River Geomorphology Study for the Merced River at the Ferguson Rock Slide

Effects of Proposed Transportation Alternatives on River Morphology and Flood Flow Behavior

Report prepared for:

California Department of Transportation

Under sub-contract with:

Parsons Transportation Group, Inc.

Prepared by:

Mark Strudley, Ph.D.

Greg Guensch, P.E., M.S.

Shawn Chartrand, C.E.G., M.S.

Benjamin Roberts, Ph.D., P.E.

Balance Hydrologics, Inc.

January, 2009

A report prepared for:

California Department of Transportation
Attention: Mr. Matthew Voss
2015 Shields Avenue, Suite 100
Fresno, California 93726
559-243-8175
Matthew_voss@dot.ca.gov

River Geomorphology Study for the Merced River at the Ferguson Rock Slide

© 2009 Balance Hydrologics, Inc. Project Assignment: 207240
by

FOR MARK STRUDLEY
Mark Strudley, Ph.D.
Hydrologist / Geologist

Greg Guensch, P.E., M.S.
Water Resources Engineer

Shawn Chartrand, C.E.G., M.S.
Principal Geomorphologist

FOR BENJAMIN ROBERTS
Benjamin Roberts, Ph.D., P.E.
Senior Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY RUSSEL GUENSCH
C 72585
Exp. 6/30/10
CIVIL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
SHAWN M. CHARTRAND
No. 2442
CERTIFIED ENGINEERING GEOLOGIST
exp. 03/10
STATE OF CALIFORNIA

Balance Hydrologics, Inc.®

800 Bancroft Way, Suite 101
Berkeley, California 94710-2227
(510) 704-1000
office@balancehydro.com

January 30, 2009

TABLE OF CONTENTS

1. EXECUTIVE SUMMARY 1

1.1 SUMMARY OF FINDINGS 3
1.1.1 Natural Conditions 4
1.1.2 Alternatives C and T 5
1.1.3 Alternative S 5
1.1.4 Alternative S2 7
1.1.5 Alternative A 7
1.1.6 No-build Alternative 8
1.1.7 Low-flow River Hydraulics Considerations Regarding Recreational Activities 8
1.1.8 Water Quality and Episodic Events Considerations 10
1.2 RECOMMENDATIONS 11

2. PROJECT INTRODUCTION AND BACKGROUND 14

2.1 PROJECT INTRODUCTION 14
2.2 PROJECT BACKGROUND: SUMMARY OF PREVIOUS WORK COMPLETED 16
2.2.1 Work conducted by Caltrans 16
2.2.1.1 Initial study/environmental assessment 16
2.2.1.2 Geotechnical reports 18
2.2.1.3 Location hydraulic study 18
2.2.2 Other agency reports 19
2.2.2.1 U.S. Geological Survey 19
2.2.2.2 Ferguson Slide runout and deposition 20
2.2.2.3 Merced River hydraulics associated with Cascade Dam removal 20
2.2.2.4 U.S. Forest Service 21
2.2.2.5 Yosemite National Park 22
2.3 ACKNOWLEDGEMENTS 22

3. ENVIRONMENTAL SETTING 23

3.1 LOCAL GEOLOGY 23
3.2 THE FERGUSON ROCK SLIDE 24
3.3 PHYSIOGRAPHY / HYDROGRAPHY 25
3.4 LOCAL AND REGIONAL CLIMATE 25

4. HWY 140 PROJECT ALTERNATIVES ANALYZED BY PRESENT STUDY 27

4.1 ALTERNATIVE C (OPEN-CUT REALIGNMENT) 27
4.2 ALTERNATIVE T (TUNNEL REALIGNMENT) 27
4.3 ALTERNATIVES T2 AND T3 (TUNNEL REALIGNMENTS) 27
4.4 ALTERNATIVE S (VIADUCT REALIGNMENT) 28
4.5 ALTERNATIVE S2 (VIADUCT REALIGNMENT) 28
4.6 ALTERNATIVE R (ROCKSHED REALIGNMENT) 28
4.7 ALTERNATIVE A 29
4.8 NO-BUILD ALTERNATIVE 29

5. TECHNICAL APPROACH OF PRESENT STUDY 30

5.1 WORK BY BALANCE HYDROLOGICS (THIS STUDY) 30
5.1.1 Description of site visits conducted by Balance Staff 31
5.2 SITE-SPECIFIC GEOMORPHIC AND HYDRAULIC ANALYSES 32
5.3 GEOMORPHIC FACIES MAPPING 32
5.4 SEDIMENT GRAIN SIZE ANALYSIS 35
5.5 SEDIMENT TRANSPORT 37
5.6 DEPOSIT DATING 39
5.7 AERIAL PHOTOGRAMMETRIC ANALYSIS 40
5.8 QUATERNARY LANDSLIDE OBSERVATIONS 41
5.9 GEOLOGICAL RECONNAISSANCE 42

6. GEOMORPHIC ANALYSES AND RESULTS 43

6.1 PLANFORM DYNAMICS (AERIAL PHOTOGRAMMETRIC ANALYSIS) 43
6.2 QUATERNARY GEOMORPHIC HISTORY 44
6.2.1 Landslide deposits 44
6.2.2 Stable channel banks 46
6.2.3 In-channel features 46
6.2.4 Channel margin deposits 47
6.2.5 Large woody debris 50

7. HYDROLOGIC ANALYSES AND RESULTS 51

7.1 AVAILABLE GAGE DATA 51
7.2 PEAK FLOW HISTORY 51
7.3 GAGE DATA ANALYSES 52
7.3.1 Gage data correlation 53
7.3.2 Log Pearson 54

8. HYDRAULIC ANALYSES AND RESULTS: 1-D AND 2-D MODLEING 56

8.1 RIVER BATHYMETRY 56
8.2 ONE-DIMENSIONAL MODEL 57
8.2.1 Model development 57
8.2.2 Model runs 57
8.2.2.1 Calibration 57
8.2.2.2 Two-dimensional model flows and boundary conditions 58
8.2.3 Model results 58
8.2.3.1 Calibration flows 58
8.2.3.2 Verification flows for the two-dimensional model 58
8.3 TWO-DIMENSIONAL HYDRAULIC MODEL 58
8.3.1 Model scenarios 59
8.3.2 High-Flow Model 60
8.3.2.1 Model development and parameterization 60
8.3.2.2 Model results 61
8.4 LOW-FLOW (RAFTING, ~Q2/BANKFULL) MODEL 63
8.4.1 Model development and parameterization 63
8.4.1.1 Model results 64
8.4.2 Discussion of limitations or issues 66

9. RIVER-FOCUSED EVALUATION OF PROPOSED HWY 140 TRANSPORTATION ALTERNATIVES: DISCUSSION AND CONCLUSIONS 67

9.1 LOW-FLOW (~Q2/BANKFULL) RESULTS 67
9.1.1 Natural conditions 67
9.1.2 Alternatives C and T 68
9.1.3 Alternatives S and the S Temporary Bridges 70
9.1.3.1 Alternative S 70
9.1.3.2 Alternative S Temporary Bridges 71
9.1.4 Alternative S2 72
9.1.5 Alternative A 73
9.1.6 No-build alternative 74
9.2 HIGH-FLOW RESULTS 75
9.2.1 Natural conditions 75
9.2.2 Alternatives C and T 76
9.2.3 Alternatives S and the Temporary S Bridges 77
9.2.3.1 Alternative S 77
9.2.3.2 Alternative S Temporary Bridges 79
9.2.4 Alternative S2 80
9.2.5 Alternative A 81
9.2.6 No-build alternative 81

9.3 BRIDGE SCOUR 82
9.4 WATER QUALITY AND EPISODIC EVENTS CONSIDERATIONS 84

10. RECOMMENDATIONS TO FACILITATE SELECTION OF THE PREFERRED HWY 140 TRANSPORTATION ALTERNATIVE 86

11. LIMITATIONS....88

12. REFERENCES 88

LIST OF TABLES

Table 1. Peak flow hydrology methods, Merced River, near Ferguson Rock Slide, California

Table 2. Geomorphic dating methods for fluvial landforms, Merced River near Ferguson Slide, Mariposa County, California

Table 3. Grain size analysis at Merced River near Ferguson Slide, Mariposa County, California

LIST OF FIGURES

Figure 3.1. Ferguson Slide, Merced River, Mariposa County, California

Figure 4.1. Ferguson Slide permanent restoration project proposed alternatives

Figure 4.2. Temporary crossings, Merced River at Ferguson Slide, Mariposa County, California

Figure 5.1. Abandoned left-bank terrace deposit in the Merced River canyon upstream of the Ferguson Slide, Mariposa County, California

Figure 5.2. Dendrochronology and lichenometry at the Ferguson Slide, Merced River, Mariposa County, California

Figure 5.3. Relative dates for terraces using lichenometry, Merced River near Ferguson Slide, Mariposa County, California

Figure 6.1. Historical aerial photo analysis, 1945 – 1998, Merced River between South Fork of Merced River and Ferguson Slide, Mariposa County, CA

Figure 6.2. Schematic section E-E, Merced River near Ferguson Slide

Figure 6.3. Schematic section F-F, Merced River near Ferguson Slide

Figure 6.4. Schematic section G-G, Merced River near Ferguson Slide

Figure 6.5. Ancestral and historical slides in the Merced River canyon upstream of the Ferguson Slide, Mariposa County, California

Figure 6.6. Bars on the Merced River at Ferguson Slide, Mariposa County, California

Figure 6.7. Cobble bar across from Ferguson Slide, Merced River, Mariposa County, California

LIST OF FIGURES (CONTINUED)

Figure 6.8. Bar-complex along left bank above upper temporary crossing, Merced River at Ferguson Slide, Mariposa County, California

Figure 6.9. Large woody debris, Merced River at Ferguson Slide, Mariposa County, California

Figure 6.10. Large woody debris, Merced River at Ferguson Slide, Mariposa County, California

Figure 7.1. Mean annual flow in Merced River at USGS Pohono Gage #11266500, 1917-2007, Mariposa County, California

Figure 7.2. Mean annual flow in South Fork Merced River at USGS gage #11268000 and in the Merced River at Briceburg (USGS gage #11268200), 1952-1975, Mariposa County, California

Figure 7.3. Annual instantaneous peak flows in Merced River at USGS Pohono Gage #11266500, 1917-2007, Mariposa County, California

Figure 7.4. Annual instantaneous peak flows in South Fork Merced River at USGS gage #11268000 and in Merced River at Briceburg (USGS gage #11268200), 1950-1975, Mariposa County, California

Figure 7.5. Mean daily and peak flows at USGS gaging station #11266500 (Merced River at Pohono Bridge), Mariposa County, California

Figure 7.6. Mean daily discharges at the Briceburg (# 11268200) and Pohono (# 11266500) gages for the period of the Briceburg Record.

Figure 7.7. Ratio of flow at the Briceburg Gage (11268200) located approximately 5 miles downstream of the project reach to the flow at the Pohono Gage (11266500) approximately 16 miles upstream.

Figure 7.8. Mean daily discharge data at the El Portal South Fork Gage (#11268000) and the El Portal Resultant of combining 11-2665 + 11-2680 (#11268001), Mariposa County, California.

Figure 7.9. Ratio of flow at the combined flow El Portal Gage (11268001) to immediately upstream of the project reach to the flow at the Pohono Gage (11266500) approximately 16 miles upstream.

Figure 7.10. Flow frequency curve for Merced River at Pohono Bridge near Yosemite, USGS gage #11266500, 1917-2007, Mariposa County, California

Figure 7.11. Flow frequency curve for South Fork Merced River near El Portal, USGS gage #11268000, 1950-1975, Mariposa County, California

LIST OF FIGURES (CONTINUED)

Figure 8.1. HEC-RAS work Map.

Figure 8.2. Manning's n values used in HEC-RAS model and FESWMS.

Figure 8.3. Example photographs of observed high water marks.

Figure 8.4. Water surface elevation profiles generated by the HEC-RAS 1-D hydraulic model for the high and low estimates of the 1997 storm discharge (43,500 and 49,000 cfs), and for the estimated 2005 peak discharge (17,000 cfs).

Figure 8.5. Example section of mesh used for the high-flow model.

Figure 8.6. Water surface elevations (ft) computed with the 2-D hydraulic model for the 42,000 cfs flow rate, plotted against HEC-RAS results, which were calibrated to the high water marks observed in the field.

Figure 8.7. Water surface elevation (upper) and depth (lower) contours for the natural channel condition at 42,000 cfs.

Figure 8.8. Velocity (upper) and Froude number (lower) contours for the natural channel condition at 42,000 cfs.

Figure 8.9. Depth (upper) and velocity (lower) contours for the temporary bridge conditions for Alternatives C and T at the upstream location and C, T, and S at the downstream location, for a flood flow of 42,000 cfs.

Figure 8.10. Depth (upper) and velocity (lower) contours for the Alternatives C and T at 42,000 cfs.

Figure 8.11. Depth (upper) and velocity (lower) contours for the temporary bridges proposed during the construction of Alternative S at 42,000 cfs.

Figure 8.12. Depth (upper) and velocity (lower) contours for the Alternative S at 42,000 cfs.

Figure 8.13. Example section of mesh used for the high-flow model.

Figure 8.14. Water surface elevations (ft) computed with the 2-D hydraulic model for the 8,800 cfs flow rate, plotted against HEC-RAS results, which were calibrated to the high water marks observed in the field.

Figure 8.15. Water surface elevation (upper) and depth (lower) contours for the natural channel condition at 8,800 cfs.

LIST OF FIGURES (CONTINUED)

Figure 8.16. Velocity (upper) and Froude number (lower) contours for the natural channel condition at 8,800 cfs.

Figure 8.17. Depth (upper) and velocity (lower) contours for the temporary bridge conditions for Alternatives C and T at the upstream location and C, T, and S at the downstream location, for a flood flow of 8,800 cfs.

Figure 8.18. Depth (upper) and velocity (lower) contours for the Alternatives C and T at 8,800 cfs.

Figure 8.19. Depth (upper) and velocity (lower) contours for the temporary bridges proposed during the construction of Alternative S at 8,800 cfs.

Figure 8.20. Depth (upper) and velocity (lower) contours for the Alternative S at 8,800 cfs.

Figure 9.1. Depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential under natural channel conditions at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.2. Comparison between Alternatives C & T and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.3. Comparison between Alternative S and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.4. Comparison between Alternative S Temporary Bridges and natural conditions prior to construction of Alternative S: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.5. Comparison between the No Build Alternative and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.6. Depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential under natural channel conditions at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California

LIST OF FIGURES (CONTINUED)

Figure 9.7. Comparison between Alternatives C & T and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.8. Comparison between Alternative S and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.9. Comparison between Alternative S Temporary Bridges and natural conditions prior to construction of Alternative S: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 9.10. Comparison between No Build Alternative and natural conditions: depth-averaged flow volume, velocity, shear stress, critical sediment diameter, and incipient motion potential at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California

Figure 10.1 Suggested pier feasibility zones for continued planning and review of Ferguson Rock Slide Transportation Restoration Alternatives, Merced River Mariposa County, California.

PLATES

Plate 1. Geomorphic map of Merced River canyon in the vicinity of the Ferguson Slide and South Fork Merced River confluence, Mariposa County, California

APPENDICES

Appendix A: ASCE scour equations

Appendix B: Aerial photographs consulted

Appendix C: Merced River at the Ferguson Slide Reach, Condition Bathymetric and Topographic Surveys: Field Data Collection Procedures

Appendix D: Individuals contacted

1. EXECUTIVE SUMMARY

The Ferguson Rock Slide is located on California State Highway 140; in between the towns of Mariposa and El Portal (El Portal is located a few miles west of the western entrance to Yosemite National Park). Highway 140 is a crucial transportation corridor for both local residents whom work in the Park, as well as visitors to the Park. For example, Yosemite National Park (2005) reports that 1,370 vehicles entered the Park via the Highway 140 entrance on a daily basis during the month of August 2004. Freely-flowing traffic along this stretch of Highway 140 was first disrupted by reactivation of the Ferguson Rock Slide beginning on April 29, 2006. Conditions quickly deteriorated thereafter, and by the end of May 2006, a 600-foot long stretch of Highway 140 was buried in rockfall debris (Harp and others, 2006) and closed indefinitely. Closure of Highway 140 in between Mariposa and El Portal by reactivation of the Ferguson Rock Slide led to organization of a multi-agency response team, to address, in part, both the slope stability issues, and re-establishment of transportation past the Ferguson Rock Slide on Highway 140. By August 2007, the California Department of Transportation had developed 5 separate transportation alternatives for re-establishment of freely-flowing traffic conditions at the Ferguson Rock Slide (Beck, 2007a). Since that time and leading up to the present study, Caltrans has focused their list of transportation alternatives for implementation consideration. The present study provides a detailed physical evaluation of each of the alternatives which were deemed to have the most risk to impact the Merced River, 6 in total. Our evaluations have been conducted within the context of the Wild and Scenic designation of the river noting the recreational value this stretch of the river provides.

Balance Hydrologics, Inc. was asked by Parsons Transportation Group Inc., on behalf of the California Department of Transportation, to prepare a River Geomorphology Study[1] in support of environmental compliance for transportation restoration of Highway 140 at the Ferguson Rock Slide in Mariposa County, California. Restoration of freely-moving traffic conditions on Highway 140 at the Ferguson Rock Slide involves consideration of nine (9) separate project alternatives, including the no build alternative, which permits traffic to bypass the recent rock slide. Six (6) of these alternatives, Alternatives C, T, S, S2, A, and the No Build, are examined in

[1] The California Department of Transportation defines this River Geomorphology Study as one which must address the affects which the proposed transportation alternatives may have on "the river course, its direction, the decrease or increase in river flow rates, river water quality or the increase and decrease of turbidity, any potential scouring effects of the river, and the topography and geology of the river bottom – including the Ferguson Slide."

this report because of their potential to impact Merced River form, function, and recreational characteristics associated with whitewater enthusiasts. In framing our alternatives analysis, Balance has provided a very thorough characterization of *existing* geomorphic, hydrologic, and hydraulic conditions of the Merced River canyon at the project site, as well as within the context of the larger Merced River watershed. Specifically, we have assessed the potential for each alternative to drive change to the form, function, and/or whitewater recreational characteristics of the Merced River at the project site by utilizing multiple lines of evidence and reasoning, rooted in both field-based studies and analyses and hydraulic modeling. Multiple lines of evidence are particularly valuable in highlighting the limitations of one analytical approach with respect to others, and more importantly, in validating conclusions reached through disparate pathways. Specifically, Balance has:

1. Carefully characterized historical and existing physical conditions through the project reach, understanding that these conditions set the baseline or benchmarks to which all alternatives will be assesses and compared;

2. Developed quasi-calibrated 1-dimensional (1D) and 2-dimensional (2D) hydraulic models to guide assessment of alternatives impacts to flood water surface elevations and reach-scale hydraulics at both a high flood stage, and a lower flood stage deemed relevant for whitewater recreational use; and

3. Developed spatial analysis visual tools of reach-scale hydraulic parameters such as velocity and shear stress to explicitly and quantitatively assess transportation alternatives impacts to reach-scale flow structure and incipient motion potential (i.e., dynamic capacity to transport sediment and change river form through erosion), and the associated likely impacts to existing channel form, function, and whitewater recreational characteristics at both modeled flow conditions.

Results developed within the above analyses provide the information necessary for California Department of Transportation staff to complete environmental review of the proposed alternatives from the perspective of hydrology and water quality. Perhaps as importantly, though, the visual tools developed through our spatial analysis of hydraulic variables has facilitated development of recommendations for design consideration, as the review process moves towards selection of the preferred project alternative. Lastly, our physical characterization of the Merced River system, and supporting broad technical analyses guides the principle objective of this report—to describe how the Merced River may respond to the proposed Highway 140 transportation alternatives, relative to changes that have occurred naturally and episodically over the established period of record (including occurrence of the Ferguson Rock Slide).

1.1 Summary of Findings

A summary of our findings is provided below. These findings describe results of our modeling and quantitative analysis of the proposed Highway 140 transportation restoration alternatives, and the potential physical impacts each alternative may pose to the Merced River in the vicinity of the Ferguson Slide. These brief descriptions outline our predicted changes in both hydraulic and sediment transport conditions specific to each alternative, and with respect to existing conditions (i.e. without the temporary bridges). These descriptions also briefly touch upon how these predicted changes (if any) compare with the expected natural change that will be driven by further movement of the Ferguson Rock Slide mass. Our analysis has been performed using two distinct flow conditions: a low-flow condition of 8,800 cfs[2] and a high-flow condition of 42,000 cfs. The low-flow discharge was chosen to evaluate potential project effects on recreational river use and safety during spring and early summer months. This flow is essentially equal to the 2-year discharge (Q2, 8,871cfs) presented in the Section 7(a) evaluation completed for the project (Drivas and others, 2008). Their value for Q2 was obtained by assuming it was equal to the bankfull[3] discharge, which was estimated based on regional curves (Drivas and Others, 2008, Appendix C). *Thus, the discussions and conclusions for the low-flow condition apply to the Q2 flow condition as well.* The high-flow condition is intended to illustrate how large, infrequent flood events, on the order of the 100-year flow, may alter the form and function of the river at the Ferguson Rock Slide location – understanding that alteration would be driven by both hydraulic and sediment transport phenomena. In reviewing the summaries below, readers are directed to **Figures 4.1**, **9.1** through **9.10**, and **Plate 1** for illustrations of (1) the proposed transportation restoration alternatives, (2) the predicted changes to hydraulic and sediment transport conditions through the project reach from modeled natural conditions due to the proposed transportation alternatives, and (3) a geomorphic map of the reach, respectively.

---

[2] cfs is an abbreviation for 'cubic feet per second', and defines a commonly used volumetric unit of measure for river discharge.

[3] In the field of fluvial geomorphology, bankfull discharge represents the flow at which rising waters just begin to overtop the floodplain and is generally presumed to be the channel forming or effective discharge for most rivers and streams. While this flow often corresponds to the 1.4 to 2.3 year event, it should be noted that 'bankfull flow' is defined morphologically. It is also worth noting that the Merced River differs substantially from the rivers used to correlate bankfull discharge to the channel forming discharge, due primarily to its boulder-sized substrate and bedrock channel, and lack of a well-developed floodplain.

1.1.1 Natural Conditions

Under modeled low flow conditions, the bed and bar materials characterizing the Merced River in the vicinity of the Ferguson Rock Slide will be at the threshold of movement (**Figure 9.1**). Mobilization may periodically occur at the upstream end of the bar across the channel from the Rock Slide toe, and portions of the large bar along the left bank[4] upstream of the Rock Slide may be mobile along its stream-side periphery. Correspondingly, sand, gravel, and cobble may become mobilized on the channel bed. Under high flow conditions, bed and bar deposits along the majority of the project reach will likely be mobilized, with the exception perhaps of bar deposits closest to the canyon walls, and upstream of the Rock Slide (**Figure 9.6**) – as was generally observed during the 1997 flood. Our work did not include dynamic sediment transport modeling and thus we cannot comment on the magnitude of possible bed elevation changes in association with modeled flood events. We would expect a moderate amount of suspended sediment transport during flood events, but unlikely at the magnitudes experienced on the California coast where rivers drain the Franciscan complex.

Future movement of the Rock Slide can be expected to have associated impacts to river morphology within the vicinity of the deposited material. Movement of coarse material into the river in April and May of 2006 shifted the river to the east. Manifestation of this is evidenced by ongoing erosion of the right margin of the bar across the channel from the Rock Slide toe. Aerial photographs suggest that the bar was originally deposited during the 1997 flood. Deposition of Rock Slide material into the river has also led to a steepening of the river reach, as the river is forced to flow over the deposits. Steepening is clearly illustrated through the reach by two active knickpoints. Continued creep of the Rock Slide mass into the river would likely drive further river movement to the east, and associated erosion of the adjacent bar. Severe restriction of river width due to continued movement of the Rock Slide mass into the river could ultimately pose significant problems for any structure located within the immediate vicinity. Problems would presumably be driven by large-scale changes to the nature of flood hydraulics through the reach. Catastrophic failure of the Rock Slide mass into the river would yield the most significant change to river morphology. This topic is discussed in detail by Gallegos and DeGraff, 2006.

---

[4] Throughout this report we will use the terms left bank, and right bank to reference particular sides of the river. In all cases, the use of these terms assumes that an observer is looking downstream. Therefore, left bank would be the bank on which the present Highway 140 was constructed while right bank refers to the bank on which Incline Road is located.

1.1.2 Alternatives C and T

Under low-flow conditions for Alternatives C and T, predicted hydraulic and sediment mobility conditions are fairly consistent with predicted natural conditions – with the exception of areas located immediately upstream of the proposed bridge locations (**Figure 9.2**). The most pronounced changes are predicted for the upstream bridge where velocities could increase by up to 1.0 to 1.5 feet per second, across roughly 100 feet of river. Changes in shear stress at these locations would reflect the predicted changes in velocity, and thus sediment mobility can be expected to be enhanced as well. Given that these changes would occur over the downstream end of a mapped riffle, it is possible that erosion of riffle material could occur. Our work also suggests expected decreases in velocity and shear stress upstream and downstream of the left-hand (as viewed looking downstream) bridge pier of the upstream bridges.

Under high-flow conditions, our work suggests flow depth, velocity and shear stress changes through the reach, with the highest gradient of change predicted within close proximity to, and between proposed bridge piers. Flow depth is predicted to increase by as much as 2.0 feet within reaches upstream of the proposed bridges, but not immediately upstream as the models predict decreases in flow depth as flow approaches the bridge decks and piers. Decreases in predicted velocity are spatially consistent with predicted increases in flow depth. The most pronounced hydraulic changes are predicted to occur at the outside of bridge piers where the river would be constricted against steeper competent banks – here velocities are predicted to increase by as much as 4.0 feet per second (**Figure 9.7**). Less significant change is predicted for areas in between bridge piers where velocities could increase by approximately as 1.0 to 2.0 feet per second. Changes in shear stress at these locations would reflect the predicted changes in velocity, and thus sediment mobility can be expected to be enhanced as well. Specifically, in the case of the proposed upstream bridges it is likely that pools would be scoured on the outside of piers, while the large pool mapped downstream of the proposed bridge location would likely enlarge or grow upstream due to the prediction of enhanced bed sediment mobility beneath the bridges and between the piers.

1.1.3 Alternative S

Under low-flow conditions for Alternative S, predicted hydraulic and sediment mobility conditions are generally consistent with those conditions predicted under natural conditions – again, with the exception of the area located immediately upstream of the upstream bridge (**Figure 9.3**). Predicted changes in flow velocity here are slightly more pronounced than those predicted for Alternatives C & T with velocity increases of up to 3 feet per second near the middle of flow, and adjacent to the proposed left-hand bridge pier. This will lead to

correspondingly enhanced sediment mobility within the middle of the river, as well as on the downstream inside portion of the bar mapped along the right-hand side of the river (**Plate 1**). Given that the proposed location for the upstream bridge is at the head of a mapped pool, it is likely that predicted hydraulic effects would result in upstream growth of the existing pool, as well as erosion at the distal end of the mapped bar along the right-hand side of the river. Predicted low-flow hydraulic conditions for the proposed upstream temporary bridge associated with Alternative S would likely lead to erosion of the mapped riffle in between the proposed bridge piers, as well as to the right of the right-hand pier (**Figure 9.4**). At these locations the 2-D hydraulic model is predicting velocity increase of up to 2 feet per second – shear stress would again accordingly increase. Predicted conditions at the downstream bridge appear to be very similar to those under natural conditions.

Under high-flow conditions for Alternative S, both the downstream and upstream bridges are predicted to have measurable effects to local hydraulics and sediment mobility (**Figure 9.8**). Modeling suggests that both bridges will cause water surface elevations to rise on the upstream side of the bridges (more pronounced at the upstream vs. the downstream), with general lowering immediately downstream. This would appear to be caused by a contraction in flow area associated with the locations of the proposed piers within the general flow field and planform geometry of the river. The model predicts that flow would be somewhat concentrated under the bridges, in between the piers, and lead to plumes of higher velocity waters (up to 3 feet per second) downstream of the bridges and generally focused within the center of mapped pools. The model also predicts an increase of velocities within the flow field left of the downstream bridge left-hand pier, where flows will be constricted along the bank below Highway 140. Within these areas we can expect enhanced sediment mobility. With the exception of the area left of the downstream left-hand pier, it is likely that existing pools would get deeper. Sediment mobility would also be enhanced on the distal end of the bar along the right-hand side of the river, upstream of the upstream proposed bridge. This could lead to erosion at this point on the bar, perhaps resulting in a permanent shortening of the bar.

Alternative S would utilize the same downstream temporary bridge as that highlighted for Alternatives C and T (see discussion under 1.1.6). However, Alternative S comes with a unique upstream temporary bridge which under both low-flow, and high-flow conditions could yield significant change to river morphology (**Figures 9.4** and **9.9**). Under low-flow and high-flow conditions, the upstream temporary bridge for Alternative S has the potential to permanently change the nature of the river there through erosion of a fairly long-term stable riffle and bar.

1.1.4 Alternative S2

Alternative S2 would utilize a similar downstream bridge to that proposed for Alternative S (**Figure 4.1**). Therefore, results discussed in the preceding section for Alternative S with respect to the downstream bridge would be valid for Alternative S2. The upstream bridge proposed for Alternative S2, however, is distinct from those proposed for Alternatives S, C, and T (**Figure 4.1**). The upstream bridge for S2 would have the longest span over the river compared to S, C, and T, and would have an alignment more oblique than that proposed for Alternative S. The upstream bridge for Alternative S2 would leave the right bank (view looking downstream) just upstream from where S leaves the right bank, and would land on the left bank just upstream from where Alternatives C and T land on the left bank.

Structure design of Alternative S2 was not completed by the time we had completed most of our modeling efforts, therefore we have not modeled Alternative S2. As a result we do not have definitive data from which to base our assessment of the upstream bridge for Alternative S2 given that it has a distinct alignment within the proposed alternatives. We have, though, provided a qualitative assessment of the proposed upstream bridge based partly on practicality of where piers are proposed and in general on model and analytical results for the other proposed upstream bridges. Unlike the upstream Alternatives for S, C, and T, Alternative S2 proposes only one bridge pier within the river, and it would be located fairly close to the far left bank, just below Highway 140 – this would put the pier on the far, or back-side of the mapped bar along the left-hand side of the river (**Plate 1**). This is positive given that the natural conditions high-flow model suggests that this location will have relatively low velocities and water depths. The only caveat here is that the proposed location of the pier may constrict flood flows along the left bank, and lead to possibly moderate bar erosion, or enlargement of the high-flow channel mapped there. Given that the bridge decks for Alternatives S, C, and T are designed to be above the high-flow water surface elevation, we would assume this to be true also for Alternative S2. Therefore the bridge alignment would have minimal effect in steering high-flow waters towards the right bank. If though the bridge deck for S2 is engaged by flood flows, it would strongly steer flows towards the right bank given its radically oblique alignment relative to the general course of the river.

1.1.5 Alternative A

Alternative A proposes a downstream and upstream bridge built with abutments, but no piers and the shortest bridge spans of all alternatives considered (**Figure 4.1**). By the time our modeling efforts were nearly complete, however, structure plans for Alternative A were not complete and thus we did not model the alternative. As with Alternative S2 though, we can lay

out some general thoughts rooted in the practicality that the bridges are planned with no piers. If the downstream bridge soffit is constructed at an elevation higher than approximately 1,366 feet above mean sea level, and the upstream bridge soffit is constructed higher than approximately 1,380 feet above mean seal level, than it is likely that the bridges will be above the high-flow (42,000 cfs) water surface elevations. The elevations cited would likely provide minimal freeboard. However, with no piers and with bridges elevated above the modeled high flow, Alternative A would perhaps have very little potential impact to river morphology, due to a lack of structure interaction with flood waters. It is also then reasonable to suggest that recreational conditions at the modeled low-flow would also be functionally no different from natural conditions.

1.1.6 No-build Alternative

Under low-flow conditions for the no build alternative, predicted hydraulic and sediment mobility conditions compare very well with natural conditions with no exceptions (**Figure 9.5**)

Under high-flow conditions for the no build alternative, predicted hydraulic and sediment mobility conditions differ considerably from natural conditions within the vicinity of the bridges (**Figure 9.10**). Both the downstream and upstream bridges are predicted to back-up water upstream from them, with general increases in flow velocity as water moves beneath the bridges and around the existing bridge piers. The downstream bridge could result in a very pronounced increase of flow velocities downstream of the bridge over approximately 125 feet of river width – velocity increase here could be as high as 4 feet per second. Given that the downstream bridge was constructed near the downstream end of a mapped riffle, it is possible that velocity changes could lead to erosion of the downstream end of the riffle and development of a pool. Predicted velocity changes at the upstream bridge are slightly less severe (up to 3 feet per second), but one field of increase would be focused on the distal end of a mapped bar along the right-hand side of the river. This would likely lead to bar erosion and a permanent shortening of the bar compared to existing conditions. A predicted increase of flow velocities along the left bank side of the river, downstream of the bridge will put more active stress on the left bank of the river during large floods and should be considered relative to the existing condition and strength of the bank. There should be little functional effect to the head of the existing pool at this location.

1.1.7 Low-flow River Hydraulics Considerations Regarding Recreational Activities

Proposed bridge abutments for Alternatives S, C and T are completely outside of the modeled wetted channel, and they are mostly out of the channel for the temporary bridge alternatives.

The left-hand abutment of the downstream bridge, however, is probably the most intrusive, but it is unlikely to pose a hazard because it is in a straight reach with a gradual transition from bank to abutment.

The primary considerations with respect to recreation use most likely pertain to proposed locations for the upstream bridge piers. These include:

1. The right-hand pier of the temporary bridge associated with Alternative S is slightly to the right of the center of the channel as flow curves gently to the left coming around the large bar. At rafting flows, most of the flow will occur where the pier would exist (**Figures 9.1** and **9.4**), as it will be too shallow on the bars to the left or to the right of the pier. Rafters and kayakers will thus have to avoid the pier by staying to the left (inside of the bend) when momentum may be pushing them to the right. This maneuver may be complicated by the fact that boaters may have as little as approximately 10 seconds to maneuver away from this pier, as average velocities will be roughly 10 ft/sec and they will have about 100 feet to make the maneuver.

2. The left-hand pier associated with the upstream bridge for Alternatives C and T is on the left-hand side of the channel at the upstream end of the pronounced meander to the north (**Figure 9.2**). Boaters may have to maneuver to avoid this pier as they come down the run between the left bar and right bar at a predicted velocity of 10 to 12 feet per second. A boat that does not clear the inside of the pier could possibly become pinned on the pier or stuck between the pier and the river bank.

3. The left-hand pier associated with the upstream bridge for Alternative S presents the same hazard that was described above for the C & T Bridge (**Figure 9.3**). The Alternative S pier is located further into the meander bend than the pier in the C & T alternatives, with even higher predicted velocities. Rafters and kayakers will have to fight even more momentum pushing them into the outside of the meander and towards the pier – it will therefore be more difficult to avoid the pier compared to Alternatives C & T. Boats that do not clear the pier also run the risk of getting pinned on the pier or between the pier and the bank.

4. All piers within the flowing channel, particularly those described above, can catch debris such as floating wood or ice. Such debris can make the piers even more hazardous to boaters.

With regards to the downstream bridges, the piers would be at the channel margins, the abutments would be outside the flow, and the reach is relatively straight. Therefore, the downstream bridge alternatives considered do not appear to present significant impacts to boaters in the downstream half of the reach at flows around 8,800 cfs or less. At higher flows, this may change because the piers will be further into the flowing channel as the stage rises, but the hazards are unlikely to be as significant as those described above.

1.1.8 Water Quality and Episodic Events Considerations

Balance has not explicitly analyzed potential project effects to river water quality (i.e. turbidity) as little background data exists. Several likely outcomes can be reasoned from practical considerations. First, project water quality considerations should be focused on how the project could affect those processes which drive water quality. Given that the local geology is characterized by metamorphic rocks, and not fine-grained sedimentary rocks characteristic of the Coast Ranges, the production of fines will likely be driven by slope failures which bring soil and pulverized rock into the river. The project could affect slope failure processes for those alternatives which would utilize an open-cut for the proposed road bed along the right side of the river. Some useful recommendations have been provided by Beck (2007b) with regards to construction considerations for open-cuts, and the U.S. Forest Service has prepared an erosion control plan for implementation during construction with a focus on reduction of construction driven erosion. If an open-cut alternative is chosen for implementation, we would recommend more careful consideration of the potential for cut to affect slope failure processes, and the possible ultimate introduction of fine material to the river.

Second, the river bed through the project reach is moderately steep and is largely composed of very coarse substrate. Fines are limited primarily to sand which has been deposited on bars along the river flanks. While erosion of the river bed has been predicted for many of the alternatives, this erosion is unlikely to affect turbidity during floods as material which will be eroded is likely to primarily range in size from gravels to boulders. Third, on many river systems bank failures represent real, and chronic sources of turbid material to rivers. Given the occurrence of bedrock or other hardened structures along much of the banks of the project reach, it seems unlikely that future bank failures would lead to a measurable increase of turbidity during floods.

The last water quality consideration deals with road runoff. Depending on how the bridges are drained, potentially serious pollutants could enter the river in the event of a spill, accident, or general mechanical failure of a motorized vehicle. We would recommend that stormwater runoff be directed away from the river and off of the bridges to properly designed features to contain contaminated liquids.

Forest fires, mass movements, and extreme flood events can lead to the introduction of large volumes of debris to rivers. Bridges built over rivers can easily become hot spots for debris accumulation or rafting. This potentiality can be elevated in the event of massive sediment deposition following a forest fire – although this risk may be minimal through the project reach

given the existing bed slope. While Balance did not explicitly analyze for the affects of debris rafting, etc., it is important to point out that large woody debris has been deposited by past floods through the project reach (**Plate 1**). Many of the risks associated with debris rafting are reflected in the discussion given above for whitewater recreational considerations. Left-hand piers associated with the upstream bridges for Alternatives S, C, and T, as well as piers for the temporary bridges all pose risk to debris rafting. Structural design of the piers and bridge decks should reflect this possibility, and some type of management plans developed for storm related observations of debris rafting development.

1.2 Recommendations

For specific environmental review and planning purposes, and to aid in the selection of the preferred transportation alternative, we have prepared **Figure 10.1**. This figure illustrates suggested bridge pier feasibility zones; zones are broken into *preferred* (blue fields) and *not recommended* (red fields) zones. Further description of the basis for developing this figure is provided in Chapter 10. While **Figure 10.1** provides a broad platform for selection of a preferred transportation alternative, we also provide the following specific observations which are pertinent to those transportation alternatives reviewed within this study. We have developed **Figure 10.1** and the following observations to support selection of perhaps the most river appropriate transportation alternative for the Highway 140 corridor in the Merced River canyon at the Ferguson Rock Slide:

Whitewater Recreational Use at the Low-flow Condition:

- The upstream piers proposed along the left bank for Alternative S, and to a lesser extent those proposed for Alternatives C and T, pose significant safety hazards for whitewater rafters and boaters. The threat is more significant with Alternative S, than it is for Alternatives C and T. However, we highly recommend that these piers be re-located for all three alternatives to be, if possible, consistent with the preferred zones illustrated in **Figure 10.1**.

- The right bank pier for the upstream Alternative S temporary bridge would pose significant temporary safety hazards for whitewater rafters and boaters. The pier would be located slightly to the right of the thalweg (**Figure 9.4**) as flow would curve gently to the left moving past the head of the large mapped bar along the left bank of the river (**Plate 1**). At the modeled whitewater flow, this pier would be located within the main flow zone and rafters and boaters would have to work against momentum to avoid colliding with the pier. The potential danger would be enhanced if debris were to raft against the pier because the danger zone would be broader. We highly recommend that a different temporary bridge design be developed to avoid the potential serious safety hazards for rafters and boaters.

- Proposed piers of downstream bridges for all alternatives appear to present minor to negligible hazards for whitewater rafters and boaters.

Protection of Present River Form (depositional architecture) and Function:

- Alternatives C, T, and S all pose some potential to permanently alter the nature of the existing river through the project reach, although all of these instances would be largely isolated to areas immediately adjacent to the bridges and piers. Most of the potential for permanent change is associated with the high-flow channel and distal end of the bar mapped along the left bank, at the upstream bridges. Less significant potential would exist for the right bank bar mapped at the upstream bridges. These changes could include development of scour holes in bars, truncating the river side or downstream ends of these bars, erosion of the riffle mapped there, and perhaps upstream enlargement of the large pool mapped through the meander mid-way through the project reach. If these potential impacts are deemed unacceptable, we would recommend revising designs to be more consistent with the preferred zones highlighted in **Figure 10.1**.

- The upstream temporary bridge for Alternative S would pose significant, temporary potential impacts to the nature of the river there. The proposed piers for the temporary bridge could lead to development of scour holes within an existing riffle, and could significantly truncate the river side of the left bank bar mapped there. While the bridge would be temporary as would the potential impacts, it is difficult to suggest the impacts likely durations. We would suggest that the impacts are significant enough to warrant a temporary bridge re-design.

- The above stated concerns are of less significance for Alternative S2, given the limited information which we presently have for this alternative.

- It would appear that Alternative A would pose minor to negligible potential impacts to the nature of the present river corridor, again based on the limited information which we presently have for this alternative.

- We would suggest that the downstream bridge alternatives for all alternatives would pose minor to negligible potential impacts to the nature of the present river corridor.

Debris Rafting Potential and Episodic Sedimentation:

- Debris due to massive floods, wild-fires, and future mass movements into the river all pose a significant threat to rafting of material on proposed bridge piers. While we have not explicitly analyzed for this risk, anyone working in mountain basins understands that the risk is real. Specific alternatives which have perhaps more risk for debris rafting on piers would include: Alternatives C, T, and S (left bank upstream piers), the upstream temporary bridge for Alternative S, and the No Build Alternative (upstream bridge pier). This risk would also hold for ice rafting, although the risk is likely relatively smaller given the steepness of the reach, the steepness of upstream reaches, as well as its

elevation in the watershed. Given these potential risks, we suggest that designs fully account for debris and ice rafting potential on the bridge piers.

- The potential for massive bed aggradation due to episodic sedimentation is always a concern in large California rivers. The most salient recommendation here could perhaps be to elevate the bridges as high as is feasible and practical.

Other Considerations:

- Key to the selection process of proposed alternatives is the acknowledgement that flood flows above those simulated in this study will have the potential to engage the proposed bridge structures. Our simulated high flow of 42,000 cfs is at a threshold above which bridge decks would appear to begin interacting with flood flows. In terms of planning needs, our simulated high flow is less than some estimates of the 100-year event. Flows equivalent to or larger than our high-flow event can trap ice and debris behind structures, and flows much larger than our high-flow event may potentially cause a bridge to fail.

2. PROJECT INTRODUCTION AND BACKGROUND

Chapter 2 provides the reader with a broad introduction to the present study, its uniqueness and it's relevance to previous work completed at the Ferguson Rock Slide, and within the larger watershed. After reviewing Chapter 2, the reader should have a solid understanding as to the purpose of the present study, and why an integrated geomorphic/hydrologic/hydraulic study was needed to both (a) complement and complete existing studies in support of choosing a preferred transportation alternative, and (b) to facilitate completion of project environmental review.

2.1 Project Introduction

In 1987, one-hundred and twenty-two (122) miles of the Merced River were federally designated as a Wild and Scenic River of the United States of America. The designation means that the river exhibits outstandingly remarkable values as a natural system within the United States, and therefore shall be protected to preserve these values and managed to enhance them, when feasible, for the enjoyment of present and future users as well as the species which depend on the river for their survival[5]. The designation as a Wild and Scenic River further means that proposed actions within the designated Wild and Scenic River reach must be conceptualized and developed to preserve the characteristics of the natural system which are core to the special management designation of the Merced River upstream of Lake McClure.

Following the completion of several initial studies and preliminary environmental review, Balance Hydrologics, Inc. (Balance) was asked in January of 2008 by Parsons Transportation Group Inc. (Parsons), on behalf of the California Department of Transportation (Caltrans), to prepare a River Geomorphology Study in support of ongoing environmental review of proposed transportation restoration of the Highway 140 corridor through the Merced River canyon at the Ferguson Rock Slide in Mariposa County, California. The Caltrans definition of a River Geomorphology Study effectively establishes our scope of services:

> *"A study which must address the affects which the proposed transportation alternatives may have on the river course, its direction, the decrease or increase in river flow rates, river water quality or the increase and decrease of turbidity, any potential scouring effects of the river, and the topography and geology of the river bottom – including future re-activation of the Ferguson Slide."*

[5] According to the Wild and Scenic Rivers Act (16 USC. 1271-1287 10/2/1968).

Our project or analysis approach reflects this definition and we provide a thorough review of our approach within Chapter 5 of this report.

Re-establishment of freely-flowing traffic conditions on Highway 140 at the Ferguson Rock Slide involves consideration of 9 previously developed and distinct transportation project alternatives:

- Alternative C (open-cut realignment);
- Alternative T (tunnel realignment);
- Alternatives T2 and T3 (tunnel realignments);
- Alternative S (viaduct realignment);
- Alternative S2 (viaduct realignment);
- Alternative R (rockshed realignment);
- Alternative A (utilizing Incline Road); and
- The no-build alternative.

Six of these alternatives (C, T, S, S2, A, and no-build) are examined in detail by this report because of their potential to impact Merced River form, function, and recreational characteristics associated with whitewater enthusiasts. The details of each of these alternatives will be discussed in more detail within Chapter 4 of this report. This River Geomorphology Study diligently acknowledges the Merced River Wild and Scenic designation by providing a comprehensive and technically sound geomorphic and hydraulic review of the six key proposed transportation alternatives developed by Caltrans. Our comprehensive and multi-disciplinary review forms the *focus* of our scope of services for this project.

To address this scope, Balance has drawn upon multiple lines of evidence and reasoning to corroborate, validate, and support the conclusions drawn, and ultimately the project recommendations provided. The use of multiple lines of evidence is a necessity for the practice of river-systems science, as many different types of physical processes occur along river corridors, each at their own rates of progress through time, and susceptibility to dynamic forcing by outside influence (i.e., construction of a road at the base of a steep slope will likely affect sediment supply more than flood peak response). Multiple lines of evidence and reasoning are also invaluable in contextualizing the findings of one discipline or approach, with respect to another. For example, hydraulic models can be developed and run for the sake of

generating results of river response during flood events. However, the validity and use of the model arguably increases when field-based evidence is used to corroborate the results of a hydraulic model, or provides the necessary input parameters by which to build a sound model of the physical system. The concept of multiple lines of evidence is by no means new to the scientific community, but is unfortunately rarely practiced at the project level.

While this report does touch upon some biologically-oriented pieces of evidence, it is by far a physically-based characterization and analysis of system processes, and thus should be regarded within that context. To understand how our work fits into the larger project and regional literature base, we will now provide discussion of some of the previously completed studies which are most relevant to our work and the Ferguson Rock Slide itself.

