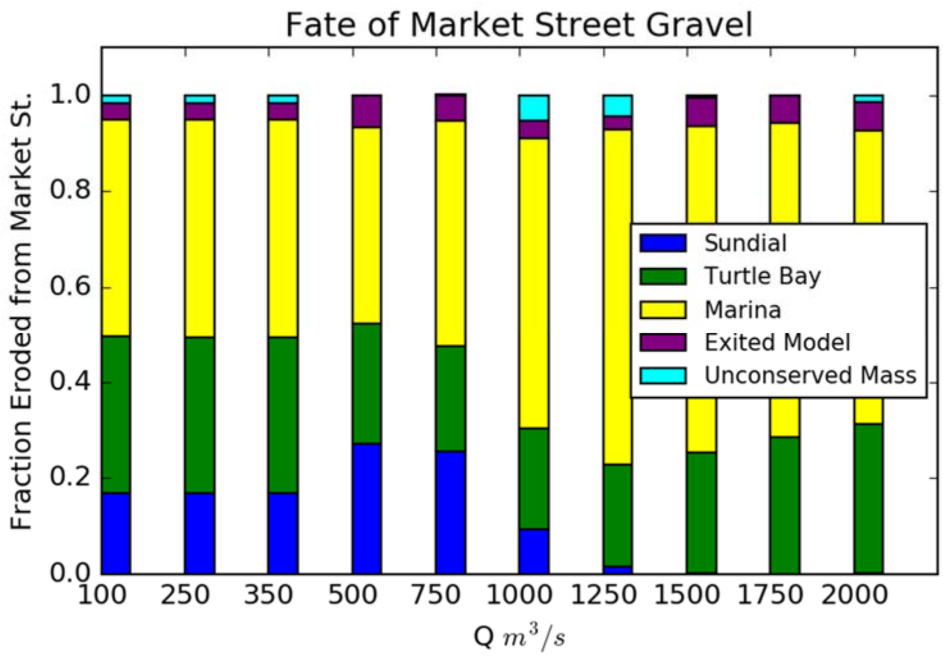


Figure 38. The fate of sediment eroded from Market Street. The amount of sediment downstream of the Marina reach is too small to show up on the plot (less than 0.01%). Most of the sediment in the Marina (~1800 m^3) is still in motion at the end of the simulation.

3.3.2 Transient Model

The transient mobile bed model simulated the Sacramento River flows for the 19 months following the 2016 gravel augmentation. The change in bed elevation at the end of the simulation (2496 hours) is shown in Figure 39. About half of the height of the augmentation pile (bed elevation + 0.42 m) was eroded and deposited in Turtle Bay and the Marina. Figure 40 through Figure 42 show how the river bed evolves over the course of the simulation. By about 1500 hours, almost half of the 6320 m^3 of gravel augmentation at Market Street was eroded (Figure 40) with little additional erosion after the second peak in the synthetic hydrograph (number 4 in Figure 23). About half of the eroded Market Street gravel was initially deposited in the Sundial reach, only to be remobilized and evacuated on the rising limb of peak 3 of the hydrograph at about 1000 hours.

The evolution of the Turtle Bay reach is complex (Figure 41). Deposition is at a maximum during the second hydrograph peak (about 300 hours) and then begins to decrease. Deposition starts to increase again on the falling limb of peak 3 as sediment in transport settles out. Ultimately, only about 300 m^3 of sediment deposited in Turtle Bay. Deposition in the Marina reach increases more or less monotonically over the first ~1600 hours of the simulation, reaching a maximum of about 1200 m^3 on the falling limb of peak 4. About 500 m^3 of sediment are still in motion in Turtle Bay and the Marina at the end of the simulation, indicating that the 350 m^3/s flow threshold used to condense the Keswick hydrograph and shorten model execution time is too high to allow all sediment to come to rest in the ~300 hours following peak 5. In hindsight, this could have been remedied by extending the falling limb to a much lower flow.