2.2 Project Background: Summary of Previous Work Completed

2.2.1 Work conducted by Caltrans

Caltrans (California Department of Transportation) has provided Balance with a number of documents pertaining to the proposed restoration of freely-flowing traffic conditions on Highway 140 at the Ferguson Rock Slide. These documents outline engineering and geotechnical design criteria, and provide information relating to environmental review of the proposed alternatives. Included in these materials are:

- An Initial Study with Proposed Mitigated Negative Declaration/Environmental Assessment (Caltrans, 2007) which outlines the impacts that the project would have on the human, physical, and biological environments in the project area;
- A Preliminary Geotechnical Report (Beck, 2007a) and a Geotechnical Design Report (Beck, 2007b), both of which describe local geology and geotechnical considerations related to each of the proposed alternatives; and
- A Location Hydraulic Study (Cipponeri, 2007) that briefly describes hydraulic modeling of the 100-year base flood flow of the project reach incorporating each of the proposed alternatives.

We provide brief synopses of these reports below.

2.2.1.1 Initial study/environmental assessment

The Initial Study completed by Caltrans in 2007 outlines the impacts that the project would have on the human, physical, and biological environments in the project area. Of particular

relevance and importance to the present study are aspects of this analysis that describe the Wild and Scenic River setting, and the effects on local hydrology, hillslope material stability and erosion, and floodplain encroachment from the proposed restoration.

The portion of the Merced River which flows through the project site is classified as a "recreational" reach under the Wild and Scenic Rivers Act (16 United States Code 1271) and the California Wild and Scenic Rivers Act (Public Resources Code Section 5093.50 *et seq*.) because of the presence of Highway 140 and Incline Road and because of recreational activities that occur in the river. Consideration of potential project effects pursuant to a Wild and Scenic River corridor, the study describes that Alternative C would generate 360,000 cubic yards of excess material that could not be used on-site. Likewise, the Initial Study suggests that the cut slopes produced from all alternatives would generate loose fill that could erode into the river. Thus, short-term impacts to surface water quality include rock falls initiated as a consequence of construction activity, and fines being introduced into the river from slope cuts, tunneling, and construction-related erosion. The study also lists short-term construction impacts associated with Wild and Scenic recreational opportunities that may impede whitewater boating and access to Incline Road, along with long-term recreational impacts associated with one to two bridge piers impeding flow within the channel. In an effort to mitigate short-term construction impacts, an erosion control plan approved by the U.S. Forest Service would be implemented to control disturbed slopes and outcroppings. The study also recommends that a 22-foot wide area should be cut adjacent to, and at the same grade as, the highway to prevent rocks from falling on the highway, and further suggests that cut slopes should be at a 1:4 ratio or flatter to avoid rock fracturing. Excess material generated during construction would be hauled off site to a disposal facility. Additionally, to protect Wild and Scenic recreational opportunities, the proposed bridges have been designed to obstruct as little of the channel as possible with minimized bridge length, while shoulder transitions are modified from standard designs to minimize encroachment on the channel.

The Initial Study also highlights potential environmental impacts to local hydrology, water quality, and adjacent floodplains due to implementation of any of the proposed alternatives. Because the study site, as well as the lower portions of the proposed structures, are within the 100-year base floodplain, both the highway and the structures would be unusable during a base flood. In particular, Alternatives C, T, and S would increase water surface elevation between the proposed bridges by 4.54 feet. However, because the Merced River canyon walls are primarily composed of bedrock, the study suggests that increases in water surface elevation and velocity would not appreciably increase present rates of erosion, and the proposed structures

are not considered longitudinal floodplain encroachments. However, as a suggested concern for a Wild and Scenic River designation, water quality could be impacted by increased delivery of fine sediments due to construction-related erosion, as well as associated increases in turbidity levels. Thus, the report outlines a number of avoidance, minimization, and mitigation measures (in addition to those listed above for Wild and Scenic Rivers) for the potential erosion of loose material at the project site. Specifically, efforts to maintain existing drainage features, limit disturbance, and incorporate check dams, drainage inlets, and energy dissipation systems are encouraged.

2.2.1.2 Geotechnical reports

In August 2007, a Preliminary Geotechnical Report (Beck, 2007a) was developed to assist project staff in evaluating the present physical condition of the project site and the Ferguson Rock Slide, and to assist in planning and scoping activity related to development of transportation alternatives. In October 2007, a Geotechnical Design Report (Beck, 2007b) was completed to provide design information relevant to transportation Alternative S. Both reports outline the environmental setting, local geology, and proposed alternatives in detail, with a focus on geological and geotechnical considerations relevant to project construction. In particular, the report suggests that local rock excavated from the cuts and tunnels is suitable for fill behind retaining walls and in embankments. At the same time, no erosion hazard is implied in the report. If material excavated from cuts and/or tunneling is used for embankment fill, the report suggests that these materials may be classified as poorly graded gravel with cobbles and boulders. This fill material, if used, will be analogous to the fill used in Incline Road and within the embankments of Highway 140, and thus future erosion potential on valley walls associated with flood flows interacting with embankment fill will not likely differ appreciably from current conditions.

2.2.1.3 Location hydraulic study

In September 2007, Caltrans completed a Location Hydraulic Study (Cipponeri, 2007) which summarizes drainage and runoff characteristics and presents HEC-RAS modeling results of water surface elevations for the proposed alternatives at that time. The modeling employed a 100-year base flood flow of 72,000 cfs, which puts the project site well within the 100-year base floodplain. Under baseline conditions, which models the river in its present condition excluding the temporary bridge structures installed in 2006, the predicted 100-year flood water surface elevations exceed those of Highway 140 by approximately 8.6 feet. Under these predicted conditions Highway 140 would obviously be impassable.

Modeling suggests that Alternative C, which would utilize a *cut* realignment, will impact the base floodplain because piers, abutments, and structures will be placed in the floodplain. Maximum water surface elevation within the base floodplain was predicted to increase in between the two proposed bridge structures by 4.54 feet. Water velocity is predicted to increase in some locations while decreasing in others, but erosion hazard was suggested to be low due to the competent bedrock-controlled channel boundaries and canyon walls. Analogously, Alternative T yields the same modeling effects on the 100-year base flood flow as Alternative C, since the same bridges, structures, and realignment have been proposed.

Alternative S will also impact the base floodplain because piers, abutments, and structures would be placed in the floodplain. Again, maximum water surface elevations within the base floodplain are predicted to increase in between the two proposed bridges by 4.54 feet during the base flood. As with Alternatives C and T, predicted flow velocities during the base flood are suggested to have negligible erosion hazards.

Alternative R (the rock shed) constitutes a floodplain encroachment since the structure would be within the 100-year base floodplain. Under this alternative, hydraulic modeling suggests a maximum backwater increase of 2.38 feet within the base floodplain. Cipponeri (2007) calculates that water velocities will increase in some locations and decrease in others, although there is no suggestion of erosion hazard due to the presence of bedrock along the river margins and valley walls.

2.2.2 Other agency reports

In response to reactivation of the Ferguson Rock Slide, government agencies have conducted a number of studies designed to characterize site conditions, the evolving slide, and conditions that may develop in the event the slide mass continues to destabilize. These reports, primarily conducted by the U.S. Geological Survey (USGS) and the U.S. Forest Service (USFS), are described below. We also provide a synopsis of an earlier study completed by the USGS which outlines conditions well upstream of the project site associated with the former Cascade Dam in Yosemite National Park.

2.2.2.1 U.S. Geological Survey

The USGS has conducted a number of studies in the vicinity of the project site. An early study on the hydraulic effects of the removal of Cascade Dam upstream of the project site near Yosemite National Park (Blodgett, 1989) provides some general hydrologic information that aids our analysis of channel-forming flow conditions at the project site. More recently, in

response to the Ferguson Rock Slide, Denlinger (2007) conducted a numerical simulation of potential runout distances and depositional geometries related to differing types of Ferguson Rock Slide mass failure (i.e. catastrophic collapse, etc.). This work defines worst-case-scenario conditions of slide-mass collapse which we have used in our analysis, described in subsequent chapters, of erosional and depositional changes that might accompany the placement of bridge structures within the Merced River gorge.

2.2.2.2 Ferguson Slide runout and deposition

Denlinger (2007) conducted simulation modeling of runout and deposition of two end-member portions of the Ferguson Slide to assess the hazard posed by the slide mass in its current configuration. The report assesses potential runout but does not assess the probability of rapid failure. The two end-members considered in the modeling analysis are (a) failure of the toe of the slide (128,000 m^3) along boundaries defined by Beck (2007a,b), and (b) failure of the entire slide mass (780,000 m^3) enveloped within boundaries again defined by Beck (2007a,b). The mechanics of failure in the model were also cast in terms of end-member conditions: (a) a bed friction angle that matches the angle of repose of the talus material of 38° was used, and alternatively (b) a bed friction angle of 25° that is sufficient for rapid acceleration of slide material. Regardless of the amount of slide material at failure, simulations employing the smaller bed friction angle of 25° result in rapid acceleration of slide material into the river channel, filling the canyon to a minimum depth of 3 meters and a maximum depth of 33 meters. Alternatively, using a bed friction angle of 38° results in a slowly failing rockslide that develops a talus cone extending back up to the position of the original slide debris. In these scenarios, regardless of the amount of material at failure, the river channel can become blocked by 30 to 66 percent. Ignoring the effects of bed friction angle, a smaller slide volume may result in only partial blockage of the river or it may dam the river with up to 22 meters (depth) of material. Failure of the entire mass, disregarding bed friction angle, the river may either become partially blocked or may be dammed by up to 33 meters (depth) of material.

2.2.2.3 Merced River hydraulics associated with Cascade Dam removal

In 1989, the USGS completed a report (Blodgett, 1989) which evaluated the possible changes in hydraulic characteristics of the Merced River in association with removal of Cascade Dam near the western boundary of Yosemite National Park. The study was conducted in conjunction with the National Park Service. Since we have not surveyed the reach near the pre-existing dam, nor have we examined grain size and hydraulic conditions in that reach, we cannot adequately utilize the data presented in this report to guide our insights on potential channel adjustment at the project site. However, the report usefully confirms that the largest flood

flows on record for both USGS gages at Pohono Bridge, and at the South Fork confluence are a result of synchronous, regional, rain-on-snow events early in the winter season. This information allows us to put constraints on our hydrologic analysis, and associated considerations of sediment transport in the project reach, which will be described in subsequent chapters.

2.2.2.4 U.S. Forest Service

In 2006, the U.S. Forest Service (USFS) completed a brief study outlining the geology and structure of the Ferguson Rock Slide (much of which is summarized in section 3.2) and described landslide dam potential and the extent to which a backwater profile would develop in the Merced River and South Fork above the slide deposit (Gallegos and DeGraff, 2006). Their work suggests a low probability of landslide dam formation because the current slide is a smaller reactivation of a much larger prior event that left no evidence of rapid failure or impoundment. The report also suggests that the rapid movement required for dam formation would require an unusual triggering event. Nevertheless, the report describes backwater profiles of Merced River water trapped behind a landslide dam with a height of either 50 or 100 feet, estimated by means independent of and predating the analysis in the USGS report above. With a 100-foot high dam, water is impounded up the Merced River to nearly river mile 106 (near the Clearinghouse Mine) and the South Fork Merced River is impounded to approximately river mile 1.5, forming a 3800 acre-foot body of water. Under this scenario, Highway 140 and Incline Road would be below water upstream from the dam closure to just below Clearinghouse Mine. A 50-foot high dam would impound water just upstream of the confluence with the South Fork and the South Fork would be minimally impounded in its lower reaches, forming a water body of 540 acre-feet. Under this scenario, Highway 140 and Incline Road would be submerged from the dam closure to the South Fork confluence.

The report does little towards describing the effects of dam overtopping, erosion through the blockage, and subsequent sediment transport dynamics downstream as floodwaters and dam material move through the system. The 100-foot high dam would require 6 hours to fill and overtop, while the 50-foot high dam would require 1 hour to fill and overtop. Once breached, flood waters would reach Lake McClure in 4.1 hours for the 50-foot high dam and 2.1 hours for the 100-foot high dam. The dam would mostly be composed of large angular blocks similar in size and shape to those in the talus cone currently protruding into the river and those in the lateral left-bank bar immediately downstream of the talus cone. Because of the coarseness of material, it is unclear how much material would be transported and distributed in the channel downstream of the dam breach and how much would be left behind rimming the channel and

valley walls. Significant reorganization of channel deposits downstream would occur, and it is likely that much of the granitic, rounded channel sediments would be temporarily replaced by the angular phyllite produced from the eroded dam. This would not only change the current configuration and character of the channel, but bar deposits would likely be much less mobile than they are currently.

2.2.2.5 Yosemite National Park

In June 2005, Yosemite National Park released their *Merced Wild and Scenic River Revised Comprehensive Management Plan and Supplemental Environmental Impact Statement*. While the revised and supplemental plan deals only with that portion of the Merced River within the jurisdiction of the National Park Service, it contains a wealth of information that provided our team with a solid understanding of natural and historic conditions within the headwaters of the Merced River. We also reviewed the report within the context of understanding how park staff reviewed proposed project alternatives in the plan with respect to the Wild and Scenic designation of the Merced River. This was completed to provide, at the very least, consistency in how our team assessed the proposed transportation alternatives again with respect to the Wild and Scenic designation.

2.3 Acknowledgements

We would like to thank Bob Ferguson and Jimmy Gado at Zephyr Whitewater Expeditions, James Eicher at the Bureau of Land Management, Jerry DeGraff of the U.S. Forest Service, and Bill Tucker of the Miwok and Paiute tribes for their candid discussions with project staff on the geomorphic history and dynamics of the Merced River at the project site. We would also like to thank Tony Cipponeri, Pat Teczon, and Ronald McGaugh at Caltrans for their assistance in providing the necessary data to complete our modeling and analysis. The help and support of these individuals facilitated a more rapid and efficient examination of the conditions at the project site in light of the proposed restoration alternatives.

3. ENVIRONMENTAL SETTING

The project site is uniquely situated in terms of its hydrography, climatology, and geology. The confluence with the South Fork Merced River is immediately upstream, precipitation and runoff exhibit distinct seasonality influenced by spring snowmelt and a California-Mediterranean climate, and local geology create an episodically unstable bedrock template through which boulders and cobbles – some glacially-derived – are transmitted in a deep, narrow canyon. We describe below, in more detail, the environmental characteristics of the project site which continue to influence the dynamics of the Merced River, and associated morphologic signatures along its canyon course.

3.1 Local Geology

The project site is located within a northwestern-trending structure, the Calaveras Complex, composed of a series of complexly deformed metasediments[6] with surfaces that predominantly dip to the northeast (Bateman, 1992). The structure is one of many that form the Sierra Nevada foothills, where eroding metamorphosed sedimentary rocks drape an uplifting and eroding fault block intruded by granitic rocks. The Calaveras Complex is bounded on the east, just upstream of the project site, by a regional shear zone called the Calaveras-Shoo Fly Thrust Fault (Beck, 2007a,b; Bateman, 1992). The shear zone separates rocks of the Calaveras Complex from the Pilot Ridge Quartzite, which is intruded locally to the northeast of the project site by the Bass Lake Tonalite, a medium gray, medium-grained granitic rock. These granitics supplement the predominantly granitic-boulder/cobble bedload of the Merced River derived from glacial deposits and eroded bedrock of the Sierra Nevada batholith[7] farther to the east in the Yosemite area.

The Calaveras Complex itself is composed of a dark, fine-grained phyllite[8], the Hite Cove Phyllite, and the Briceburg Phyllite, both of which exhibit inclusions of chert[9]. The latter crops

---

[6] The term 'metasediments' refers to metamorphosed sedimentary rocks, or sedimentary rocks (in this case, fine-grained rocks such as siltstones and shale) that have undergone deformation and partial re-mineralization by high temperatures and pressures associated with mountain building and regional tectonics in the Sierra Nevada system.

[7] A 'batholith' is a large body of intrusive igneous rock with an exposed surface area of more than 100 km^2 that develops in the subsurface and intrudes other local rocks through buoyancy, melting, and filling of voids opened by faulting and fracturing.

[8] A 'phyllite' is a fine-grained, foliated metamorphic rock with a silky, glossy, or silvery metallic luster and a wavy foliation with a wrinkled appearance.

out west of the project site, while materials at the project site are that of the Hite Cove Phyllite. Just downstream from the project site and the downstream temporary Highway 140 detour bridge, grain size and fabric within the phyllite transition to a more schistose[10] and gneissic[11] texture. Reconnaissance below the project site and along the Merced River demonstrates that this transition occurs at other locations, although the lithologic differences are not sufficient to permit drastically different styles in weathering, hillslope morphology, or river erosion.

3.2 The Ferguson Rock Slide

The Ferguson Rock Slide (**Figure 3.1**) is a complex hillslope failure that initially occurred on April 29, 2006 along the left bank side of the Merced River, just downstream of the northward bend that takes the river around Ferguson Ridge. The main body of the 1.5-2.0 million-cubic-meter failure is a rock-block slide with numerous internal slumps (Harp and others, 2006). An alternate estimate of slide volume is 700,000 cubic yards (535,000 cubic meters) – 650 feet wide by 1000 feet long by 90 feet thick (Beck, 2007b). Approximately 70,000 cubic meters of material, 100 meters long, forms a talus slope that presently buries Highway 140 and protrudes roughly 10 meters into the active width of the Merced River (Harp and others, 2006; Beck 2007b). This talus slope is still considered unstable and periodically produces small rockfalls as the toe area over-steepens, dilates, and fails (Beck, 2007b). The slide produced and continues to produce mostly large, angular, elongate blocks of phyllite that have accumulated along the talus slope, within the river channel, and slightly downstream along the left bank. Few blocks have been transported, either by rockfall or by the river, to the opposite bank and bar.

The present failure is the most recent in a sequence of downslope movements of appreciable geologic age that have occurred along strike within the Hite Cove Phyllite through which the Merced River continues to cut (Beck, 2007a,b; Harp and others, 2006). The hillslope and slide area morphology itself bear evidence of at least two prehistoric slides at the same location (Beck, 2007b). Other evidence of recurrent landslides at or near the present failure are (a) the

---

[9] 'Chert' is cryptocrytalline quartz with an opaque gray or white color and a waxy luster. It is mineralogically identical to quartz, but elemental impurities and mode of formation make it distinct.
[10] 'Schistose' refers to the texture of a schist, or medium- to coarse grained foliated metamorphic rock formed by intermediate-grade metamorphism of phyllite. The texture itself is defined by subparallel alignment of medium- to coarse-grained platy minerals, such as micas.
[11] 'Gneissic' refers to the texture of a gneiss, or medium- to coarse-grained foliated metamorphic rock with distinct layers or lenses of contrasting mineralogy. Generally, the light-colored layers are quartz and feldspars, which alternate with dark-colored layers of biotite mica and hornblende.

well-vegetated left bank bar immediately downstream of the Ferguson Rock Slide supporting willows at least 12-15 years old and composed of large angular slide debris with sand-blasted, rounded edges, (b) exposure of disorganized, matrix-supported and clast-supported slide breccia in the Highway 140 road-cut immediately upstream of the most recent failure, and (c) a depression in the bedrock surface above the roadway and upstream temporary bridge along the left bank (slide side) where material has failed leaving a void in the dip-slope surface roughly 5 meters wide by 30 meters long.

3.3 Physiography / Hydrography

The project site is in the Merced River canyon approximately 0.5 miles downstream of the mouth of the South Fork Merced River, a major tributary which has cut a valley with similar morphology to that of the main fork. The town of Mariposa is roughly 12 miles to the southwest, while El Portal is located approximately 7 miles east of the project site. Drainage area at the project site is 661 square miles (Cipponeri, 2007). Canyon walls are steep and generally mantled with a thin veneer of sediments that support mostly grasses on southern exposures and forests of oak, pine, and other trees on northern exposures. The Highway 140 roadbed elevation in the immediate vicinity of the project site is at an elevation of 1375 feet, while surrounding ridge tops approach 3000 feet (Beck, 2007a, 2007b).

The Merced River traces a relatively straight path from the Yosemite Valley area to the Central Valley, with the exception of the bend to the north around Ferguson Ridge at the project site. The deflection is the most prominent change in the river's course as far downstream as the head of deposition in Lake McClure. The channel planform is bedrock controlled, and significant bends are likely a result of active or inactive faults and knickpoints, and changes in structure and lithology.

The walls of the valley at the approximate level of the design flood are composed of (a) bedrock, (b) blast rock, (c) imported (and often cemented or fortified) rip rap, (d) vertically-cemented rock walls, and (e) slide debris and derivative talus, much of which appears to be angular rock clasts supported in a sandy to clayey matrix. All wall types listed are generally stable, with the obvious exception of slide debris.

3.4 Local and Regional Climate

The local climate at the project site is characterized by a wet winter season (October to March) during which approximately 90% of total precipitation falls, and a warm, dry season (May to September) with temperatures which can reach 100 degrees Fahrenheit. The average annual

maximum temperature is 69°F, while average annual minimum temperature is 34°F. Lows during the winter season are generally around 20°F and annual average precipitation at the project site is 37 inches. Mean annual rainfall for the entire Merced River watershed is 42 inches, although higher elevations receive closer to 60 inches annually in the form of snow, while lower elevation areas near the project site receive 37 inches.

4. HWY 140 PROJECT ALTERNATIVES ANALYZED BY PRESENT STUDY

The California Department of Transportation has proposed to restore freely-flowing traffic conditions along the 0.7 mile stretch of Highway 140 that was affected by the Ferguson Rock Slide of April 2006. Presently, the highway bypasses the Ferguson Rock Slide with a detour realignment that routes a single-lane roadway across the river to a roadbed originally used as a railway (Incline Road). Two temporary bridges facilitate the detour and reconnect the detour alignment with Highway 140 upstream and downstream of the slide deposit. The proposed transportation restoration of Highway 140 includes several alternative realignments and a *no-build* alternative, which are described below.

4.1 Alternative C (Open-cut Realignment)

Alternative C, or open-cut realignment, would bypass the Ferguson Rock Slide by realigning Highway 140 to the northeast. This would be accomplished by spanning the Merced River downstream of the rockslide, constructing an open-cut roadway across the mountain's nose above the right bank, and reconnecting with the original highway alignment via another bridge upstream. Two permanent bridges are included with this alternative. This alternative requires four bridge piers. This proposed alignment is shown in **Figure 4.1**.

4.2 Alternative T (Tunnel Realignment)

Alternative T, or tunnel realignment, would bypass the Ferguson Rock Slide by realigning Highway 140 to the northeast. This would be accomplished by spanning the Merced River downstream of the rockslide, tunneling 725 feet through the mountain's nose on the right bank side, and reconnecting with the original highway alignment via another bridge upstream. Two permanent bridges are included with this alternative. This alternative requires four bridge piers and retains the same proposed bridges and alignment as Alternative C. This proposed alignment is shown in **Figure 4.1**.

4.3 Alternatives T2 and T3 (Tunnel Realignments)

Alternatives T2 and T3, both tunnel realignments, would bypass the Ferguson Rock Slide by realigning Highway 140 completely underneath Ferguson Ridge for one mile for the former, or underneath the Ferguson slide mass and west canyon wall for the latter (**Figure 4.1**). These alternatives do not require bridges and do not impinge on the river corridor, and are thus not considered in our hydraulic modeling or geomorphic analysis.

4.4 Alternative S (Viaduct Realignment)

Alternative S, or viaduct realignment, would bypass the Ferguson Rock Slide by realigning Highway 140 to the northeast. This would be accomplished by spanning the Merced River downstream of the rockslide, constructing a hillside viaduct with retaining wall across the mountain's nose above the right bank, and reconnecting with the original highway alignment via another bridge upstream. Two permanent bridges and one new temporary bridge are included with this alternative. The new temporary bridge would replace the current upstream temporary bridge. This alternative requires four bridge piers. The proposed viaduct crosses the hillside on the right bank below the proposed open-cut alignment in alternative C, and the proposed bridges in this alternative require longer spans at more oblique angles to the Merced River's channel alignment. Additionally, the upstream proposed bridge crosses the river slightly below the proposed upstream bridges in Alternatives C and T. This proposed alignment is shown in **Figure 4.1**.

4.5 Alternative S2 (Viaduct Realignment)

Alternative S2, or viaduct realignment 2, would bypass the Ferguson Rock Slide by realigning Highway 140 to the northeast. This would be accomplished by spanning the Merced River downstream of the rockslide, constructing a hillside viaduct with retaining wall across the mountain's nose above the right bank, and reconnecting with the original highway alignment via another bridge upstream. Two permanent bridges are included with this alternative. This realignment differs from Alternative S in that the proposed upstream bridge crosses the Merced River at a more oblique angle, resulting in a longer span reconnecting with Highway 140 at PM 42.7 (**Figure 4.1**). At the time of the development of this report, structure design has not been developed, but information on the number and placement of bridge piers and abutments is available, and is similar to that of Alternative S. Our analysis for Alternative S2 relies on inferences based on potential geomorphic response from Alternative S and the No-build Alternative (below), as requested by Caltrans.

4.6 Alternative R (Rockshed Realignment)

Alternative R, or Rockshed Realignment, would cut through the Ferguson Rock Slide along the original trace of Highway 140 by utilizing a tunnel with rockshed through the slide debris. The rockshed would support slide debris and talus, preventing existing and future accumulations of slide debris from blocking the roadway. Because this alternative does not affect the river channel, our analysis does not include Alternative R.

4.7 Alternative A

Alternative A consists of a realignment similar to that currently comprising the temporary one-lane bypass, and completely spans the river like the temporary bridges currently in place. The proposed upstream bridge for this alternative will be slightly below (downriver) of the current temporary structure, while the downstream bridge for this alternative will be placed just upstream of the current lower temporary bridge. A significant portion (0.5 miles) of this bypass alternative consists of an at-grade roadway along the right (east) bank of the river on Incline Road. Structure designs for this alternative have not yet been developed, so the analysis of potential geomorphic effects from this alternative will rely heavily on inferences based on the No-build Alternative (see below).

4.8 No-build Alternative

The National Environmental Policy Act and the California Environmental Quality Act require a No-build Alternative, which would leave Highway 140 in its current alignment and state of damage from the Ferguson Rock Slide. The temporary detour, consisting of two temporary, one-lane bridges spanning the Merced River obliquely[12] to its channel alignment, would become the permanent Highway 140 alignment (**Figure 4.2**). The lower one-lane bridge spans the river by virtue of two concrete abutments resting on predominantly bedrock and road-bed blast materials of local rock. Rock gabion wing walls support peripheral bank materials surrounding either side of the concrete abutments. These abutments appear to interact little with the river as the channel in this location is a straight bedrock reach. However, the right bank abutment protrudes slightly onto a small surface only inundated during very large floods. The upper one-lane temporary bridge spans the river with two concrete abutments at either end, and one set of piers supporting the bridge roughly half-way across the channel. Rock gabion wing walls were also installed to support loose materials between the bridge abutments and Incline Road on the north bank and the Highway 140 roadway on the south bank. These piers rest on bedrock covered by a few meters of boulder- and cobble-bar material that are part of a large lateral bar-floodplain-terrace deposit along the right bank. These surfaces are described in more detail later in the report.

[12] Both the upper and lower temporary crossings were originally built perpendicular to the Merced River's channel alignment, but were revised with obliquely-oriented structures at the same locations during Summer 2008 to accommodate larger tour buses and vehicles longer than the previous vehicle length restriction of 28 feet.

5. TECHNICAL APPROACH OF PRESENT STUDY

Our analysis of potential river adjustment to the 6 proposed key transportation alternatives for Highway 140 at Ferguson Rock Slide relies on a strong mix of considering previously developed literature coupled with our multi-disciplinary, integrated analytical approach focused on geomorphology, hydrology, and hydraulics. Agency reports produced in rapid response to the Ferguson Rock Slide have provided the much needed background information from which we have built the present study. Specifically, the present study has been developed to highlight questions not yet touched upon, and provide the remaining physical information necessary to complete project environmental review. To that end, Chapter 5 is devoted to providing readers of this report with the details necessary to understand the technical and analytical approach which we have developed to address the project scope. We describe below, in extensive detail, the field methods, modeling techniques, and analytical approach that we have utilized in developing this comprehensive River Geomorphology Study.

5.1 Work by Balance Hydrologics (This Study)

We have developed a comprehensive, inter-disciplinary analytical approach to guide our river systems impact assessment of the 6 key transportation alternatives presented in Chapter 4. As such, our approach utilizes multiple lines of evidence to elucidate reliable and corroborated answers to the following key questions which are the focus of our scope or services:

- What constitutes the present-day geomorphic characteristics of the Merced River at the project site?
- How have floods, fires, and other large-scale forcing processes affected geomorphic characteristics and processes of the river at the project site?
- What are the estimated hydraulic characteristics of flood flows at the project site under existing river conditions?
- Under what hydraulic conditions will existing river morphologic features be mobile and subject to adjustment?
- Did the Ferguson Rock Slide affect river morphology and function, and how may subsequent re-activation phases interact with the river, and potentially implemented transportation alternatives?

We describe in detail in subsections below each method of analysis, and how the information it garnered was used in our evaluation. But first, we will provide a brief description of the activities associated with each of the field visits conducted by Balance geomorphologists and engineers.

5.1.1 Description of site visits conducted by Balance Staff

Geomorphologists and engineers at Balance Hydrologics visited the project site on four separate occasions. The first visit, on March 12, 2008, entailed an on-site discussion with Caltrans personnel in which Balance staff was provided a description of the project site, alternatives, project history and objectives, and slide history. During the latter half of this trip, Balance staff were able to conduct some initial site reconnaissance, which included qualitative surveys of slide and river deposits, initial floodplain surface dating using lichenometry and dendrochronology (described below), and photographic documentation.

Beginning on March 25, 2008, Balance geomorphologists spent two days at the project site conducting (a) detailed geomorphic mapping and semi-quantitative cross section surveys extending from a point roughly 500 meters upstream of the South Fork confluence to a point roughly 400 meters downstream of the lower temporary bridge; (b) bed-surface and bed-core sediment sampling and grain size analysis within the channel, on the floodplain, and in terrace deposits; (c) extensive surface dating using lichenometry and dendrochronology; (d) Quaternary landslide observations and geologic reconnaissance; and (e) identification and measurement of high water marks and other hydrodynamic flow features useful for hydraulic model calibration.

On April 10, 2008, staff engineers visited the project site to further identify and quantify hydrodynamic features and topographic conditions pertinent to modeling the project reach with, and without the proposed bridge structures.

Lastly, on July 9, 2008, a Balance geomorphologist accompanied personnel from Environmental Data Solutions to perform reconnaissance of the study reach in preparation for a Class I bathymetric survey (see section 8.1 and Appendix C). This visit coincided with removal of the temporary bridges which were oriented perpendicular to the river. Removal of these structures was completed by Caltrans and Teichert staff. These temporary bridges were immediately replaced with similar structures, but oriented obliquely to the river. This change facilitates use of the temporary bridges by longer vehicles. These activities permitted Balance staff to learn more about subsurface grain size distribution in point bars beneath the temporary bridges, and about the geomorphic history at the project site through discussions with a) Jerry DeGraff, a Forest Service geologist, b) Bill Tucker, a construction monitor for the Miwok and Paiute Tribes, and c) a foreman for Teichert (name not provided). In addition, water levels were appreciably lower during this visit compared with prior visits, which exposed more surfaces to observation and so allowed Balance staff to test geomorphic inferences based on prior observations.

5.2 Site-specific Geomorphic and Hydraulic Analyses

Our analysis of river geomorphology, and the potential for change or adjustment within the Merced River canyon near the Ferguson Rock Slide, consists of:

- Complete historic flow analysis of all locally available USGS gaging data including the development of flow peak predictors for the recent floods of 1997 and 2005;
- Field-based geomorphic facies mapping of alluvial deposits through the project reach in order to describe present-day river-bed architecture, and to correlate river form and depositional surfaces with fluvial processes;
- Field-based surface and near-surface sediment grain-size analysis to characterize the composition of alluvial deposits;
- Incipient motion or bed mobility analysis to highlight the potential for bed sediment movement and associated geomorphic change of alluvial deposits occurring within the project reach;
- Quantitative dating of flood deposits and geomorphically-significant surfaces utilizing lichenometry and dendrochronology with results placed in context of historic floods for results validation;
- Aerial photogrammetric analysis to assess the frequency of geomorphic change and/or resilience of channel features within the Merced River gorge in response to large-scale forcing processes (floods, fires, etc.);
- Field-, and literature-based geologic reconnaissance of the area local to the project reach to place the river system within the larger geologic construct including extrapolations of landslide activity through the project reach, based on published maps and professional studies; and
- Development of one- and two-dimensional hydraulic models to verify predicted water surface elevations and profiles against observed high water marks and simulate river hydraulics at design flows

The methods used for each type of analysis are described below.

5.3 Geomorphic Facies Mapping

The centerpiece of this report from a geomorphic perspective is our geomorphic facies map of the project reach. This map illustrates the suite of alluvial and colluvial[13] deposits that mantle

[13] The term 'colluvial' refers to hillslope deposits that are transported by gravity rather than by water flow, resulting in colluvium. Deposits left by water, or alluvial deposits, are called 'alluvium'. Here,

the bedrock floor and channel margins within the Merced River canyon. It depicts the relative age of each deposit, which was determined by stratigraphic[14] and onlapping[15] relationships observed on-site along with lichenometric and dendrochronometric sampling that provide an approximate absolute age. Lichenometry and dendrochronology are described in a subsequent subsection, and essentially permits a reasonable correlation between historic flow events and deposits left by, or modified by, those events. The geomorphic map can be viewed as a graphical compendium of the remnants of past fluvial activity combined, with a depiction of the mobile and ephemeral alluvial landscape through which the river flows. Or, in more general terms, it is a view of the ancestral Merced River superimposed by deposits and features left by recent activity.

Developing the geomorphic map in the field entailed walking the channel from a point roughly 500 meters upstream of the South Fork confluence to a point roughly 400 meters downstream of the downstream temporary bridge, with a detailed topographic base map in hand. We also walked approximately 400 meters up the South Fork to map and compare the alluvial deposits found there with those of the main stem. While walking down-channel, we identified different deposits and features and marked their location and extent on the topographic base map. The location of boundaries between map units were closely approximated based on features shown on our high-resolution topographic base map (generated by LIDAR and plotted with a contour interval of 2 feet), or by using triangulation and resection. Horizontal error in boundary placement is estimated at ± 2 feet near channel margins to ± 4 feet for channel bars where LIDAR could not resolve bedforms and topographic expression beneath the water surface during the fly-over survey. This mapping technique developed a palette of map units, or units discriminated by age and process-derivation, that are displayed by different color and pattern on the geomorphic map. We also identified and mapped the location of trees and lichen used for absolute dating, large woody debris (LWD) left by flood flows, sediment sampling sites

---

colluvial deposits include landslide deposits associated with Ferguson Slide and other identified historic and/or pre-historic landslide deposits.

[14] The term 'stratigraphic' is derived from the discipline of Stratigraphy, which studies the arrangement of rock strata in terms of geographic position within a landscape and the chronologic order or sequence of deposition.

[15] The term 'onlapping' refers to the overlap of layers of sediment characterized by regular and progressive pinching out toward the margins of the depositional body.

where grain size analyses were performed, knickpoints[16], riffles and pools, and cross section traces where we developed depictions of cross-valley surfaces and features to enhance the third (vertical) dimension of our characterization.

Deposits, or geomorphic facies[17], of different age and developed by different processes were identified based on grain size, texture, landscape position, degree and age of vegetation development, and topographic correlation across the channel. Our map units fall into 5 general categories:

- *Active channel and floodplain features* which are principally composed of granitic materials ranging from fine sands and silts to very large boulders derived from channel and moraine deposits upstream. These features may also contain minor amounts of phyllite fragments and boulders depending on proximity to landslide source areas and sediment transport conditions. This category includes (a) in-channel medial cobble/boulder bars, (b) sandy floodplain deposits and associated sandy flood-channel deposits on floodplain surfaces, and (c) longitudinal cobble-boulder bar and floodplain surfaces with associated lower-stage surfaces closer to the channel centerline and boulder ribs along margins next to the base-flow channel. On the geomorphic map we refer to 2-stage surfaces, which are the lower-stage (lower elevation) surfaces of longitudinal bar deposits (and of landslide deposits, see below) that have a clear topographic expression indicating that a particular flow regime is responsible for their form. Ages of active channel and floodplain features were determined, in part, by the age of willows and other riparian trees growing on these surfaces.

- *Abandoned and relict terraces* which are not generally active, but may be inundated by infrequent extreme events (such as flood flows in water years 1956 and 1997). This category includes older channel margin terraces composed of either cobbles and boulders or a mixture of sands with cobbles and boulders, and topographically-high gorge-wall terrace deposits that appear sporadically throughout the mapped reach and the greater Merced River gorge. These latter deposits are chiefly a grain-supported assemblage of well-rounded granitic cobbles in a matrix of sand and silt left plastered on gorge walls well above the Highway 140 roadway (approximately 15-20 feet in some locations; see **Figure 5.1**) and correlative positions on the opposite side of the channel. Channel margin terraces were often ornamented with trees of significant age that could

[16] 'Knickpoints' are linear, cross-channel topographic expressions of discontinuities in a channel's longitudinal profile derived from faults, changes in lithology and erodibility, or localized changes in sediment transport or storage.

[17] The term 'facies' refers to a distinctive sediment body (or sedimentary rock type in the practice of Stratigraphy) that is diagnostic of a depositional process and sedimentary environment at the field site. Many of the geomorphic facies we identify are derived from identical depositional processes, but are distinguished from one another based on age and their position in the landscape, which separates them functionally from other mapped deposits and surfaces.

be dated using an increment-borer and by measuring diameter of the trunk at breast height, and then applying dendrochronometric relationships. Channel margin terraces often also contained boulders with lichens that could be measured and aged using lichenometry.

- *Landslide deposits* which are identified based on their chaotic, brecciated texture, their topographic association with Ferguson Slide or older landslide scarps and hillside breccias, and their lithology, which is predominantly of local phyllite. Landslide deposits within the channel are mostly associated with ancestral and recent slide activity at the Ferguson Slide location, and form a large, reworked left-bank longitudinal bar just downstream of the present-day slide. Part of this deposit is reworked closer to the channel centerline, indicating that more frequent flows have been able to modify this surface. This lower-stage surface is identified based on its topographic expression.

- *Bedrock* which is chiefly a highly competent phyllite, although near the downstream extent of our mapping area the phyllite grades to a more gneissic texture (and we indicate this with a separate map unit). This category also includes phyllite rip rap derived from local blast material and debris left along channel margins to stabilize, protect, and support road beds (Highway 140) and abandoned rail beds (Incline Road). The erosion of bedrock is limited by the rate at which both fine and large particles transported by river flow can abrade surfaces and dislodge fragments. Phyllite rip rap is largely immobile except during very high flows that can overcome both the submerged weight of large blocks and the frictional resistance afforded by interlocking and/or partially buried angular protuberances of this material. It is distinguished from imported rip rap based on color, rock type, and angularity.

- *Man-made materials* of various degrees of stability, which include cemented (also called 'fortified') rip rap and cemented vertical rock walls, and imported unconsolidated rip rap (rock type not of local origin). Imported rip rap is grey to blue in color and generally smaller and less angular than the locally-derived phyllite rip rap and blast rock.

5.4 Sediment Grain Size Analysis

Sediment grain size analysis was performed to assess the size distribution of sediments deposited on the bed and along channel margins within the project reach. This information was then fed into incipient motion and scour formulae (described in the next subsection) to calculate approximate flow conditions under which the channel substrate will begin to move. Grain size information also provides a greater understanding of flow history and background river hydraulics through characterization of depositional processes. We focused our grain size sampling protocol at select locations within the project reach that corresponded to areas at, and adjacent to, the locations of bridge piers in the proposed alternatives described in Chapter 4. Based on our detailed observations and geomorphic mapping, we would suggest that grain size data obtained from sampling locations serve as a proxy for describing similar surfaces throughout the project reach (portrayed on the geomorphic map). We feel this statistical assumption is valid because (a) our grain size sampling at specific locations was dense (three

transects at each of four sites with 100 counts per site), (b) lithology and river sediments are somewhat homogeneous throughout the project reach, (c) lithology and river sediments upstream of the project site and in the South Fork appear analogous to those in the project reach, (d) river sediments are derived from an appreciable source of moraine sediments upstream that is not yet exhausted, and (e) sampled surfaces were not influenced by localized sources of slide material or imported rip rap.

Caveat 'e' above requires specific attention and discussion because one large longitudinal bar located along the left bank just downstream of the Ferguson Rock Slide is of importance to project reach morphology. This bar is presently being supplemented by material from the most recent slide of 2006, and represents a surface upon which to place a left bank bridge pier in the proposed alternatives. Because this bar supports mature willows, as well as other riparian trees and shrubs, transect sampling would have been very difficult. More importantly, we chose not to sample this bar because it appears relatively stable (based on degree of vegetation development, soil formation, rounding of once-angular phyllite blocks, etc.) and would appear to be only marginally mobile in extreme events. The older slide blocks that comprise this bar are generally far larger and more angular (and hence have higher friction angles [defined in the next subsection] and are more well-packed) than the granitic particles that characterize most mobile surfaces in the project reach that were sampled. We provide discussion based on observations outside of grain size sampling on the relative mobility and geomorphic history of this bar in Chapter 6.

The grain size analysis consisted of three styles of field sampling: (a) pebble count surveys along linear transects over bar and floodplain surfaces; (b) point-sampling of bed surface samples on bars and within the submerged channel (base flow during the time of sampling); and (c) point-sampling of subsurface (bed core) sediments on bars and within the submerged channel. Pebble count surveys and point sampling were performed at the following locations:

- Along the left bank longitudinal bar upstream of the upstream temporary bridge;
- Along the right bank point bar immediately upstream and immediately downstream of the upstream temporary bridge (the point bar extends beneath the bridge);
- Within the submerged channel along the right bank margin adjacent to the point bar immediately downstream of the upstream temporary bridge; and
- Along the point bar across from the Ferguson Rock Slide that has recently been separated from the right bank.

At all locations except the point bar across from the Ferguson Rock Slide, we sampled surface sediments at regular intervals along three 200-foot transects stretched parallel to the flow direction. The point bar opposite the slide could only accommodate 2 transects because of its narrow width. Transects were generally separated by approximately 20 feet and were designed to capture any lateral differences in substrate between the base flow channel, any 2-stage surfaces (if present), and floodplain surfaces. Transects did not cut across different surfaces. Traditional *Wolman* pebble counts require that sampling occur with eyes averted so that the sampler does not bias the process through selective sampling. Because all surfaces at the project site exhibit boulders and uneven walking surfaces, we chose to select grains that rested beneath regular intervals along the taped transect. Intervals were far larger than grain diameters encountered along transects, preventing size bias at the upper end of the grain size spectrum. Grains were sampled along each transect and measured with ruler and tape to half phi-size[18] increments.

Point sampling on subaerial and submerged surfaces consisted of placing a 1-meter square grid on the surface and sampling all particles within the grid square at the surface. Once surface particles were removed, subsurface particles were measured, or if the subsurface consisted of a large proportion of fines (gravels and finer, indicating a well-armored surface), an estimate of the modal percentage of fines was made relative to the remaining large particles. For samples obtained in the submerged channel, the sampling grid floated in place on the water surface, held static by rebar pins. Care was taken to prevent river flow from biasing the procedure by eroding fines during sampling. In some cases, this required creating and installing flow obstructions. Sampled particles were placed downstream of the sampling grid after removal and measurement.

5.5 Sediment Transport

We have used a bed sediment mobility analysis to evaluate how the different transportation alternatives may affect sediment mobility relative to existing conditions. Enhanced mobility under any transportation alternative could lead to adjustment of the existing bed morphology

[18] The 'phi scale' applied to grain size distributions of riverine (and other) sediments is based on factors of two, a scale which appears to reflect the natural distribution of sedimentary particles. This distribution is therefore logarithmic with base '2', and the Greek letter 'phi' is used as the unit for this scale: $\varphi = -\log(\text{grain diameter in mm})$. The negative is used because it is convenient to represent grain sizes on a graph as decreasing from right to left.

through truncation of bars, to deepening of pools. This analysis has been carried out by determining the dimensionless critical shear stress of the median grain size (D_{50}) which is likely to be mobile under the different flow and transport conditions. We have used our hydraulic modeling results (described below) to generate discrete distributions of boundary shear stress across the study reach, which we then substitute into the standard incipient motion relationship to solve for a critical median sediment diameter:

$$D_c = \frac{\tau_0}{\tau_c^*(\rho_s - \rho)g}$$

D_c is the critical median diameter of sediment that would mobilize under the applied boundary shear stress τ_o, given *a priori* values for the Shields' parameter (dimensionless critical shear stress), τ_c*, and density of sediment, ρ_s, and water, ρ. (We assume a density of 2.65 g/cm^3 for sediment, appropriate for the abundant quartz-rich granitic sediment supply of the Merced River in this vicinity; gravity is standard at g = 9.81 m/s^2.) If the calculated critical median sediment diameter is appreciably greater than the median diameter measured in our grain size pebble counts, then the alluvial surfaces characterized by such a grain size distribution are likely to be mobile under modeled conditions. Otherwise, a critical diameter less than that measured would suggest bedform and/or channel margin stability under the flow regime generating the critical shear stress applied. Our analytical model calculates critical shear stress for input into the incipient motion relationship using flow velocities and channel geometry output from the associated hydraulic model:

$$\tau_0 = \frac{\rho \bar{u}^2}{\left[5.75 Log\left(12.27 R/k_s\right)\right]^2}$$

where $\bar{u}$ is the depth-averaged flow velocity and R is the hydraulic radius (defined as the cross-sectional area, A, divided by the wetted perimeter, P; here we use flow depth as an approximation to the hydraulic radius). The term k_s is the boundary roughness length scale, which is generally greater than D_{50} (Buffington and Montgomery, 1997); here we use k_s = $3.5*D_{84}$, consistent with HEC-18 (Richardson and Davis, 2001) and others (e.g., Whiting and Dietrich, 1990), where D_{84} is the sediment diameter below which 84% of the sampled grains are finer. The relationship for the boundary shear stress is based on moment stability analysis for a cohesionless non-uniform grain size distribution resting on inclined beds and embankments under hydraulically rough flow conditions. For rough turbulent flow, the commonly quoted value for the Shields' parameter is usually between 0.045 to 0.06 (Buffington and Montgomery, 1997), but we will apply a value of 0.03 to be consistent with the guidance for scour assessment under bridges provided in HEC-18 for gravel- and cobble-bedded rivers (Richardson and Davis,

2001). Model runs under natural conditions (no bridges) produce critical boundary Reynolds numbers well within the range of hydraulically rough conditions, supporting the use of dimensionless critical shear stress values at the lower end of the spectrum of published values for hydraulically rough conditions. Additionally, the use of a lower Shields' parameter is appropriate given that 1) larger bed surface grains, like those in the study reach, may have lower mobility thresholds because of greater grain protrusion and lower friction angles (Buffington and others, 1992), 2) poorly sorted and mixed grains tend to have lower friction angles and thus lower incipient motion thresholds (Buffington and others, 1992; Buffington and Montgomery, 1997), and 3) Shields' parameters developed in mixed-grain studies (real-world streams) tended to generate lower incipient motion thresholds than those from flume studies with uniform grain sizes (Buffington and Montgomery, 1997).