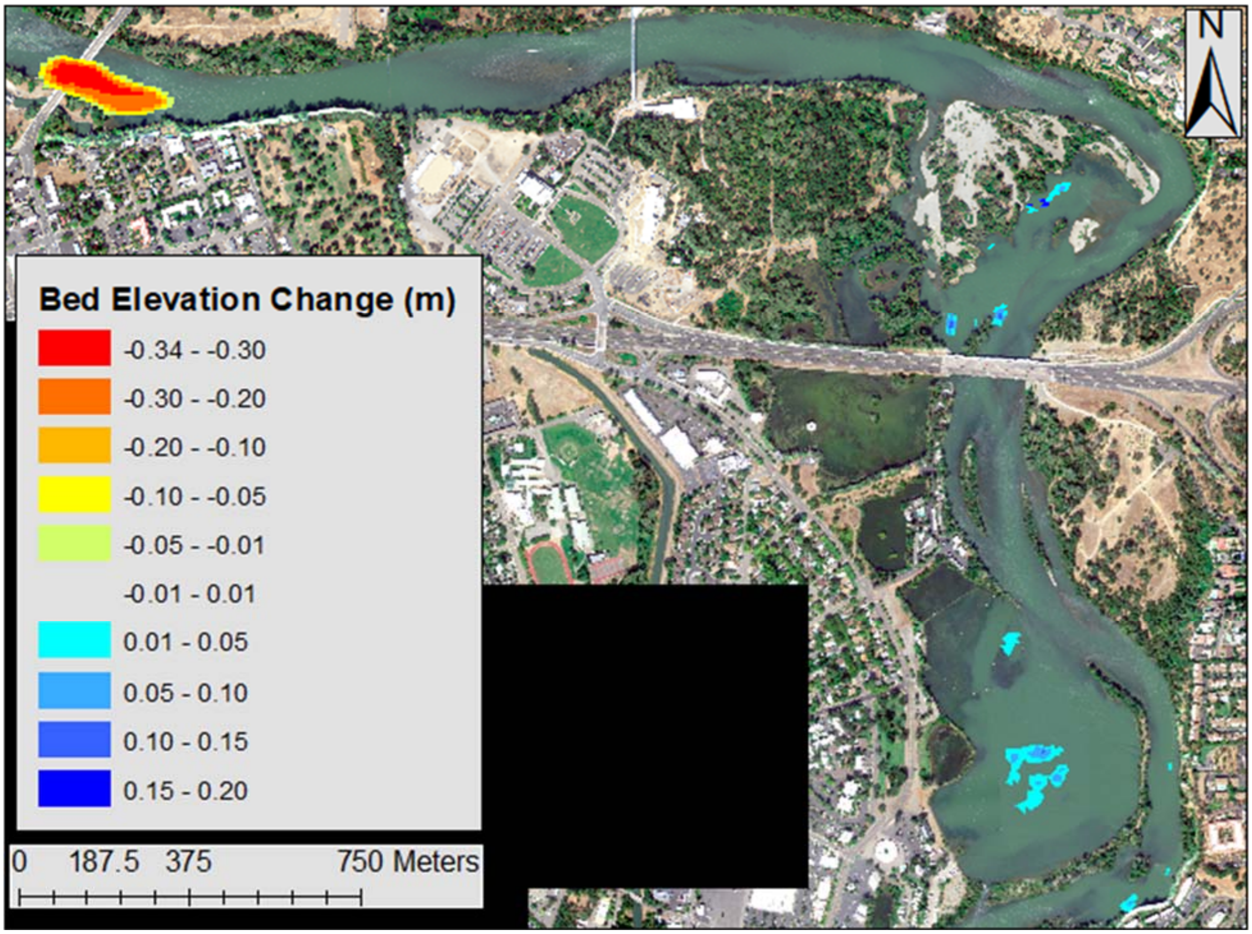