To complement the incipient motion studies applied to bar surfaces in the project vicinity, we also conduct bridge-scour calculations provided by HEC-18 (Richardson and Davis, 2001) and by the ASCE in their manual on toe scour evaluation. Scour analyses recommended by the ASCE include Blodgett (1986), Lacey (1930), Blench (1969), and Competent/Limiting Velocity Control (Pemberton and Lara, 1984). A listing of these equations, along with definitions and derivations for their input parameters, is given in **Appendix A**.

5.6 Deposit Dating

Measurable changes within a channel are typically associated with periods of large-scale river bed erosion or deposition. These processes can be caused by events such as rain-on-snow floods, wild fires, or other significant hydrologic events. To better understand the frequency and magnitude of channel-changing events we used Quaternary dating methods to construct a history of depositional features found throughout the project reach. Two simple methods employed in this study include (a) dendrochronology and (b) lichenometry. Below, we define these two methods and describe how they were used.

- *Dendrochronology:* Young vegetation tends to colonize floodplains and bars after a flood, only to be scoured and removed during subsequent floods. Willows are typically the first to colonize along the Merced River after major floods. Coniferous trees tend to occupy less disturbed areas above the active channel. We selected some well-known species that exhibit annual growth increments (in cross-section) to give minimum ages of depositional surfaces—a technique known as dendrochronology. Balance collected core samples from pines and oaks on defined terraces by using a 2-foot long, 0.5-inch-diameter increment-borer (see **Figure 5.2**, **panel A**). Young willows on lower floodplains and bars were felled near the base to expose a cross-section through the trunk. Samples were cleaned and tree-rings were counted to establish minimum ages of the depositional surfaces upon which they grew. The diameter (of the trunk) at breast

height was also measured to corroborate our age determinations using tree-ring counts. This method assumes that the trees sampled had established themselves within a year of the formation of the surface upon which they grow, or grew, in the case of felled willows.

- *Lichenometry:* Several former floodplain deposits or terraces along the Merced River, at or near the project site, were dated using a combination of dendrochronology and lichenometry. Typically, once a floodplain is abandoned and left undisturbed (as a terrace), boulder surfaces can be colonized by lichens. Lichenometry is a method of banded-age dating that uses the size of lichen colonies on a rock to determine the surface's age.

- The two most common assumptions employed in using lichenometry are: 1) lichen growth is indicative of elapsed time since deposition, and 2) the largest lichen (by diameter) generally represents the oldest and fastest growing lichen on a substrate. The exposed surfaces of partially buried boulders on Merced River terraces are colonized by several species of lichen including *Rhizocarpum geographicum*. Using the second assumption above, Balance geomorphologists identified *R. geographicum* on at least two terrace deposits near the project site[19] and measured multiple individuals on boulders of similar lithology (usually granitic boulders) and identified the maximum-diameter individual for analysis (see **Figure 5.2**, **panel B**). We compared our results to published literature and size-growth rate relationships to estimate a range of plausible ages of the lichens, and hence, the age of the terrace surfaces (Table 2**; Figure 5.3**). This method assumes that rocks sampled for lichenometry possess lichen individuals that began their growth within a year or so after the rock, and geomorphic surface, was originally deposited. As such, this method provides a minimum absolute age of a geomorphic surface; reworking of the surface's substrate may allow for lichen individuals to become established after the surface originally formed, or, lichen individuals may not have began growing within a year after surface formation.

5.7 Aerial Photogrammetric Analysis

We examined a number of historical aerial photographs (1945, 1955, 1973, 1982, 1988, 1993, and 1998; listed in detail below, with available aerial photographs listed in **Appendix B**) in order to understand historic channel dynamics at the project site through time. We evaluated channel planform adjustments, vegetation changes, and the position of bars, riffles, and pools relative to major flood events (including rain-on-snow events) to assess the effects of major flows on sediment movement and planform adjustment. Our photos were obtained from USGS EarthExplorer as .TIF files, and include the following:

[19] Several Balance geomorphologists have experience with the identification and measurements of *R. geographicum* in Alaska and British Columbia.

- 1998 (August 20) 3.75-min b/w Kinsley, SE corner coordinate 37°37′30.00″N, 119°52′30.00″W
- 1993 (July 16) 3.75-min b/w Kinsley, SE corner coordinate 37°37′30.00″N, 119°52′30.00″W
- 1988 (July 14) 1:32,000 vertical reconnaissance CIR, center coordinate 37°39′59.55″N, 119°55′05.44″W
- 1988 (July 14) 1:32,000 vertical reconnaissance CIR, center coordinate 37°40′51.61″N, 119°49′42.42″W
- 1982 (July 14) 1:31,133 vertical reconnaissance CIR, center coordinate 37°40′51.10″N, 119°51′53.99″W
- 1982 (July 14) 1:31,133 vertical reconnaissance CIR, center coordinate 37°39′06.10″N, 119°51′52.28″W
- 1973 (June 29) 1:32,000 vertical reconnaissance CIR, center coordinate 37°39′23.52″N, 119°53′12.89″W
- 1955 (Sept. 1) 1:47,000 vertical cartographic b/w, center coordinate 37°42′01.49″N, 119°53′31. 07″W
- 1955 (Sept. 1) 1:47,000 vertical cartographic b/w, center coordinate 37°40′02.80″N, 119°51′45.18″W
- 1955 (Sept. 1) 1:47,000 vertical cartographic b/w, center coordinate 37°39′35.36″N, 119°53′24.67″W
- 1945 (Dec. 15) 1:39,200 low oblique b/w, center coordinate 37°39′15.02″N, 119°53′10.35″W

5.8 Quaternary Landslide Observations

One of the tasks of this study is to compare potential adjustments of the Merced River, due to the proposed transportation alternatives, to that which could be due to the effects of slide debris runout into the river by the Ferguson Rock Slide. There is also evidence, both in the channel and in road cuts, that Quaternary[20] landslides in the vicinity of the project site are not restricted to the Ferguson Rock Slide (Beck, 2007a; 2007b). As such, we examined the age, perceived

[20] The 'Quaternary' period, as defined by the geologic time scale, extends from the present day back to 1.8 million years ago. The term 'Quaternary' is often used to refer to the most recent geologic activity pertinent to present-day deposits and morphology, and is commonly used in geomorphic and neotectonic circles when describing surface expression and hazards relevant in environmental planning.

stability, and extent of Quaternary landslide deposits in the project reach that have affected valley or channel morphology. In some cases, landslide materials are evident in the channel, whereas in other cases landslide debris is absent along channel margins below scarps or valley-wall breccia deposits. We have included these landslide deposits and scarps on our geomorphic map, and in Chapter 6 we discuss the importance of these features relative to channel forming processes.

5.9 Geological Reconnaissance

Because the Merced River's form through the project site is not only a reflection of localized sediment transport and storage processes, but also a reflection of the regional geologic character and structure, we examined regional geology of the area using published geologic maps (Strand, 1967; Bateman and Krauskopf, 1987; Bateman 1992) and through on-site reconnaissance. Our on-site reconnaissance supplements published geologic maps by identifying unmapped, fine-scale changes in mineralogy, lithology, and structure across the project reach. We also examined potential correlations between these fine-scale changes and alterations in grade, planform, and hydraulic geometry in the project reach. River form and dynamics is also often a product of processes and conditions originating far upstream in the contributing watersheds, which are frequently referred to as cumulative effects. It is clear that sediment transport and storage conditions at the project site are a product of the granitic materials derived from lithologies and glacial deposits that are 6 miles or more up-river. Thus, examining these cumulative effects from a geologic standpoint in a largely unregulated catchment required study of geologic materials in the upper watershed. We utilized the geologic map of Strand (1967) to gain this regional perspective. Our discussion of the effects of local and regional geology on river form and process in the project reach are provided throughout Chapter 6.

6. GEOMORPHIC ANALYSES AND RESULTS

6.1 Planform Dynamics (Aerial Photogrammetric Analysis)

A sequence of aerial images of the project reach illustrating geomorphic change during the past 7 decades is provided in **Figure 6.1**. Based on these photographs and our historical interpretations, we provide the following observations of historical planform dynamics in the project reach:

Visually, the main stem channel exhibits different bedform patterns (i.e., riffle, bar locations) between the confluence and the Ferguson Rock Slide (approximately 5,000 feet of channel) relative to channel reaches upstream and downstream of it. This is typical for confluence zones. The South Fork appears to contribute considerably volumes of sediment. Point bars are much more developed and dynamic immediately downstream of the confluence. These features shift, augment, and scour in the most significant events (i.e., 1938, 1950, 1956, and 1997), but largely retain the overall form of the channel through the project reach. A contributing factor in sediment availability between the two watersheds may be physiography; while the geologies of both the main stem and the South Fork are roughly similar, their valley shapes differ significantly. For example, the size and characteristics of Yosemite Valley on the main stem suggests the Valley may retain much of the sediment that originated in the upper reaches of its watershed. Alternatively, the South Fork is a much steeper and narrower valley with very few opportunities for sediment storage; thus, most of the sediment entering the South Fork is likely transported to the main stem.

Although the project site is located within a transitional reach where active adjustment due to South Fork contributions occurs, historical photos dating as far back as 1945 suggest *little change* in the overall planform of the river within the project reach. The valley topography and bedrock exposure constrain horizontal channel migration. The channel exhibits a single-threaded planform with alternating riffle and pool sequences. Given what we know about the size of bar materials (*i.e.*, cobble and boulder sized), the riffle and pool locations migrate or re-organize only after significant rain-on-snow events—flows which are much larger than those annual peak flows derived solely from snowmelt. For example, upstream of the proposed upstream bridge alternatives, the large flood of 1950 appears to have shifted the riffle transition downstream from pre-event conditions, and changed the bar characteristics as well (comparing 1945 and 1955 photos). Also, the 1956 flood and to a lesser extent, the 1965 flood, likely influenced bar bifurcation at the same location (1973 photo).

The bar located along the left bank (looking downstream), immediately downstream of the current Ferguson Rock Slide, appears unchanged throughout the last 63 years. Field observations suggest this bar is derived from former slide deposits, and boulder size and angularity may provide stability against transport. Furthermore, tree abundance and size suggest and corroborate the notion of relative bar stability during the photographic record. It remains to be seen how the most recent activation of the Ferguson Rock Slide influences future stability of this bar.

It is important to note that the 1997 rain-on-snow event may have driven the most adjustment within and of the river corridor over the record of aerial photographs reviewed. The 1998 photograph suggests many of the point bars were scoured of vegetation with potentially new bar augmentation. In addition, a bar located opposite the Ferguson Rock Slide, which historically had bifurcated flows to either side (Ferguson, B., pers. comm.., 2008), became a zone of large-scale flood sedimentation as following the 1997 flood the bar was large enough that it forced flows toward the rockslide, and reduced boaters access downstream.

6.2 Quaternary Geomorphic History

Plate 1 illustrates the latest Quaternary geomorphic history of the Merced River at the Ferguson Rock Slide; the detail offered by **Plate 1** is supported by three representative and conceptual cross sections through the reach which are intended to add detail to the vertical dimension (**Figures 6.2, 6.3, 6.4**). The legend lists the various map units described in Section 5.3, each accompanied by a different map pattern and/or color.

6.2.1 Landslide deposits

Mapped deposits in this category include ancient and recent landslide deposits, and 2-stage surfaces formed atop landslide deposits left in the canyon bottom. We were able to identify three ancient landslide deposits in the project reach, all upstream of the Ferguson Slide. Proceeding upstream from the Ferguson Slide, a large landslide deposit of significant but indeterminate age drapes local bedrock immediately adjacent to the current Ferguson Slide mass (**Figure 6.5**, panel A). The slide is most evident in exposures left by the Highway 140 road cut along the left bank, where both matrix-supported and clast-supported deposits of local phyllite lay unconformably over wall rock. The deposits are heavily vegetated, and extend a significant distance upslope, inferred by hillslope morphology. The reworked landslide deposits in the channel immediately below the Ferguson Slide (mapped with a 2-stage surface; see **Figure 6.6**, panel A) are likely products of this ancestral slide, although recent and minute contributions from the Ferguson Slide have been added to the upstream end of the deposit

body. Based on the size (and age) of hillslope trees growing on the slide mass upstream of the Ferguson Slide, the age of the unit is at least 100 years (it is likely much older, possibly by an order of magnitude or more), which sets a limit on the transport rates of coarse, angular slide material that enters the channel system. The stability of the 2-stage landslide deposit bar left immediately downstream of the Ferguson Slide is both corroborated and afforded by its texture, degree of soil development, and vegetative cover. The deposit, which forms an episodically-inundated bar along the left channel margin, consists of large, angular phyllite clasts with sporadically well-developed soil patches and willows that are approximately 12 – 15 years old. The infrequent movement of this deposit has allowed soil development and sustained vegetative growth, both of which enhance resistance to flow during large runoff events. As previously mentioned, the bar includes a 2-stage surface, with morphology suggestive of two distinct flow regimes that inundate the bar. One is a low flow regime, exemplified during our visit on March 25-26, 2008 of approximately 500-600 cfs (estimated from real-time flow data from the USGS South Fork and Pohono gages during the days and hours immediately preceding the field excursion), which percolates through and over rocks at the upstream margins of the 2-stage surface. Larger flows, equivalent to those that would just fill the channel above the elevation of local bar deposits likely reworks the upper, more downstream portion of this landslide deposit bar.

Approximately 2000 feet upstream, just below of the South Fork confluence, another ancestral slide covers an extensive portion of hillslope above Highway 140 on the south side of the channel. Again, the antiquity of the slide mass is judged on the extensive vegetative cover and on its texture, which is similar to that of the ancestral slide immediately upstream of the Ferguson Slide. Breccia deposits from the slide are clearly visible in the Highway 140 road cut. Unlike downstream slide masses, there is no evidence of reworked slide debris in the channel associated with this mass movement, an observation that may also attest to its appreciable age.

A shallow-seated slide of limited area can be found on the right bank margin just upstream of the South Fork confluence (**Figure 6.5**, panel B). Unlike other slides in the study reach, the slide scarp on the hillslope has been evacuated of all colluvium, leaving an oblate bedrock scar above the channel. Debris from this slide has accumulated along the right bank channel margin, partially intruding the low flow channel but more dominantly covering Incline Road and the steep channel bank. Slide debris is not vegetated and is highly angular, suggesting a relatively young landslide deposit.

6.2.2 Stable channel banks

Stable channel banks, in the form of local bedrock, blast material derived from local sources, imported rip rap, cemented rip rap, and rock walls can be found at various locations throughout the project reach. Where located, cemented or fortified rip rap predominantly supports the outside bends of meanders on the left bank, while locally-derived and imported rip rap stabilize point bars and cut banks on both sides of the river. There is only one vertical rock wall in the project reach, just downstream of the large upper landslide deposit on the south side of the canyon. The wall supports a steep cut bank just below the Highway 140 roadway. Both Highway 140 and Incline Road are frequently supported by blast material derived from local rock (phyllite) that was excavated and moved towards the channel margins. This material is clast supported and is composed of generally very large boulders with interstitial fill of varying texture.

A history of channel adjustments based on relative ages of these varying types of stable materials is difficult to ascertain. We assume that the imported rip rap, and possibly the fortified rip rap (but not the masonry-rock wall), post-date the local blast material (coeval with road construction). This suggests that the bar and cut bank on the right-bank in the vicinity of the South Fork confluence, along with the cut banks on the south side of the river below the South Fork confluence, have all experienced some planform adjustment in response to large flow events. Historically-recent intense storm events likely caused the destabilization along the north-bank point bar and above Incline Road across the channel from the South Fork confluence, leading to the small slide found there, which covers imported rip rap along the channel banks.

6.2.3 In-channel features

In-channel features consist of medial cobble and boulder bars and riffles. Medial bars were difficult to exclude from the general categories of riffles and longitudinal point bars (along channel margins). This is because the impression of a medial bar during low flows can easily become a riffle feature at higher flows. Additionally, many of the medial bars were near point bar deposits, but were separated from them by chutes or alternate flow paths distinct from the main flow cell. However, the medial bars mapped in Plate 1 had vegetation growing on them (establishing them as bars rather than exposed riffles) and a longitudinally-graded surface rather than a laterally-graded surface (suggestive of a medial feature rather than a disarticulated point bar). The medial bars appear to be of two types. In one, the bar forms an elongate, tear-drop shaped deposit tapering downstream. These types are generally one channel length wide and are closer to one channel bank than another, and are usually

associated with the downstream margins of point bar deposits. Because of this latter observation, they are likely depositional locales created by a surfeit of sediment supply that overwhelms the storage capacity of the associated point bar. Alternatively, they may be derived from reworked point bar deposits translated downstream during large floods. There are two medial bars of this type within the project reach: one upstream of the South Fork confluence and one downstream. The second type of medial bar tends to inhabit most of the channel width as a marginally subaerial feature composed of cobbles and boulders and stretching downstream for approximately 2-3 channel widths. These features also appear to be associated with excess sediment supply. One is located at and just downstream of the upper ancestral landslide deposit just below the South Fork confluence, while the other is located at the downstream margin of the large left bank point bar above of the upper temporary crossing. The former is most likely associated with reworked slide deposits or contributions from the South Fork that have not yet reached equilibrium, while the latter is likely a consequence of redistribution of the point bar deposit as it translates downstream.

Riffles in the project reach are spaced at relatively regular intervals in areas other than those where medial bars of the second type mentioned above are found. The spacing is roughly 500 feet, while the riffle is usually slightly shorter than 500 feet, except where they join medial bars or are affected by recently introduced material from the Ferguson Slide. At the large left-handed bend upstream of the upper temporary crossing, a knickpoint appears to control the location of a riffle head with a noticeable drop in grade. A bedrock step protruding from the low-water surface creates the knickpoint, and is coincident with what appears to be an unmapped fault line extending up the drainage on the hillslope to the north. A similarly exaggerated step, possibly fault-controlled, heads the first mapped riffle in the lower reaches of the South Fork. The remaining riffles in the study area are geomorphically controlled.

6.2.4 Channel margin deposits

Channel margin deposits consist of older terrace deposits, ancient gorge-wall terrace deposits, floodplain deposits (including flood channel deposits within other surfaces), longitudinal bars and associated boulder ribs, and 2-stage bar surfaces. Ancient gorge-wall terrace deposits consisting of grain-supported, well-rounded granitic cobbles in a matrix of sand and silt have been left plastered on gorge walls well above the Highway 140 roadway (approximately 15-20 feet in some locations) and correlative positions on the opposite side of the channel. However, these deposits have only been mapped in one discrete location within the project reach above the Highway 140 roadway upstream of the upper temporary crossing (see Plate 1 and **Figure 6.5**).

Rather than systematically describing the locations where the other channel margin deposits are found within the field area, we present a description of 4 point bar deposits and one cut bank deposit that dominate the in-channel depositional suite and that will be important in assessing pre- and post-project geomorphic change. Starting downstream, the first point bar deposit of importance is directly across the stream from the Ferguson Slide (see Figure 6.7). It consists of a partially-armored cobble and boulder longitudinal bar bordered by a subsidiary channel along the right bank margin and intact bedrock at Incline Road. The bar is primarily reworked glacial deposits from upstream rather than slide debris, and appears armored at the downstream right-bank edge. The bar deposit was heavily modified during flood flows of 1997 that reorganized once channel-wide distributions of material that originally formed three separate raftable routes through the reach (called "Let's Make a Deal"; J. Gado and B. Ferguson, pers. comm.., 2008). Reorganization in 1997 carved a single route through the debris, depositing and supplementing material at the right bank (J. Eicher, pers. comm., 2008). Exhumation by a subsidiary chute channel along the right bank bedrock margin occurred in 2005, with greater incision occurring through this new channel as flows were impinged by Ferguson Slide material from the left bank after 2006. Continued exhumation of the chute channel has exposed a body of sediment approaching 2-3 meters thick at the downstream end, showing abundant clast-supported cobbles and boulders with interstitial sand which falsifies the apparent armoring observed on the bar surface. According to local boaters and resource agency staff, this is the only bar surface that was modified in the project reach during the 1997 flood (J. Gado, B. Ferguson, and J. Eicher, pers. comm.., 2008).

The second point bar deposit of importance is along the right bank at the site of the current upstream temporary crossing (see **Figure 6.6**, panel B). It consists of 2-stage longitudinal bar and upper bar surfaces extending from the water's edge to approximately 40 feet below Incline Road and an older terrace deposit just below Incline Road, consisting of cobbles and boulders along most of its length and a small, but homogeneous, sand deposit at its downstream margin. The terrace deposit just below Incline Road is largely inactive, and only experiences fluvial activity during the most extreme events. Several galleries of oaks and pines of approximately 40+ year's age have colonized this surface, and organic debris has began mixing with silts and sands from floods to create patches of partially soil mantled bedrock. The longitudinal bar surface below the terrace consists of a flight of surfaces that correspond to morphological adjustment at two separate flow frequencies. The lower surface, mapped as the 2-stage surface, appears to be inundated perhaps annually, while the upper bar surface appears to be inundated less frequently, perhaps every 3 to 5 years. What is important about these surfaces is that they are 1) inundated annually to super-annually, 2) composed of cobbles and boulders that are

sporadically imbricated (see **Figure 6.6**, panel B), and 3) only adjusted morphologically by large events (see incipient motion calculations below).

Just upstream of the upper temporary crossing on the left bank is an extensive and complex point bar and associated terrace deposits (Figure 6.8). Older sandy terrace deposits border Highway 140 just below the roadway around the apex of the bend. Below the terrace deposit by approximately 10-15 feet are longitudinal bar deposits with associated 2-stage surfaces, sculpted locally by sandy floodplain deposits and flood-channel deposits. The downstream end of the complex is bordered with a boulder rib at the water's edge at low flows. Two reentrants just downstream of the bend's apex created during flood flows introduce sands and rising flood waters to the left margin of the bar-complex against the toe-slope of the Highway 140 roadway. These flows have produced discontinuous channel "stringers", which are depicted in Plate 1 as narrow, yellow, cross-hatched units just below Highway 140 downstream of the bend apex (also see **Figure 6.8**, panel A). The lower 2-stage longitudinal bar surface has a similar morphology along the lower margins of the bar-complex, where an upstream 2-stage surface protrudes into the bar surface towards the left bank margin. The 2-stage and associated bar surfaces in this bar-complex are correlative with analogous surfaces within the point bar just downstream on the opposite bank at the upper temporary crossing.

At the next bend upstream a small point bar exhibits a combination of sandy floodplain deposits near the water's edge and more marginal cobble and boulder longitudinal bar deposits that extend far upstream along the right bank as a narrow body of coarse material. At the upstream extent of this narrow ribbon of coarse sediment but across the channel and just downstream of the South Fork confluence exists a substantial suite of deposits left along the outside bend of the river. This suite consists of an older sandy terrace deposit just below the Highway 140 roadway, and lower longitudinal bar, 2-stage, and sandy floodplain deposits. It is unclear if these 2-stage and/or terrace deposits correlate with those mapped down- or upstream. The 2-stage and longitudinal bar surfaces along the right bank upstream of the confluence likely correlate, but the relationship was largely untested. The cut-bank deposits represent a considerable amount of potentially mobile debris, although thick riparian vegetation (mostly willows) has apparently stabilized this body of material for at least a decade, based on the approximate age of willows on the upper terrace and portions of the floodplain surface. Additionally, partially rotten large woody debris is trapped within the thick riparian stand, indicating a modicum of stability and retention of materials deposited there. This is also a likely depositional locale for excess sediment debouching from the South Fork, and essentially forms an extension of the point bar deposit at the mouth of the South Fork.

6.2.5 Large woody debris

Numerous pieces or accumulations of large woody debris (LWD) exist in the project reach (**Figures 6.9** and **6.10**). Forty instances in all were mapped, but most are individual pieces rather than debris accumulations. Pieces are predominantly aligned with the local flow direction with rootwads, if present, pointing upstream. All documented pieces are less than one channel width in length, which reduces their potential to create blockage at proposed crossings. Only three pieces were found on active bar surfaces near the channel center line, while five more pieces became trapped within the thick vegetation on the 2-stage landslide deposit below the Ferguson Slide. All other pieces were located along channel margins near thick stands of riparian vegetation and trees. Although wood is an important component of the material transported through the project reach, it appears to have a subsidiary rather than dominant role in developing channel morphology.

7. HYDROLOGIC ANALYSES AND RESULTS

7.1 Available Gage Data

A total of 5 gages relevant to the project were identified and evaluated. General information about these gages and their records is presented in Table 1. Flow records are available for the Merced River at both the Pohono Bridge approximately 16 miles upstream in Yosemite National Park (USGS gage #11266500, 1917 to present) and approximately 5 miles downstream near Briceburg (USGS gage #11268200, 1966 to 1974). Records are also available for the South Fork Merced River at the confluence with the mainstem of the Merced River near El Portal (USGS gages 11268000 and 11268001, 1952 to 1975). The gage 11268001 record indicates that it is the resultant of combining gages 11-2665 and 11-2680. Although specific descriptions of these gage numbers were not found, internet searches indicate that 11-2665 corresponds to the Merced River and 11-2680 corresponds to the South Fork of the Merced River. This suggests gage 11268001 may represent the total flow in the Merced River at the confluence.

7.2 Peak Flow History

The records demonstrate a history of moderate flows punctuated by episodic large floods in each drainage. Mean annual flows at the Pohono Bridge gage range between 126 and 1466 cubic feet per second (cfs) (**Figure 7.1**), while those for the South Fork at the confluence range between 114 to 814 cfs (**Figure 7.2**). The relatively short (8 year) gaging record at Briceburg (station #11268200) exhibits mean annual flows approximately 3 times as large as those at the South Fork confluence (**Figure 7.2**). Major floods occurred in water years 1938, 1951, 1956, 1965, and 1997 (**Figures 7.3, 7.4,** and **7.5**). The largest flood on record at the project site occurred in December 1955 (water year 1956), which resulted in flows at the Pohono Bridge of 23,400 cfs and 46,500 cfs on the South Fork Merced at the confluence.

The peak flow records visually suggest the presence of a threshold—the extremely large events for both the main stem (gage #11266500) and the South Fork (gage #11268000), with values labeled on **Figures 7.3** and **7.4**, are appreciably larger than most other annual peak flow events. The mean peak flow, excluding flows above 10,000 cfs, for the main stem gage at Pohono Bridge (#11266500) is 4502 cfs, which concurs with a visually-estimated threshold flow on the annual peak flow series (**Figure 7.3**) of ~5000 cfs. An analogously-calculated mean peak flow (excluding flows above 10,000 cfs) for the South Fork Merced River (gage #11268000) is 3366 cfs. Flow events above these thresholds (or an approximate threshold of 5000 cfs) predominantly occur earlier in winter, in December and January, while most peak flows occur from spring snowmelt in late April to mid-June. We suggest that these extremely large flood events on

record occur in mid-winter during rain-on-snow events, and constitute episodic events that are likely very important to the mobility of channel substrate and the morphology of the canyon bottom.

An analysis of these peak flow events suggests that unit-discharges are larger for the South Fork watershed (241 square miles) than for the main stem watershed (321 square miles). An evaluation of each watersheds' physiographic characteristics provides evidence that supports such a hypothesis. Both the South Fork and the mainstem were glaciated, resulting in shallow depths to bedrock along river corridors. Poor growing conditions produce limited vegetative soil cover which combines with low infiltration rates to increase the amount of runoff while shortening concentrations times. Said another way, rainfall is not retained by ground cover or soil, runs off quickly, and produces relatively fast, sharp flood peaks. These runoff peaks are minimally retarded in the watershed, so that a smaller, more directly connected watershed such as the South Fork will produce peak discharges relatively fast, while a larger, more indirectly connected watershed such as the main stem will produce a more extended, more delayed peak discharge.

The form of the precipitation is also very important. In the higher elevation North Fork watershed, precipitation is more likely to be in the form of snow, which is retained in the watershed until a snowmelt event occurs. Cold winter storms produce moderate runoff from the lower elevation watersheds, the primary source for the South Fork of the Merced, but produce snow and little runoff at higher elevations, the major source for the main stem of the Merced. Similarly, a relatively warm winter or early spring storm will rapidly melt accumulated snow in the South Fork watershed, while causing little runoff in the main stem watershed. A very warm mid-winter storm, an infrequent occurrence, will cause rapid melting of snow in both watersheds, producing the highest discharges below the confluence. For most rain on snow events, the South Fork of the Merced likely generates greater runoff than the main stem. Hydrologic work on the main stem watershed reported by the National Park Service (Yosemite National Park, 2005) supports the above interpretation of watershed behavior …

7.3 Gage Data Analyses

It was necessary to estimate discharge values within the project reach based on the available gage data so that design flows could be put into perspective and models could be calibrated using observed high water marks from known events. Estimation of peak discharges within the project reach using the available gage data was complicated by the short duration of the

Briceburg and South Fork records, and the upstream distance of the El Portal gage. Two primary components of the flow system were unknown:

1. The phasing of the peak discharge recorded at Pohono versus the phasing of the peak of the South Fork at the confluence. Presumably, the South Fork peak will usually have passed before the peak on the mainstem occurs, but the potential for the peaks to coincide is not known.

2. The additional flow and phasing of the discharge to the mainstem that occurs within the 16-mile reach between the Pohono gage and the confluence with the South Fork.

In order to obtain reasonable flow estimates without creating a hydrologic model of the entire watershed and attempting to accurately reproduce the 2005, 1997 and other recurrence interval storm events, the discharges at the gages near the project reach were correlated to the Pohono gage.

7.3.1 Gage data correlation

Since the Briceburg gage record (**Figure 7.6**) does not include any extreme flood events, these gage data are most appropriate for the lower threshold flows, such as occurred in 2005. The analysis consisted of calculating the ratio of the Briceburg flow to the Pohono flow, for both the annual peak flows and the full record of daily mean flows. These ratios were then plotted against the flow at Pohono, and a clear correlation between the Briceburg and Pohono flows of approximately 1.64 was evident for increasing discharges at Pohono (**Figure 7.7**). This value was used to scale the 10,200 cfs peak discharge measured at Pohono in 2005 to obtain a discharge estimate of 16,500 cfs at the Briceburg gage (Table 1). The lower range of the flow estimate for the 1997 event was estimated by using this ration to adjust the 24,000 cfs flow at Pohono for the 43,250 at Briceburg. The flow at the Briceburg gage is considered to reasonably approximate the flow in the project reach because watersheds that contribute to the river flow between the project and the gage are not large relative to the total watershed size at that point, and because the peak discharge from the watershed between the South Fork confluence and Briceburg will have passed before the peak for the total watershed arrives.

To assess the high end of the peak flow range at the project site, which is probably influenced by rain on snow events, the longer record of gage 11268001('combined' flow at the South Fork, **Figure 7.8**) was used. This record included daily mean discharges and annual peaks. The ratios between the daily means at the combined South Fork gage and the Pohono gage were calculated and plotted against the flow at Pohono as described previously (**Figure 7.9**). In this case, however, the record includes a number of higher peak discharges at Pohono. Using only

the data points for flows at Pohono that exceeded 8,000 cfs, flow at the combined gage was 2.0 times the discharge at Pohono. This ratio was used to estimate 49,250 cfs as the upper bound for the 1997 discharge in the study reach (Table 1).

7.3.2 Log Pearson

Flood frequency analysis using annual peaks and both the Weibull Plotting Position and Log-Pearson Type III distribution indicates that the 100-year recurrence interval event at the Pohono Bridge on the Merced River main stem is approximately 24,000 cfs (**Figure 7.10**), while it is 65,560 for the South Fork Merced River at the confluence (**Figure 7.11**). The magnitude of the flow distribution at the south fork is most likely skewed upwards because the El Portal record is relatively short and includes a number of relatively high peak discharges.

Flows with recurrence intervals of 1.4 – 2.3 years are generally assumed to represent bankfull discharge in perennial alluvial streams[21]. These flows range between 3400 – 5000 cfs at Pohono Bridge and 2700 – 5300 cfs for the South Fork at the confluence. The peak discharge in the mainstem just upstream of the confluence between with the South Fork at the El Portal gage should occur significantly later than the peak at the Pohono gage for two reasons. The first is that the drainage area at the Pohono gage is 321 square miles, while the area just upstream of the confluence is 420 square miles. The second is that the confluence at El Portal is 16 miles downstream of the Pohono gage, adding significant travel time and attenuation of the peak discharge at Pohono. Further the peak discharge at Pohono should occur later than the peak of the South Fork at the confluence at the El Portal gage. This is because the South Fork has an area of 241 square miles verses 321 at Pohono, and the South Fork watershed is lower and more susceptible to late winter/early spring rain-on-snow events. Thus, the peak of the South Fork watershed should significantly precede the peak in the mainstem.

Due to the phasing and attenuation of the hydrographs, it overestimates the flow at the study reach to combine the flow values at Pohono with those from El Portal. Our flow proportioning calculations between the Briceburg and Pohono records suggest a 1.4-year flow of 5,700 cfs and a 2.3 year flow of 8,375 cfs at the Briceburg Gage downstream of the study reach, which should represent a reasonable approximation of the range of bankfull discharge estimates within the

[21] Bankfull discharge is the flow at which rising waters just begin to overtop the floodplain. While this flow often corresponds to the 1.4 to 2.3 year event, it should be noted that 'bankfull flow' is defined morphologically, not on the basis of recurrence.

study reach. **Note that the upper estimate (8,375 cfs) is similar to the bankfull estimate of 8,871 cfs that Drivas and Others, 2008 equated to the two-year flow event (Q2) in the Section 7(a) Evaluation for the project.** Subsequent sections of this report will expand upon the notion of bankfull discharge at the project site, but we submit here that these discharges may not be representative of channel-filling flows nor channel-forming flows (an 'effective discharge') at the project site.

8. HYDRAULIC ANALYSES AND RESULTS: 1-D AND 2-D MODLEING

Two-dimensional hydraulic modeling was performed for this project to support the geomorphic analyses of the river features and the evaluation of the potential affects of the bridge alternatives on recreational activities. A one-dimensional model of the project reach was developed for the natural condition to estimate the maximum flow that would pass under the bridges, and estimate boundary conditions and verification data for the two-dimensional model.

The one dimensional hydraulic model provides a water surface elevation and an average velocity for the main channel and left and right overbank areas at a cross section. Two dimensional models provide a field of depth-averaged velocities, depths and other hydraulic variables throughout the simulated reach.

8.1 River Bathymetry

Two surveys were initially available for the project reach. Caltrans had performed aerial photogrammetry for the canyon and river corridor, and a ground survey of the channel features above the water line of the river. In order to reasonably simulate the flow dynamics in the reach, however, additional topographic information was necessary to better represent the river channel. As a result, Caltrans supported a hydrographic survey to supplement the existing topographic information.

In mid-July 2008, Environmental Data Solutions (EDS) performed a detailed Class I hydrographic and bathymetric survey of 2,654 feet of the Merced River at the Ferguson Slide to fill data gaps in the previously available LIDAR survey data provided by Caltrans. The survey focused on in-channel bathymetry and channel margin topography below Highway 140 and Incline Road, with particular attention paid to the depth and configuration of pools. For bathymetry, EDS utilized a custom 14.0-ft Zodiac outfitted with two *Odom Hydrotrac* fathometers coupled with a 3°, 200 kHz depth transducer, calibrated using an *Odom Digi-Bar Pro* speed-of-sound probe and standard barcheck techniques. Depth data were corrected in real-time for vessel heave, pitch, and roll using a *TSS CMS-25* digital motion sensor. Position data were collected using a *Leica* Real Time Kinematic (RTK) GPS rover unit, providing horizontal accuracy of ± 0.10 ft. Survey planning and data acquisition was managed using *Hypack Max* (2008). A topographic survey of river banks, cobble bars, and the channel bottoms inaccessible by survey vessel complemented the bathymetric survey, and utilized a *Leica System 1200* RTK

base station (with rover unit) set up over a known Caltrans control point. For more details on survey techniques, controls, and QA/QC, please see Appendix C.

The aerial, ground, and hydrographic surveys were merged into one dataset with duplicate points and bad data removed using ESRI ArcMap. Topographic contour lines were then generated in ArcMap. The resulting elevation contour coverage was then used to create model geometry data for both the one- and two-dimensional models (**Figure 8.1**).

8.2 One-dimensional Model

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) was used to conduct the one-dimensional hydraulic modeling for the project reach. The software package was developed and is supported by the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center. It is the standard analysis tool for one-dimensional hydraulic modeling. The model represents the creek channel using cross-sections and computes friction (Manning's equation), and expansion/contraction losses. The momentum equation is used for situations where the water surface rapidly varies such as hydraulic jumps and confluences. It can be run for a steady-state (constant flow rate) or for an unsteady state (varied flow rate). Steady state simulations were used for this project. Only the natural condition was simulated.

8.2.1 Model development

Cross sections, shown in **Figure 8.1**, were extracted from the elevation contour map generated in ArcGIS using the geospatial data capabilities of the software (GeoRAS). Manning's 'n' values were estimated in the field based on substrate roughness and vegetation characteristics according to established guidelines (Chow, 1959; Haan and others, 1981) and are summarized in **Figure 8.2**. These 'n' values were then distributed appropriately across each cross section.

8.2.2 Model runs

8.2.2.1 Calibration

To assess the reasonableness of the hydraulic model results, the model was run for the 1997 and 2005 peak flow rates estimated in Section 7.1.3.1, and the estimated water surface elevations were compared with the high water marks observed in the field. The locations of these high water marks are depicted on **Figure 8.1**, and examples of stranded debris used to estimate high water marks are shown in **Figure 8.3**. As described in Section 7.3.1, the flow rates used were estimated to be 43,25 0 – 49,000 cfs for the 1997 storm and 16,500 cfs for the 2005 storm using available gage data (Table 1).

8.2.2.2 Two-dimensional model flows and boundary conditions

The verified model was then run for the flows to be used in the two-dimensional hydraulic model. These flows were set at 42,000 cfs – the maximum flow estimated to pass under the downstream bridge soffit for the S Alternative - and 8,800 cfs – the upper end of discharges suitable for recreational rafting. The results of these model runs were used to determine the downstream water surface elevation for the two-dimensional hydraulic model, and also to provide water surface profiles with which to compare the two-dimensional hydraulic model.

8.2.3 Model results

8.2.3.1 Calibration flows

Model results were compared with high water marks observed in the field for the 1997 and 2005 flood events, producing very good correspondence (**Figure 8.4**). Although there is uncertainty in the flood discharges calculated for these events, the agreement between the model using the 'n' values estimated in the field and the high water marks demonstrates reasonable accuracy of the model results.

8.2.3.2 Verification flows for the two-dimensional model

Results verified that 42,000 cfs would pass under the low point of the downstream bridge for Alternative S, establishing this flow as the upper value used in the two-dimensional model. The downstream water surface elevation corresponding to this flow rate was 1359.5 ft. Results for the 8,800 cfs specified a downstream water surface elevation boundary condition of 1347 ft for the two-dimensional model.

8.3 Two-dimensional Hydraulic Model

The two-dimensional modeling was done using the Finite Element Surface Water Modeling System – Two Dimensional Hydrodynamic (FESWMS-2DH) coupled with the Surface-water Modeling System (SMS) graphical pre- and post-processor software package. Developed for the Federal Highway Administration and the U.S. Geological Survey, this modeling platform is widely accepted and recommended by Caltrans for use in modeling highway river crossings where complex hydraulic conditions exist (Caltrans, 2006).

FESWMS-2DH applies the finite element method to solve a system of equations that describe two-dimensional depth-averaged surface water flow. Inputs to the model include a network of nodes and elements referred to as the model mesh that describes the physical geometry of the river reach, conveyance parameters within the mesh, and upstream and downstream boundary conditions defining starting water surface elevations and flow rates (FHWA, 2003). For this

application the model was set to analyze steady-state conditions, with water surface elevations drawn down incrementally from a fully inundated initial condition.

Due to the different nature of the low-flow and high-flow channels, two different meshes were used. This minimized the amount of drawdown and cell drying, and allowed a finer grid pattern on the channel bottom which was necessary for the low-flow model. An artificial inlet section was used for both meshes to enhance model stability at the upstream boundary. This section consisted of a narrowed, flat-bottomed initial cross section that widened to match the actual channel width and transitioned to meet the channel bottom topography. The upstream-most cells in the model, in the tapered section of the mesh, do not represent the real channel – model results in this section are not used to predict river behavior.

8.3.1 Model scenarios

The following scenarios were simulated for both the 42,000 cfs (high flow) and 8,800 cfs (rafting flows):

1. Natural Condition with no bridges or abutment fill prisms in the river channel
2. No Build Alternative (the existing temporary bridges)
3. Alternatives C and T (equivalent)
4. The temporary bridges associated with Alternative S
5. Alternative S

Alternatives A and S2 were not modeled because design data (bridge deck, abutments and pier locations) were not available.

In order to check for approximate agreement between the one-dimensional and two-dimensional models, the water surface elevation results of the Natural Condition model are compared to the HEC-RAS model at various locations through the length of the reach. The depth, velocity, and Froude number will also be presented for the natural condition results. Only the depth and velocity results will be shown for the other alternatives in this chapter, however, because the other variables do not change enough to warrant in-depth presentation and discussion.

8.3.2 High-Flow Model

8.3.2.1 Model development and parameterization

The high flow channel mesh extended up the banks to approximately the level of the Highway 140 road surface. The actual road surface was excluded to reduce problems associated with the cells drying out during the model draw down. A rectilinear mesh was used on the channel bottom, where flow was primarily in the downstream direction, wetting and drying was not occurring, and a very small cell size was not necessary. The cells in this portion of the mesh were generally 8 to 12 feet long on a side depending on the complexity of the flow at that location, with the long axis oriented parallel to the channel. A triangular mesh was used on the steeply sloped, irregular bank surfaces because it proved to be more stable than the rectilinear mesh where cell drying was occurring and flow directions were more variable. Dense patches of small cells were created at pier locations to more accurately capture pier shapes. An example section of the mesh is shown in **Figure 8.5**. Curved bank, water edge, bridge and abutment features were also more easily represented by triangular cells than rectangular ones. Elevation data from the elevation contours were assigned to the nodes at each mesh vertex using mesh generation functions within SMS.

Once created, the mesh cells must be attributed appropriately to reasonably represent the channel and flow conditions. The primary parameters include: Manning's 'n' values, eddy viscosity, storativity, wetting and drying depth tolerance. The Manning's 'n' values used ranged from 0.035 for the main channel to 0.07 for the rough, rocky, vegetated surface on the rock slide debris. Bank areas were typically assigned 'n' values of 0.05 to 0.06. In some cases Manning's 'n' values were increased in locations where the results were showing unrealistically high velocities, in shallow areas where roughness features would be large relative to the water depth, and in areas of substantial turbulence. A kinematic eddy viscosity (V_o) of 20 ft^2/s was uniformly used to represent turbulent energy losses based on available research and other studies (Vionnet and others, 2004; and Huizinga, 2007).

The bridge abutments and piers were represented by disabling cells. This created a barrier to the flow and in the case of piers was more effective than the pier analysis function built into the model, which basically estimates the drag and scour on a pier, but does not create a flow obstruction. Since the pier designs show that they will be placed without substantial change to the river bottom topography, disabling of the cells reasonably represents the effect of the pier. To allow better representation of the pier shapes, patches of higher density cells were created in the mesh at the locations of the bridge piers.

The boundary conditions generated with HEC-RAS were used for all modeled scenarios. These boundaries consist of a downstream water surface elevation and a flow rate to input at the upstream end of the mesh. The downstream water surface elevation used was 1359.5 ft for a flow rate of 42,000 cfs.

8.3.2.2 Model results

*Natural Condition (***Figures 8.6 - 8.8***)* – The water surface elevation results for the FESWMS model correspond well to those estimated using HEC-RAS. This is illustrated in Figure 8.6 showing the WSE computed in the FESWMS model plotted against the WSE computed in HEC-RAS (Observed). These calibration points are located at the low point of each bridge alternative and at the toe of the slide area, and are shown on the WSE color contour plot (Figure 8.7, top) as small box plots. The bar on the box plot represents the difference between the FESWMS model WSE and the corresponding value from the HEC-RAS results. The error bars on the box plots represent 1 foot. Note that the one-dimensional HEC-RAS model does not account for lateral variation in WSE while the two-dimensional FESWMS model does, so some deviation between the two models is expected. This verification enables a good level of confidence in the FEWMS model results when used to evaluate alternative bridge designs.

Other observations include:

- The water surface is 1-3 feet below the road level at the downstream end of the reach.
- The bars and deep pools are evident in the depth plot, (**Figure 8.7** lower), with maximum depths (dark blue) of approximately 30 feet in the thalweg at the downstream portion of the reach.
- The velocities (shown as blue to red with increasing magnitude) range from 0 ft/s at the edges, 10 to 18 ft/s in the straight runs, and up to 27 ft/s in the rapid at the toe of Ferguson Slide (**Figure 8.8**, top).
- Some super-elevation of the water surface on the outside of the bend in the river can be seen, particularly in the approach to the bend. Flow arrows also demonstrate direction of some of the flow, particularly in the main channel (Thalweg) towards the left bank in the approach to the bend.
- The flow directions are primarily parallel with the channel, with the exception of a few eddies at the channel edges. The largest of these is on the left bank at the toe of the slide.
- The influence of some channel features, particularly bars, is minimized by the large depths of flow associated with this high discharge.

- The Froude number plot shows that the flow is subcritical through most of the reach, with some areas of possible critical flow showing in yellow, and patches of supercritical flow in the rapid at the toe of the slide (**Figure 8.8**, bottom).
- The results described above match both expect behavior based on experience and eye witness accounts of river behavior during major flood events, providing addition confidence in the veracity of the model predictions.

*Temporary Bridge Alternative (**Figure 8.9**)* – The temporary bridges, which are presently in place, have abutments that encroach on the channel and the upstream bridge has a pier in the center of the channel just upstream of the first right-hand bend. Backwater areas and eddies are shown behind the abutments on the left bank at the downstream bridge and on the right bank at the upstream bridge, and an increase in velocity can be seen where the channel is constricted at the downstream bridge (**Figure 8.9**, bottom). The low-velocity region behind the pier at the upstream bridge is evident as well.

*Alternatives C and T (**Figure 8.10**)*– The bridges associated with Alternatives C&T do not have abutments that encroach on the channel, but have 11-foot piers located on the sides of the channel outside the main flow path. The left upstream bridge pier is located on the downstream end of the left-side bar and the right pier is located on the point bar upstream of the right-hand bend. Consequently, impacts of the bridges for these alternatives on channel hydraulics are negligible.

*Alternative S Temporary Bridges (**Figure 8.11**)* – An alternative upstream temporary bridge is necessary for the construction of the S alternative. This bridge is upstream of the other bridges and crosses the channel near the midpoint of the large bar on the left. The bridge has two 4-foot piers in the channel, one in the thalweg and one on the bar. Since this is a straight, relatively high-velocity reach of channel, these piers are in high-velocity flow in the range of 12 to 14 ft/s. The wake downstream of the piers is evident in the velocity plot and a higher velocity is shown between the piers where the flow is concentrated. The downstream bridge is the same as the Temporary Bridge scenario.

*Alternative S (**Figure 8.12**)* – This alternative does not have abutments that encroach on the channel. Both the upstream and downstream bridges have 11-foot diameter piers located in the channel. These upstream bridge piers are outside the main flow path in relatively low-velocity areas, and do not have a substantial effect on depth or velocity. The downstream bridge piers are more exposed to higher velocities and impose localized effects, seen as a lower velocity "wake" downstream and a significant redirection of flow both towards the center of the channel

and the bank. Increased velocities and turbulence at the banks, particularly the left bank, should be considered in terms of bank stability impacts for both of the downstream bridge piers. Increased velocities and turbulence at the right downstream pier should be considered in terms of bar stability also.

8.4 Low-flow (Rafting, ~Q2/Bankfull) Model

8.4.1 Model development and parameterization

The low-flow (rafting, bankfull) channel boundaries were extended only part of the way up the channel banks, well below the Highway 140 road surface. This was done for three reasons:

1. The cell size within the channel and bar features was too large in the high flow model to capture the complicated flow patterns under rafting conditions, and reducing the cell size over the extent of the high-flow model grid resulted in too many cells for efficient model performance.

2. A smaller channel area was necessary to reduce the cell size enough to adequately represent flow patterns at the rafting flow without exceeding the maximum number of cells for which the model would write output.

3. The drying of cells was excessive when drawing the model down from the high banks for the low flow condition, which made it nearly impossible to achieve a stable and valid simulation at the desired flow and downstream water surface elevation.

A triangular mesh was used for the entire low-flow channel because this mesh type was more effective for the multidirectional flow and cell drying that occurs on the relatively flat bar surfaces (**Figure 8.13**).

Once created, the mesh cells were attributed to reasonably represent the channel and flow conditions using generally accepted guidelines (Chow, 1959 and Haan 1981). The Manning's 'n' values used ranged from 0.04 for the main channel to 0.075 for the rough, rocky, vegetated surface on the rock slide debris. Bank areas were typically assigned 'n' values of 0.05 to 0.06. In some cases Manning's 'n' values were increased in locations where the results were showing unrealistically high velocities, in shallow areas where roughness features would be large relative to the water depth, and in areas of substantial turbulence. It was necessary to use slightly higher 'n' values than in the high flow model because the relative depths are less at low flows, and model stability was a problem with lower 'n' values. Variable 'n' values were also used to approximate an increase in roughness below a certain threshold depth. As an example, the 'n' value for the upstream bar surface was set to be 0.04 for depths greater than 2 feet and 0.05 for depths less than 1 foot, with a linear transition in between. A kinematic eddy viscosity

(V_o) of 20 ft^2/s was used to represent turbulent energy losses (Vionnet and others, 2004; and Huizinga, 2007).

The bridge abutments were outside of the wetted channel for all scenarios except the Temporary Bridges for the low-flow condition. The obstruction associated with the temporary bridge abutment was represented by disabling cells. The piers that were located within the low-flow mesh were represented using the built in pier function and by disabling a cell. The pier function creates a graphic of the pier and estimates drag on the pier, and the disabled cell creates a flow obstruction. Since the pier designs show that they will be placed without substantial change to the river bottom topography, disabling of the cells reasonably represents the effect of the pier.

The boundary conditions generated with HEC-RAS were used for all modeled scenarios. These consist of a downstream water surface elevation and a flow rate to input at the upstream end of the mesh. The downstream water surface elevation used was 1347 ft for a flow rate of 8,800 cfs.

8.4.1.1 Model results

*Natural Condition (**Figures 8.14 – 8.16**)* –The natural condition results were again compared to the water surface elevations computed by the HEC-RAS model by plotting the FESWMS (Computed) results against the HEC-RAS (Observed) results (**Figure 8.14**). The water surface elevations show a good correlation to those from HEC-RAS, although the FESWMS model is approximately a foot higher in 5 of the 12 locations. The difference at each calibration point is shown on the WSE plot (**Figure 8.15** upper), with the two largest occurring at the location of the left abutment of the downstream temporary bridge and the location near the right pier of the alternative S bridge (bridge alternatives are not part of the natural condition model). These differences are likely due in part to the use of slightly higher Manning's n values in the FESWMS model, which was helpful in reducing model instabilities in some places. Also, the FESWMS model captures lateral variation in WSE and HEC-RAS does not, which may result in discrepancies. Other observations include:

- Deep pools exist at the upstream end of the reach on the right, in the straight run between the bends, and downstream of the slide debris deposit (**Figure 8.15** lower).
- A series of riffles are shown, with the largest at the toe of the slide and adjacent to the large upstream bar.
- The velocities range from 0 to 17 ft/s, with the highest velocities in the riffle at the toe of Ferguson slide (**Figure 8.16** upper).

- The bar at the upstream left side of the channel and the bar on the right side across from the toe of the slide are each partially exposed (shown as white outlines on the plots) and there is a distinctive back channel adjacent to each.
- The Froude number shows supercritical flow at the upstream end of the toe of the slide and a few other locations (**Figure 8.16** lower).
- Edge effects of the model are evident in the Froude number plot where cells are going dry leading to high values. These are artifacts of the depth going to zero and should not be relevant to the interpretation of results or comparison of alternatives.

*Temporary Bridge Alternative (**Figure 8.17**)* – The left abutment for the downstream temporary bridge protrudes only slightly into the wetted channel at the 8,800 cfs flow, which redirects flow and increases the velocity in the channel slightly. The abutments of the upstream bridge are outside of the wetted channel. The pair of piers in the center of the bridge is on the right edge of the flow in a relatively low-velocity area.

*Alternatives C & T (**Figure 8.18**)* –The abutments for Alternatives C & T are outside the flow and the piers are located in the outer edges of the wetted channel. The left pier of the upstream bridge is located on the left side of the channel in higher velocity flow and in line with the back channel flow along the left side of the upstream bar. Velocities in this region are moderate (8-12 ft/sec) and will generate some turbulence and flow diversion around the left pier.

*Alternative S Temporary Bridges (**Figure 8.19**)* – The downstream bridge is the same as previously discussed for the Temporary Bridge Alternative. The abutments of the upstream bridge do not encroach on the wetted channel at this low flow. The two sets of piers are located in the channel however. The right pier appears to be located in roughly the center of the channel thalweg in relatively high velocity conditions. The wake of the pier and higher velocities in the channel to the left can be seen in the model results. This may be a concern to rafters. The left pier is located on the bar, and will not significantly impact channel hydraulics since the bar is almost fully exposed.

*Alternative S (**Figure 8.20**)* – All of the abutments are out of the channel for the Alternative S bridges. The piers on each side of the bridges are in the wetted portion of the channel, however. The right and left pier for the downstream bridge and the right pier for the upstream bridge are in shallow, low-velocity flow on the wetted channel margin. These piers have little impact on channel hydraulics. The left pier of the upstream bridge is in high velocity flow on the outside of the first right-hand bend. This position makes it vulnerable to passing debris and an obstruction to rafters. An increase in center channel velocities and a wake below this pier can

be seen in the model results. Model detail is insufficient to represent flow between the pier and the bank; however, it is possible that the pier is also directing flow into the left bank. These conditions could impact boating safety during a flow at this magnitude or higher and need to be considered in channel and bank stability assessments.

8.4.2 Discussion of limitations or issues

A number of aspects of the modeling methods or model characteristics warrant discussion. Because this is a relatively steep reach with steep banks, it was necessary to draw the model down up to 30 feet from the "flat water" initial condition, which required a significant number of cells to go dry and complicated the achievement of a stable and representative model of the hydraulics of this reach. Edge effects due to drying of cells created high velocity perturbations in some locations that are probably not representative of real behavior. Artificial river geometry was created at the upstream end of the reach in order to solve stability problems during model drawdown. The results in the section of the model upstream of the first left bar (the tapering channel section) are, therefore, not representative of actual stream behavior. The extent of critical and supercritical flow also contributed to model stability and required judicious adjustment of model parameters. Slightly higher 'n' values were used in the FESWMS model than in the HEC-RAS model in some locations to increase the FESWMS model stability and control unrealistic velocities. Lastly, the resolution of any numerical model is limited by both computation capacity and number of cells that can be created. Therefore, hydraulic behavior that is more detailed than the model mesh will not be represented accurately. While the FESWMS model capacity was fully exercise to produce as much detail as possible, details of flow around bridge abutments and piers, for example, are limited. Therefore, interpretation of small scale changes in hydraulics predicted by the model should be done with caution.

Despite the challenges of creating a 2-dimensional hydraulic model of the reach, model results can be used with reasonable confidence – particularly with respect to medium to larger scale flow patterns, velocities and water surface elevations. The calibration and verification process used in this study demonstrated good correspondence between observed high water marks and model results and good correspondence between one-dimensional model results and two-dimensional model results. In addition, model behavior matches both experience with similar rivers and eye witness reports of river behavior during large discharge events.

9. RIVER-FOCUSED EVALUATION OF PROPOSED HWY 140 TRANSPORTATION ALTERNATIVES: DISCUSSION AND CONCLUSIONS

Chapter 9 focuses on evaluation of each proposed transportation alternative at both the low (8,800 cfs) and the high flow (42,000 cfs) modeled, for potential impacts to the Merced River with respect to the Wild and Scenic designation. While Chapters 5 through 8 provided the background on our technical approach, as well as the general results of our geomorphic, hydrologic, and hydraulic analyses, Chapter 9 utilizes these results to highlight specific and potential impacts to the river from each proposed alternative. Prior to presenting results for the alternatives, we provide a synopsis of hydraulic conditions under natural conditions (i.e. no bridges or piers in the river, permanent or temporary) to establish the baseline, and then we provide results for each alternative. The discussion is broken out by the modeled low flow and the modeled high flow. For ease of comparison between alternatives, we have developed template panels within an 11x17 page format which illustrates how hydraulic and sediment mobility conditions may change under each alternative, with respect to natural conditions. Within the discussion of each alternative, we provide clear guidance on the potential recreational use, and river impacts as well as which aspect of the present designs (i.e. pier locations) will drive the highlighted impacts. Flow direction in all panels is from right to left.

9.1 Low-flow (~Q2/bankfull) Results

9.1.1 Natural conditions

Plots of flow depth, flow velocity, shear stress, critical grain size diameter, and incipient motion potential under low-flow conditions (8,800 cfs) are shown for the project reach in its natural state (no crossings, with existing Highway 140 roadway above west bank only) in **Figure 9.1**. Flow depths are predominantly 2 – 12 feet deep in most locations, with depths reaching up to nearly 18 feet in pools. Depths are generally less than 1 foot over much of the bar deposits within the project reach, and therefore likely to just begin to engage channel margin secondary or cut-off channels. However, flows do not inundate the entirety of the left-bank landslide bar just downstream of the Ferguson Rock Slide, where boulders and roadway talus below Highway 140 are still above water line at the left-bank margin.

Hydraulics through the project reach for the natural condition at the low flow creates an irregular patchwork of high velocity fields separated by more extended domains of lower velocity zones. Velocities approach 17 feet/second (from heron ft/sec) through riffles, rapids and grade drops associated with bar deposits and landslide debris. Some portions of pools

exhibit velocities of equivalent, or slightly less magnitude while in general pools are characterized more commonly by velocities of around 7 – 9 ft/sec.

Shear stress calculations are performed over geomorphic units composed of material whose grain size distribution was measured in the field by Balance geomorphologists. As **Figure 9.1** shows, shear stress is relatively low over much of the left-bank bar upstream of the slide, with values usually much less than 1 lb/ft^2 (pound-force per foot squared). Shear stress intensifies at the downstream end of this bar surface to approximately 5 – 8 lb/ft^2 as flow velocities and depth increase closer to the channel center. Computed shear stress on the right bank bar just downstream remains low, with values below 1 lb/ft^2, while the bar across from the slide exhibits varying shear stress conditions. The upper end of this bar exhibits shear ranging between 10-15 lb/ft^2 as flows coming around the bend upstream first interact with it. Flow separation and friction losses across the top of the bar result in reduced depth and velocity at the downstream margin of this bar, and shear stresses drop to values generally less than 1 lb/ft^2.

Areas of locally elevated shear stress (> 2-4 lb/ft^2) illustrated in the shear stress panel exhibit critical sediment size distributions generally much higher than the median sediment sizes sampled at these locations. Therefore we would expect these areas of the bars to be mobile under the 8,800 cfs scenario. On the other hand, areas characterized by lower shear stresses (< 2-4 lb/ft^2) result in critical sediment size distributions generally commensurate with existing grain size distributions. For these areas we would expect stability or immobility, rather than mobility (see the incipient motion panel). Under natural conditions, the downstream end of the left-bank bar in the upstream reach of the panels is susceptible to transport, as are deposits on the upstream nose of the slide bar and along the eastern margin. We expect river deposits within the actively flowing portions of the river to be mobile, since flow depth and velocity will be roughly equivalent or higher than conditions within the mobile domains over bar surfaces shown in **Figure 9.1**.

9.1.2 Alternatives C and T

Panels provided in **Figure 9.2** illustrate the differences between flow depth, flow velocity and shear stress relative to natural conditions, and critical grain size diameter and incipient motion conditions for Alternatives C and T under low flow conditions. Under Alternatives C and T, flow depth is shown to increase along the left side of flow by slightly less than 1 foot upstream of the upstream proposed bridge. This is due to the pier being located in a relatively high-velocity (8 – 10 ft/s) portion of the channel on the outside of a bend where flow naturally tends

to accelerate. Minor predicted flow depth increases are shown upstream of the right bank pier for the downstream bridge. The right bank pier for the upstream bridge would be located just at the edge of the predicted water surface, so predicted hydraulic changes there due to the presence of the pier are negligible. The left bank pier for the downstream bridge is predicted to have negligible effects to flow hydraulics. The predicted increase in flow depths upstream of the left bank pier for the upstream bridge will also be associated with a slight depression in water surface elevation of generally less than one foot within the wake downstream of the pier.

Changes in flow velocity from natural conditions for Alternatives C and T are generally minor and focused at the locations of the bridges and piers. In between the piers at both the upstream and downstream bridges velocity will increase by approximately 1 to 1.5 ft/sec, with slightly higher values predicted to the left of the left bank pier at the upstream bridge. Velocity is predicted to decrease to zero immediately upstream and downstream of 3 of the 4 piers (excluding the right bank pier of the upstream bridge as it is located at the right-edge of flow). Changes in shear stress follow a pattern similar to that for velocity (where the fields overlap), with increases of 0.5 to 1.0 lb/ft^2 focused to the left and the right of the left bank pier of the upstream bridge. Despite some predicted changes in the velocity and shear stress fields in the vicinity of the bridges and piers, critical grain diameter for zones where the fields overlap are largely unchanged, with the exception of the area immediately right of the left bank pier at the upstream bridge. Here, critical sediment diameter is predicted to increase from cobble size material to boulder size material. From field observations of bed material size within the active or main flow portions of the river (thalweg), we would expect similar sized material to be mobile under the modeled low-flow scenario (cobble to boulder size material).

Because of the proposed locations of bridge piers for Alternatives C and T, under low-flow conditions we would expect minor to negligible changes to the present extents and locations of mapped bars. The real potential for bar change would be to the downstream or distal portion of the bar mapped along the left bank at the upstream bridge. This change would be driven by the pier location – however it would likely be minor. We do anticipate enhanced mobility of the bed in between the piers at the upstream bridge. This zone is at the transition from a mapped riffle to a pool so it is possible that the pool could enlarge, and the riffle shrink as flows accelerate into the pool. These possibilities would have little noticeable effect to the character or nature of the river in this location. Whitewater enthusiast could be affected by the left bank pier of the upstream bridge as they would need to steer away from the pier under flow velocity conditions of 10 to 12 ft/sec (**Figures 9.1** and **9.2**). Failure to steer away from the pier could

results in boaters becoming pinned, or colliding with the pier. Movement of the pier approximately 15 feet to the southeast could greatly reduce this potential impact.

9.1.3 Alternatives S and the S Temporary Bridges

Plots of the difference between the natural and Alternative S model results under low flow conditions are shown in **Figure 9.3**. **Figure 9.4** shows the difference between natural conditions and those under temporary conditions that would exist during the time of construction of Alternative S, which includes an alternative upstream temporary bridge upstream. It is important to point out that the downstream temporary bridge under the Alternative S temporary condition is the same as that for the no-build alternative at the downstream location. These temporary conditions incorporate an upstream crossing similar to that used in the no build alternative, but located farther upstream away from the proposed location of the permanent Alternative S crossing. As with **Figure 9.2**, **Figures 9.3** and **9.4** illustrate changes in flow depth, flow velocity, and shear stress from natural conditions, along with critical grain size diameter and incipient motion potential.

9.1.3.1 Alternative S

The obliquely-oriented upstream bridge for Alternative S creates slight (~1 ft) increases in flow depth upstream of the upstream bridge (**Figure 9.3**), similar to that observed in **Figure 9.2** for Alternatives C and T. The depth increase is predicted to extend approximately 200 feet upstream, which is slightly more extensive than shown for Alternatives C and T. This is most likely due to the proposed placement of the left pier of Alternative S left further into the meander bend, and within higher velocity flow. Because the piers for the downstream proposed bridge in Alternative S are largely out of the water, depth increases are negligible to zero. Under low-flow conditions, the upstream bridge right bank pier is sheltered by the right bank bar located just upstream – thus relatively minor hydraulic changes are predicted. Slight depressions in water surface elevation of generally less than one foot are predicted for the wake zones of the upstream bridge piers. These depressions decay within perhaps 10-20 feet downstream of the piers.

Flow velocity increases of approximately 2 ft/sec are shown between the piers at the upstream bridge due to a loss in the effective width of the river at the left bank pier and the resultant focusing of flow in between the bridge piers at the upstream bridge. Minor flow velocity decreases are predicted upstream and downstream of the left bank pier at the upstream bridge (**Figure 9.3**). The predicted decrease downstream of the pier would be due to flow separation and eddy re-circulation at and downstream of the pier. As was the case for flow depth, the

downstream bridge piers would have a minimal effect on flow velocities because they would be largely out of the water.

Increases in flow velocity between the piers at the upstream crossing would drive associated increases in shear stress there, with shear stress increasing by 1 lb/ft^2 or more compared to natural conditions (**Figure 9.3**). The increase in shear is only apparent, however, along a narrow strip of the right bank bar just upstream of the upstream bridge towards the channel center. We would expect this shear stress increase to extend out into the channel center, and under the upstream bridge, as suggested by the increase in flow velocities in between the piers. Shear stress remains relatively unchanged across and around the slide bar, downstream. These minor changes in shear stress across the bar surfaces within the project reach are reflected by virtually no change in critical sediment diameter and incipient motion potential between natural conditions and Alternative S. We do, however, expect enhanced sediment mobility in the channel center associated with velocity and shear stress increases in between the piers of the upstream bridge for Alternative S. Thus, there may be enhanced mobility of sediments along the left margin of the right bank bar at the upstream crossing possibly leading to permanent morphologic changes to the distal end of the bar. Because the field of increased shear stress is relatively small, we would expect the area of bar impacted to be relatively small.

As with Alternatives C and T however, Alternative S poses serious challenges to whitewater enthusiasts as the left bank pier of the upstream bridge has been located within and at the margin of some of the highest velocity fields predicted at low-flow conditions. As boaters and rafters approach this pier they will likely be experiencing flow velocities of 12 to 14 ft/sec with momentum carrying them towards the pier as they enter the meander. As a result, they will need to work to avoid the pier or risk becoming pinned against, or colliding with the pier. The danger of this potential outcome could be heightened if debris has rafted against the pier. Moving the pier approximately 10 to 15 feet to the east could put the pier out of the water, and thus more or less eliminate the risk to boaters under the modeled low-flow conditions.

9.1.3.2 Alternative S Temporary Bridges

Modeled flow depths under the temporary crossings for Alternative S (**Figure 9.4**) suggest a backwater effect at the upstream temporary crossing, compared to natural conditions. Flow depth increases by approximately 1 to possibly 2 feet upstream of the pier which is in the thalweg (the pier closest to the right bank). This backwater effect at the upstream temporary crossing decays within one-hundred feet upstream. No changes in flow depth are predicted upstream of the downstream temporary bridge crossing, presumably because the downstream

bridge has no piers, and the abutments do not cause a major construction in flow width under low-flow conditions. A small eddy is shown downstream of the left bank abutment, although its extent appears to be amplified due to an artifact of the model solution.

Flow velocity increases of up to 2.5 ft/sec are shown as flow is forced around the right pier of the upstream temporary bridge, which is in the river thalweg (**Figure 9.4**). The flow velocity decreases immediately upstream and in the wake downstream of the pier within the thalweg. The left pier, which is on the left bar in very shallow, low-velocity water, has no observable effect on the depth or velocity. Changes in flow velocity at the downstream temporary bridge are only evident in the flow separation downstream of the left bridge abutment, where there is a velocity decrease of 2 to 3 ft/s in the backwater, and an increase of up to 1 ft/s towards the center of the channel. As mentioned above, the extensiveness of this change may be exaggerated due to a localized divergence of the model.

Shear stress increases of up to 2 lb/ft^2 are shown between the piers of the upstream temporary bridge as a result of the increase in velocity. This shear stress increase would be located on the left edge of the channel thalweg and the right edge of the upstream mapped bar. Calculations of change in shear stress, again, are limited to bar surfaces with grain size characterization obtained in the field, but we expect similar increases in shear stress to exist within the full extent of flow between piers at the upstream temporary crossing. Although the results indicate an increase in the shear stress at the downstream portion of the left bank bar associated with flow between piers at the upstream temporary Alternative S crossing, the distribution of critical sediment diameter is qualitatively very similar to that calculated for natural conditions (**Figures 9.1** and **9.4**). As such, the incipient motion conditions are nearly identical to that developed under natural conditions. It is clear, however, that enhanced flow velocities and shear stresses through the upstream temporary bridge openings will potentially create enhanced sediment mobility at flood flows during the construction of the permanent structures associated with alternative S. As such, it is possible that the nature of the river bed at the upstream temporary bridge could see a short-term impact due to enhanced sediment transport (erosion).

9.1.4 Alternative S2

Distributions of changes in flow depth, flow velocity, and shear stress, along with illustrations of critical sediment diameter and incipient motion potential are not included for Alternative S2. However, we can provide generalized statements about the potential effects of this alternative on flow depths, flow velocity, shear stress, and incipient motion based on results obtained for Alternative S, as requested by Caltrans. Alternative S2 utilizes a similar downstream crossing

to that of Alternative S, while the upstream crossing is more similar to that for Alternatives C and T. Thus, we would expect flow depths and flow velocities downstream of the rock slide to be similar to those shown in **Figure 9.3** for Alternative S, with flow depths and velocities remaining very similar to natural conditions. Similarly, the depiction of flow depths and velocities upstream would likely be somewhat similar to results obtained for Alternatives C and T (**Figure 9.2**). However, the drawing provided for Alternative S2 from Caltrans depicted only 1 pier which would be located in the river, as opposed to 2 as is the case with Alternatives C, T and S. The proposed pier for Alternative S2 would be located on the mapped bar along the left bank of the river (**Figure 10.1** and **Plate 1**). Flow velocities and shear stresses would likely be predicted to increase to the right of the proposed pier location, and possibly to the left if flood flows are squeezed between the pier and the left bank below the existing Highway 140 road bed. Possible flow depth dynamics are more difficult to estimate without having completed some level of hydraulic modeling.

Alternative S2 would likely have minor to negligible potential impacts to river form and function at the modeled low-flow condition. This suggestion is primarily supported by the present proposed location for the pier which would be constructed within the river. As stated above, this pier would be placed on top of the mapped bar along the left bank of the river. Based on model results for Alternatives C and T, if the bridge and piers for Alternative S2 do not affect flow depths much, it is likely that at low-flow conditions the S2 pier would be inundated by 0 to 2 feet of water with fairly low velocities. This also supports our suggestion of minor to negligible potential impact to river form and function. The only caveat here would be with respect to the high-flow channel located at the back side, or road side, of the mapped bar. While velocities here could reach 4.0 ft/sec. during the low-flow condition (**Figure 8.18**), it remains to be seen how much the proposed pier location would effect hydraulics along the extreme left side of the river. If this becomes a serious concern than our models should be updated with the complete structure plan for Alternative S2 in order to provide a more informed answer to the question. Whitewater recreational impacts at the low-flow condition are also likely to be negligible compared to stated concerns for Alternatives C, T, and S, also due to the location of the pier well upstream of the meander entrance and well out of the primary thread of flow.

9.1.5 Alternative A

Distributions of changes in flow depth, velocity, and shear stress, along with depictions of critical sediment diameter and incipient motion potential are not included for Alternative A. However, we can provide generalized statements about the potential effects of this alternative

on predicted changes to flow depths, velocity, shear stress from natural conditions, as well as incipient motion based on results obtained for the no build alternative, as requested by Caltrans. We would expect flow depths and velocities to be generally similar to those shown in **Figure 9.5** for the no build alternative. Specifically, we would expect minor to negligible flow depth changes on the upstream side of the proposed bridges as the proposed Alternative A bridges would have no piers in the river. It is our understanding that as of now, Alternative A would include bridge abutments located well up and onto the left and right banks. Additionally, as long as the downstream bridge soffit is higher than roughly 1,366 feet above mean sea level, and the upstream bridge soffit is higher than roughly 1,380 feet above mean sea level, it is likely that the proposed bridges would be completely out of flood flows at both the low-, and high-flow condition. This explicitly assumes no major net increase in the elevation of the river bed within the vicinity of the proposed bridges. It is also important to point out that based on the information provided by Caltrans, we would anticipate no potentials effects to whitewater recreational use as at the low-flow condition the proposed bridges and abutments should be clear of flow.

9.1.6 No-build alternative

The no-build alternative utilizes the current temporary crossings in place for the Highway 140 detour. Plots of the difference between natural conditions and those under the no-build alternative under low-flow conditions are shown in **Figure 9.5,** utilizing the same panels previously illustrated in **Figures 9.1** through **9.4**. No depth increases are evident for the low-flow condition immediately upstream of both the downstream and upstream bridges for this alternative. At the upstream bridge, however, a narrow band of water surface decrease is predicted along the far right bank and right-edge of flow. This is most likely an effect of the right bank abutment currently in place. At the downstream bridge, a small flow separation is shown, as described in Section 9.1.3.2, where the depth decreases between 1 and 2 feet.

Changes in flow velocity are generally minor under this alternative, with slightly (~1 ft/s) higher velocities predicted around the left side of the center pier of the upstream bridge, and in the center of the channel downstream of the downstream bridge. These changes in flow velocity produce negligible changes to the shear stress field over bar features in the project reach. **Figure 9.5** shows that critical sediment diameter and incipient motion potential for this alternative are largely unchanged from natural conditions over bar surfaces in the project reach. Potential impacts to whitewater recreational use are greatly reduced from that for Alternatives C, T, and S. The only issue to possibly contend with here is that the center pier for the upstream bridge is within the flow field at the low-flow condition. However, it is located along the right

side of the flow field as well as on the inside of the meander bend. As such, boaters would *not* likely need to fight momentum to avoid colliding with the pier – this is a positive aspect when compared to Alternatives C, T, and S. At the upstream bridge, boaters would have approximately 100 feet of river width to use in boating past the pier and bridge.

9.2 High-flow Results

9.2.1 Natural conditions

Plots of flow depth, flow velocity, shear stress, critical grain size diameter, and incipient motion potential under high-flow conditions (42,000 cfs) are shown for the project reach in its natural state (no crossings, with existing Highway 140 roadway above west bank only) in **Figure 9.6**. Flow depths reach up to nearly 30 feet in the thalweg below the Ferguson Rock Slide and in discrete portions of the thalweg in the bend immediately upstream of the slide. The left-bank bar upstream of the upstream temporary crossing currently in place would be inundated by at least 6 feet of water, as is the periphery of the right-bank bar just downstream and the mid-channel bar across from the slide. Flow velocity is predicted to reach roughly 20 – 27 ft/sec. over the riffle and grade drop at the slide, in the pool just upstream of the slide, and at the downstream boundary of the project reach within the thalweg. In the channel center, flow velocities are generally much less than these isolated high-velocity cells, with values of approximately 10 – 15 ft/s.

Shear stress calculations are performed over geomorphic units composed of material whose grain size distribution was measured in the field by Balance geomorphologists. As **Figure 9.6** illustrates, shear stress is relatively low over much of the left-bank bar upstream of the slide, with values ranging between 0 – 10 lb/ft^2. The right-bank bar upstream of the slide maintains slightly higher shear stress values along its inside margin, with values around 17-24 lb/ft^2. A progressive increase in shear stress values is expected towards the interior of the channel from these two bars, since flow depths and velocities increase towards the thalweg. The mid-channel bar across from the slide exhibits much higher predicted shear stresses because of its position relative to increased flow velocities under high-flow conditions over this relatively elevated bar feature. Shear stresses range generally between 17 and 51 lb/ft^2.

The plotted shear stresses result in critical bed sediment size distributions generally much higher than median sediment sizes sampled on the three bars shown. As a result, we would anticipate that most of the material along channel margins, on the river bed, and on bars not sampled, will be mobile under the simulated high-flow condition. On both the left-bank and right-bank bars upstream, the critical bed sediment sizes generally represent cobbles and

boulders (up to ~2000 mm diameter), except along the outside margins of these bars just below adjacent roadways (Highway 140 on the left bank and Incline Road on the right bank). In contrast, critical sediment size distribution on the medial bar across from the slide is much higher, ranging between 1000 – 5000 millimeters. This is a product of the large predicted shear stresses which would be exerted over this bar during the modeled high-flow.

The above shear stress and critical sediment size distributions yield potential incipient motion conditions which would provide for complete mobilization of the river bed and banks within the project reach. Most bar surfaces portrayed in **Figure 9.6** exhibit mobile conditions (shown in red), whereas only narrow ribbons characterized by geomorphically stable deposits remain along the periphery of the upstream bar deposits. The mobile character of these channel margin bar deposits would most likely extend into the middle of the channel, where no calculations have been performed, but where even higher flow velocity, depth, and shear stress conditions are being exerted on substrate of similar character.

9.2.2 Alternatives C and T

Plots of the difference between natural conditions and Alternatives C and T under high-flow conditions are shown in **Figure 9.7**. Specifically, changes in flow depth, velocity, and shear stress are shown, along with critical grain size diameter and incipient motion potential. The obliquely-oriented bridges for Alternatives C and T create slight increases in flow depth upstream of the two crossings. The predicted depth increases appear to be on the order of one foot and dissipate gradually a few hundred feet upstream of each crossing. Isolated increases are also predicted upstream of piers. Large portions of the channel downstream of each crossing exhibit no change in depth, suggesting a return to hydraulic conditions similar to the natural flow regime. **Figure 9.7** also illustrates localized depressions in water surface elevation of up to 4 feet downstream of the proposed bridge piers.

Changes in flow velocity are the direct result of flow being channeled through bridge openings between piers, and between proposed piers and the river banks. Flow velocity increases of up to 1 ft/sec are predicted for flow zones between piers at both the upstream and downstream bridges, while decreases in flow velocity are created at the wakes or eddies downstream of all of the piers. The most significant velocity increase of approximately 3 to 4 ft/sec is shown between the left bank pier of the upstream bridge and the left bank. This is simply and likely due to a loss in flow area due to the presence of the pier, coupled with the pier location within a velocity zone under natural conditions of 9 to 12 ft/sec on the high end (**Figure 9.6**). Flow separations and eddy re-circulations are also created in the wakes downstream of all of the

piers, resulting in velocity decreases that dissipate gradually within a few dozen feet downstream of each pier.

Increases in flow velocity between piers result in increases in localized shear stress, with values increasing by 1 lb/ft^2 or more compared to natural conditions. The increase in shear stress associated with piers is most pronounced on either side of the right bank pier at the upstream bridge, but shear stress increases are also expected at the downstream piers and in the channel center in upstream reaches, as suggested by the changes to the velocity field in the project reach. Shear stress has declined slightly in many locations on the left-bank bar at the upstream bridge, because the backwater effect of the upstream bridge is most pronounced along the left channel margin.

Fields of high critical sediment diameter have contracted slightly for Alternatives C and T, compared to natural conditions. This is a result of decreasing shear stress over much of the project reach under Alternatives C and T compared to natural conditions, except at locations of flow constriction between proposed piers. These reductions in critical sediment diameter yield contracted fields of potentially mobile sediment on bar surfaces, despite the increases in shear stress at localized areas between proposed piers. We still expect sediment in the channel center and around piers to experience enhanced mobilization compared to natural conditions, whereas in other locations that experience backwater effects and decreased flow velocities, sediment mobility will be reduced.

9.2.3 Alternatives S and the Temporary S Bridges

Plots of the difference between natural conditions and Alternative S under high-flow conditions are shown in **Figure 9.8**. **Figure 9.9** shows the difference between natural conditions and those under temporary conditions that would exist during the time of construction of alternative S. These temporary conditions incorporate an upstream crossing similar to that used in the no build alternative, but displaced farther upstream away from the proposed location of the permanent Alternative S crossing. In addition, the S temporary-condition modeling also incorporates the downstream crossing currently in place and incorporated into the no build alternative. Again, these two figures illustrate changes in flow depth, velocity, and shear stress, along with critical grain size diameter and incipient motion potential.

9.2.3.1 Alternative S

The obliquely-oriented bridges for Alternative S create moderate increases in flow depth upstream of the two crossings (**Figure 9.8**). The depth is shown to increase upstream of the

upstream bridge, starting with the maximum increase of approximately 1.5 feet occurring immediately upstream of the left bank upstream bridge pier. This depth increase gradually dissipates along a distance of nearly 600 feet upstream of the bridge. We would suggest that the depth increase shown for Alternative S in **Figure 9.8** is significantly higher in magnitude and extent than shown for Alternatives C and T (**Figure 9.7**), although the general pattern of the increases are similar. It would appear, based on these results that the proposed location of the upstream left bank pier is relatively important for flood conveyance within the model solution and as such leads to a model solution which would bring elevated water surface elevations upstream. This suggests the concept that the proposed upstream bridge for Alternative S will have a more significant hydraulic effect than that proposed for Alternatives C and T. More localized depth increases of approximately 1 foot are shown upstream of the other piers. Large portions of the channel downstream of each crossing exhibit no change in depth, suggesting a return to hydraulic conditions similar to the natural flow regime. Depressions in water surface elevation up to four feet are predicted to occur downstream of the bridge piers; these depressions are predicted to decay within 10 to 20 feet downstream of the piers.

Changes in flow velocity are the direct result of flow being forced around piers, in between pier openings, and between piers and the left and right banks. **Figure 9.8** shows that flow velocity increases of up to 1.7 ft/sec could occur across the right bank bar upstream of the upstream bridge, up to 2.5 ft/sec on each side of the right bank pier of the upstream bridge, and up to 2 ft/sec on each side of the left bank pier of the downstream bridge. These again are more significant than that illustrated in **Figure 9.7** for Alternatives C and T. Localized decreases in flow velocity are created at backwater features upstream of the piers at each crossing (**Figure 9.8**). Flow separation and eddy re-circulation also occurs downstream of the piers, resulting in velocity decreases on the order of 3 ft/s that dissipate within 20 to 40 feet downstream of each pier, depending on the velocity of the incoming flow.

Increases in flow velocity between piers result in increases in shear stress between piers, with shear stress increasing by 1 lb/ft^2 or more compared to natural conditions (**Figure 9.8**). The increase in shear stress associated with piers is most pronounced just upstream of the right bank pier at the upstream bridge, where flow is accentuated in anticipation of the flow constriction between piers. Shear stress increases are also expected at downstream piers and in the channel center in upstream reaches, as suggested by changes to the velocity fields in the project reach. Shear stress has decreased slightly in many locations on the left-bank bar upstream of the upstream bridge, due to the backwater effect of the upstream bridge, and because of increased bed friction associated with the bar surface.

Fields of high critical sediment diameter have contracted slightly for alternative S (**Figure 9.8**), compared to natural conditions. This is a result of decreasing shear stress over much of the project reach under alternative S, compared to natural conditions, except at locations of flow constriction in between piers. These reductions in critical sediment diameter yield contracted fields of potentially mobile sediment on bar surfaces, despite the increases in shear stress at localized areas in between piers. We would still expect sediment on the river bed, and in and around the proposed piers to experience enhanced mobilization compared to natural conditions. However, locations that are predicted to experience backwater effects and decreased flow velocities, sediment mobility will be reduced. Compared with the incipient motion field developed for Alternatives C and T, the potentially mobile domain on the point bar along the right-bank bar upstream of the upstream bridge would increases in size slightly, creeping up the bank. This is due to the placement of the pier along the right bank at the upstream crossing, which is farther downstream than that for the analogous pier in Alternatives C and T. Thus, material on this bar surface may experience enhanced mobility under this alternative compared with conditions for Alternatives C and T.

9.2.3.2 Alternative S Temporary Bridges

Modeled flow depths under the temporary crossings for Alternative S (**Figure 9.9**) show a backwater effect at the upstream temporary bridge that extends a few hundred feet upstream. The flow depth is shown to increase by up to 2 feet immediately upstream of the two piers, which are located within the channel with the right bank pier located in the thalweg.

The downstream temporary crossing for Alternative S uses the same bridge as the No-build Alternative, the left abutment of which significantly constricts the channel width. Model results for the downstream Alternative S temporary bridges show that the channel constriction would cause a depth increase ranging from 1 to 2 feet immediately upstream of this crossing, with a backwater effect that extends nearly to the rapid at the toe of the slide debris.

The velocity increases associated with the upstream temporary S bridge are significant, which is primarily due to the fact that the proposed piers would be located in high velocity flow zones coupled with the notion that the right bank pier would be located within the thalweg of the river. Velocity increases of up to 4 ft/sec are shown between the pier openings and between the right bank pier and the right bank, at the upstream temporary crossing for Alternative S (**Figure 9.9**). Flow velocity also decreases immediately upstream of, and in the wake downstream of piers. The results suggest that the wake downstream of the right bank pier gradually decays over a distance of nearly 200 feet. As in the no build alternative, changes in flow velocity at the

downstream temporary crossing are less pronounced, with an increase of approximately 1 ft/sec in the constricted portion of the channel at the left-bank bridge abutment.

Shear stress increases of a few lb/ft^2 are predicted for zones in between piers, and in between piers and the left and right banks. Calculations of change in shear stress, again, are limited to bar surfaces with grain size characterization obtained in the field, but we would expect similar increases in shear stress to occur within the full extent of flow in between piers.

Because of only slight changes in shear stress on most bar surfaces away from the piers supporting the upstream temporary crossing, the distribution of critical sediment diameter (**Figure 9.9**) is away from the piers qualitatively very similar to that calculated for natural conditions (**Figure 9.6**). As such, the incipient motion potential is nearly identical to that developed under natural conditions away from the vicinity of the piers. It is clear, however, that enhanced flow velocities and shear stresses in the upstream temporary bridge openings will potentially enhance sediment mobility at high flow during the construction of permanent structures associated with Alternative S.

9.2.4 Alternative S2

Because information was not available to model Alternative S2, distributions of changes in flow depth, flow velocity, and shear stress, along with depictions of critical sediment diameter and incipient motion potential are not included for Alternative S2. However, we can provide generalized statements about the potential effects of this alternative on flow depths, flow velocity, shear stress, and incipient motion based on results obtained for Alternative S, as requested by Caltrans. Alternative S2 utilizes a similar downstream crossing to that of Alternative S, while the upstream crossing is more similar to that for Alternatives C and T. Thus, we would expect flow depths and flow velocities downstream of the slide to be similar to those shown in **Figure 9.8** for Alternative S, with flow depths increasing upstream of the crossing due to suggested backwater effect. This would lead to associated flow velocity increases within flow zones in between piers. Similarly, the depiction of flow depths and velocities upstream would be closely related to results obtained for Alternatives C and T (**Figure 9.7**). Shear stresses would increase in between piers for both the upstream and downstream crossings. One potential distinct difference is that the proposed upstream crossing for Alternative S2 is placed even further upstream than that proposed for Alternatives C and T. Thus, the shear stress enhancement, and consequent sediment mobility, will more strongly affect the large bar along the left bank rather than the right-bank bar. In particular, the small high-flow channel along the left bank margin, just below the Highway 140 roadway (see **Plate**

1) may experience significant erosion and scour due to focused flow in between the left bank pier and the stabilized left bank. This potential is compounded by the fine-grained alluvium (mostly sands) that exists at this location.

9.2.5 Alternative A

Because information was not available to model Alternative A, distributions of changes in flow depth, flow velocity, and shear stress, along with depictions of critical sediment diameter and incipient motion potential are not included for Alternative A. However, we can provide generalized statements about the effects of this alternative on flow depths, flow velocity, shear stress, and incipient motion based on results obtained for the No Build Alternative, as requested by Caltrans. Thus we would expect flow depths and flow velocities to be similar to those shown in **Figure 9.10** for the No Build Alternative. Specifically, flow depths would increase upstream of the proposed bridges, due to a slight backwater effect and flow velocities would likely increase in between proposed abutments. Shear stresses would reflect velocity dynamics and likely increase in between proposed abutments for both the upstream and downstream crossings. One potential distinct difference between the No Build Alternative and Alternative A is that the proposed upstream crossing for Alternative A is placed further downstream than the upstream crossing currently utilized in the No Build Alternative. Thus, shear stress enhancement, and consequent sediment mobility might be reduced for Alternative A along the right bank bar upstream of the proposed bridge. Potential bed scour may be accentuated through the meander at the upstream crossing for Alternative A, but modification of the right-bank bar may be minor to negligible.

9.2.6 No-build alternative

Plots of the difference between natural conditions and the No Build Alternative under high-flow conditions are shown in **Figure 9.10**. Specifically, changes in flow depth, flow velocity, and shear stress are shown, along with critical grain size diameter and incipient motion potential. The No Build Alternative utilizes the current temporary crossings in place for the Highway 140 detour. The upstream crossing has one pier located approximately in the middle of the high-flow channel. The downstream bridge is the same as the temporary bridge for Alternative S that was previously discussed. It would constrict the channel significantly under high-flow conditions but does not have any piers. At the upstream bridge, a localized increase in depth is illustrated immediately upstream of the mid-channel pier, and the bridge would cause a backwater that seems to influence the depth for approximately 500 feet upstream. Small, localized depressions in water surface elevation of up to 2 feet are shown immediately

downstream of the right bank pier and right bank abutment of the upstream bridge, and the left bank abutment of the downstream bridge.

Flow velocity is predicted to increase by as much as 3 feet per second under the high-flow condition as flow is forced around the right bank pier and abutment of the upstream bridge, and through the constricted channel at the downstream bridge (**Figure 9.10**). These changes in flow velocity produce slight changes to the shear stress field over bar features in the project reach. **Figure 9.10** shows that increases in velocity in between the piers at the upstream crossing would result in increases in shear stress of several lb/ft^2. Alternatively, slight decreases in flow velocity over the left-bank bar upstream of the upstream bridge, and the medial bar across from the slide would result in slight decreases in shear stress. As a result of the decreases in shear stress, critical sediment diameter correspondingly decreases compared to natural conditions. In isolated locations on the medial bar across from the slide, critical sediment diameter reaches into the 1000s of millimeters (large boulders), while much of the upstream bars are characterized by critical sediment sizes ranging between cobbles and boulders. Because critical sediment size has decreased, a smaller portion of the bars within the project reach are potentially mobile, as shown by the larger proportion of stable locations (shown in yellow) on bars in **Figure 9.10**. However, the medial bar across from the slide remains largely mobile.

9.3 Bridge Scour

We assessed contraction and pier scour associated with the proposed bridges for the high-flow (42,000 cfs) condition. As stated earlier, we relied on scour analysis guidance from HEC-18 (Richardson and Davis, 2001) in addition to utilizing scour equations developed by Blodgett (1986), Lacey (1930), Blench (1969), and Competent/Limiting Velocity Control (Pemberton and Lara, 1984).

Our results for bridge scour vary widely, owing to the empirical nature of the various equations used, and the inherent inaccuracy of measuring required input parameters, such as the bottom width of the contracted section prior to scour. Nevertheless, our scour estimates may highlight temporary bed elevation change in the vicinity of the proposed crossings, and supplement our estimates of sediment transport along channel bars depicted in **Figures 9.1 – 9.10**. Although our scour calculations pertain to the entire cross section at bridge crossings, bridge crossings used for analysis are limited to those close to channel deposits where grain size information was collected. This limits most of our scour analysis to the proposed upstream crossings. Scour

estimates at the downstream crossings may be viewed within the context and range of values associated with the following results for the upstream crossings.

For the upstream temporary crossing currently in place as part of the No Build Alternative, Laursen's clear-water scour equation for contraction scour yielded an estimate of 0.88 ft, which is roughly equivalent to estimates using Lacey (1930) of 0.95 ft. Estimates using Blodgett (1986) and Blench (1969) were an order of magnitude higher, at 10.56 and 19.05 ft, respectively. Using the competent/limiting velocity control relationship gave unrealistically large magnitudes of scour. The higher estimates ignore the actual depth to bedrock, which is generally within 10-15 feet of the river bed surface, so an upper estimate of 10 feet seems reasonable. Lower estimates suggest nearly negligible bed scour. Pier scour estimates using the CSU local pier scour equation yield 6.9 feet of scour. These estimates most likely represent temporary scour depths because it is well documented that these bed elevation changes are usually not permanent, as sediment usually fills some or all of the scoured volume during the falling limb of a hydrograph (Richardson and Davis, 2001).

For Alternative C and T, Laursen's clear-water scour equation for contraction scour yielded negligible scour, while Lacey (1930) yielded an estimate of 0.94 ft. Estimates using Blodgett (1986) and Blench (1969) were an order of magnitude higher, at 10.56 and 18.67 ft, respectively. Using the competent/limiting velocity control relationship gave unrealistically large magnitudes of scour. As above, the higher estimates ignore the actual depth to bedrock, which is generally within 10-15 feet of the bed surface, so an upper estimate of 10 feet is reasonable.

As for the temporary crossing under the No Build Alternative, lower estimates suggest nearly negligible bed scour. Pier scour estimates using the CSU local pier scour equation yield 7.8 feet of scour. These estimates again should represent temporary scour conditions because it is well documented that these bed elevation changes are usually not permanent, as sediment usually fills some or all of the scoured volume during the falling limb of a hydrograph.