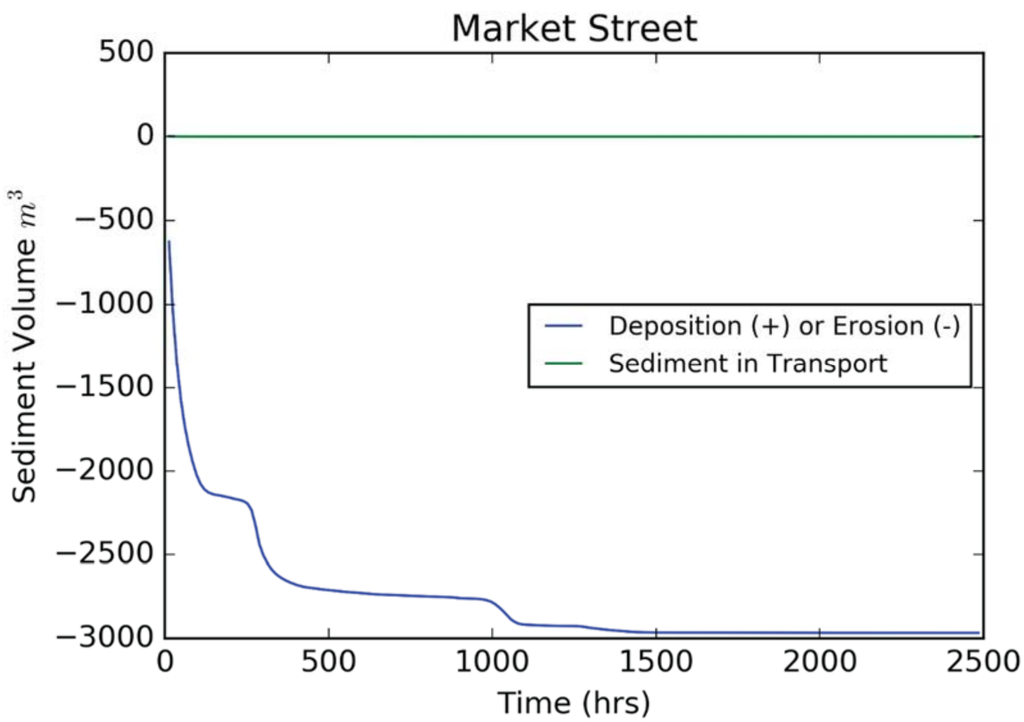