For Alternative S, Laursen's clear-water scour equation for contraction scour yielded negligible scour, while Lacey (1930) yielded an estimate of 1.14 ft. Estimates using Blodgett (1986) and Blench (1969) were an order of magnitude higher, at 11.31 and 24.16 ft, respectively. Using the competent/limiting velocity control relationship gave unrealistically large magnitudes of scour. Again, we would suggest an upper scour estimate of 10 feet due to the general depth to bedrock through the project reach. As for the temporary crossing under the No Build Alternative, lower estimates suggest nearly negligible bed scour. Pier scour estimates using the CSU local pier

scour equation yield 9.15 feet of scour. We would again expect scour conditions to be temporary in nature as sediment would likely fill some or all of the scoured volume during the falling limb of a hydrograph.

For the temporary crossings associated with construction of Alternative S, Laursen's clear-water scour equation for contraction scour yielded 3.47 ft, while Lacey (1930) yielded an estimate of 1.09 ft. Estimates using Blodgett (1986) and Blench (1969) were an order of magnitude higher, at 11.62 and 20.24 ft, respectively. Using the competent/limiting velocity control relationship gave unrealistically large magnitudes of scour. Pier scour estimates using the CSU local pier scour equation yield 8.8 feet of scour. Again, maximum scour depth and scour persistence parallels that provided above.

Because the scour results for each alternative above are relatively similar, we expect that scour estimates would be similar for alternatives A and S2. An upper bound on scour may reach approximately 10 feet, while scour may also be negligible under certain flow regimes and bed conditions. Pier scour for alternatives A and S2 may reach 5 feet or so, as suggested by pier scour estimates above.

The scour estimates above may be intensified, and more permanent for the case of drift hazard and accumulation at bridge piers during flood events. This may be a particular concern for the upstream temporary crossing proposed for Alternative S, which has a lower soffit elevation combined with a pier placed in the thalweg and another over a relatively mobile bar. This configuration will be more likely to capture large woody debris that could be transported during a large flood event. The resulting debris jam would obstruct flow area and possibly increase the scour potential at the structure. The temporary nature of this upstream crossing renders the hazard less likely to occur than for a more permanent structure, but the hazard should be acknowledged by design engineers and planners involved in the selection and construction of the Highway 140 permanent restoration alternative.

9.4 Water Quality and Episodic Events Considerations

Balance has not explicitly analyzed potential projects effects to river water quality (i.e. turbidity) as little no background data exists, and several likely outcomes can be reasoned from practical considerations. First, project water quality considerations should be focused on how the project could affect those processes which drive water quality. Given that the local geology is characterized by metamorphic rocks, and not fine-grained sedimentary rocks characteristic of the Coast Ranges, the production of fines will likely be driven by slope failures which bring soil

and pulverized rock into the river. The project could affect slope failure processes for those alternatives which would utilize an open-cut for the proposed road bed along the right side of the river. Some useful recommendations have been provided by Beck (2007b) with regards to construction considerations for open-cuts, and the U.S. Forest Service has prepared an erosion control plan for implementation during construction with a focus on reduction of construction driven erosion. If an open-cut alternative is chosen for implementation, we would recommend more careful consideration of the potential for cut to affect slope failure processes, and the possible ultimate introduction of fine material to the river. Second, the river bed through the project reach is moderately steep and is largely composed of very coarse substrate. Fines are limited primarily to sand which has been deposited on bars along the river flanks. While erosion of the river bed has been predicted for many of the alternatives, this erosion is unlikely to affect turbidity during floods as material which will be eroded is likely to primarily range in size from gravels to boulders. Third, on many river systems bank failures represent real chronic sources of turbid material to rivers. Given the occurrence of bedrock or other hardened structures along much of the banks of the project reach, it seems unlikely that future bank failures would lead to a measurable increase of turbidity during floods. The last water quality consideration deals with road runoff. Depending on how the bridges are drained, potentially serious pollutants could enter the river in the event of a spill, accident, or general mechanical failure of a motorized vehicle. We would recommend that stormwater runoff be directed away from the river and off of the bridges to properly designed features to contain contaminated liquids.

Forest fires, mass movements, and extreme flood events can lead to the introduction of large volumes of debris to rivers. Bridges built over rivers can easily become hot spots for debris accumulation or rafting. This potentiality can be elevated in the event of massive sediment deposition following a forest fire – although this risk may be minimal through the project reach given the existing bed slope. While Balance did not explicitly analyze for the affects of debris rafting, etc., it is important to point out that large woody debris has been deposited by past floods through the project reach (**Plate 1**). Many of the risks associated with debris rafting are reflected in the discussion given above for whitewater recreational considerations. Left-hand piers associated with the upstream bridges for Alternatives S, C, and T, as well as piers for the temporary bridges all pose risk to debris rafting. Structural design of the piers and bridge decks should reflect this possibility, and some type of management plans developed for storm related observations of debris rafting development.

10. RECOMMENDATIONS TO FACILITATE SELECTION OF THE PREFERRED HWY 140 TRANSPORTATION ALTERNATIVE

Our field observations and measurements, coupled with aerial photography interpretation, suggest that the project reach is at a critical location within the larger Merced River canyon. The project reach is located immediately downstream of the South Fork confluence, as well as downstream from a considerable supply of glacially-derived sediment and debris mobilized from deposits near and within Yosemite Valley. This geographic paradigm corroborates field-derived evidence which suggests that the project reach characteristically exhibits large-scale sediment deposition and erosion cycles. These cycles are tied to episodic events such as very large floods, fires, and debris flows – events such as these have the propensity to deliver large volumes of sediment to the river corridor. As such, the project reach represents a zone of mixing and storage of sediment and river debris, where deposits along channel margins have potentially long lifetimes. It may be reasonable to expect at least one large-scale cycle of sedimentation to occur during the lifetime of the implemented transportation project, as we have seen with the re-activation of the Ferguson Rock Slide, and passage of the 1997 rain-on-snow flood.

In this dynamic setting, regular features associated with rivers such as floodplains, terraces, or continuous bars may be short lived, and more dynamic than the norm. Moreover, common fluvial geomorphic concepts such as bankfull, meander geometry, or pool-riffle spacing may be neither as articulated, nor as useful in predicting future river adjustment. However, geomorphic mapping, aerial photography interpretation, and hydraulic and bed sediment mobility modeling all suggest relative stability of channel deposits, with only large, infrequent events being capable of evacuating materials from this reach. Additionally, large events may replace or even supplement materials eroded, as was demonstrated during the 1997 event.

Results from our 2-D hydraulic model coupled with results from our sediment mobility analysis suggests that the 6 proposed transportation alternatives will likely influence river morphology in very localized ways, rather than across the reach as a whole. Specifically, all of the proposed transportation alternatives will likely result in local bed erosion due to enhanced flow velocities and shear stresses through bridge openings. However, erosion depths will likely be limited due to the presumed coarse nature of deposits one to two grain diameters beneath the present day river bed. Correspondingly, adjustment and erosion of bar deposits in the vicinity of piers is likely as the piers would certainly affect the flow field and generally enhance velocities and shear stresses in bed areas around the piers. Across all transportation alternatives, bar erosion

at the upstream bridge locations is a very likely outcome as the piers are located within or at the margins of present-day bars. The landslide bar immediately downstream of the Ferguson Rock Slide on the left bank is likely to be a long-lived feature regardless of the effects of the selected Highway 140 transportation restoration alternative.

From a planning perspective, the most significant finding could be that the present design location of piers along the left bank at the upstream end of the project could pose significant threats for the safety of whitewater enthusiasts at the modeled whitewater flow (8,800 cfs). These piers generally occur where flow is converging on the left bank as it accelerates into the near 90-degree meander mid-way through the project reach. This hydraulic condition will make it difficult for whitewater enthusiasts to safely navigate the river past the piers as they will be fighting momentum and the overall current pulling them towards the left bank. This hydraulic condition is also generally valid at the modeled flood flow (42,000 cfs) condition, however it is highly unlikely that their will be recreational use of the river at the modeled flood flow.

For specific environmental review and planning purposes, and to aid in the selection of the preferred transportation alternative, we have prepared **Figure 10.1**. **Figure 10.1** illustrates suggested bridge pier feasibility zones; zones are broken into *preferred* (blue fields) and *not recommended* (red fields) zones. The basis for developing this figure stems from the results of our cross-disciplinary, and multiple lines of evidence approach. Preferred zones would generally (1) be located above the water surface elevation for the low-flow condition, and (2) place piers out of hydraulically important areas such as the left side of the river as it approaches the meander mid-way through the project reach. These two considerations factor importantly when reviewing potential impacts to whitewater recreational use and existing river form and function. Utilizing this basis for development of suggestions highlighted in **Figure 10.1** should provide for limited to negligible impact to whitewater recreational use at the modeled low-flow condition, and should decrease the highlighted potential impacts to the nature and extents of mapped bars and riffles. The extent of the preferred zones may decrease if whitewater use is tested against the proposed alternatives at flows that are much higher than our modeled flow of 8,800 cfs – we suggest the upper limit to this would be about 9,500 to 10,000 cfs.

To supplement **Figure 10.1** and close out points raised in Chapter 9 above, we provide the following specific observations to further guide selection of perhaps the most river appropriate transportation restoration alternative for the Highway 140 corridor in the Merced River canyon at the Ferguson Rock Slide:

Whitewater Recreational Use at the Low-flow Condition:

- The upstream piers proposed along the left bank for Alternative S, and to a lesser extent those proposed for Alternatives C and T, pose significant safety hazards for whitewater rafters and boaters. The threat is more significant with Alternative S, than it is for Alternatives C and T. However, we highly recommend that these piers be re-located for all three alternatives to be, if possible, consistent with the preferred zones illustrated in **Figure 10.1**.

- The right bank pier for the upstream Alternative S temporary bridge would pose significant temporary safety hazards for whitewater rafters and boaters. The pier would be located slightly to the right of the thalweg (**Figure 9.4**) as flow would curve gently to the left moving past the head of the large mapped bar along the left bank of the river (**Plate 1**). At the modeled whitewater flow, this pier would be located within the main flow zone and rafters and boaters would have to work against momentum to avoid colliding with the pier. The potential danger would be enhanced if debris were to raft against the pier because the danger zone would be broader. We highly recommend that a different temporary bridge design be developed to avoid the potential serious safety hazards for rafters and boaters.

- Proposed piers of downstream bridges for all alternatives appear to present minor to negligible hazards for whitewater rafters and boaters.

Protection of Present River Form (depositional architecture) and Function:

- Alternatives C, T, and S all pose some potential to permanently alter the nature of the existing river through the project reach, although all of these instances would be largely isolated to areas immediately adjacent to the bridges and piers. Most of the potential for permanent change is associated with the high-flow channel and distal end of the bar mapped along the left bank, at the upstream bridges. Less significant potential would exist for the right bank bar mapped at the upstream bridges. These changes could include development of scour holes in bars, truncating the river side or downstream ends of these bars, erosion of the riffle mapped there, and perhaps upstream enlargement of the large pool mapped through the meander mid-way through the project reach. If these potential impacts are deemed unacceptable, we would recommend revising designs to be more consistent with the preferred zones highlighted in **Figure 10.1**.

- The upstream temporary bridge for Alternative S would pose significant, temporary potential impacts to the nature of the river there. The proposed piers for the temporary bridge could lead to development of scour holes within an existing riffle, and could significantly truncate the river side of the left bank bar mapped there. While the bridge would be temporary as would the potential impacts, it is difficult to speculate on the likely durations of the impacts. We would suggest that the impacts are significant enough to warrant a temporary bridge re-design.

- The above stated concerns are of less significance for Alternative S2, given the limited information which we presently have for this alternative.
- It would appear that Alternative A would pose minor to negligible potential impacts to the nature of the present river corridor, again based on the limited information which we presently have for this alternative.
- We would suggest that the downstream bridge alternatives for all alternatives would pose minor to negligible potential impacts to the nature of the present river corridor.

Debris Rafting Potential and Episodic Sedimentation:

- Debris due to massive floods, wild-fires, and future mass movements into the river all pose a significant threat to rafting of material on proposed bridge piers. While we have not explicitly analyzed for this risk, anyone working in mountain basins understands that the risk is real. Specific alternatives which have perhaps more risk for debris rafting on piers would include: Alternatives C, T, and S (left bank upstream piers), the upstream temporary bridge for Alternative S, and the No Build Alternative (upstream bridge pier). This risk would also hold for ice rafting, although the risk is likely relatively smaller given the steepness of the reach, the steepness of upstream reaches, as well as its elevation in the watershed. Given these potential risks, we suggest that designs fully account for debris and ice rafting potential on the bridge piers.
- The potential for massive bed aggradation due to episodic sedimentation is always a concern in large California rivers. The most salient recommendation here could perhaps be to elevate the bridges as high as is feasible and practical.

Other Considerations:

- Key to the selection process of proposed alternatives is the acknowledgement that flood flows above those simulated in this study will have the potential to engage the proposed bridge structures. Our simulated high flow of 42,000 cfs is at a threshold above which bridge decks would appear to begin interacting with flood flows. In terms of planning needs, our simulated high flow is significantly below some estimates of the 100-year event. Flows equivalent to or larger than our high-flow event can trap ice and debris behind structures, and flows much larger than our high-flow event may potentially cause a bridge to fail.

11. LIMITATIONS

This study is not intended to provide information on the status or stability of the Ferguson Slide, nor does it assess the probability of another mass movement occurring—only how the river may respond to the supplemental source of material. The study addresses how canyon-bottom morphology may change with the introduction of the proposed alternative Highway 140 restoration elements, relative to the effect of the slide mass interacting with the river.

Although our assessment of geomorphic change within the Merced River canyon at bridge locations partly utilizes empirical scour calculations included as part of the HEC-18 analysis specified by the U.S. Army Corps of Engineers and the Federal Highway Administration for bridge scour analysis, we do not intend to conduct a full scour analysis nor do we intend our scour results to be interpreted as part of a scour analysis. Our use of the scour relationships provided in HEC-18 is solely for the purpose of examining transient bed elevation change after the introduction of Highway 140 restoration elements, and to estimate how temporary bed elevation change may influence overall geomorphic change within the river corridor at the project site. Associated with this effort is the use of empirical scour equations not included in the HEC-18 manual, which will provide sensitivity testing for an assessment of transient incision at bridge crossings.

We also stress that the calculation of incipient motion used to assess channel-bed, bar, and terrace stability at the project site is not meant to imply bed elevation change (temporary or permanent, although this may occur). Our sediment transport calculations are used to measure how transport conditions may change after Highway 140 restoration, and how this may lead to temporary or permanent morphologic change in the canyon bottom.

Finally, although we use fully-calibrated 1D (HEC-RAS) and 2D (FESWMS) hydraulic modeling to ascertain flow conditions and sediment transport driving forces, this study is not intended to serve as a basis for flood management or detailed floodplain planning, both of which are conducted by well-defined and separate procedures, and which frequently require multiple lines of evidence not included in the data collection and analyses performed here. Our "high-water" flows should not be construed as the 100-year flood, and the use of these results for purposes other than those identified above can lead to significant environmental, public-safety, or property losses. We are also not examining the safety of the Highway 140 and Incline roadways under our modeled scenarios, nor are we assessing the stability of the roadways or bridges under the flow conditions simulated.

We ask that Balance Hydrologics be contacted should there be any questions regarding how our findings may be validly used. Readers who have additional pertinent information, who have observed changed conditions, or who may note material errors are respectfully asked to contact any of the authors with their findings at the earliest possible date, so that timely changes may be made. New information could influence computations, analyses, or recommendations.

12. REFERENCES

Aplin, P.S., and Hill, D.J., 1979, Growth analysis of circular lichen thalli: Journal of Theoretical Biology, 78, p. 347-363.

Armstrong R.A., 1983, Growth curve of the lichen *Rhizocarpon geographicum.* New Phytol 94, p. 619-622.

Bateman, P.C. and Krauskopf, K.B., 1987, Geologic map of the El Portal quadrangle, west-central Sierra Nevada, California: U.S. Geological Survey MF-1998.

Bateman, P.C., 1992, Pre-Tertiary bedrock geologic map of the Mariposa 1° by 2° quadrangle, Sierra Nevada, CA: United States Geological Survey, Miscellaneous Investigations Series Map I-1960.

Beck, T.J., 2007a, Preliminary Geotechnical Report: Department of Transportation, Division of Engineering Services, Geotechnical Services – MS 5, 13 p. + plates + appendices.

Beck, T.J., 2007b, Geotechnical Design Report: Department of Transportation, Division of Engineering Services, Geotechnical Services – MS 5, 11 p. + plates + appendices.

Blench, T., 1969, Mobile-Bed Fluviology: University of Alberta Press, Edmonton, Canada.

Blodgett, J.C., 1986, Hydraulic characteristics of natural open channels, *in* U.S. Geological Survey Water-Resources Investigations Report 86-4127: Rock riprap design for protection of stream channels near highway structures, Volume 1.

Blodgett, J.C., 1989, Assessment of hydraulic changes associated with removal of Cascade Dam, Merced River, Yosemite Valley, California: U.S. Geological Survey Open-File Report 88-733, 19 p.

Buffington, J.M. and Montgomery, D.R., 1997, A systematic analysis of eight decades of incipient motion studies, with special reference to gravel-bedded rivers: Water Resources Research 33(8), p. 1993-2029.

Buffington, J.M., Dietrich, W.E., and Kirchner, J.W., 1992, Friction angle measurements on a naturally formed gravel streambed: Implications for critical boundary shear stress: Water Resources Research 28(2), p. 411-425.

Caltrans (State of California Department of Transportation), 2007, Ferguson Slide Permanent Restoration Project, Initial Study with Proposed Mitigated Negative Declaration/Environmental Assessment: 111 p.

Chow, V. T, 1959, Open-channel Hydraulics, McGraw-Hill, St. Louis , Mo, 678 p.

Cipponeri, A.., 2007, Location Hydraulic Study, Merced River at Ferguson Slide area: State of California Department of Transportation, 8 p. + attachments.

Denlinger, R.P., 2007, Simulations of potential runout and deposition of the Ferguson Rockslide, Merced River Canyon, California: U.S. Geological Survey Open-File Report 2007-1275, 11 p. + figures.

Drivas, T.A., Gould, D., Burkindine, S., Eicher, J., Stone, K.A., Clines, J., Strand, P., DeGraff, J., and Mogge, M., 2008, Section 7(a) Evaluation, Wild and Scenic Rivers Act, Merced River, Ferguson Slide temporary bridges restoration project 10-OP440, 36 p.

Gallegos, A.J. and DeGraff, J., 2006, Ferguson Rock Slide geology report: United States Department of Agriculture, Forest Service, 15 p. + plates + appendices.

Haan, C.T., Barfield, B.J., and Hayes, J.C., Design Hydrology and Sedimentology for Small Catchments, Academic Press, San Diego, CA, 588 p.

Harp, E.L., Reid, M.E., and Godt, J.W., 2006, USGS memorandum re: Ferguson Rock Slide, Mariposa County, California: United States Department of the Interior, U.S. Geological Survey, 6 p.

Huizinga, R.J., 2007, Two-dimensional hydrodynamic modeling and analysis of the proposed channel modifications and grade control structure on the Blue River near Byram's Ford Industrial Park, Kansas City, Missouri, Scientific investigations report 2007-5098 by the United States Geological Survey (USGS), 45 p.

Kondolf, G.M. and Cook, S.S., 1986, Reconnaissance-level studies of geomorphic and hydrologic questions relevant to assessment of environmental impacts of the proposed El Portal Hydroelectric Project (FERC no. 3581): Consulting report prepared for the Environmental Sciences Division of Oak Ridge National Laboratory, 53 p.

Lacey, G., 1930, Stable channels in alluvium, *in* Proceedings of the Institution of Civil Engineers, Vol, 299.

McGaugh, R.L., 2007, Office of Structures, Final Hydraulic Report: State of California Business,

National Park Service, 2005, Merced Wild and Scenic River revised comprehensive management plan and supplemental environmental impact statement. Parts 1 and 2 plus Appendices. Multi-paged.

Pemberton, E.L. and Lara, J.M., 1984, Computing degradation and local scour: prepared for the U.S. Department of the Interior, Bureau of Reclamation, Publication Number 7-2090.

Richardson, E.V. and Davis, S.R., 2001, Evaluating scour at bridges, 4th Edition: U.S. Department of Transportation, Federal Highway Administration Publication FHWA NHI 01-001, Hydraulic Engineering Circular No. 18, 378 p.

Strand, R.G., 1967, Geologic map of California, Olaf P. Jenkins edition, Mariposa Sheet: Division of Mines and Geology, State of California.

Vionnet, C.A, Tassi, P.A., Martin Vide, J.P., 2004, Estimates of flow resistance and eddy viscosity coefficients for 2D modeling on vegetated floodplains: Hydrologic Processes 18, p. 2907 – 2926.

TABLES

Table 1. Peak Flow Hydrology Metrics, Merced River, near Ferguson Rock Slide, California

Gaging Station (USGS ID#)	Site Description (USGS)	Location (mi from slide)	Watershed Area (mi^2)	Period of Record (year - year)	Mean Annual Flow (cfs)	1951 Peak Flow Discharge at Station (cfs)	1956 Peak Flow Discharge at Station (cfs)	1997 Peak Flow Discharge at Station (cfs)	2005 Peak Flow Discharge at Station (cfs)
11268500	Merced R. at Bagby, CA	20 DS	911	1923-1966	1182	83,000	92,500	n/a	n/a
11268200	Merced R. at Briceburg	5 DS	691	1965-1974	1222	n/a	n/a	n/a	n/a
11266500	Merced R. at Pohono Bridge, NR Yosemite CA	16 US	321	1917-2007	627	23,000	23,400	24,600	10,200
11268000	SF Merced River NR El Portal, CA	0.8 US	241	1950-1975	350	37,600	46,500	*42,500 - 44,500* [1]	n/a
11268001	Resultant of combining 11-2665 and 11-2680 at SF [2,3]	0.8 US	661	1951-1975	858	n/a	n/a	n/a	n/a
	Watershed area between project site and upstream gages:		*99*	--	--	--	--	--	--
	Merced River at Ferguson Rock Slide (project site) [4,5]:		661	--	--	*40,400 - 45,950*	*41,100 - 46750*	*43,250 - 49,150*	*17,000 - 20,380*

Notes:

1. The range of flows calculated for SF Merced during 1997 flood is based on results from two methods: 1) regression of rain-on-snow events between gaging stations (SF Merced NR El Portal = 2.4158 * (Merced R. at Pohono Bridge) - 16892); and 2) an average ratio of the peak flows (mulitplier) between SF Merced nr El Portal to Merced R. at Pohono Bridge using the two known flows recorded in 1951 and 1956. This ratio was 1.81.
2. Peak flow data was not available for this gage. Mean daily flows were used.
3. Watershed area assumes that gage 11268001(Described as "Resultant of combining 11-2665 and 11-2680") represents total flow of Mainstem and South Fork at El Portal.
4. Watershed area at Ferguson Rock Slide, obtained from Caltrans Location Hydraulics Study, 2005
5. The lower range of the flows at Ferguson Rock Slide was estimated based on correlation of peak and mean daily flow data at Briceburg with peak flow data at Pohono for flows at Pohono greater than 5000 cfs. The high range of flood discharges at the Ferguson Slide were estimated based on correlations between mean daily discharges at gage 11268001 and 1126650 (Pohono) for flows at 1126650 greater than 8000 cfs (ratio ~ 2).

©2008 Balance Hydrologics, Inc.

Table 2. Geomorphic dating methods for fluvial landforms, Merced River near Ferguson Slide, Mariposa County, California

	Cross-section	Channel Location	Landform	Landform surface elevation (relative)	*Rhizocarpum geographicum* (lichen)	Trees	High Water Marks	Remarks
					size (mm)	*(Type, inches DBH)*	*(type)*	
UPSTREAM	AB-AB'	Main stem, 1,000-ft above confluence	Bank	3 ft below HWY 140	--	--	LWD	Recent, likely 1997 HWM
	--	Main stem, 500-ft above confluence	Floodplain	8-ft above WSE	--	--	LWD	Large mature Pondorosa pine growing on floodplain; approx. 3+ ft diameter
	--	Main stem, 500-ft above confluence	Terrace	14-ft above WSE	25	pine, 4 - 6	--	Cobbles with R. geog. are sub-rounded granite
	B-B'	Main stem, 200-ft above confluence	Floodplain	10-ft below HWY 140, ~8-ft above WSE	--	pine, 36+	LWD	Abundant moss on boulders, subrounded granite
	A-A'	South Fork, 500-ft above confluence	Bar	4-ft above WSE	--	willow, 2 - 3	--	Estimated age of willows 8+ years
	--	Main stem, 150-ft below confluence	Floodplain	9-ft above WSE	--	willow, 3+	LWD	Estimated age of willows 12-20 years
	C-C'	Main stem, 500-ft below confluence	Floodplain	7-ft above WSE	--	pine, 42+	LWD	Sandy deposit around angular to sub-rounded boulders; willow decrease in age down bank to water surface; LWD growing new willow sprouts est. age approx 10+ years
	D-D'	Main stem, 1,500-ft below confluence	Bank	2.5-ft below HWY 140	--	--	LWD	Outside bend, recent, likely 1997 HWM
	E-E'	Main stem, 2,300-ft below confluence	Floodplain	12-ft above WSE	--	--	deposit, LWD	Distinct break-in-slope, larger boulders above; LWD at same elevation on opposite bank; likely a 'bankfull' elevation
	E-E'	Main stem, 2,300-ft below confluence	Terrace	6-ft below HWY 140	--	pine, 24+	deposit	Mostly sand terrace; 24-ft above WSE
	F-F'	Main stem, 3,100-ft below confluence	Bar	18-ft below HWY 140	--	willow, 3+	LWD	Willows were cut to count rings; 11-12 years old; bar is composed of 75% cobble-sized material, 25% boulders; metamorphics--sub angular, granitics--well rounded
PROJECT AREA	G-G'	Main stem, 4,000-ft below confluence	Floodplain	--	--	--	LWD	Break-in-slope, transition from coarse to small sized material; 4-5 ft below terrace
	G-G'	Main stem, 4,000-ft below confluence	Terrace	4-ft below HWY 140	--	oak, 12+	LWD	Abundant LWD 4-5 ft high against oak, sand
	H-H'	Main stem, 4,800-ft below confluence	Bar	2-4 ft above WSE	--	--	--	No vegetation, active bar immediately across from slide; actively eroding along backwater channel
	I-I'	Main stem, 5,500-ft below confluence	Floodplain	2-5 ft above WSE	--	willows, 3-10+	LWD	Abundant willows growing in former slide deposits; angular boulders, frequently inundated
DS	J-J'	Main stem, 7,200-ft below confluence	Bedrock bank	10-15 ft below HWY 140	--	--	LWD	Geologic transition to gneiss, bedrock channel, steep with equal interval rapids, straight reach

Notes:

Cross-sections are labeled from geomorphic mapping and were determined in the field

WSE = water surface elevation on March 25, 2008, 667 cfs at Merced R. near Pohono Bridge

Rhizocarpum geographicum is a species of lichen widely used for dating Quaternary-aged surfaces; diameter is often correlated with age; size reported is maximum size measured of multiple individuals measured

LWD = large woody debris

©2008 Balance Hydrologics, Inc.

TABLE 3. GRAIN SIZE ANALYSIS AT MERCED RIVER NEAR FERGUSON SLIDE, MARIPOSA COUNTY, CALIFORNIA

SITE INFORMATION

Project Name:	Merced River/Ferguson Slide
Project No.	207240
Stream:	Merced River
Observers:	MS, BKH
Water Year:	2008
Date:	3/26/2008

Segment:

Riffle	
Pool	x
Glide	
Bar	x

Method:

Tape and offset	x
Pace and toe touch	
Pace and offset	
Grid sampling	x

PEBBLE COUNT DATA TAKEN IN FIELD [1]

	Site Name	RB bar D/S of UTB [2]						RB bar U/S of UTB [3]					LB bar U/S of UTB [4]				RB bar at slide [5]			
	Transect No. [6]	1	2	3	Total	Grid [7,9]	Grid 2 [8,9]	1	2	3	Total	Grid 2 [8,9]	1	2	3	Total	1	2	Total	Grid 2 [8,9]
	Size Class mm	#	#	#	#	#		#	#	#	#	#	#	#	#	#	#	#	#	#
boulder	1448	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
	1024	0	0	0	0	0	0	5	0	1	6	0	0	0	0	0	0	1	1	0
	724	0	1	0	1	0	0	7	3	0	10	0	0	1	0	1	0	3	3	0
	512	0	1	0	1	1	0	5	9	5	19	0	1	5	0	6	2	2	4	0
	362	5	5	2	12	1	0	4	5	6	15	0	3	4	1	8	5	9	14	0
cobble	256	6	8	1	15	4	0	6	5	2	13	5	4	8	3	15	6	4	10	3
	180	10	10	6	26	2	3	3	6	2	11	3	7	5	4	16	11	4	15	4
	128	7	4	2	13	3	10	2	5	1	8	4	0	4	5	9	7	5	12	9
	90	1	1	1	3	7	31	1	3	2	6	2	2	2	4	8	2	3	5	7
gravel	64	1	4	3	8	5	40	0	1	0	1	4	1	2	0	3	0	0	0	8
	45	0	3	0	3	4	25	3	1	2	6	0	0	1	2	3	0	1	1	10
	32	2	2	0	4	3	17	0	1	0	1	2	0	1	2	3	1	1	2	4
	22.6	0	0	2	2	0	2	1	1	0	2	0	0	0	0	0	1	2	3	3
	16	0	1	1	2	0	0	1	0	0	1	0	0	0	0	0	2	2	4	0
	11.3	0	0	0	0	0	0	1	0	0	1	0	0	1	1	2	0	0	0	0
	8	1	0	0	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0
	5.6	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0
Sand	4	6	0	3	9	56	0	0	0	0	0	0	1	7	18	26	4	4	8	0
	Subtotals																			
	>256 (boulders)	5	7	2	14	2	0	21	18	12	51	0	4	10	1	15	7	15	22	0
	>64 (cobbles)	24	23	10	57	16	44	12	19	7	38	14	13	19	16	48	26	16	42	23
	>4 (gravels)	4	10	6	20	12	84	7	4	2	13	6	1	5	6	12	4	6	10	25
	<4 (sand)	6	0	3	9	56	0	0	0	0	0	0	1	7	18	26	4	4	8	0
	Organics	1	1	0	2	0	0	1	0	0	1	0	0	0	2	2	0	0	0	0
	Total Counts	40	41	21	102	86	128	41	41	21	103	20	19	41	43	103	41	41	82	48
	D-10:	n/a	35.9	n/a	2.0	n/a	28.2	20.2	64.7	65.1	40.8	38.1	61.8	n/a	n/a	n/a	11.5	11.5	11.5	26.4
	D-16:	6.1	46.9	12.8	26.9	n/a	32.7	42.2	86.0	81.0	77.5	50.2	76.6	n/a	n/a	n/a	19.4	17.6	18.2	32.8
	D-50:	134.8	152.2	98.7	136.8	n/a	53.8	279.2	215.3	279.2	256.0	107.6	168.1	152.2	24.7	107.6	142.9	159.0	147.0	61.3
	D-84:	238.3	266.9	177.3	244.4	78.9	83.2	675.6	463.9	434.7	524.2	205.1	286.0	344.9	145.0	249.2	263.9	354.3	318.9	124.7
	D-90:	276.3	315.2	247.3	287.4	119.4	91.1	776.0	510.0	474.4	648.1	222.9	326.3	413.0	179.5	316.5	313.0	503.2	360.3	154.9

Notes :

1) Method described in detail in the accompanying text.
2) Right bank bar downstream of upper temporary bridge.
3) Right bank bar upstream of upper temporary bridge.
4) Left bank bar upstream of upper temporary bridge.
5) Right bank bar at slide location.
6) Transects were placed in ascending order away from the channel (i.e., transect 1 is closest to the channel, while transect 3 is the farthest up the bank).
7) 'Grid' refers to a 1 m^2 sampling space (or grid) chosen randomly within a shallow portion of the channel adjacent to the sampled bar, within which the surface grains were catalogued.
8) 'Grid 2' refers to a 1 m^2 sampling space (or grid) chosen randomly on the bar, floodplain, or terrace surface within which the surface grains were catalogued.
9) Bed core analysis was usually not possible due to the coarse texture of the subsurface. Mechanized excavation would have been necessary to obtain samples, except for one obtained at RB bar D/S of UTB.

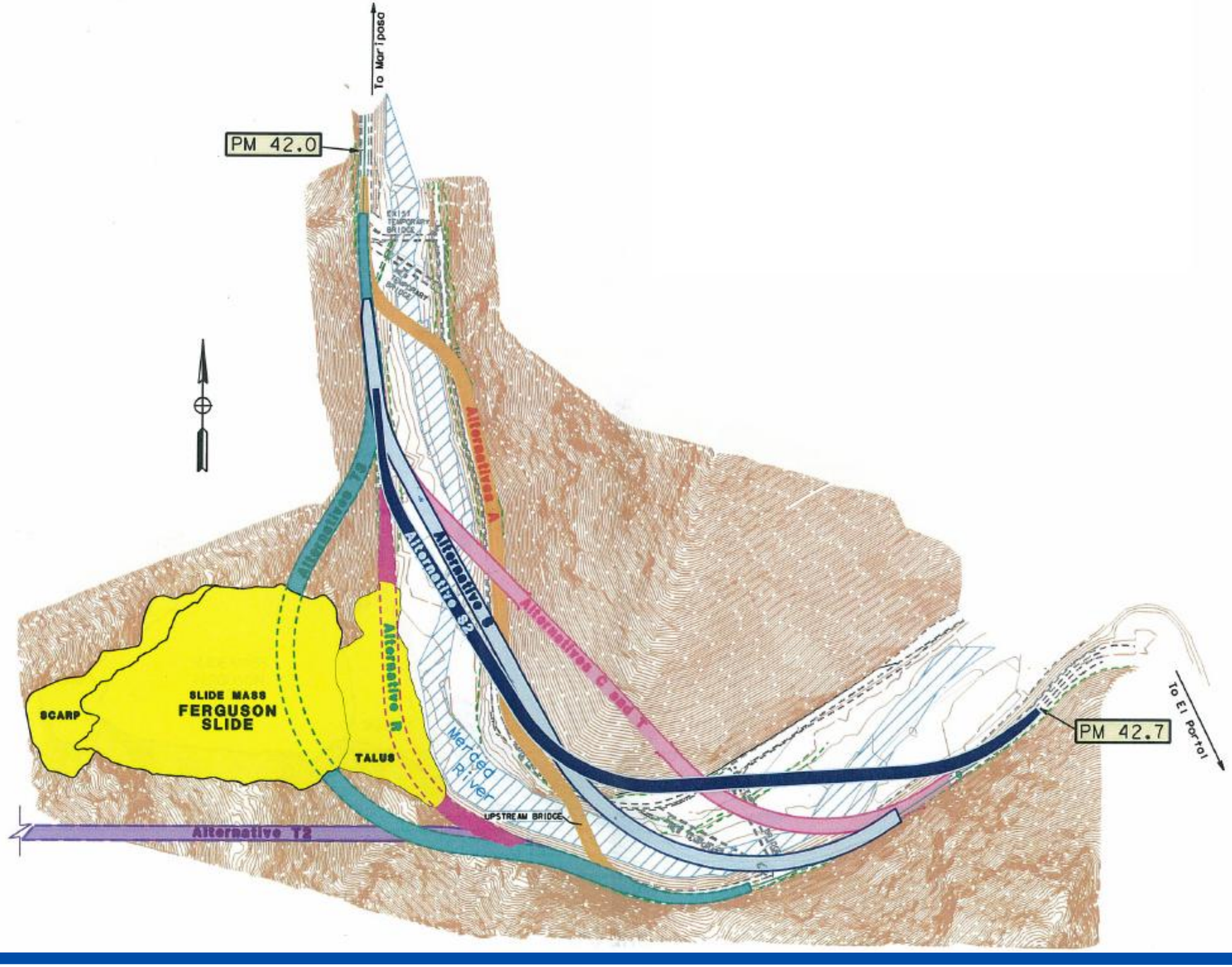
© 2008 Balance Hydrologics, Inc.

FIGURES

Balance Hydrologics, Inc.

Figure 3.1 Ferguson Slide, Merced River, Mariposa County, California. 'A' shows the slide from a point downstream, looking upstream. Note cobble bar in left background across from slide and cobble/boulder bar in right foreground (channel left) created by ancestral slide debris from upstream (ancestral slide not visible in photo). 'B' shows the slide from upstream, looking downstream from the temporary bypass along Incline Road.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 4.1 alternatives.doc

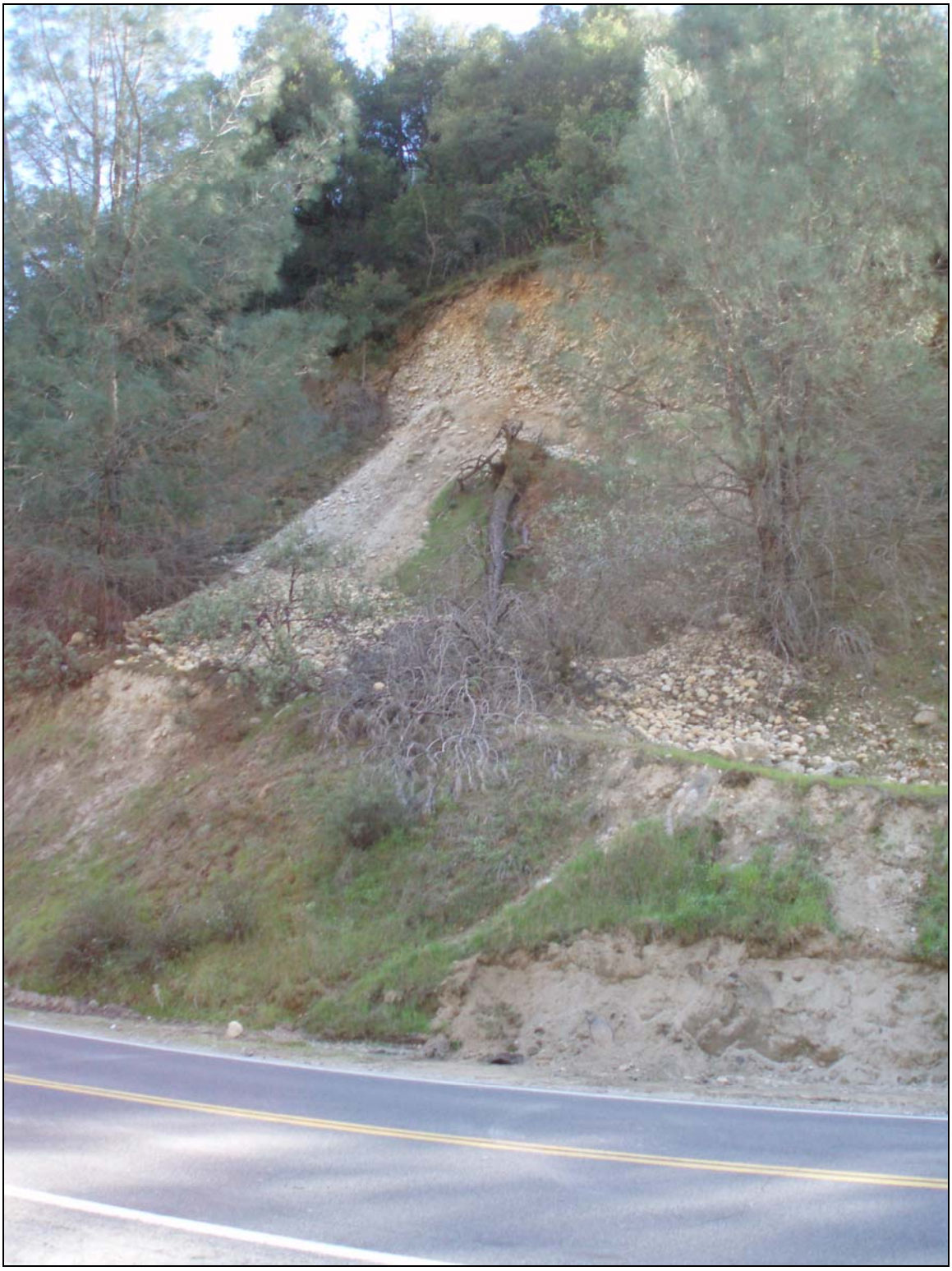
Figure 4.1 **Ferguson Slide permanent restoration project proposed alternatives.** As of the date of this report, the skewed (oblique to valley axis) temporary crossings, shown with dashed lines, are installed; the perpendicularly-oriented temporary crossings have been removed. Alternative C entails a “Cut” through the eastern ridge; alternative T entails a “Tunnel” through the eastern ridge; alternatives S and S2 require viaducts across the eastern ridge and S requires a new temporary crossing upstream during construction; alternative A includes clear-span bridges and utilizes Incline Road; alternative R is a “rockshed” through the slide debris; alternative T3 is a “Tunnel” under the slide mass; alternative T2 is a “Tunnel” through Ferguson Ridge. Drawing courtesy of CALTRANS, from “Ferguson Slide Permanent Restoration Project Proposed Alternatives, 10-Mpa-140-42.0/42.7, 10-0p9200” .

© 2008 Balance Hydrologics, Inc.

Balance Hydrologics, Inc.

Figure 4.2 Temporary crossings, Merced River at Ferguson Slide, Mariposa County, California. 'A' shows the downstream temporary bridges looking upstream, with the older, perpendicularly-oriented bridge in the foreground and the newer oblique bridge behind it. 'B' shows the newer oblique upstream temporary bridge. Slide in background; older temporary bridge is just upstream to the right, out of the photo.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

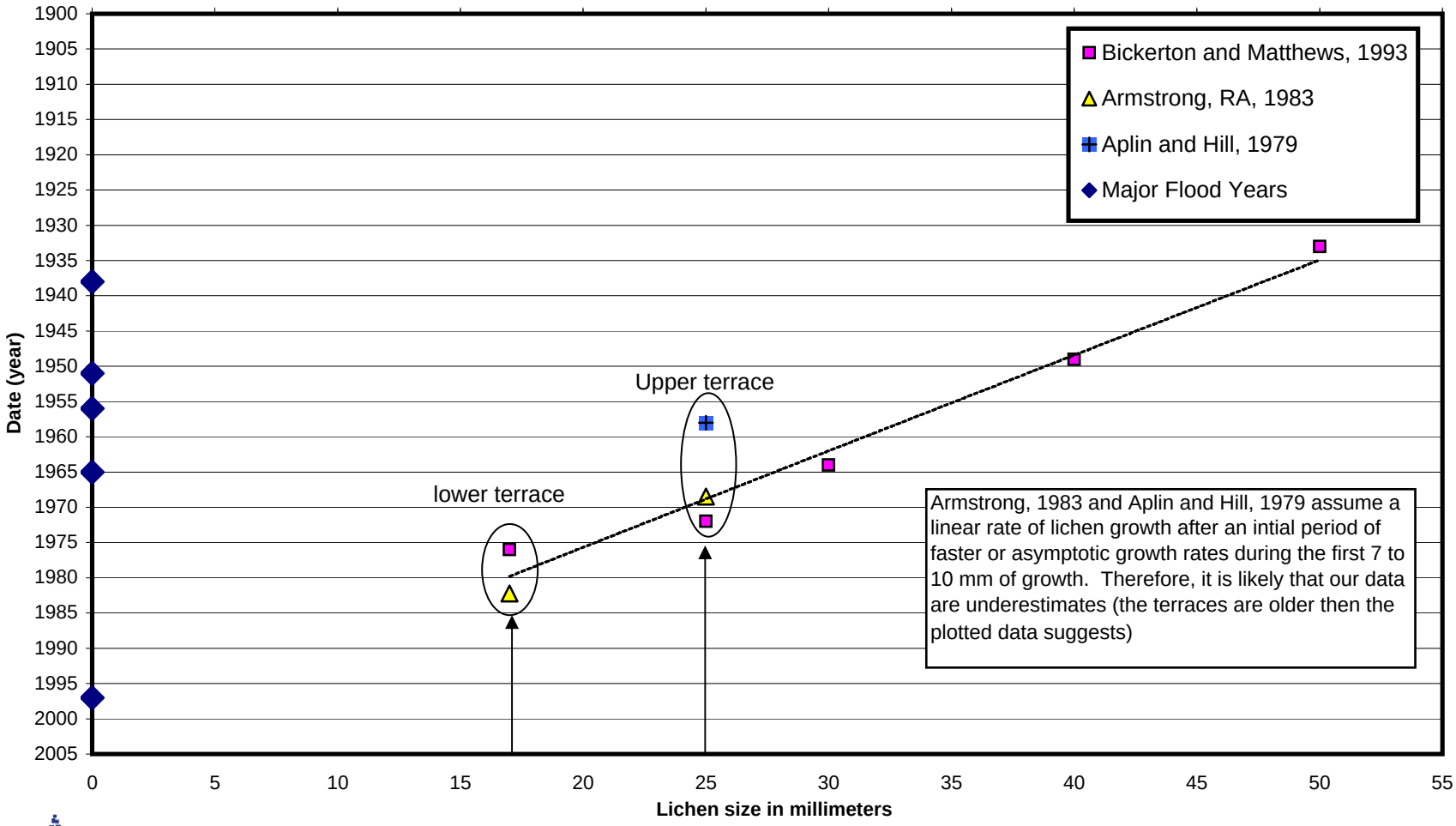
Figure 5.1 Abandoned left-bank terrace deposit in the Merced River canyon upstream of the Ferguson Slide, Mariposa County, California. Highway 140 roadway in foreground, downstream direction is to the right. Elevation of the terrace deposit is at and above the fallen pine tree; cobbles on the hillside below are from erosion.

© 2008 Balance Hydrologics, Inc.

Balance Hydrologics, Inc.

Figure 5.2 Dendrochronology and lichenometry at the Ferguson Slide, Merced River, Mariposa County, California. 'A' shows Balance staff using an increment borer to obtain a tree-ring core sample from a willow to date the left-bank floodplain above the South Fork confluence. 'B' shows measurement of *Rhizocarpum geographicum* (grey lichen with white halos) of left-bank terrace upstream of the confluence.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 5.3. Relative dates for terraces using lichenometry, Merced River near Ferguson Slide , Mariposa County, California

© 2008 Balance Hydrologics, Inc.