Figure 39. The change in bed elevation at the end of the Market Street transient simulation. Erosion or deposition of less than 1 cm is not shown.

Less than 1 m^3 deposited in the Cypress Avenue reach over the entire simulation (Figure 42) and at any particular time, there is almost no sediment in motion in the reach. There are two possible interpretations of the lack of sediment in the Cypress Avenue reach: 1) it could be a transport reach that rapidly passes any sediment or 2) Turtle Bay and the Marina trapped all sediment and none made it to the Cypress Avenue reach over the course of the simulation. Figure 43 resolves this question. About 16% (470 m^3) of the sediment eroded from Market Street passed through the Cypress Avenue reach and exited the model domain. Of the remaining sediment, 56% was deposited in the Marina Reach, and about 13% in Turtle Bay. About 16% of the sediment disappeared altogether as unconserved mass, a much higher fraction than in any of the steady discharge model runs (Figure 38). The reason for unconserved mass is likely in the bed mixing algorithms and that they are decoupled from the transport of the sediment within a given time step. There could also be surfaces that become wet and then dry during unsteady flow and in this process, the sediment in these cells could be lost. The mass lost is not ideal, but is expected to only affect the results in a proportional sense, meaning that the mass of deposition and exiting would increase by about 16%.



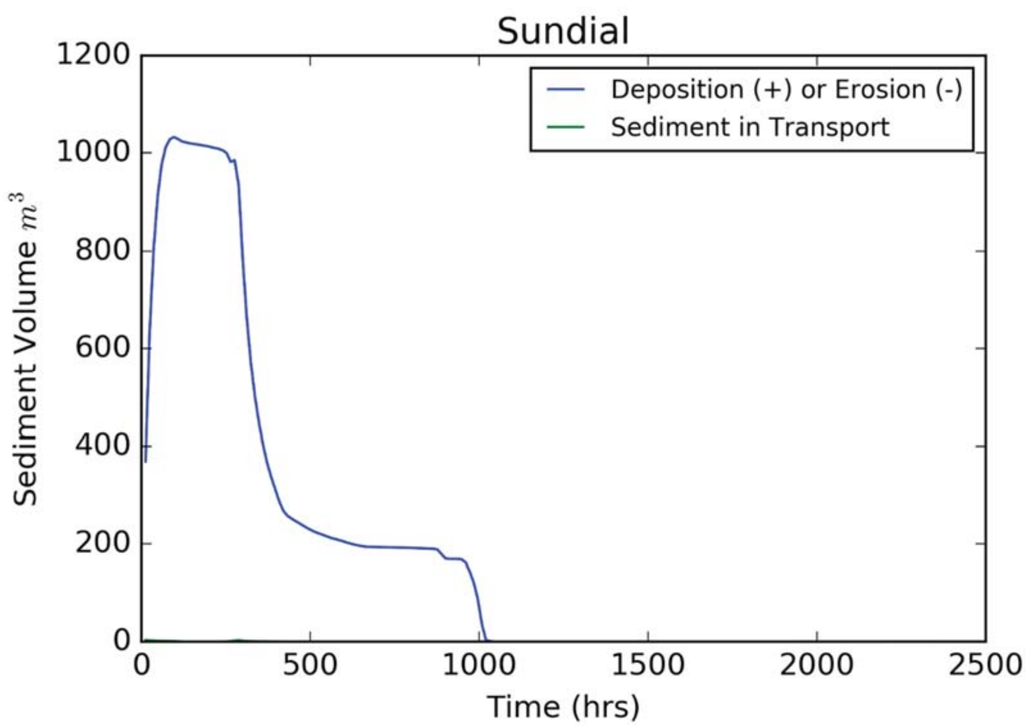


Figure 40. The evolution of the Market Street gravel augmentation pile over the course of the transient simulation (top). The bottom panel shows deposition in the Sundial reach early in the simulation that is remobilized and removed from the reach later in the model run.