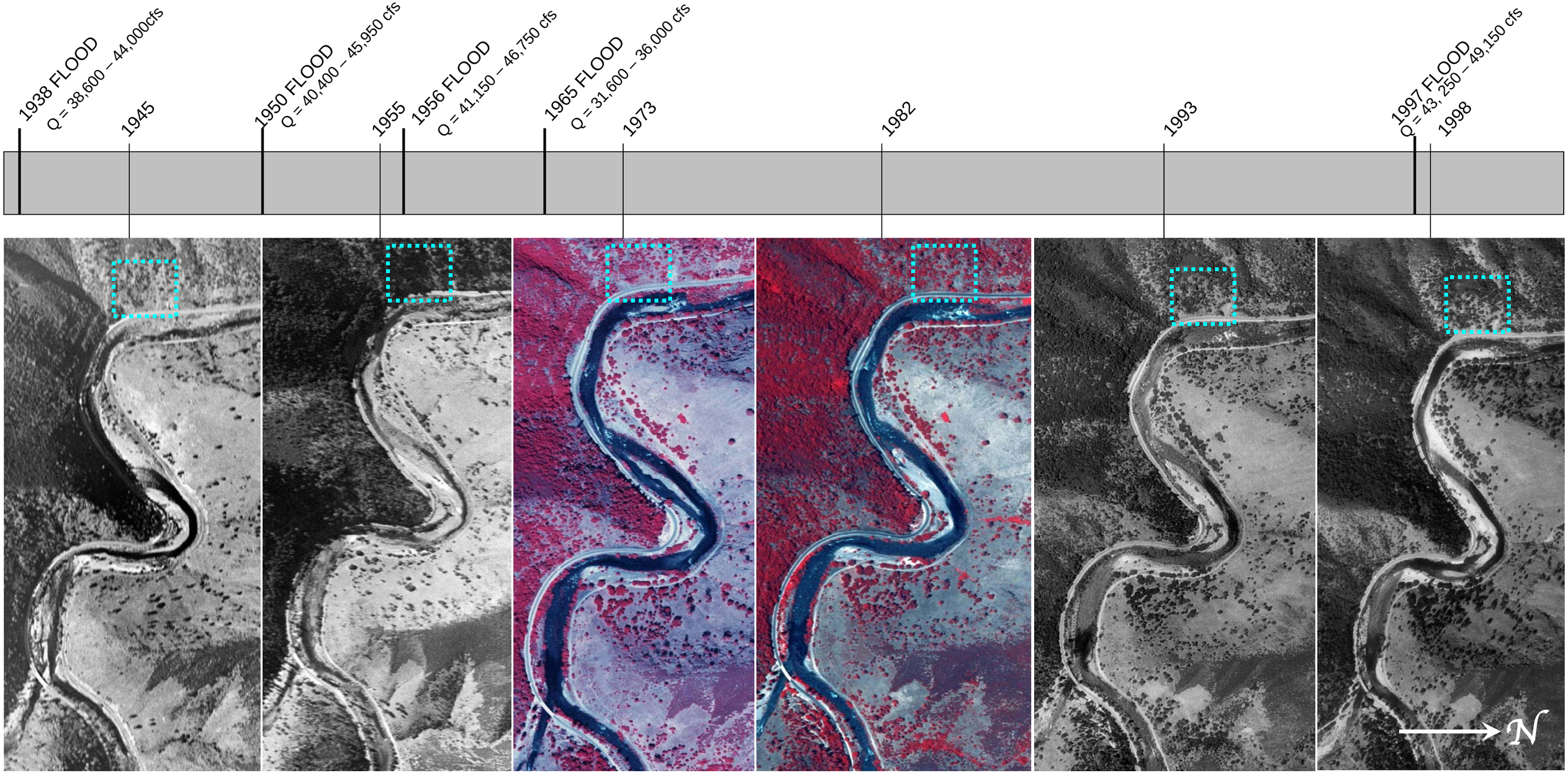
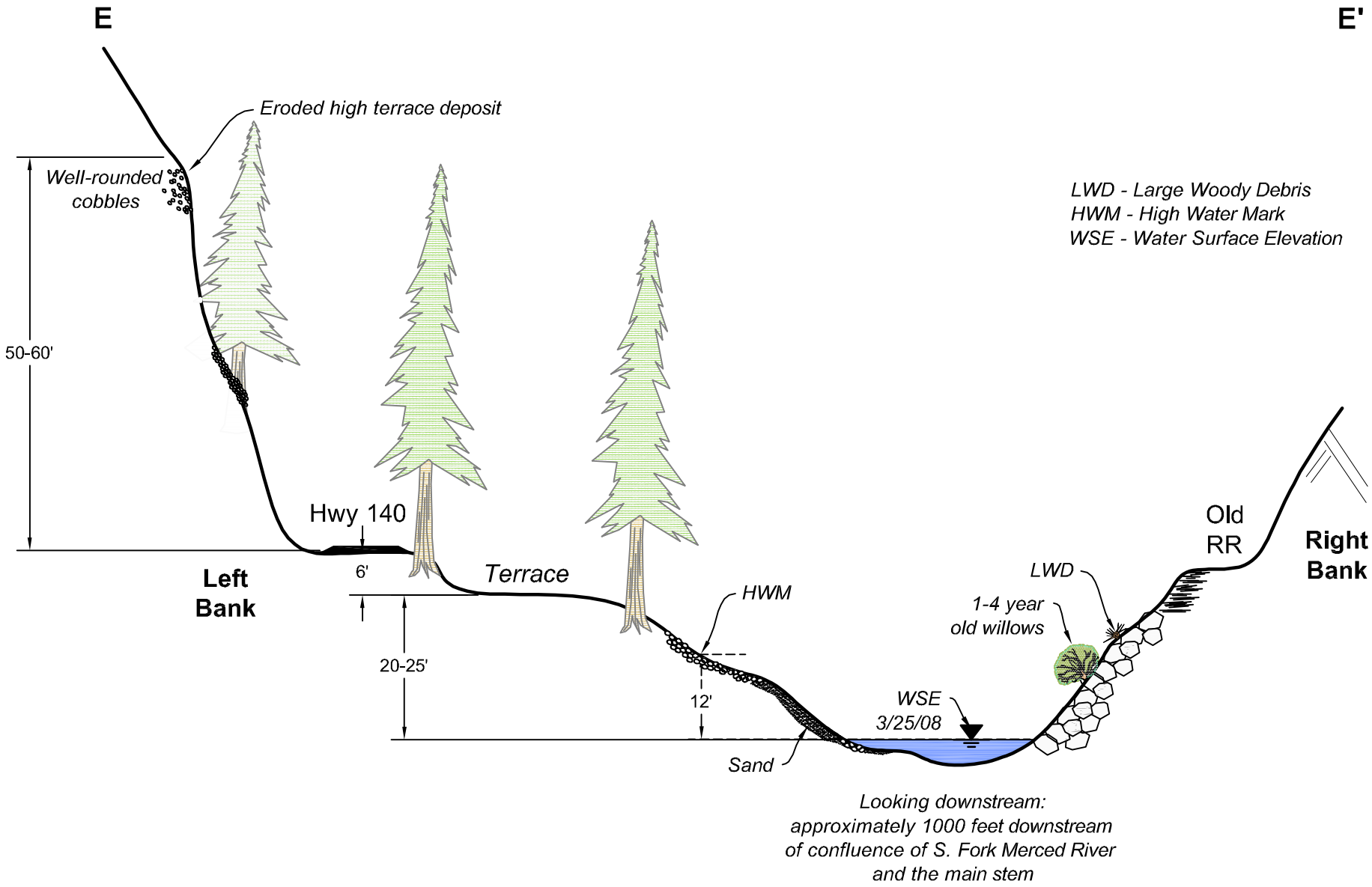


Photo Source: United States Geological Survey

Balance Hydrologics, Inc.

Figure 6.1. Historical aerial photo analysis, 1945 – 1998, Merced River between South Fork of Merced River and Ferguson Slide, Mariposa County, CA
These sequential historical photos depict the reach where Highway 140 bridge-crossing alternatives are proposed. North is to the right of page; the SF of the Merced River is shown in the lower left corner of each photo; the current-day Ferguson Slide is located at the top-center of each photo, with an approximate location outlined by the dotted blue box. A timeline of major rain-on-snow events is provided at the top of page. These photos suggest major floods (rain-on-snow events) re-organize bars and shift channel thalwegs; however channel planform remains unchanged largely due to the fact that the reach is bedrock confined and constrained by a narrow, steep-walled canyon.

© 2008 Balance Hydrologics, Inc.

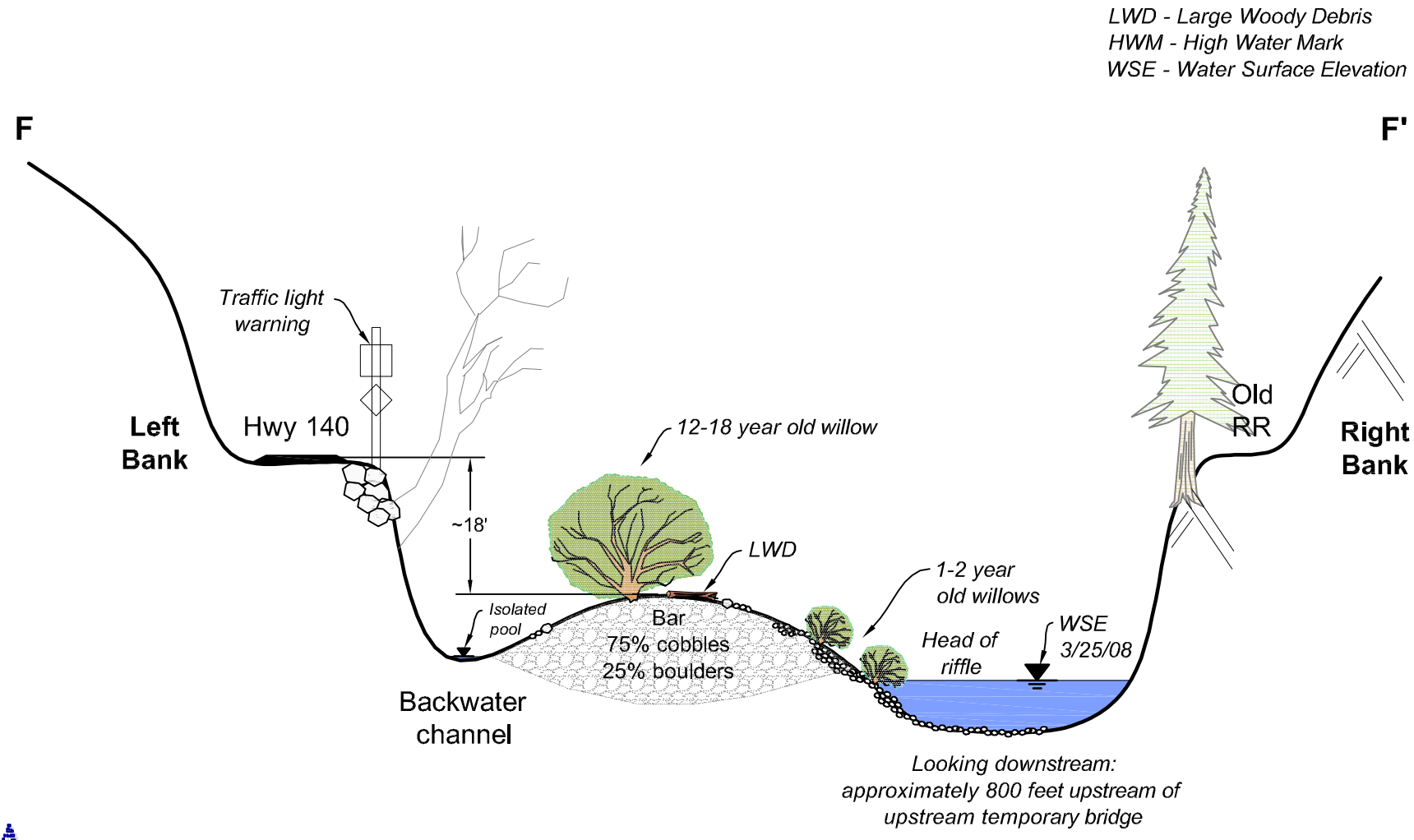


Balance Hydrologics, Inc.

207240 Merced River near Ferguson Slide.dwg

Figure 6.2. Schematic river section E - E', Merced River near Ferguson Slide.
See Plate 1 for section location.

©2008 Balance Hydrologics, Inc.

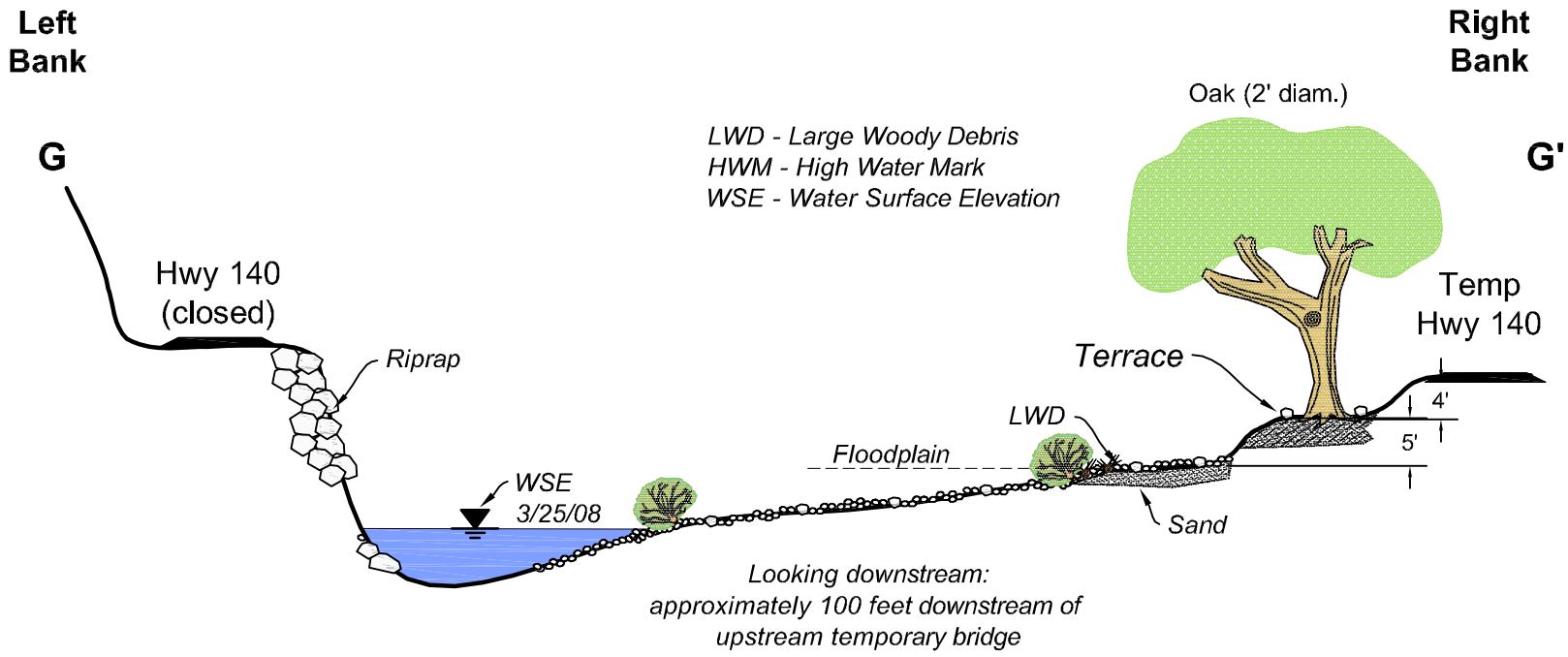


Balance Hydrologics, Inc.

207240 Merced River near Ferguson Slide.dwg

Figure 6.3. Schematic river section F - F', Merced River near Ferguson Slide.
See Plate 1 for section location.

©2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

207240 Merced River near Ferguson Slide.dwg

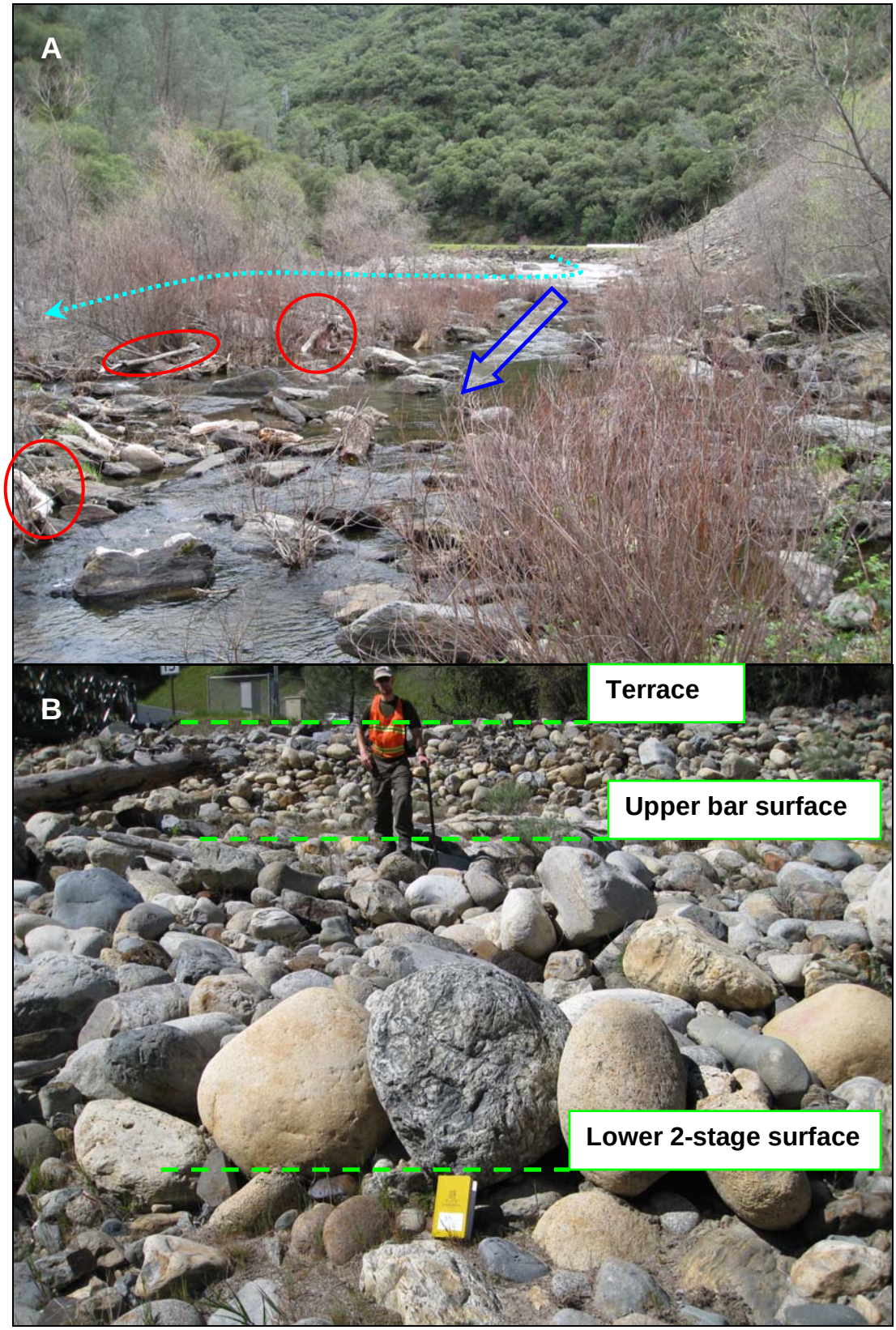
Figure 6.4. Schematic river section G - G', Merced River near Ferguson Slide.
See Plate 1 for section location.

©2008 Balance Hydrologics, Inc.

Balance Hydrologics, Inc.

Figure 6.5 Ancestral and historical slides in the Merced River canyon upstream of the Ferguson Slide, Mariposa County, California. 'A' shows an ancient slide deposit (breccia) left along the Highway 140 roadcut just upstream of the Ferguson Slide. 'B' shows a smaller historical slide on the hillslope above Incline Road that covers imported rip rap (blue-ish pieces at left below roadway) along the right bank above the South Fork confluence.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

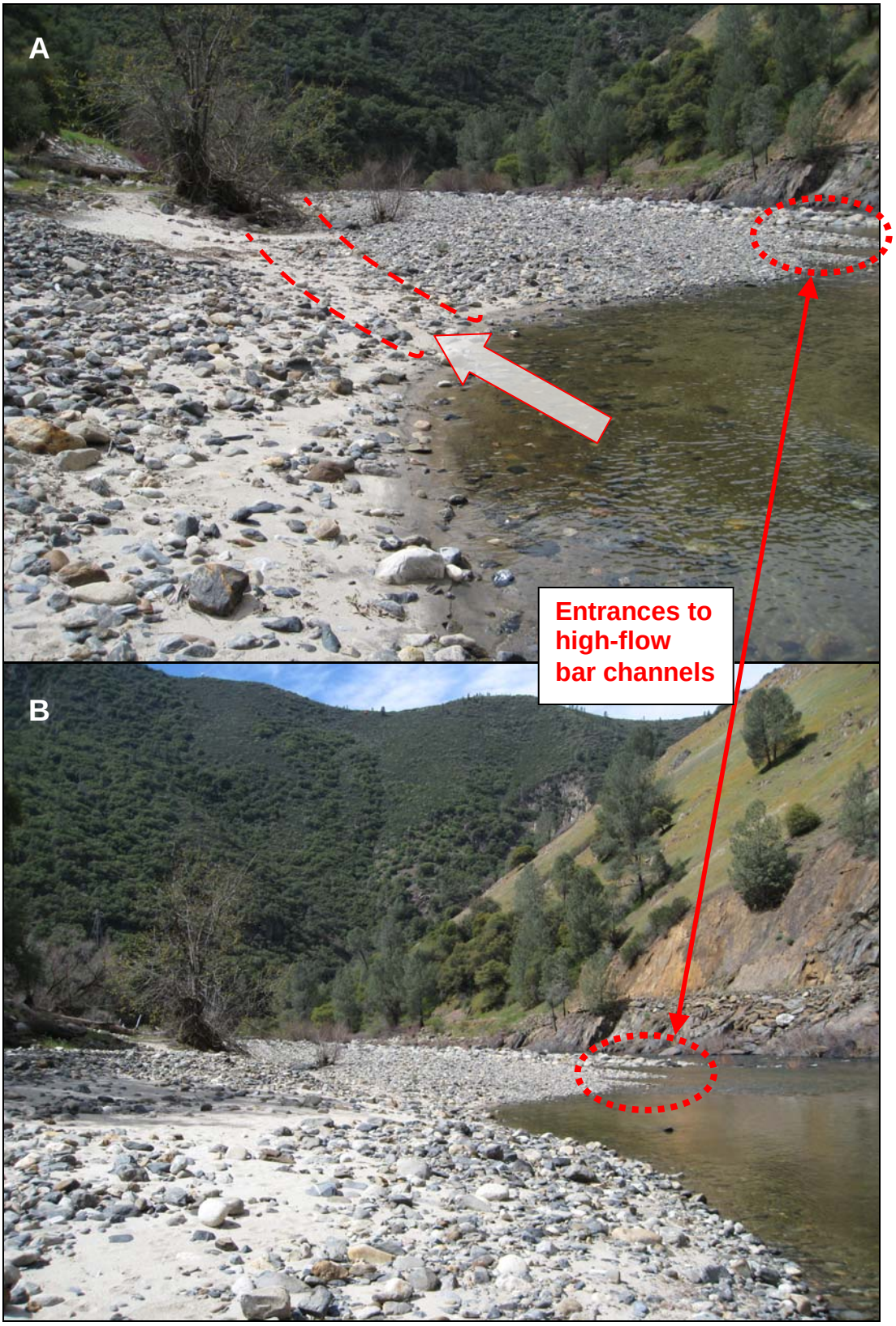
Figure 6.6 Bars on the Merced River at Ferguson Slide, Mariposa County, California. 'A' shows the 2-stage bar composed of ancient slide debris below the Ferguson Slide. Dotted arrow shows main channel flow; block arrow shows 2-stage flow across bar surface; LWD circled. 'B' shows terrace, upper bar surface, and lower 2-stage surface just upstream of upper temporary bridge. Note imbricated boulders.

© 2008 Balance Hydrologics, Inc.

Balance Hydrologics, Inc.

Figure 6.7 Cobble bar across from Ferguson Slide, Merced River, Mariposa County, California. 'A' shows cobble bar from Incline Road on the right bank, with the "chute" channel in the foreground and main-flow channel in background. 'B' shows a view looking downstream through the "chute" channel along the right bank. Note rapid change in grade through chute channel, exposing bar substrate.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 6.8 Bar-complex along left bank upstream of upper temporary crossing, Merced River at Ferguson Slide, Mariposa County, California. 'A' shows gradation in bar texture from left (sandy flood channel deposits fed by flows following block arrow) to right (cobbles and boulders). 'B' shows more clearly the entrances to high-flow bar channels along the margin of the bar-complex, where flood flows enter the bar surface.

© 2008 Balance Hydrologics, Inc.

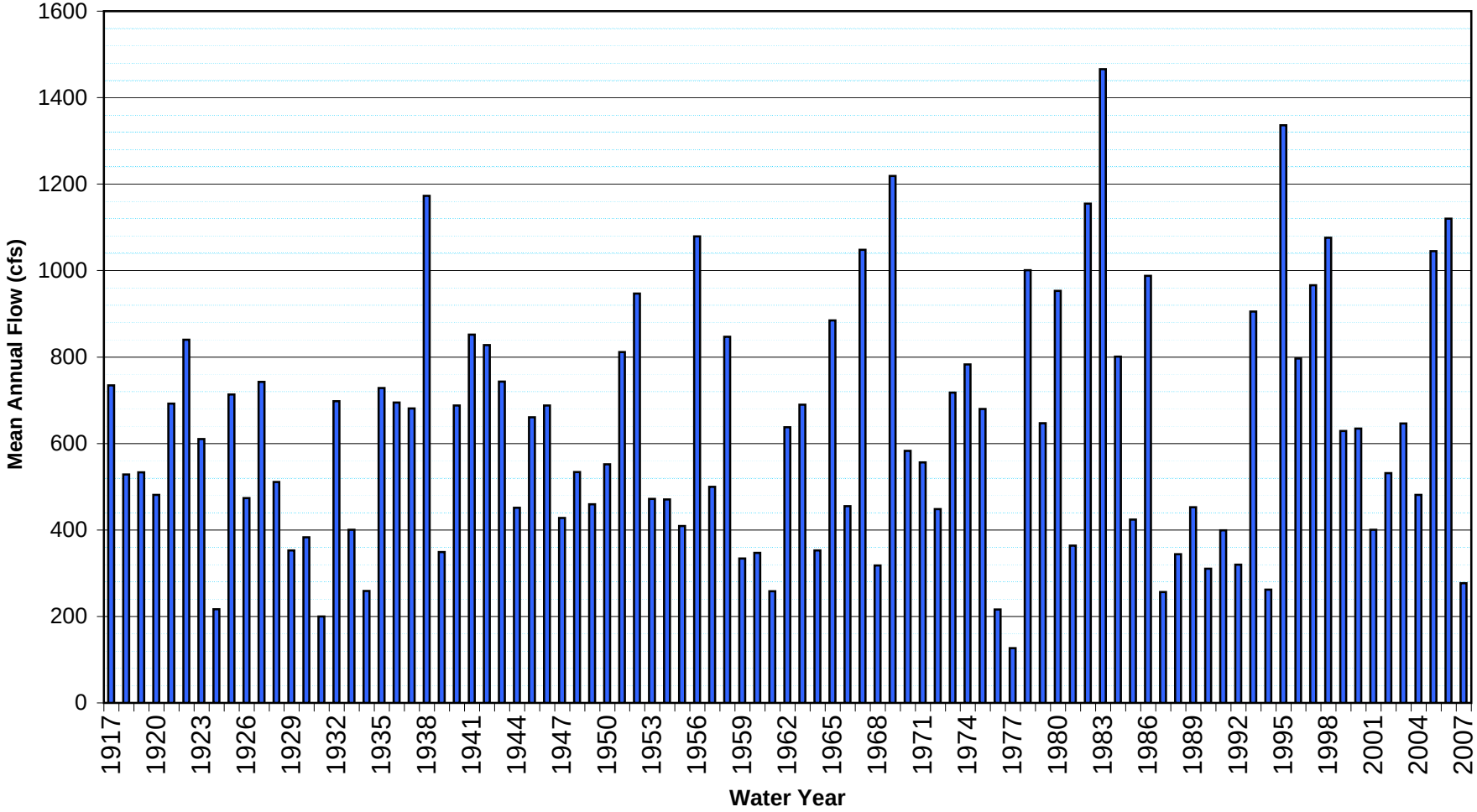
Figure 6.9 Large woody debris, Merced River at Ferguson Slide, Mariposa County, California. 'A' shows individual piece laying atop bar-complex upstream of the upper temporary crossing. 'B' shows large woody debris (circled) beneath upper temporary crossing, for comparison with pier spacing. In general, large woody debris observed was 10 to 25 feet in length.

© 2008 Balance Hydrologics, Inc.

Balance Hydrologics, Inc.

Figure 6.10 Large woody debris, Merced River at Ferguson Slide, Mariposa County, California. 'A' shows large woody debris accumulation approximately 3 feet below the Highway 140 road surface. South Fork Merced River confluence bridge visible in background. 'B' shows large woody debris on the right bank just downstream of the South Fork confluence.

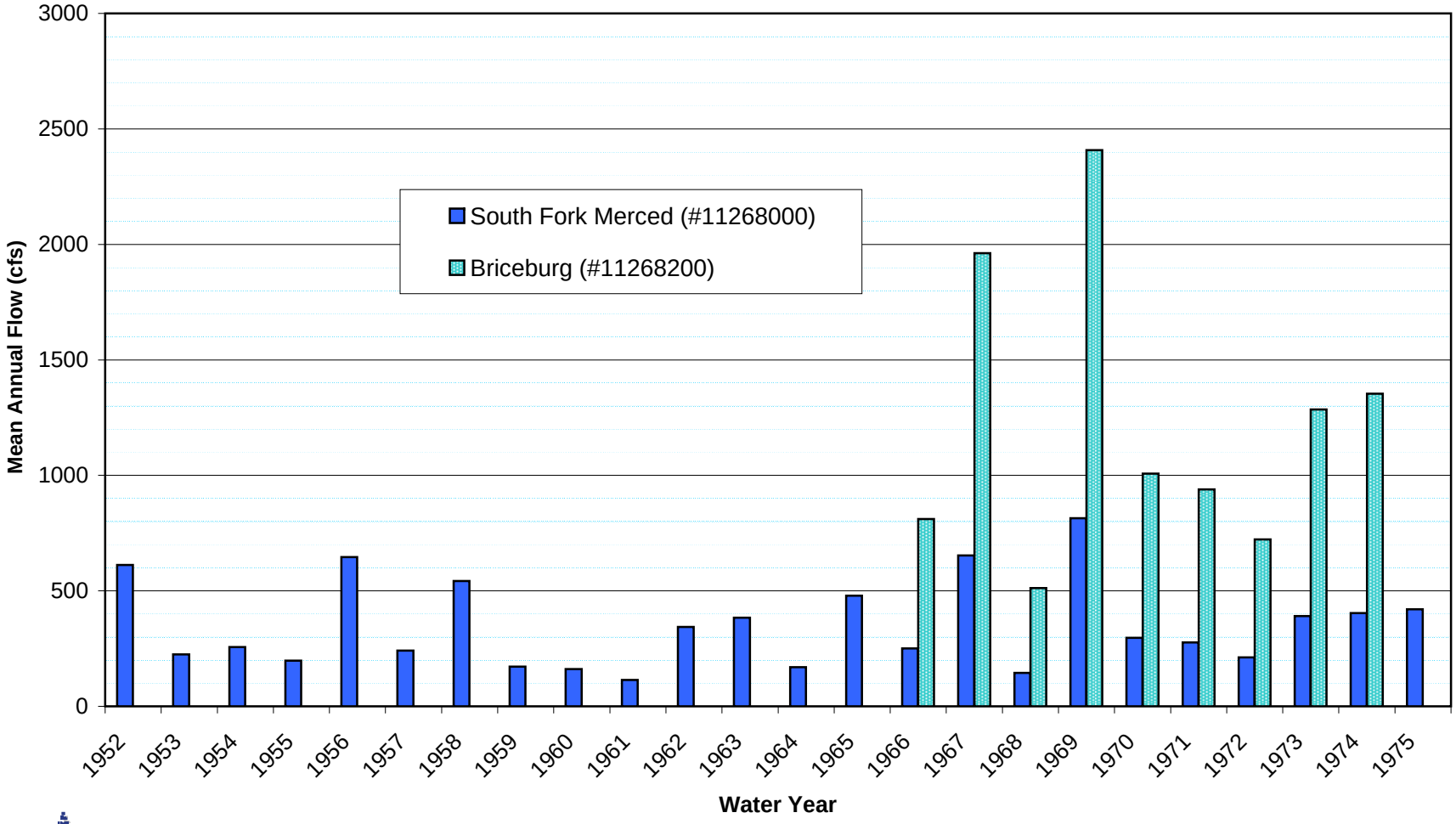
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.1. Mean annual flow in Merced River at USGS Pohono Gage #11266500, 1917-2007, Mariposa County, California.

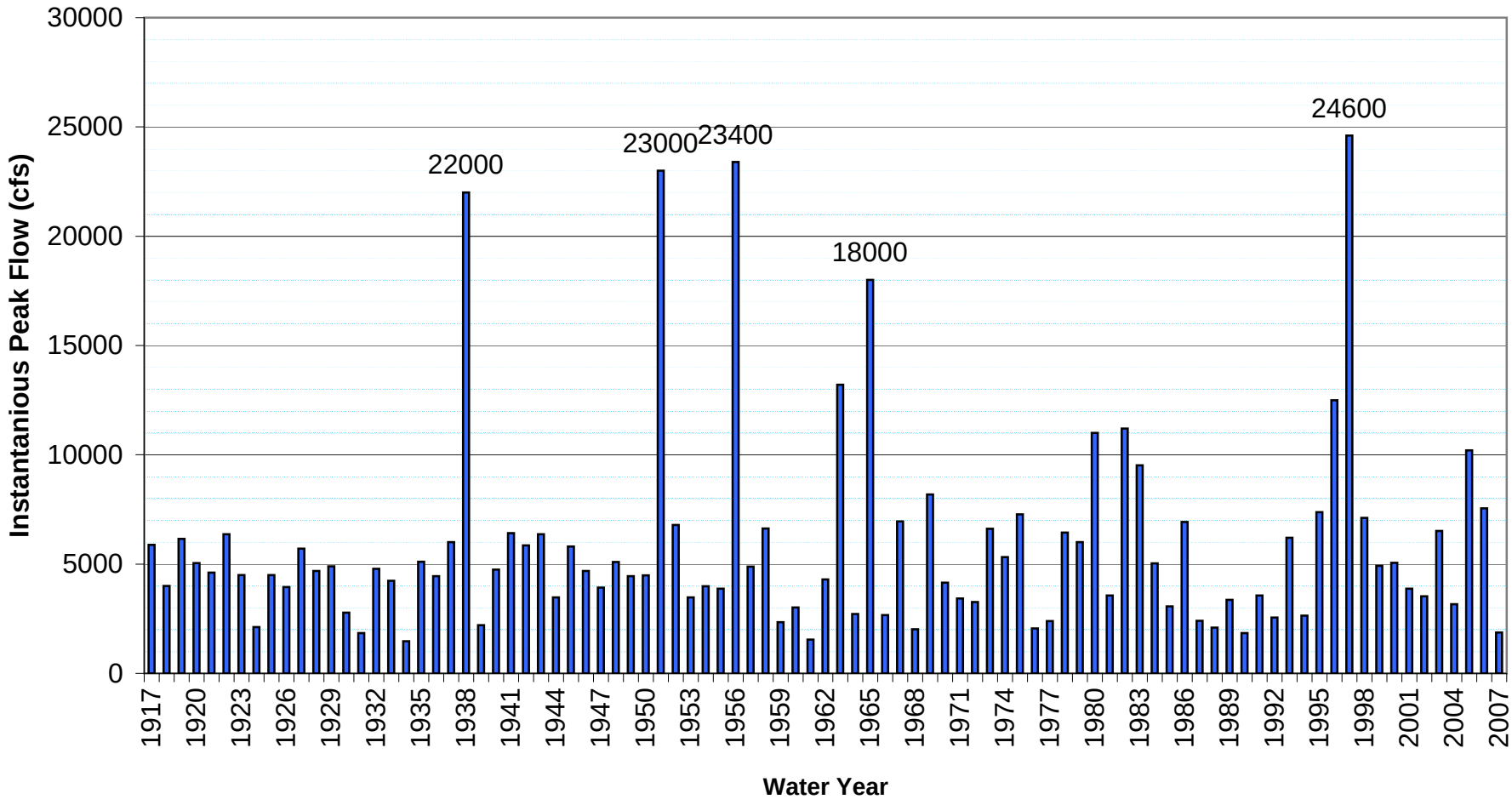
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.2. **Mean annual flow in South Fork Merced River at USGS gage #11268000 and in the Merced River at Briceburg (USGS gage #11268200), 1952-1975, Mariposa County, California.**

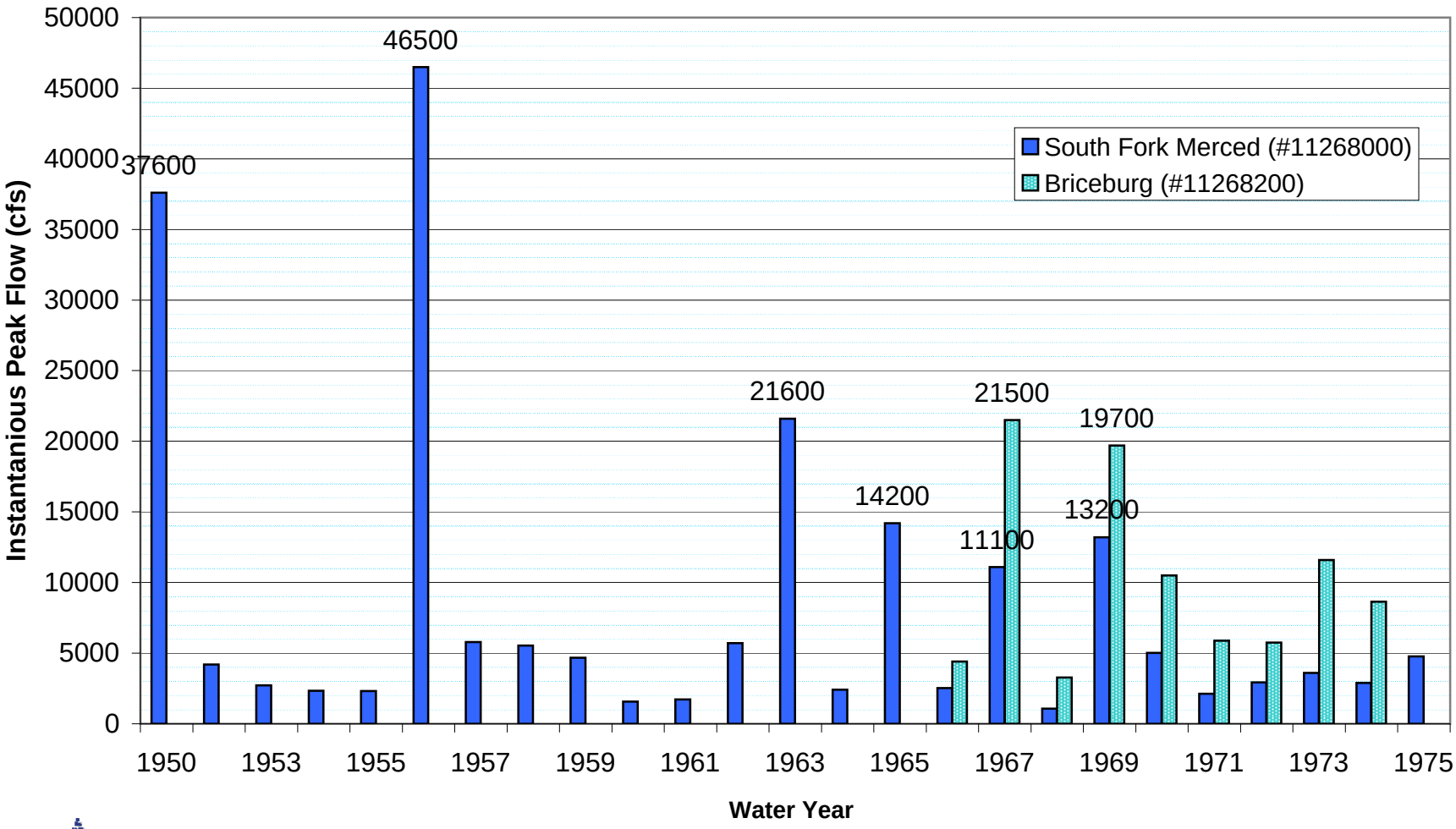
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.3. Annual instantaneous peak flows in Merced River at USGS Pohono Gage #11266500, 1917-2007, Mariposa County, California.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.4. Annual instantaneous peak flows in South Fork Merced River at USGS gage #11266800 and in Merced River at Briceburg (USGS gage #11268200), 1950-1975, Mariposa County, California.

© 2008 Balance Hydrologics, Inc.

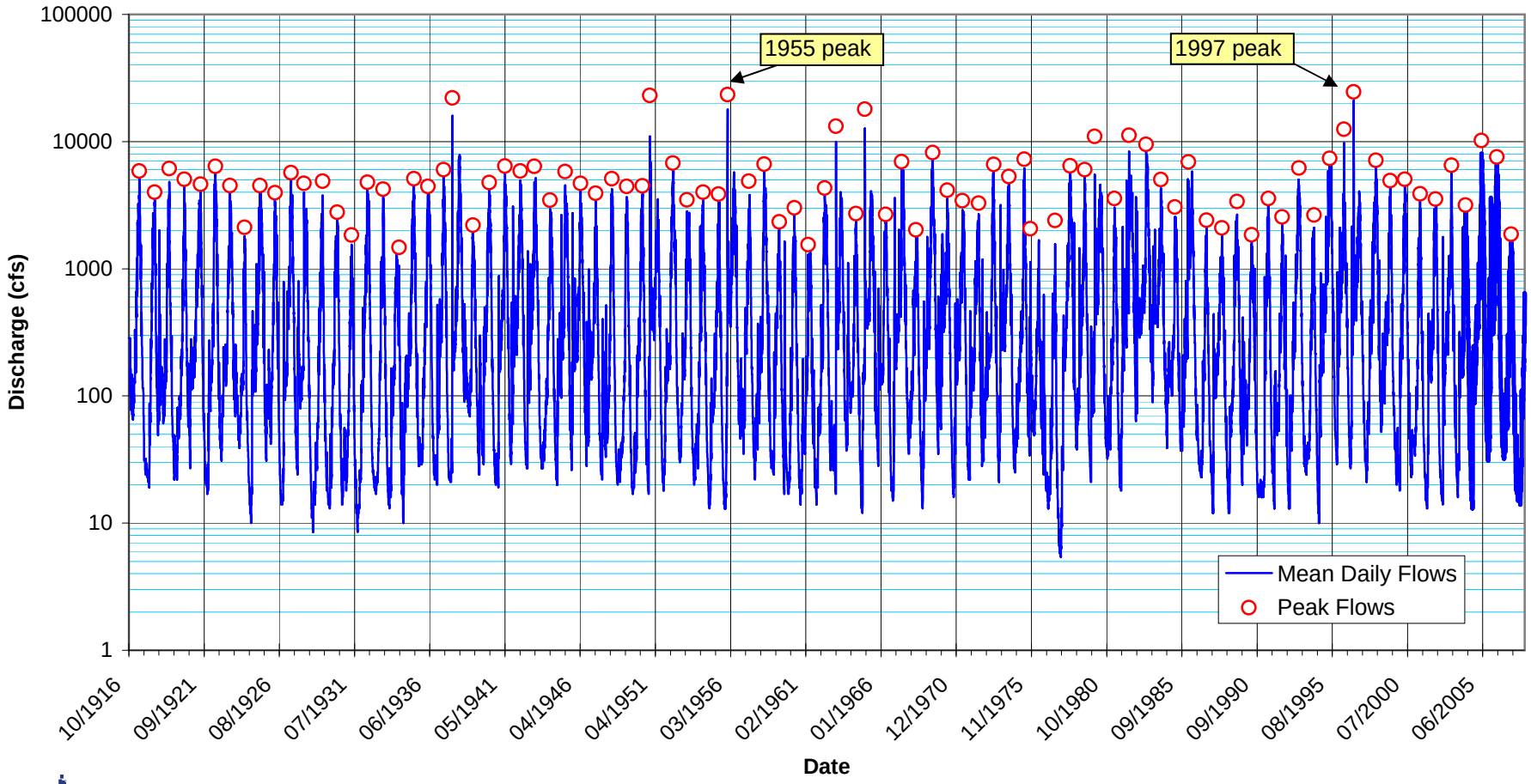
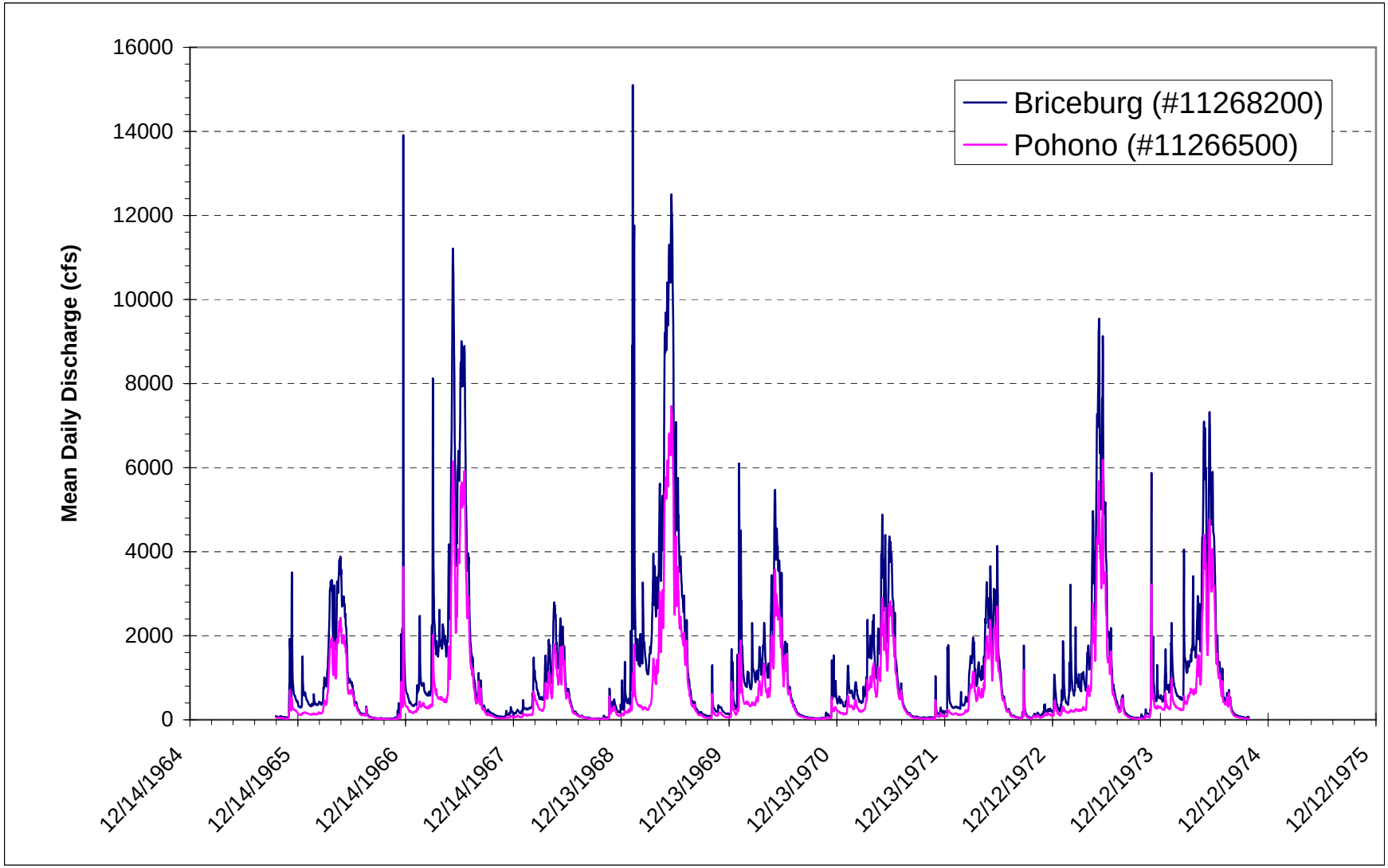


Figure 7.5. Mean daily and peak flows at USGS gaging station #11266500 (Merced River at Pohono Bridge), Mariposa County, California.