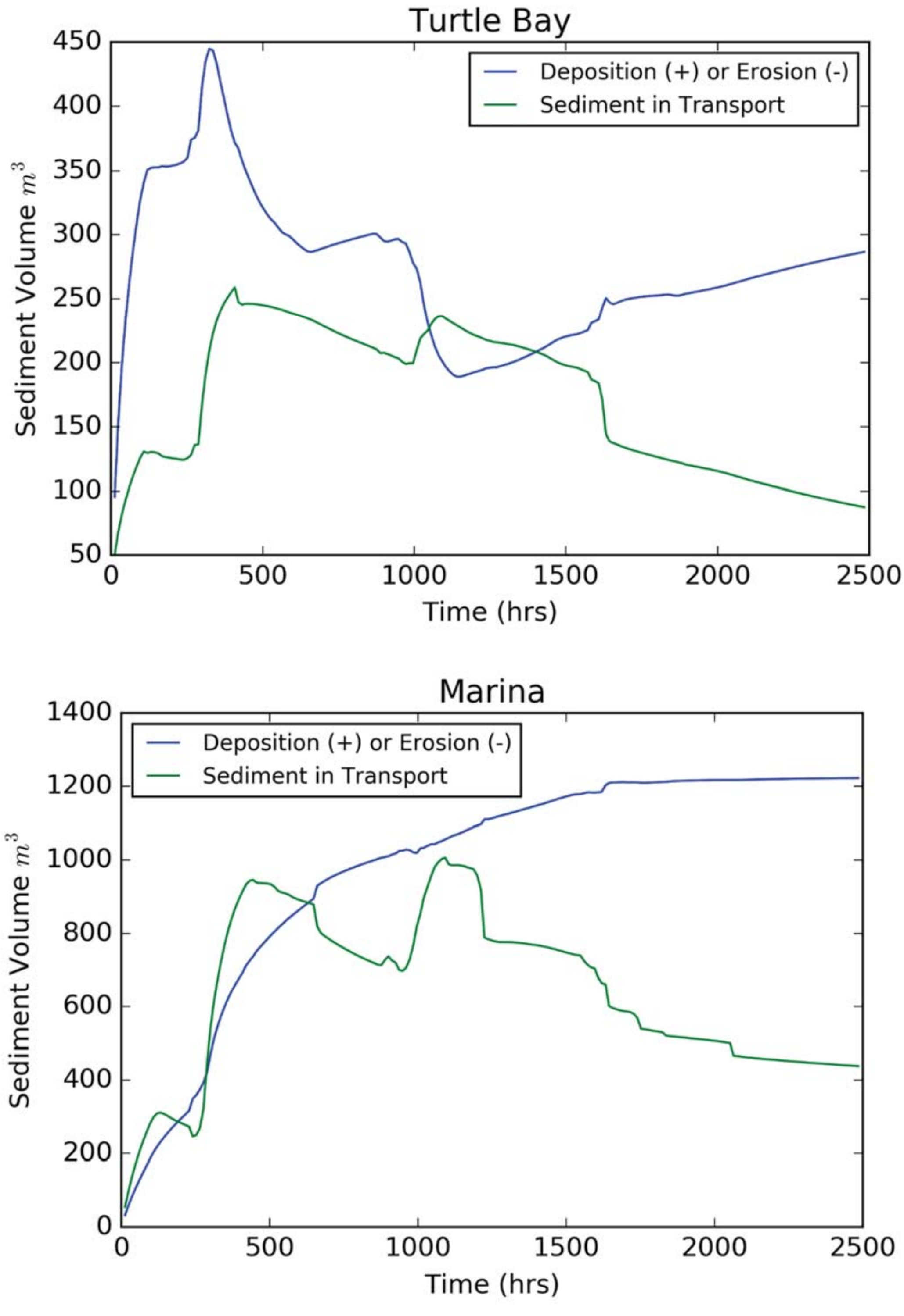


Figure 41. The evolution of sediment transport and deposition over the course of the simulation in the Turtle Bay (top) and Marina (bottom) reaches.