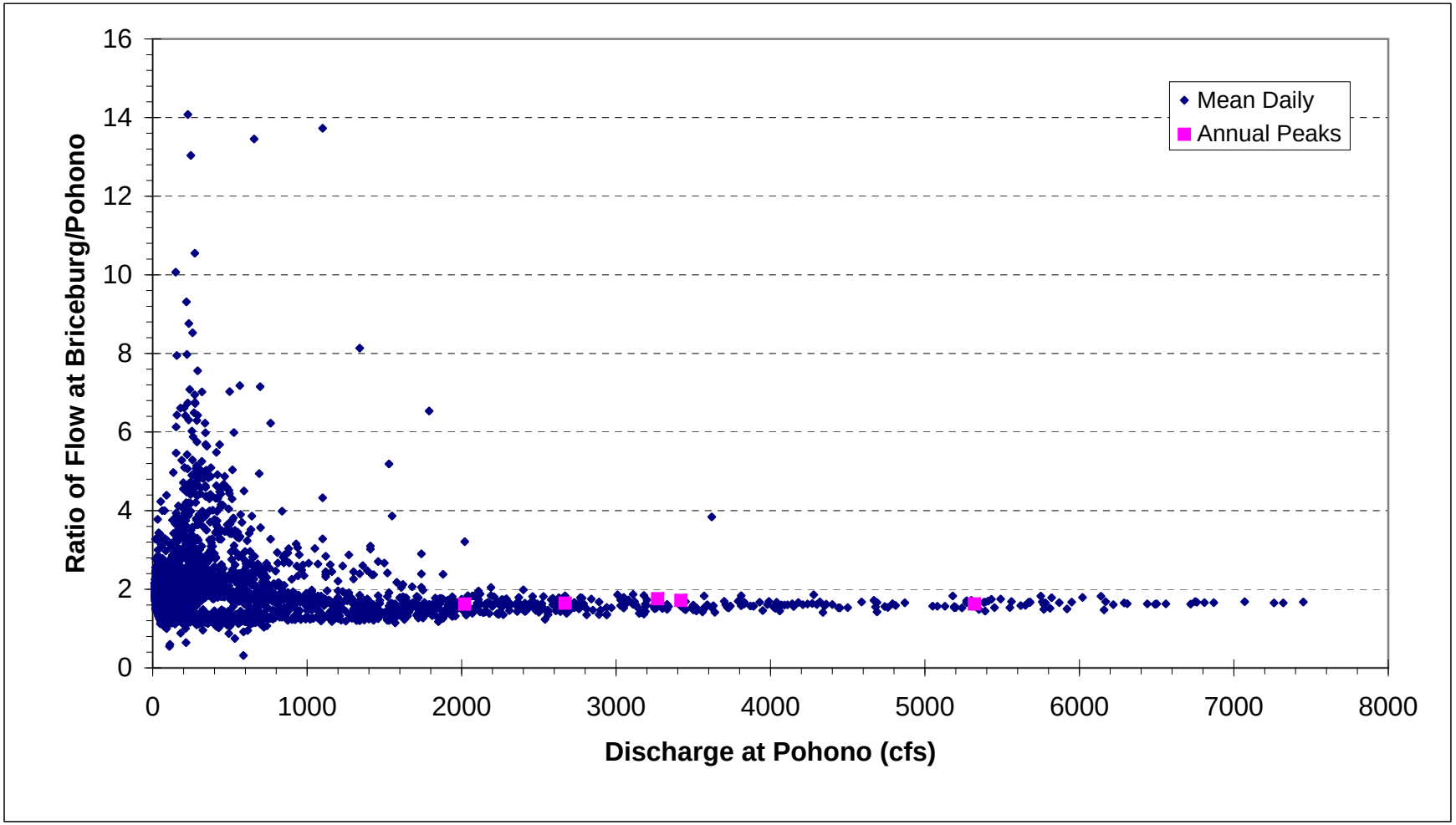
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.6. Mean daily discharges at the Briceburg (# 11268200) and Pohono (# 11266500) gages for the period of the Briceburg Record. The Briceburg gage is approximately 5 miles downstream of the slide and Pohono is 16 mile upstream.

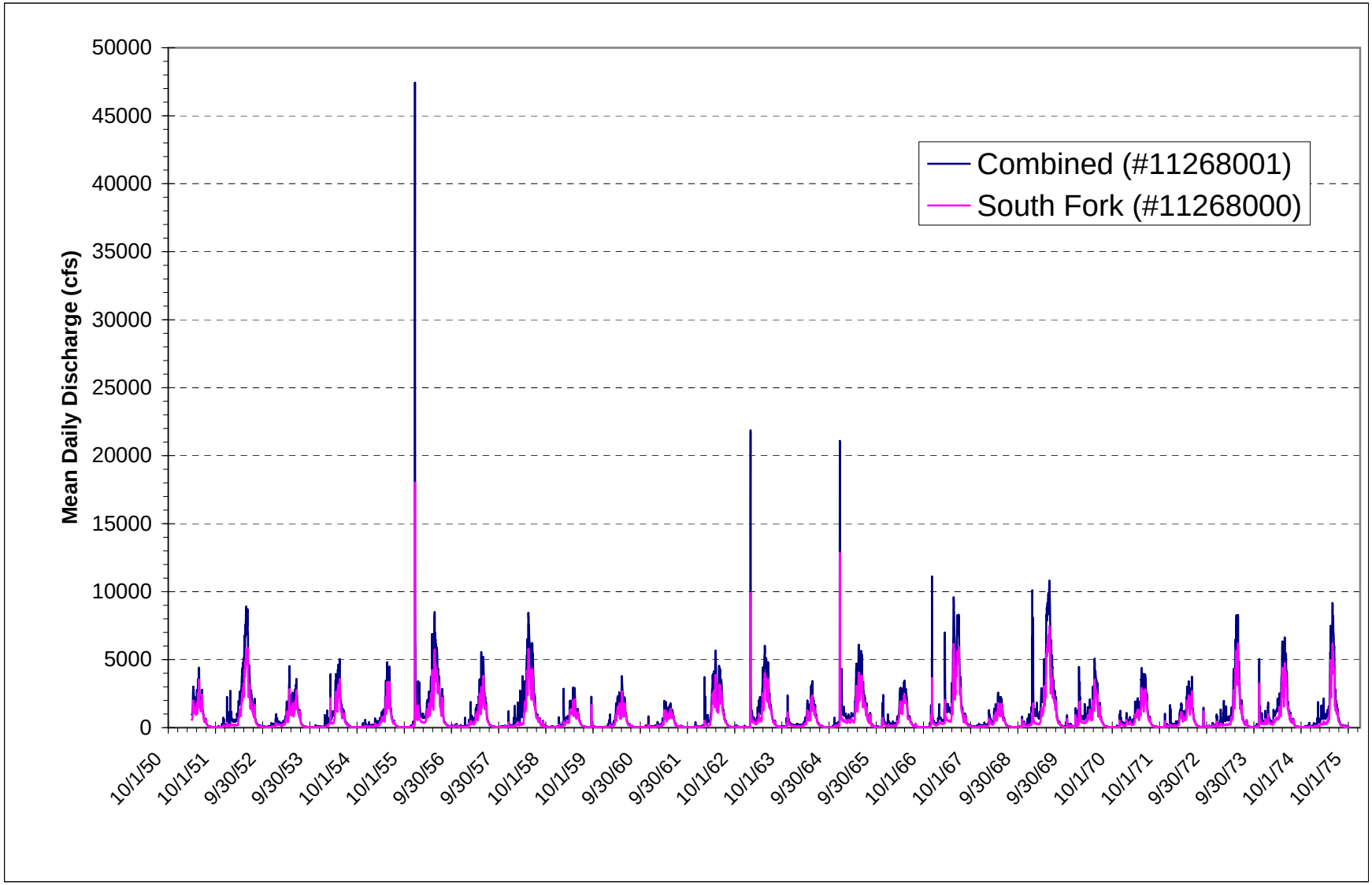
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.7. Ratio of flow at the Briceburg Gage (11268200) located approximately 5 miles downstream of the project reach to the flow at the Pohono Gage (11266500) approximately 16 miles upstream. Note that there are no extreme flood events during the Briceburg period of record (1965 - 1974).

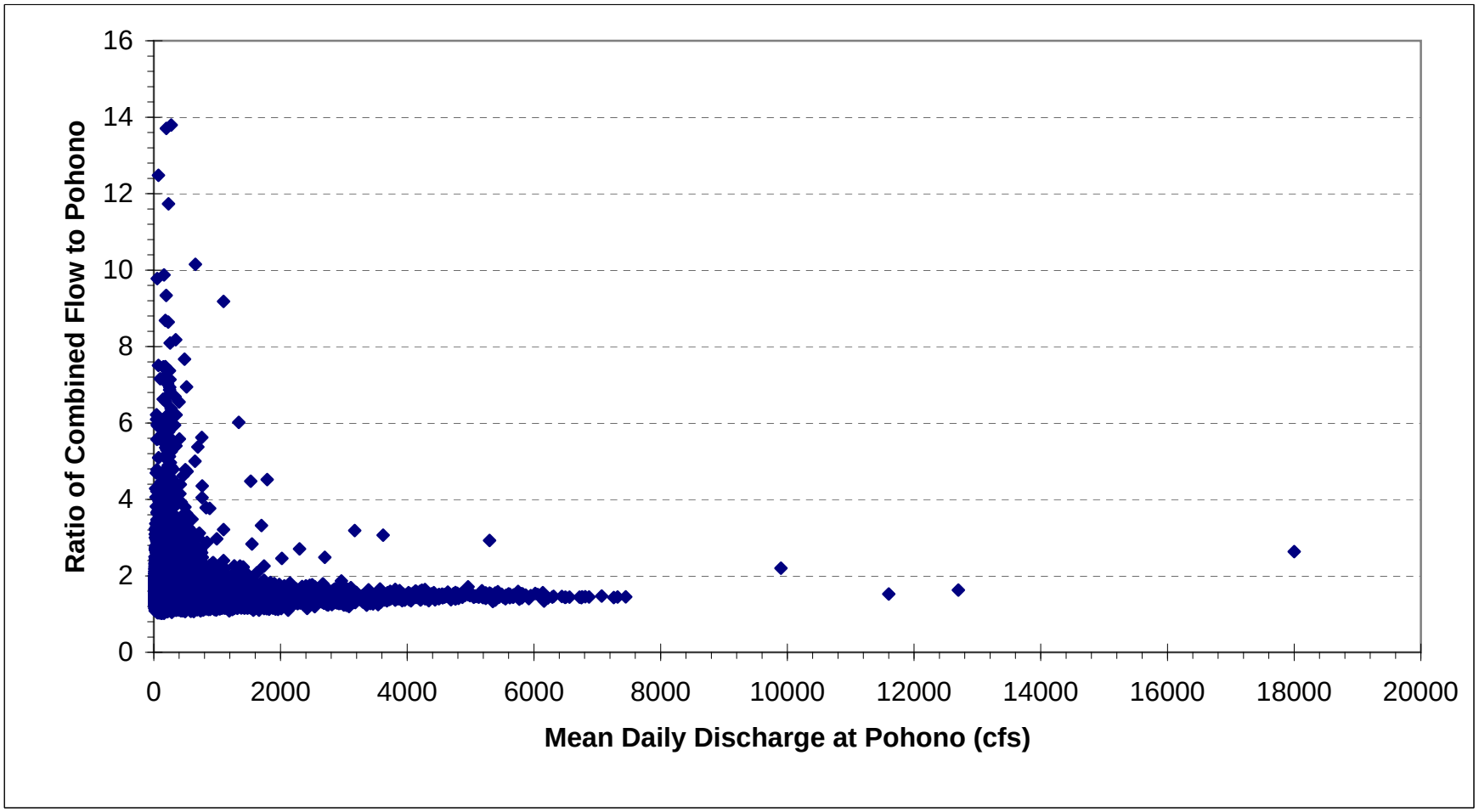
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.8. Mean daily discharge data at the El Portal South Fork Gage (#11268000) and the El Portal resultant of combining 11-2665 + 11-2680 (#11268001), Mariposa County, California. The combined data was considered to represent the flow in the mainstem of the Merced River based on references to gages 2665 and 2680.

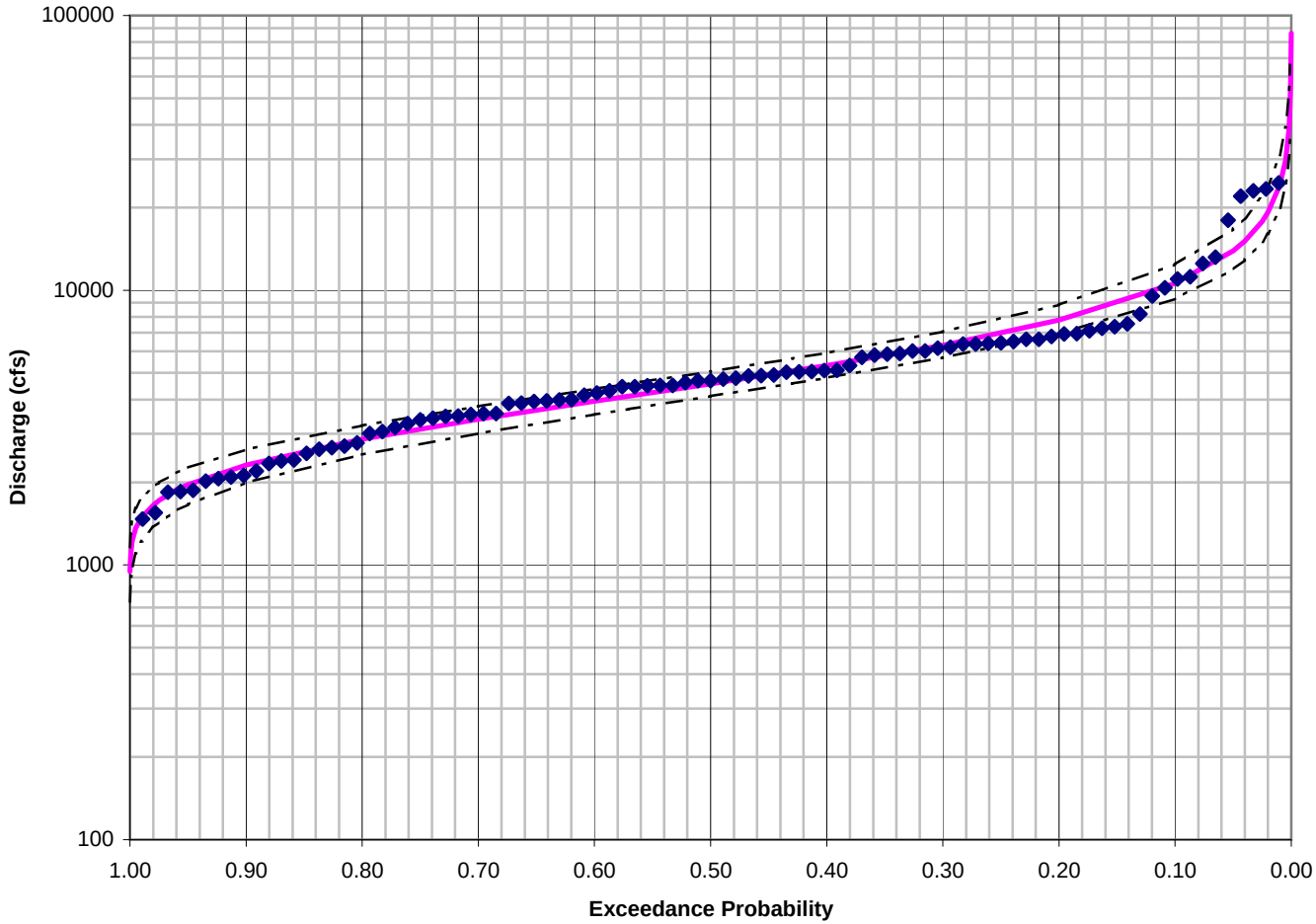
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 7.9. Ratio of flow at the combined flow El Portal Gage (11268001), located immediately upstream of the project reach, to the flow at the Pohono Gage (11266500), located approximately 16 miles upstream. Note that there are 4 high flow events during the period of record (1951 - 1975).

© 2008 Balance Hydrologics, Inc.



MERCED RIVER AT POHONO BRIDGE NR YOSEMITE, CA
(USGS #11266500)

Basin Area = 321 sq. mi.
Elevation: 3861.66 ft NGVD29
Water Years in Record
1917-2007

- Weibull Plotting Positions
- Flow Freq
- 95% Confidence Limit
- 5% Confidence Limit

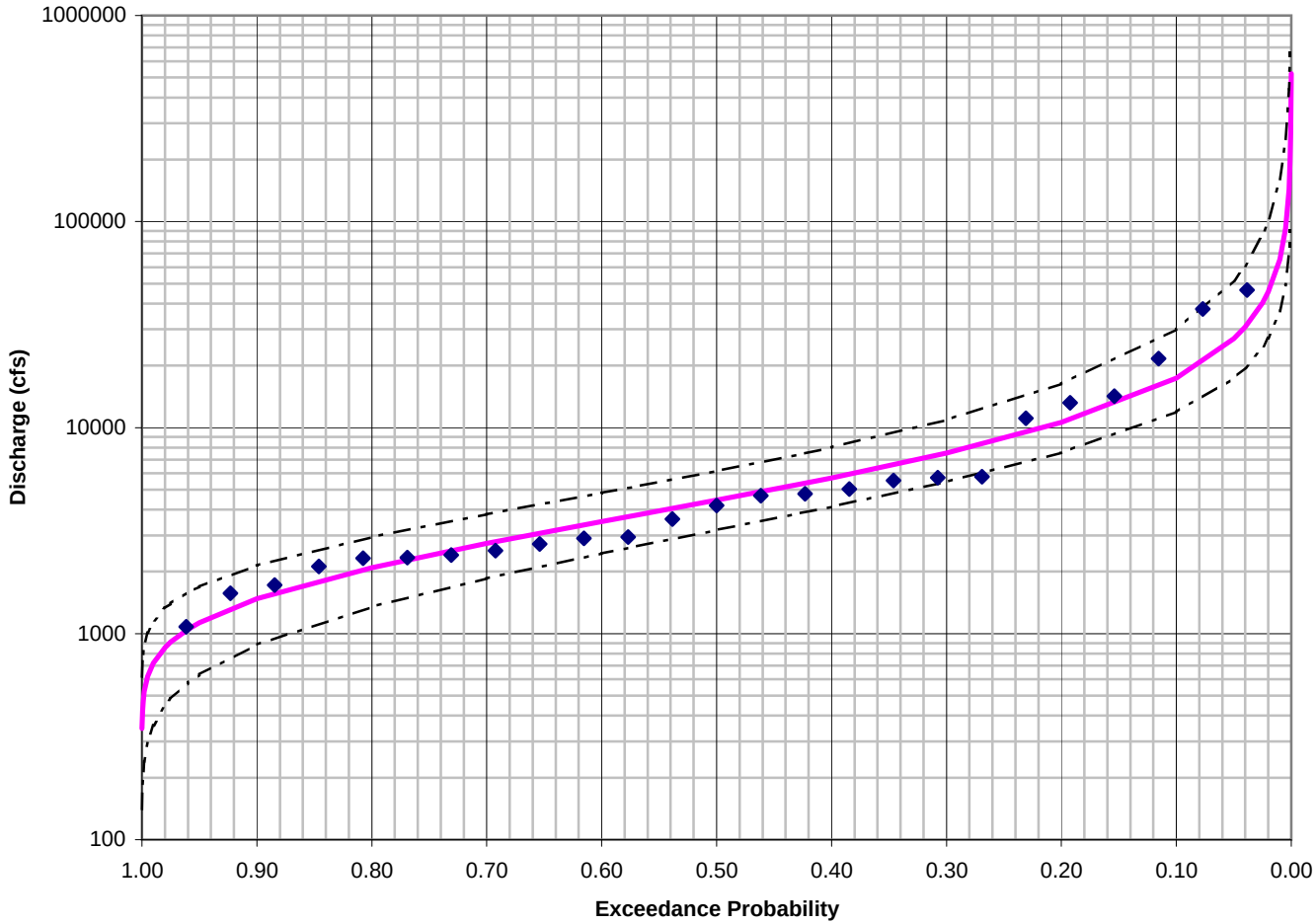
DISCHARGES FOR SPECIFIC PROBABILITIES

Exceedance Prob	Q (cfs)
0.9999	947
0.7 (1.4 yr event)	3392
0.50	4564
0.43 (2.3 yr event)	5077
0.20	7786
0.10	10600
0.04	15069
0.02	19158
0.01	23983
0.002	38773

Balance Hydrologics, Inc.

Figure 7.10. Flow frequency curve for Merced River at Pohono Bridge near Yosemite, USGS gage #11266500, 1917-2007, Mariposa County, California.

© 2008 Balance Hydrologics, Inc.



SF MERCED RIVER NR EL PORTAL, CA
(USGS #11268000)

Basin Area = 241 sq. mi.
Elevation: n/a
Water Years in Record
1950-1975

Weibull Plotting Positions
Flow Freq
95% Confidence Limit
5% Confidence Limit

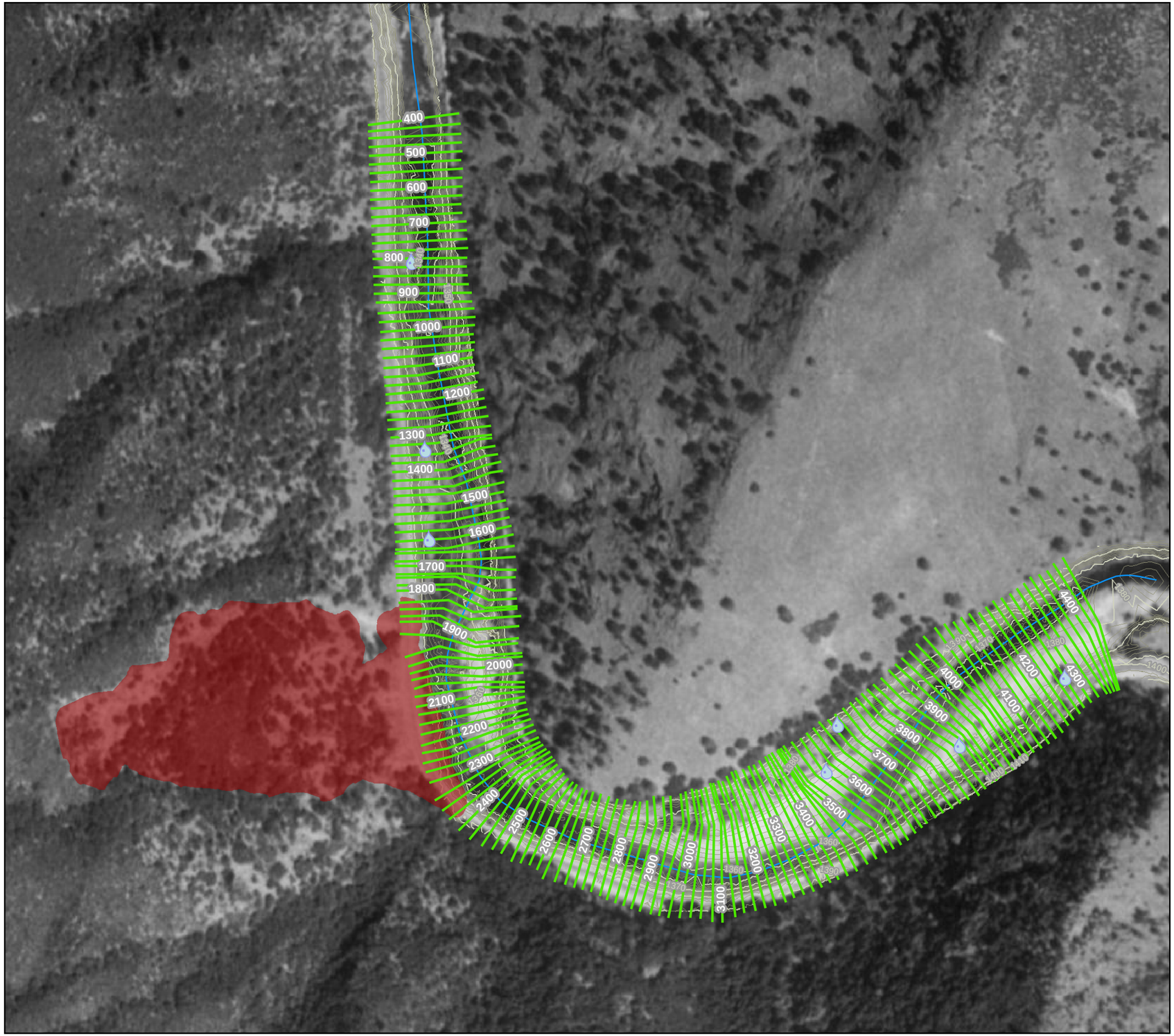
DISCHARGES FOR SPECIFIC PROBABILITIES

Exceedance Prob	Q (cfs)
0.9999	346
0.7 (1.4 yr event)	2743
0.50	4441
0.43 (2.3 yr event)	5279
0.20	10569
0.10	17428
0.04	30844
0.02	45534
0.01	65560
0.002	142939

Balance Hydrologics, Inc.

Figure 7.11. Flow frequency curve for South Fork Merced River near El Portal, USGS gage #11268000, 1950-1975, Mariposa County, California.

© 2008 Balance Hydrologics, Inc.



N

300 150 0 300 600

FEET

LEGEND

HEC-RAS Cross-sections

HEC-RAS River alignment

Observed High Water Marks

Ferguson Rock Slide Extents

Major 50-ft Contours

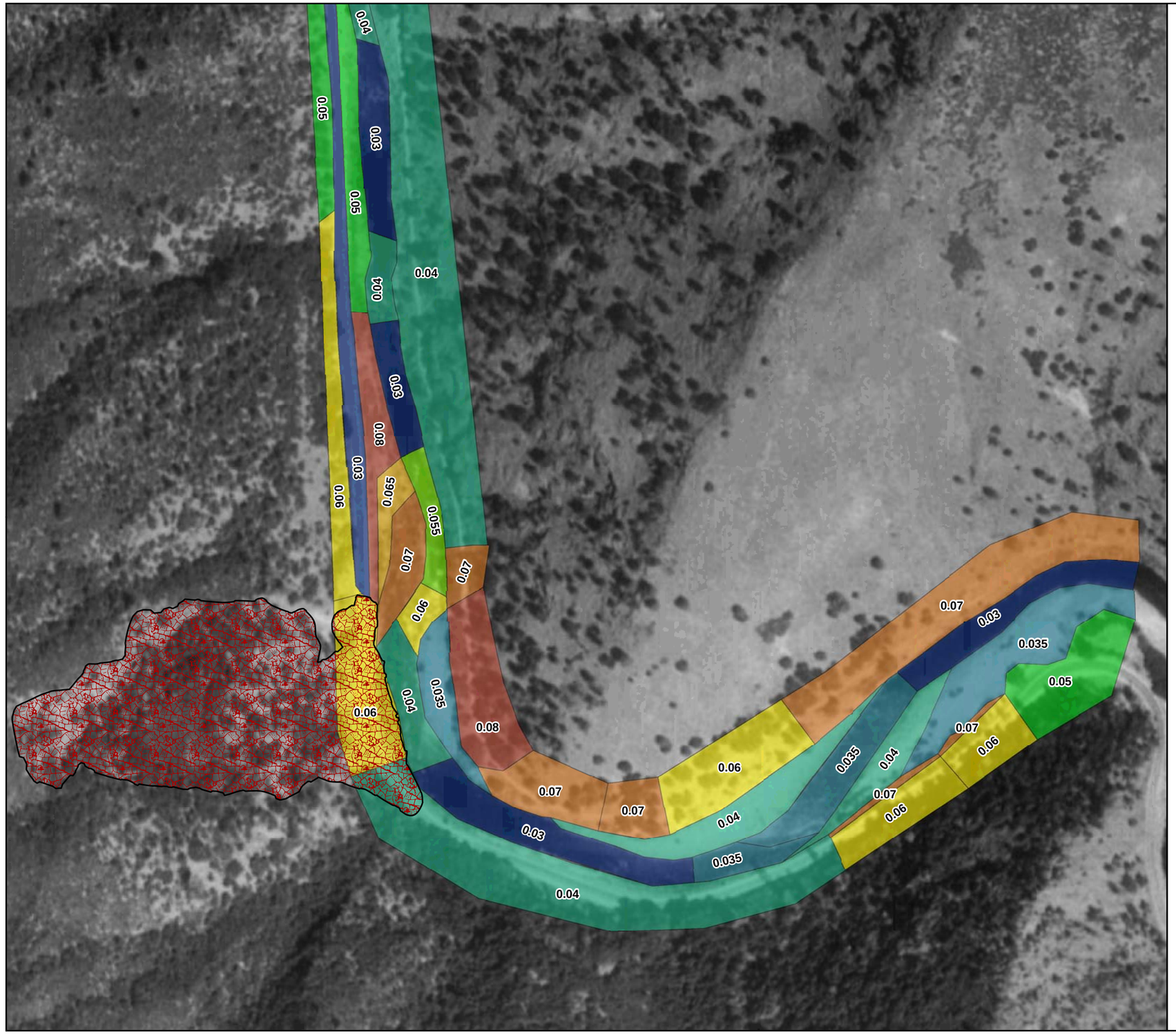
Minor 10-ft Contours

Figure 8.1 **HEC-RAS model of the Merced River at Ferguson Slide, Mariposa County, California**

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_HECRAS_110708.mxd



N

300 150 0 300 600

FEET

LEGEND

Manning's n-Value

- 0.03
- 0.035
- 0.04
- 0.05
- 0.055
- 0.06
- 0.065
- 0.07
- 0.08

Ferguson Rock Slide Extents

Figure 8.2 Manning's n-values used for hydraulic modeling of the Merced River at Ferguson Slide, Mariposa County, California

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_ManningsN_110708.mxd

Figure 8.3. Example photographs of observed high water marks.

© 2008 Balance Hydrologics, Inc.

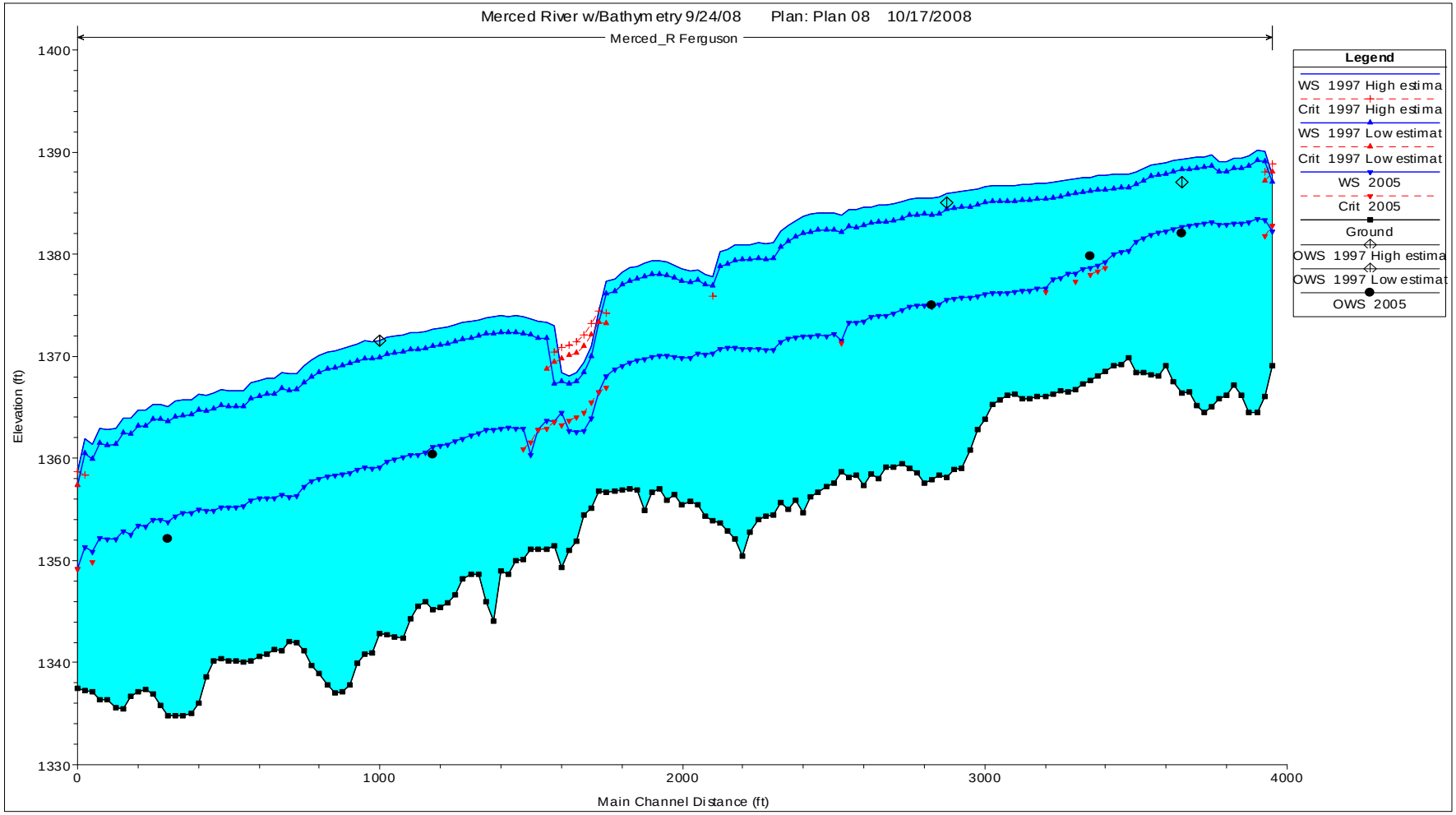


Figure 8.4. Water surface elevation profiles generated by the HEC-RAS 1-D hydraulic model for the high and low estimates of the 1997 storm discharge (43,500 and 49,000 cfs), and for the estimated 2005 peak discharge (17,000 cfs). Observed high water marks for each event are also shown. The model results corrospond well to the observed high water marks.

© 2008 Balance Hydrologics, Inc.

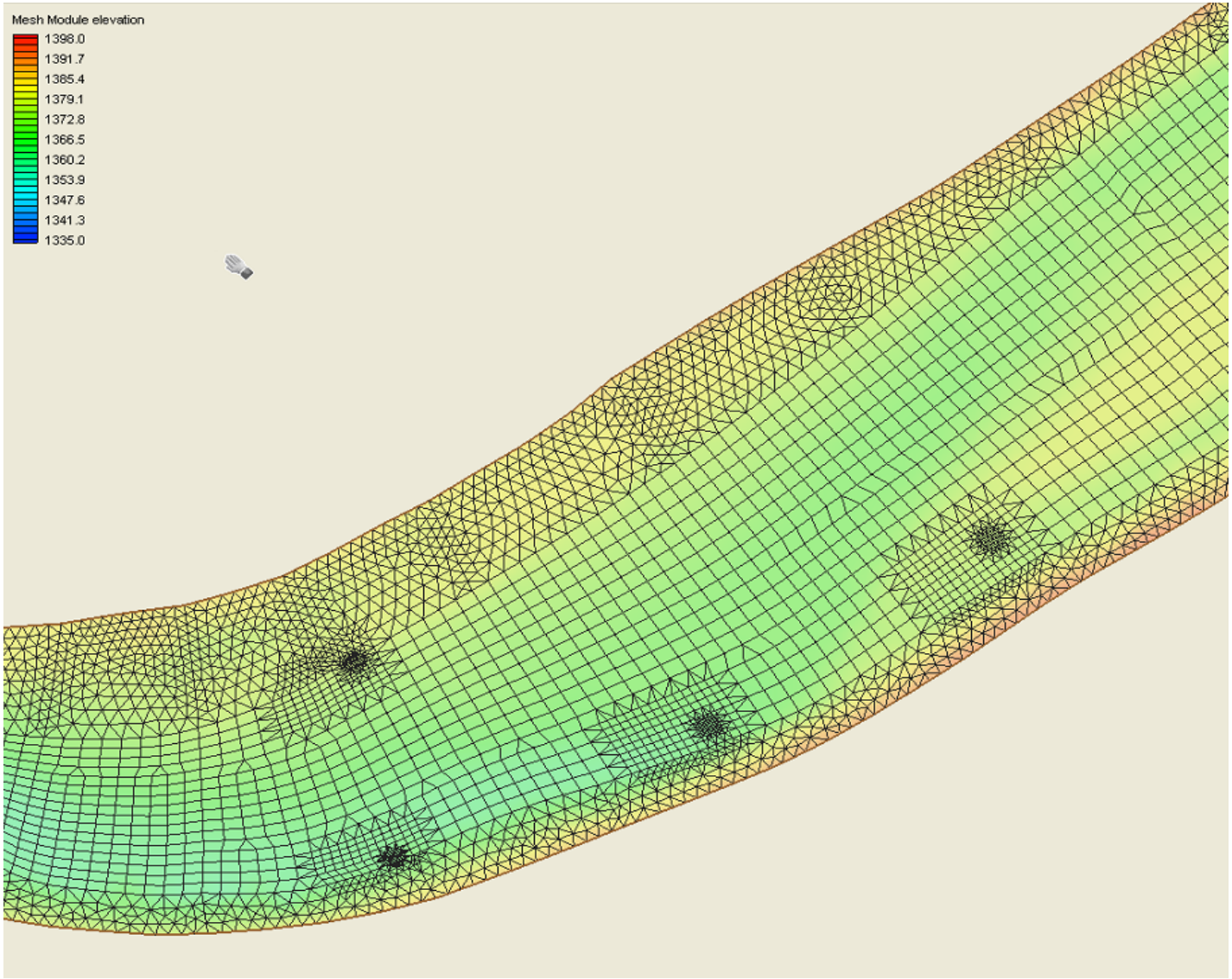
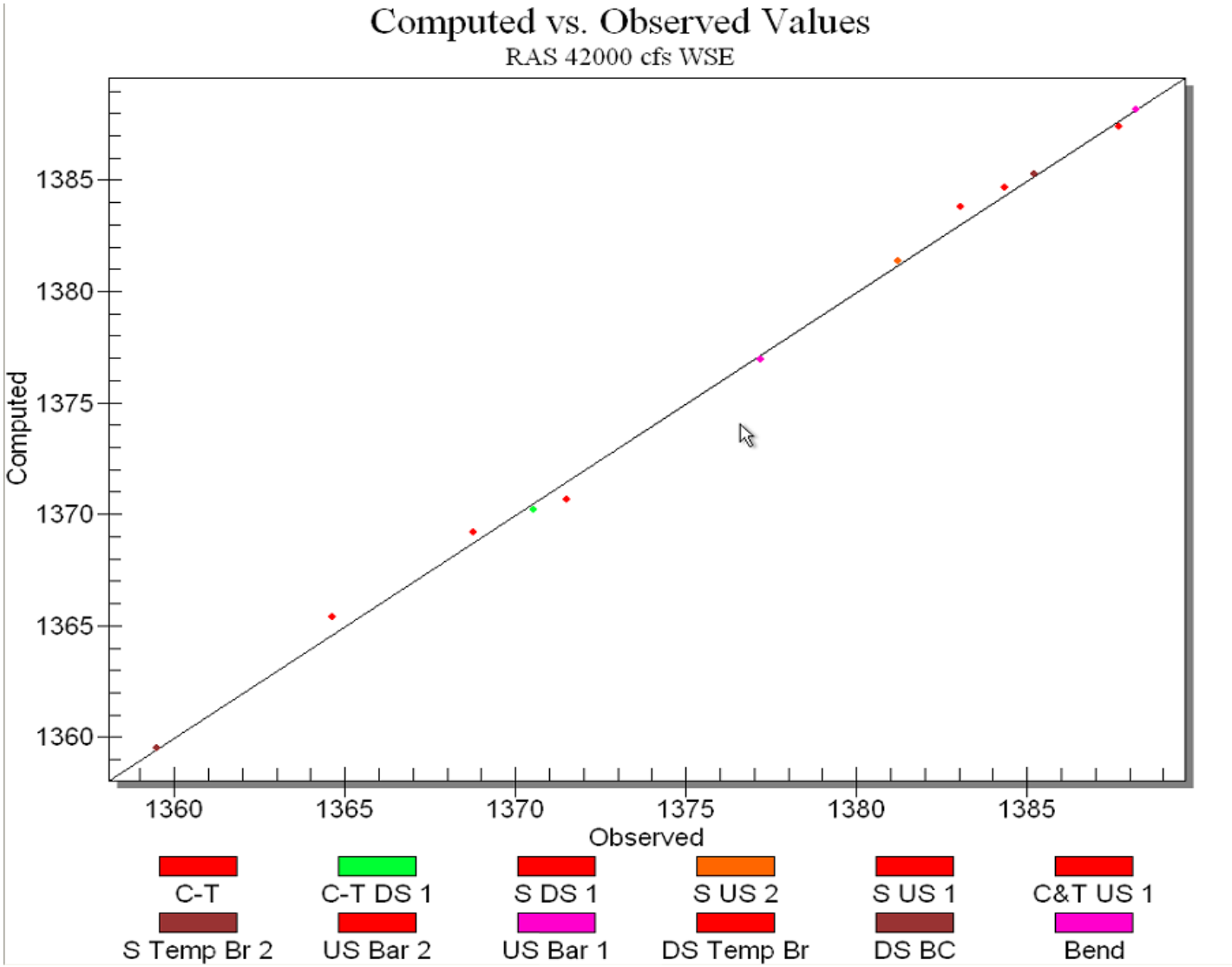


Figure 8.5. Example section of mesh used for the 2-dimension high-flow hydraulic model. The main channel is rectilinear and the banks and bars are triangular. Dense patches of cells are for representing piers.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 8.6. Water surface elevations (ft) computed with the 2-D hydraulic model for the 42,000 cfs flow rate, plotted against HEC-RAS results, which were calibrated to the high water marks observed in the field.

© 2008 Balance Hydrologics, Inc.