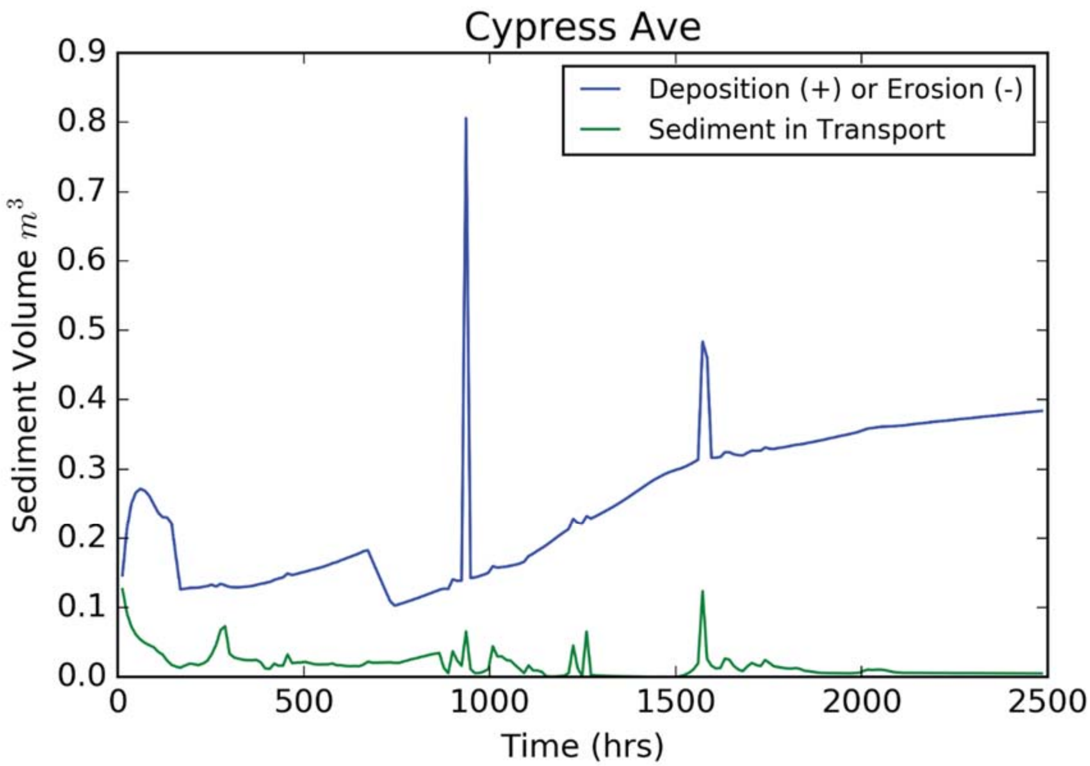


Figure 42. The evolution of sediment transport and deposition over the course of the simulation in the Cypress Avenue reach.

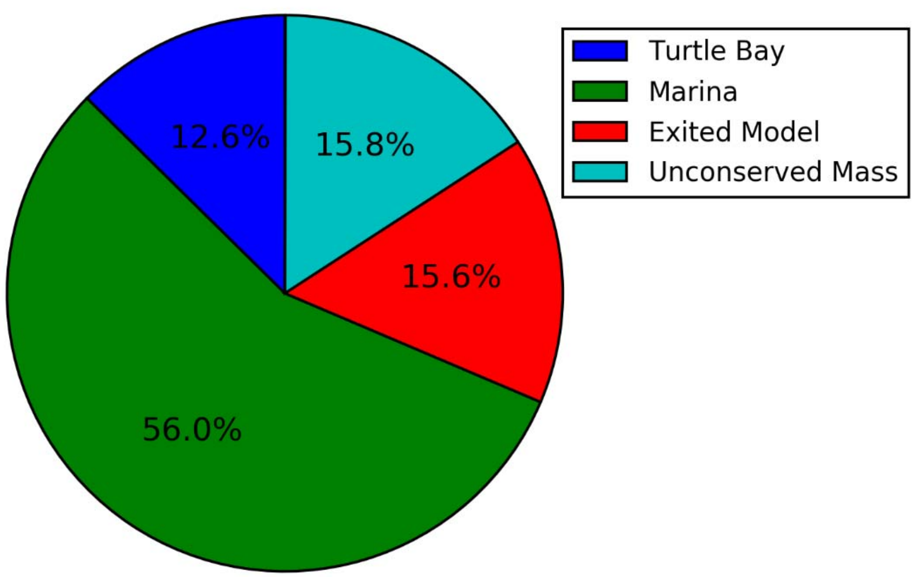


Figure 43. The fate of the approximately 3000 m^3 of sediment eroded from the Market Street site. The graph includes sediment deposited on the bed and sediment still in motion at the end of the simulation.

4. Discussion and Conclusions

The Project Management Plan (PMP) defined several questions that this study attempted to address. Those questions are addressed next.

- *At what locations and in what manner does spawning gravel need to be placed within the study reach to augment and restore spawning and rearing habitat and ultimately reestablish the long-term gravel mass balance?*

The reach averaged sediment transport capacity (Figure 31) of the Upper Sacramento River is relatively uniform over the study area, but the mobile bed simulations indicate that gravel placed at the Market Street sited mostly winds up in the Marina Reach and to a lesser extent in Turtle Bay. In the transient model run, about 16% of the sediment eroded from Market Street exited the model all together. It is likely that any gravel added to the river upstream of Turtle Bay would experience the same fate, deposition mostly in Turtle Bay and the Marina reach. The transient model results indicate that gravel placement strategy used at Market Street is an effective location if supplying gravel to the downstream reach up to Turtle Bay is the objective, with roughly 50% of the pile eroded in the 19 months following placement. There is some gravel being transported through Turtle and Marina Bay reaches, but it is a relatively small portion of the augmented gravel.

- *How much spawning gravel input into the study reach is required to reduce the sediment median diameter of the riverbed?*

This question was not addressed directly by the modeling described in this study, but some inferences can be made from the results. The low flow wetted area of the study is on the order of 3×10^6 m^2 (Table 6). To cover this entire area with a single grain thickness of 30 mm gravel would require 90,000 m^3 of gravel, or about 15 times the amount added at Market Street in 2016.

- *How often does spawning gravel need to be added to the river channel to maintain a stable balance between gravel input, transport, and storage?*

The average annual sediment transport capacity of the Upper Sacramento is on the order of 10^4-10^5 m^3 (Table 7). This is about 1.6 to 16 times the volume of the 2016 Market Street augmentation. The results from the transport capacity estimate and mobile bed models suggest that the river could accommodate projects of similar size on an annual basis. However, if gravel is augmented upstream of Turtle Bay, most of the sediment is likely to be trapped in Turtle Bay and the Marina Reach. Gravel should be augmented downstream of Cypress Avenue to restore gravel transport processes there.

- *How much gravel/coarse sediment is needed to restore transport processes?*

Model results indicate that the average annual transport capacity is on the order of 10^4-10^5 m^3 (Table 7). Additional comparisons with measured data could help to make more accurate estimates, but it is certain that the capacity is much larger than current augmentation rates.

- *Where can we place gravel for mobilization?*

It depends on where gravel is needed. The mobile bed models indicate that majority of gravel augmented at the Market Street site is deposited in Turtle Bay and the Marina reach. Though not modeled explicitly, Shields stresses derived from the hydraulic model (Figure 29) suggest that gravel augmented anywhere upstream of Turtle Bay is likely to behave similarly to the simulated Market Street augmentation (Figure 38 and Figure 43). The mobile bed model results indicate that sediment from upstream passes rapidly through the area downstream of the Cypress Avenue bridge, suggesting that sediment placed there might be likely to mobilize. However, deposition in the channel downstream of the bridge is visible in the aerial imagery, so further consideration would be necessary.

- *Where is gravel likely to accumulate?*

Gravel placed upstream of Turtle Bay is likely to accumulate mostly in Turtle Bay and the Marina reach upstream of the Cypress Avenue bridge (Figure 38 and Figure 43).

- *Where can we place gravel such that it would be unlikely to move?*

This question was not directly addressed by the modeling in this study, but results suggest that gravel is likely to be mobile over most of the study reach, with the exception of areas where the channel widens at Turtle Bay and the Marina.

- *What is the appropriate size of gravel to be adding?*

85% of the gravel used in the Market Street augmentation project was smaller than about 50 mm (Table 4). The mobile bed models indicated that half of this volume is eroded by a flow of 2000 m^3/s within 400 hours (Figure 35) or within 1500 hours by the synthetic hydrograph derived from the 2016 to 2017 Keswick gage record (Figure 40). The Shields stress results (Figure 29) show that gravel smaller than 40 mm is likely to be mobile over large areas at a wide range of flows, supporting the assertion that the Market Street gravel size distribution is appropriate.

- *What type of flows are needed for transport?*

Flows of 500 m^3/s and above are capable of moving sediment smaller than 50 mm (Figure 29 and Figure 35).

- *How much effect does the armoring in the river have on movement of gravel?*

The modeling in this study is not able to address this question directly. The mobile bed models treated the entire river bed outside of the Market Street project site as armored and immobile. Gravel moved over the armored bed, but the real world complexities of grain-to-grain interaction are not well represented by the model.

- *Could the armoring be broken up? What would the impacts be on gravel movement?*

The armoring probably could not be broken up over large areas by the range of flows modeled in this study. The Shields stress analysis (Figure 29) suggests that the highest flows, 2000 m^3/s and above, are capable of mobilizing sediment larger than 60 mm over less than 50% of the study area. Sediment 80 mm and larger is mobile over less than 10% of the study area outside of the bedrock reach downstream of Keswick. Based upon pebble counts of the bed material [Stillwater Sciences, 2007, Figures 21 and 22], the armored bed median grain diameter in the study reach is approximately 100 mm. Therefore, the flow would likely have to be significantly greater than 2,000 m^3/s for existing bed material to be mobilized.

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