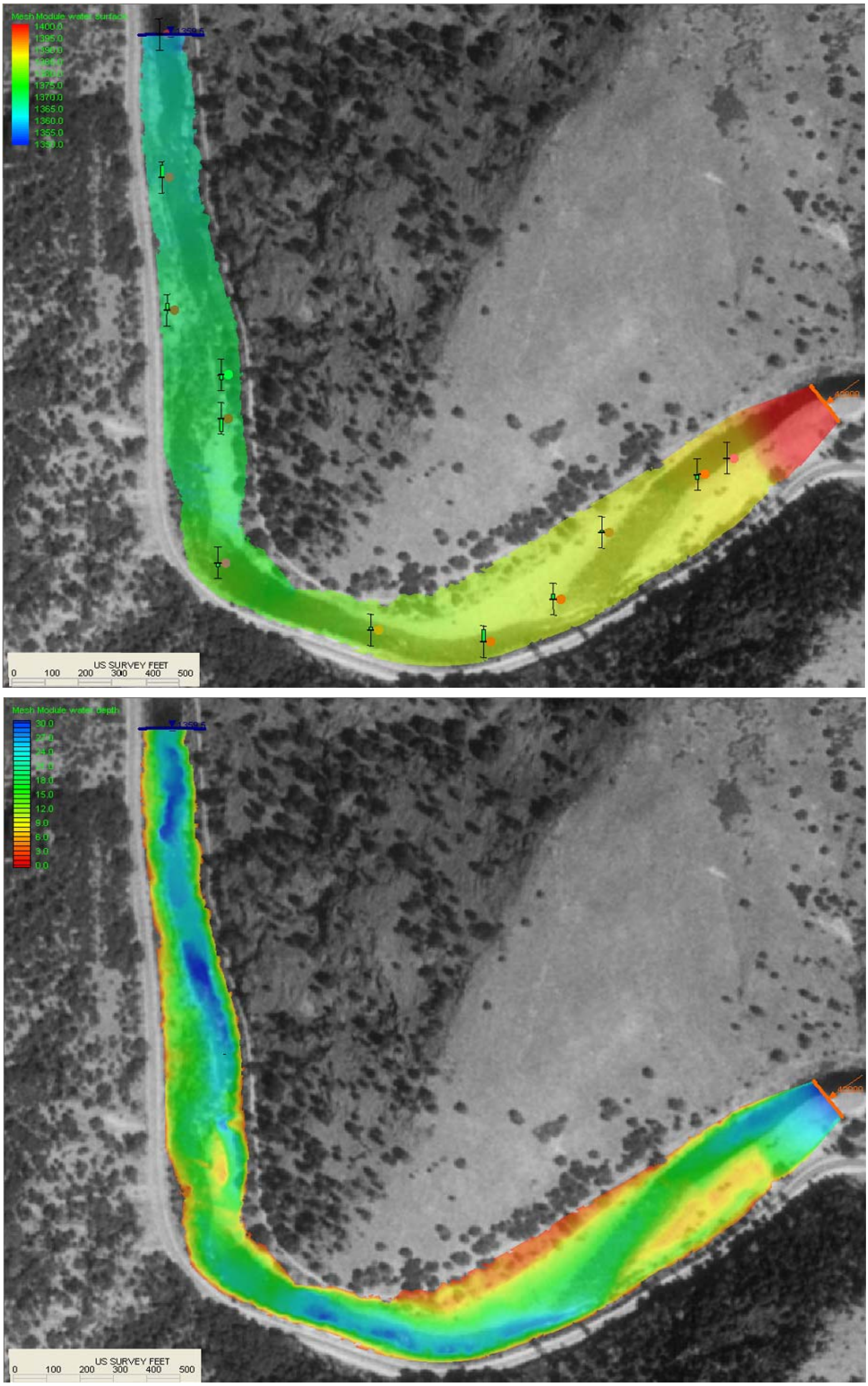
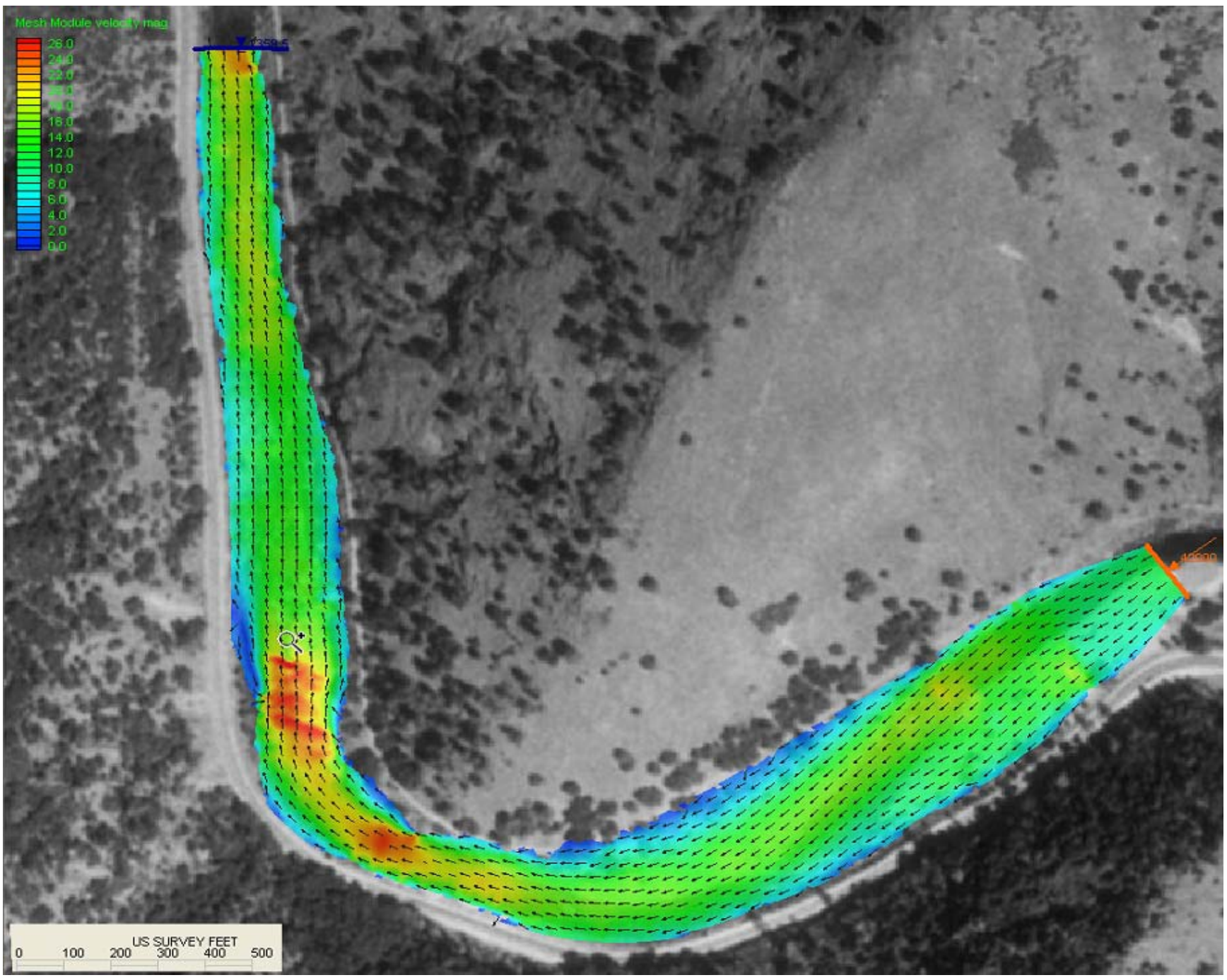
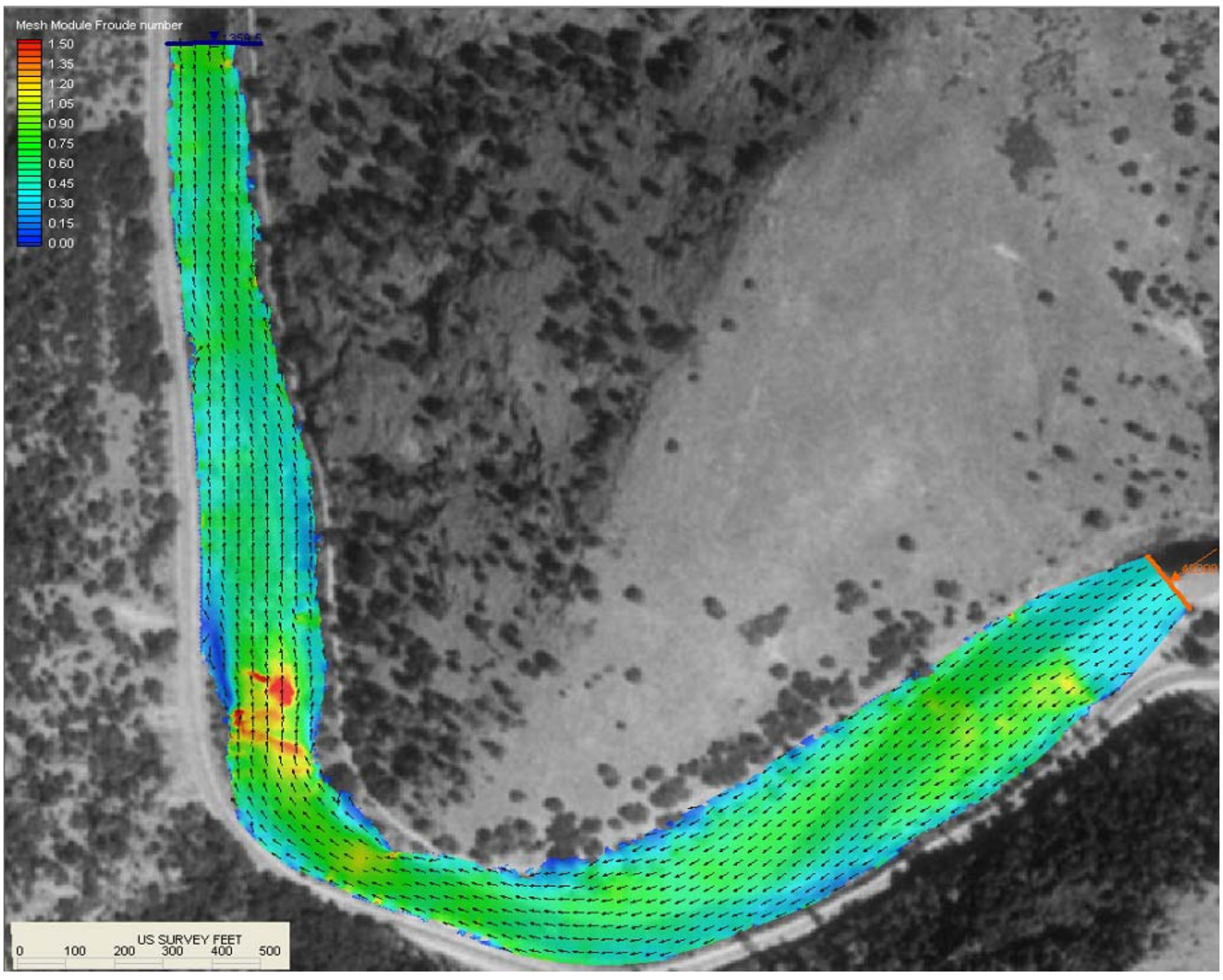


Figure 8.7. Water surface elevation (upper) and depth (lower) contours for the natural channel condition at 42,000 cfs. The box plots represent the deviation of the water surface elevation from the value estimated in the calibrated HEC-RAS model.

© 2008 Balance Hydrologics, Inc.

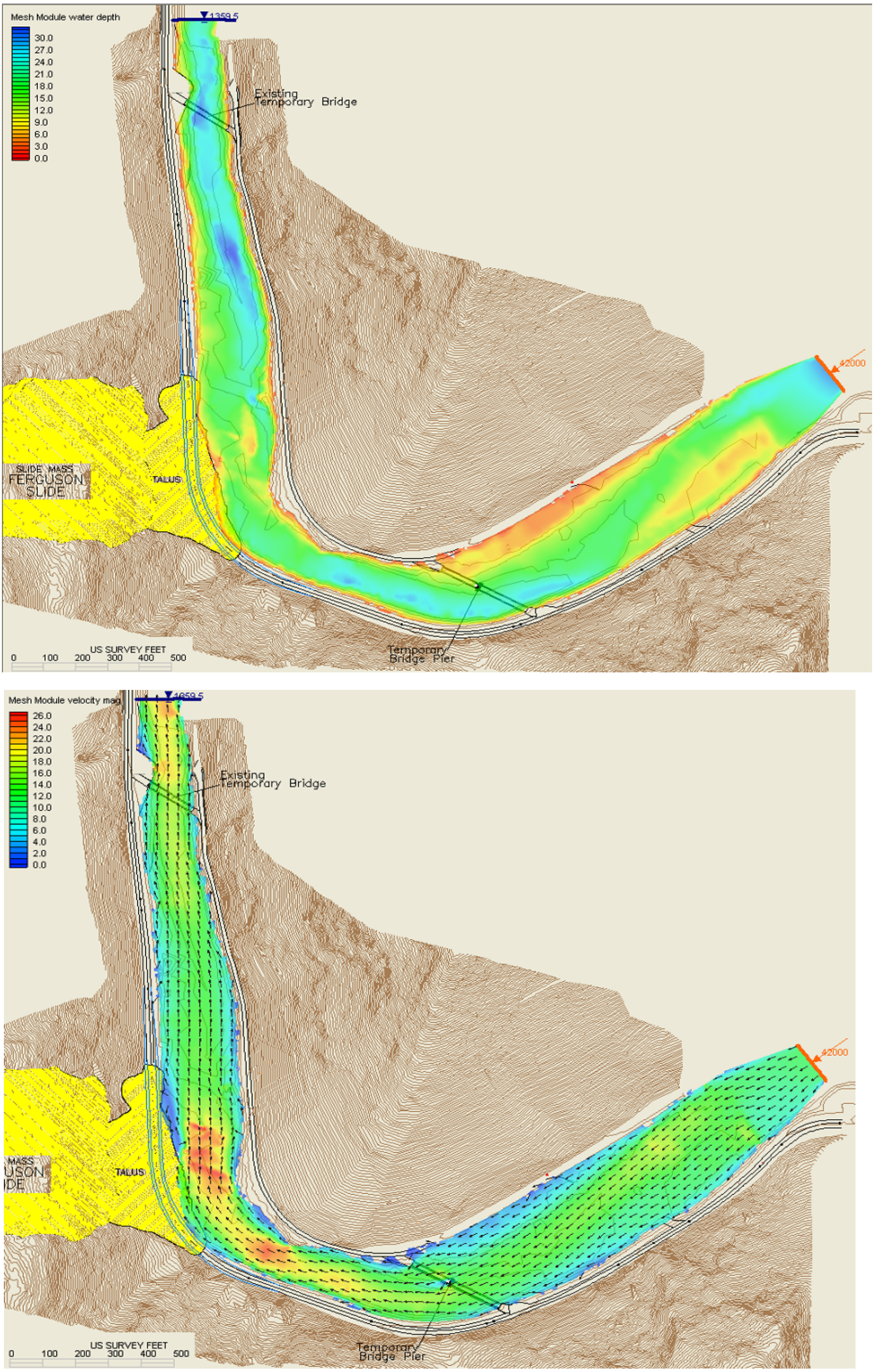




Balance Hydrologics, Inc.

Figure 8.8. Velocity (upper) and Froude number (lower) contours for the natural channel condition at 42,000 cfs.

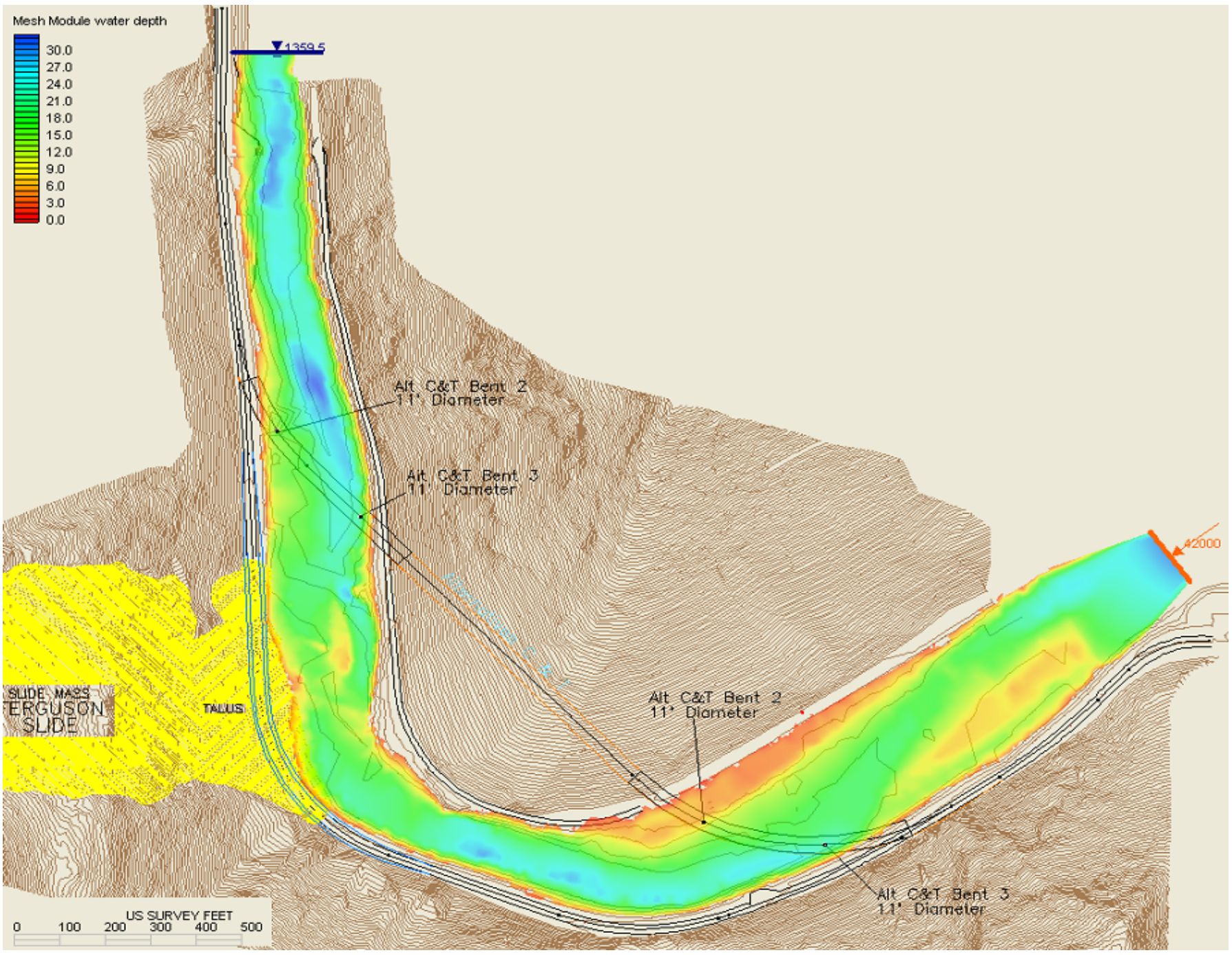
© 2008 Balance Hydrologics, Inc.

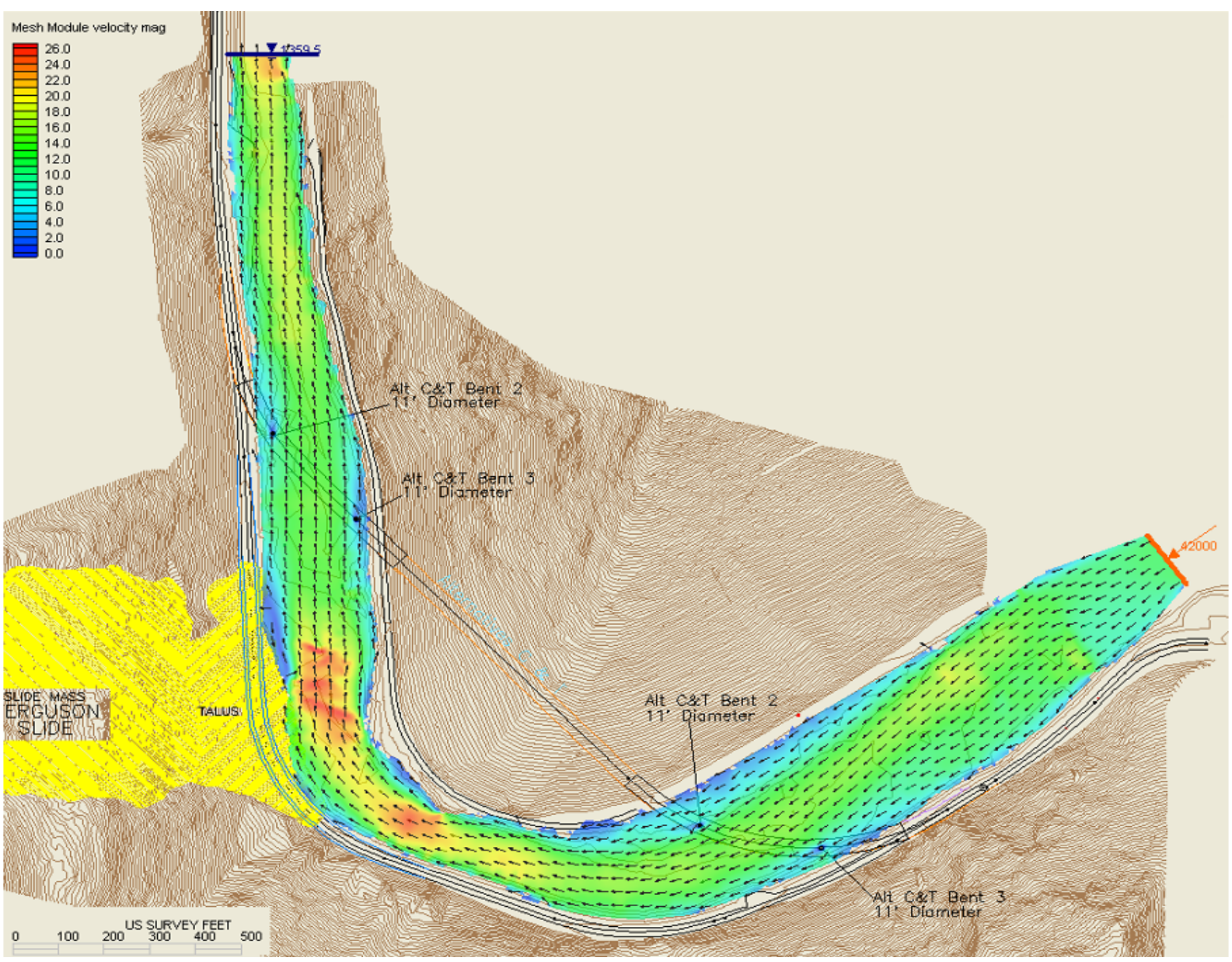


Balance Hydrologics, Inc.

Figure 8.9. Depth (upper) and velocity (lower) contours for the temporary bridge conditions for Alternatives C and T at the upstream location and C, T, and S at the downstream location, for a flood flow of 42,000 cfs.

© 2008 Balance Hydrologics, Inc.

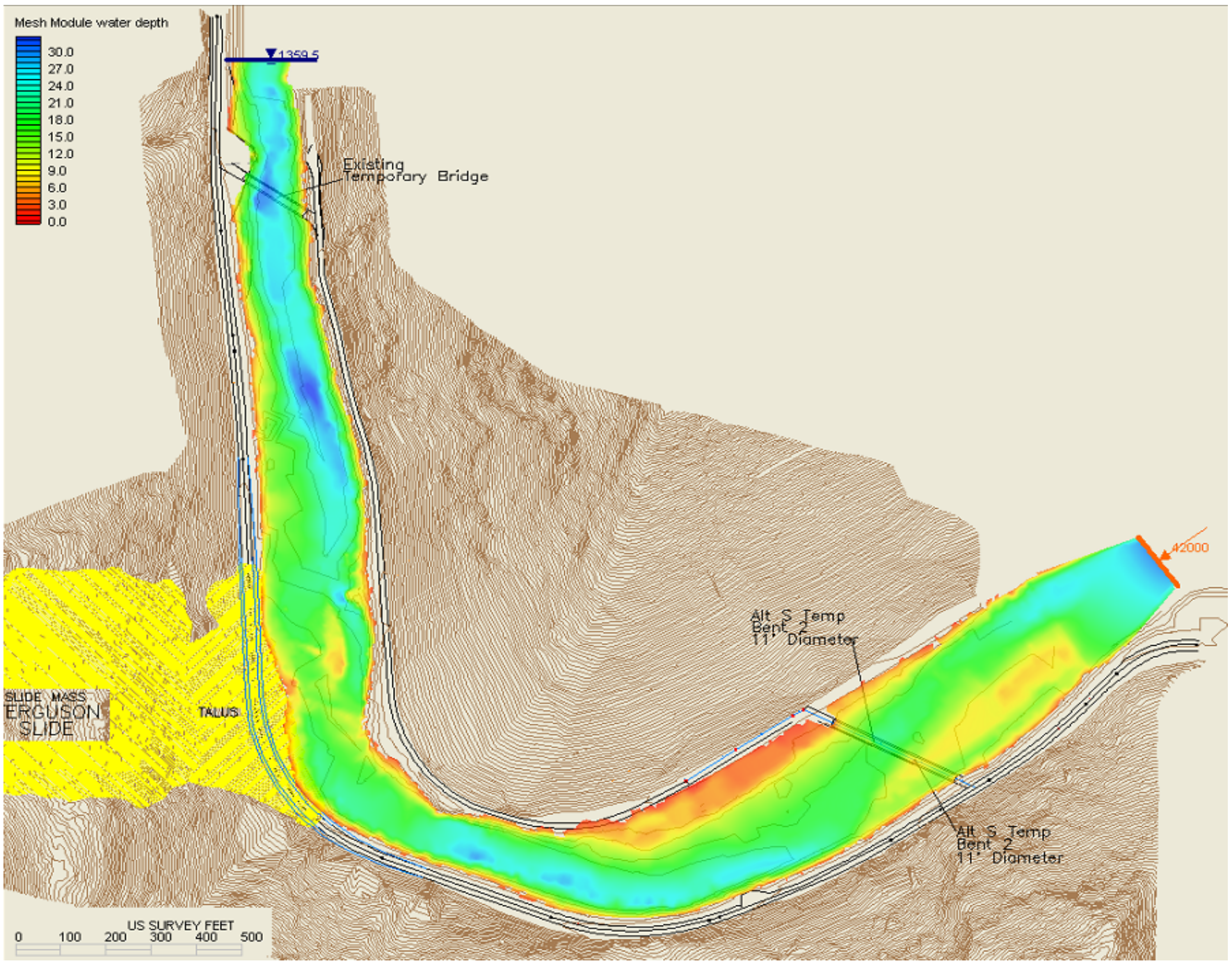


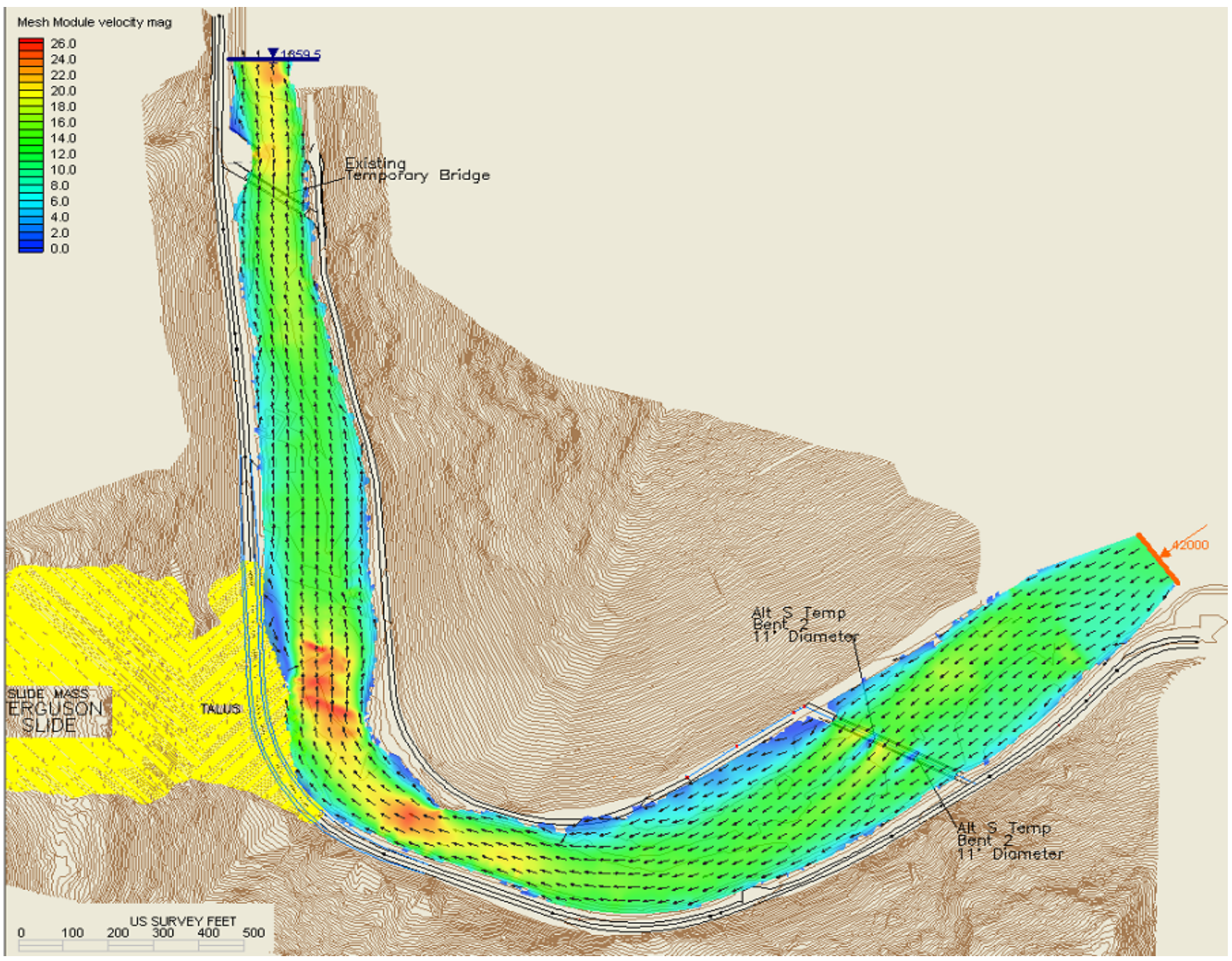


Balance Hydrologics, Inc.

Figure 8.10. Depth (upper) and velocity (lower) contours for the C and T bridge alternatives at 42,000 cfs.

© 2008 Balance Hydrologics, Inc.

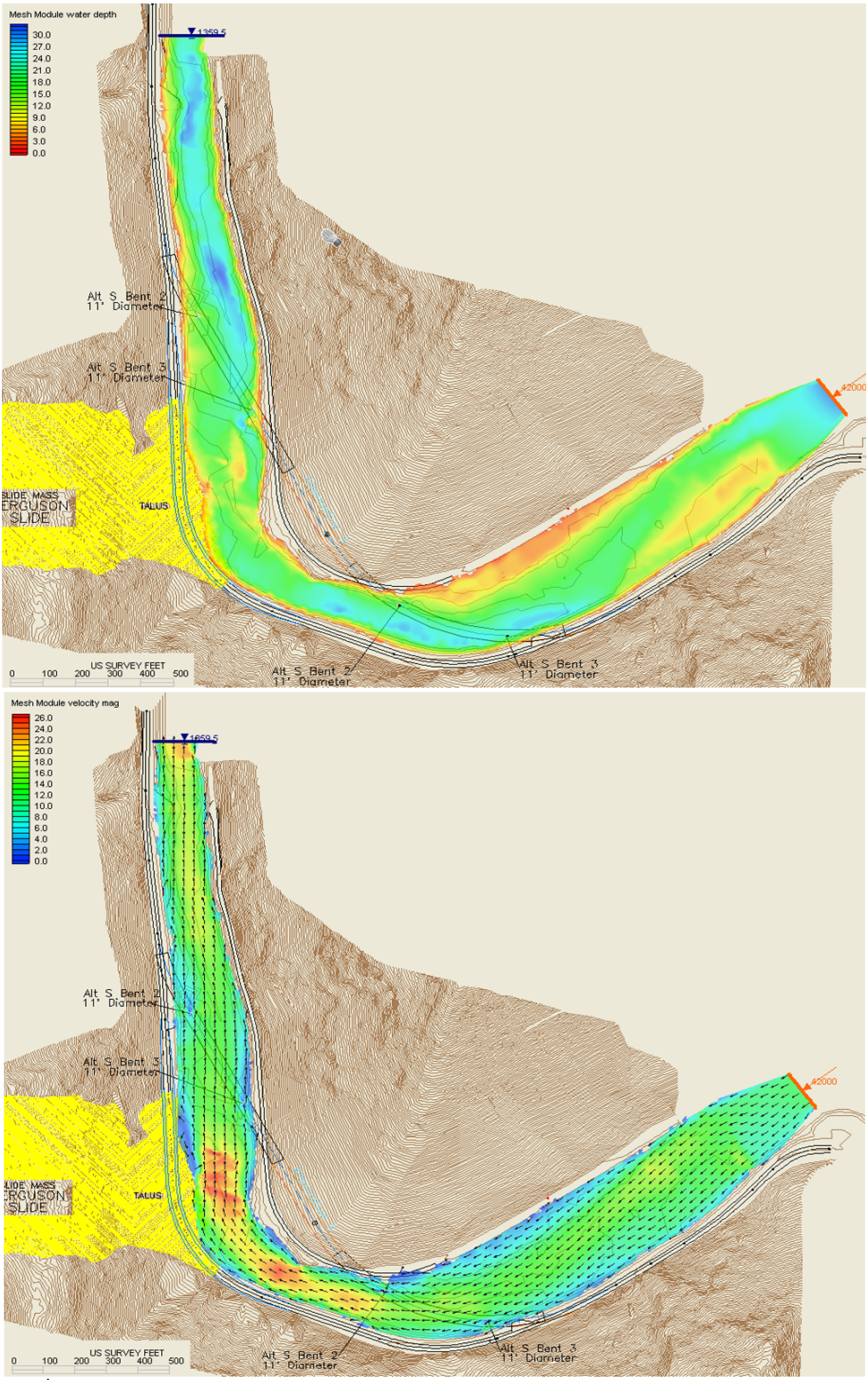




Balance Hydrologics, Inc.

Figure 8.11. Depth (upper) and velocity (lower) contours for the temporary bridges proposed during the constructin of Alternative S at 42,000 cfs.

© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 8.12. Depth (upper) and velocity (lower) contours for the Alternative S at 42,000 cfs.

© 2008 Balance Hydrologics, Inc.

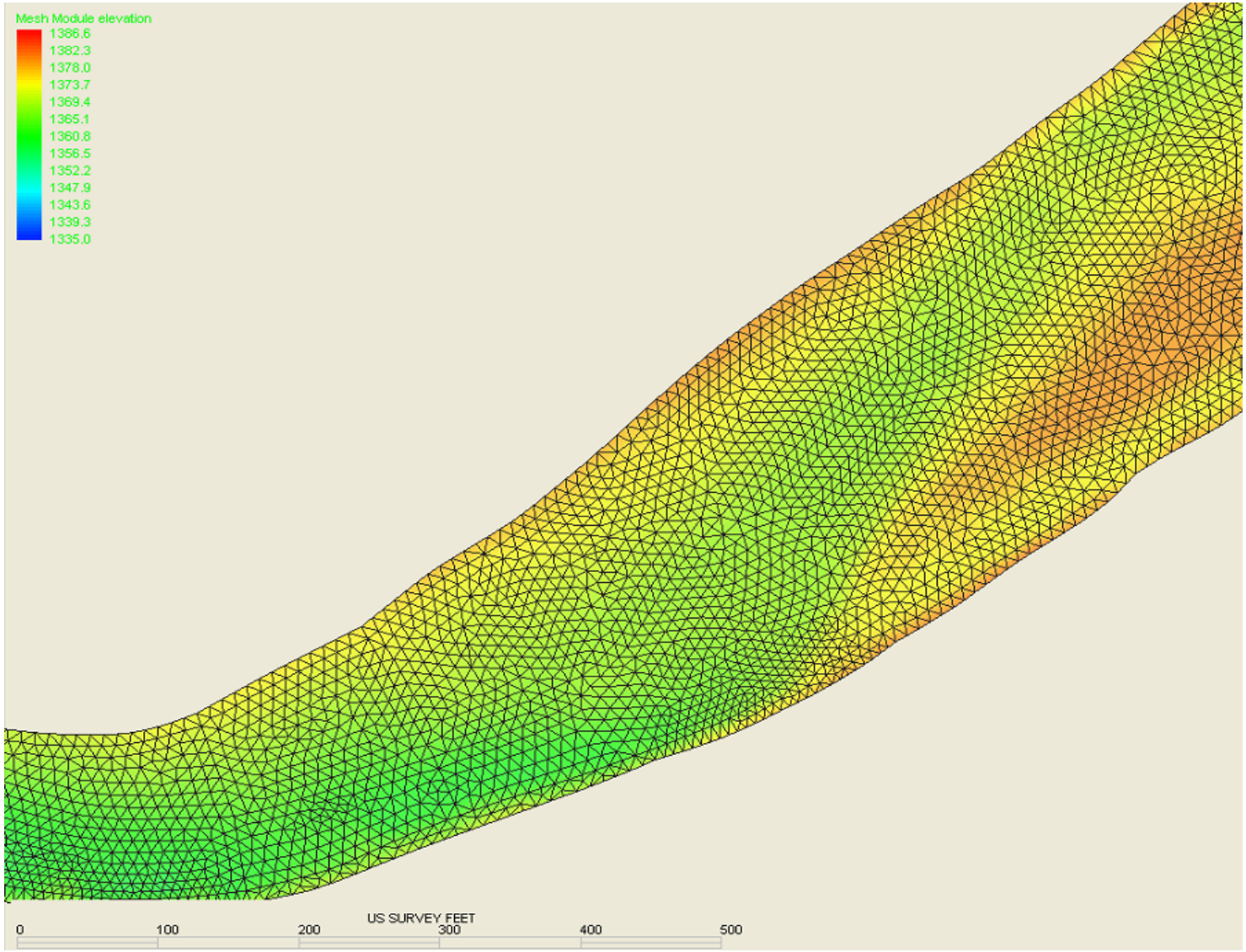
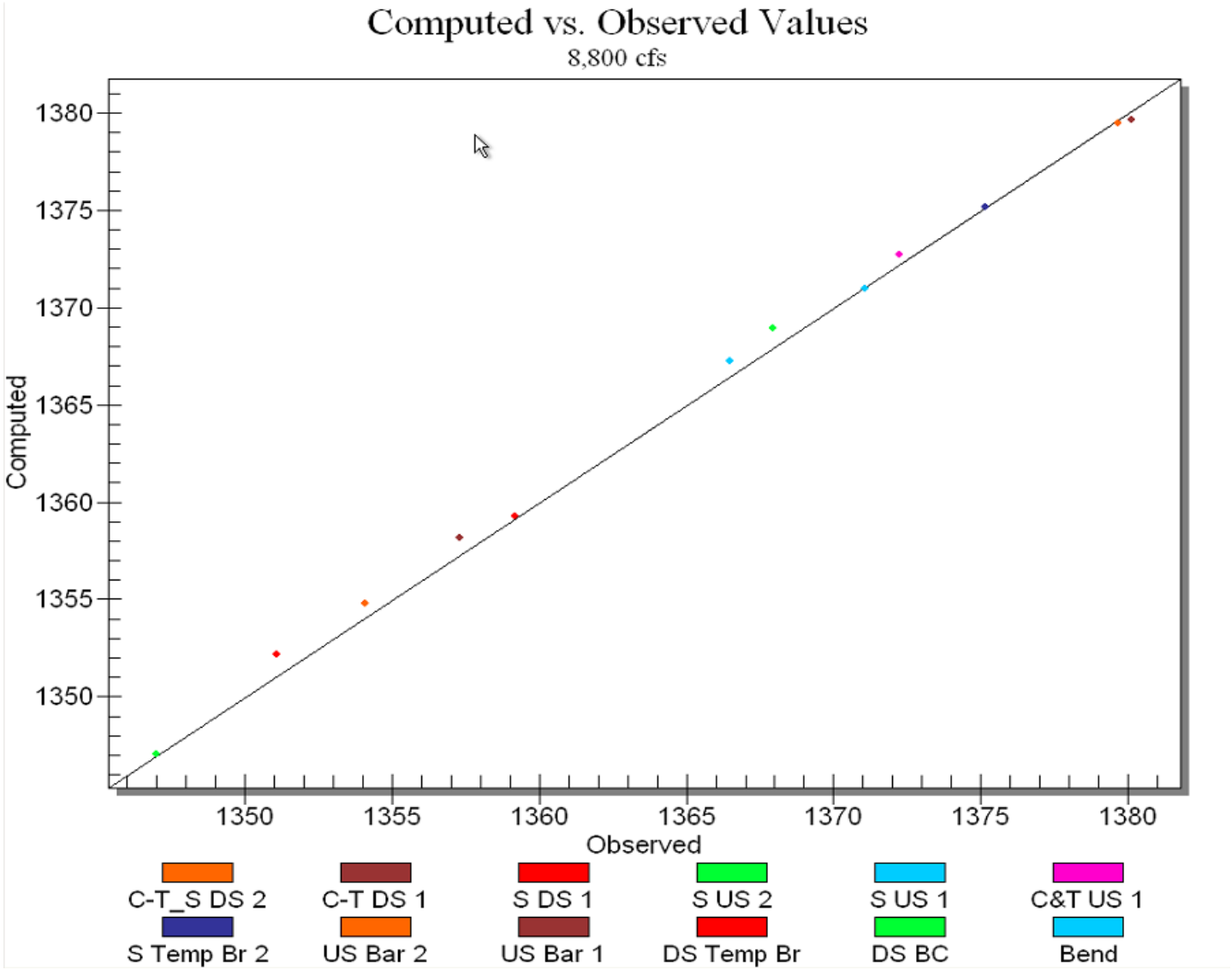


Figure 8.13. Example section of triangular mesh used in the 2-dimensional low-flow (8,800 cfs rafting flow) hydraulic model.

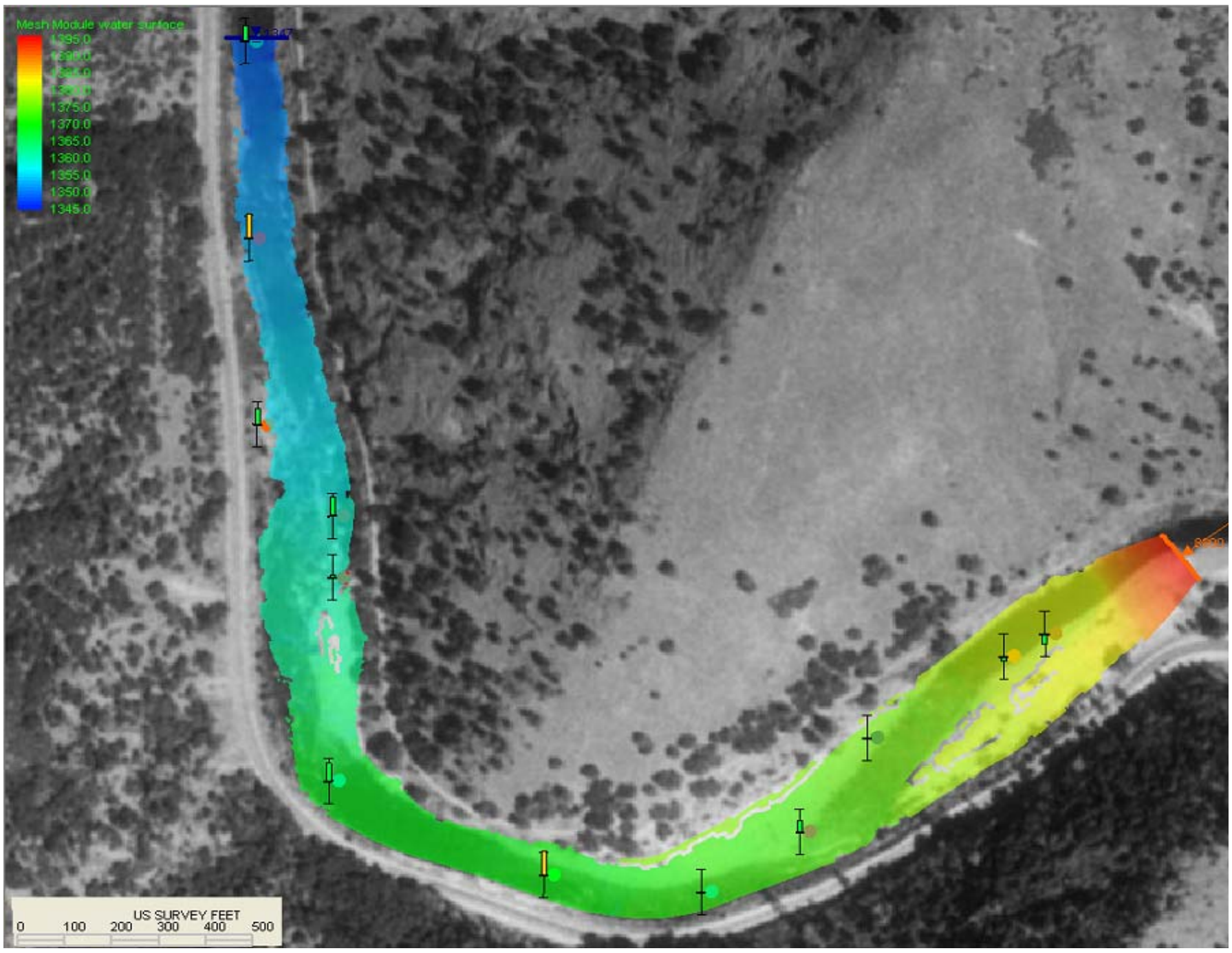
© 2008 Balance Hydrologics, Inc.

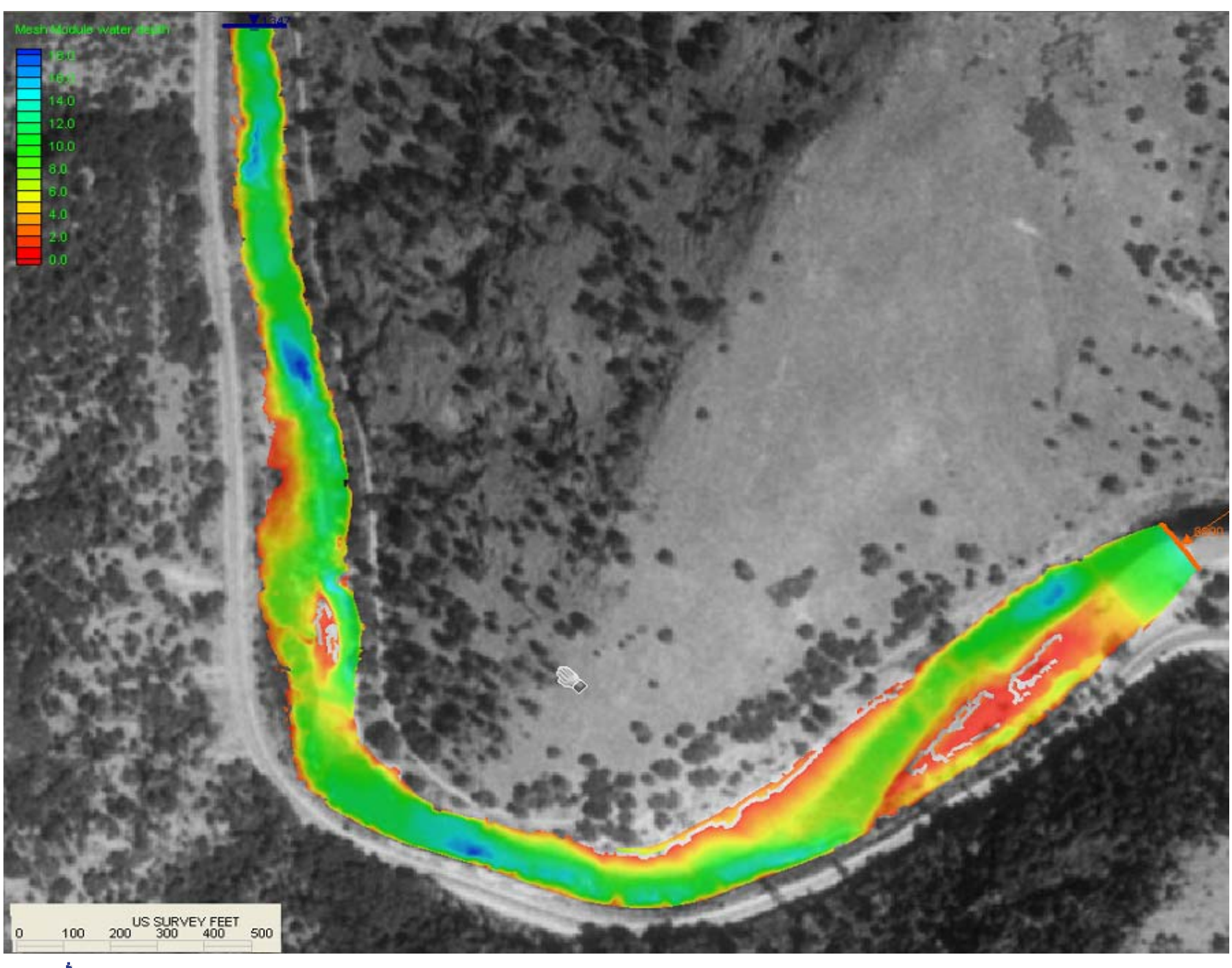


Balance Hydrologics, Inc.

Figure 8.14. Water surface elevations (ft) computed with the 2-D hydraulic model for the 8,800 cfs flow rate, plotted against HEC-RAS results, which were calibrated to the high water marks observed in the field.

© 2008 Balance Hydrologics, Inc.

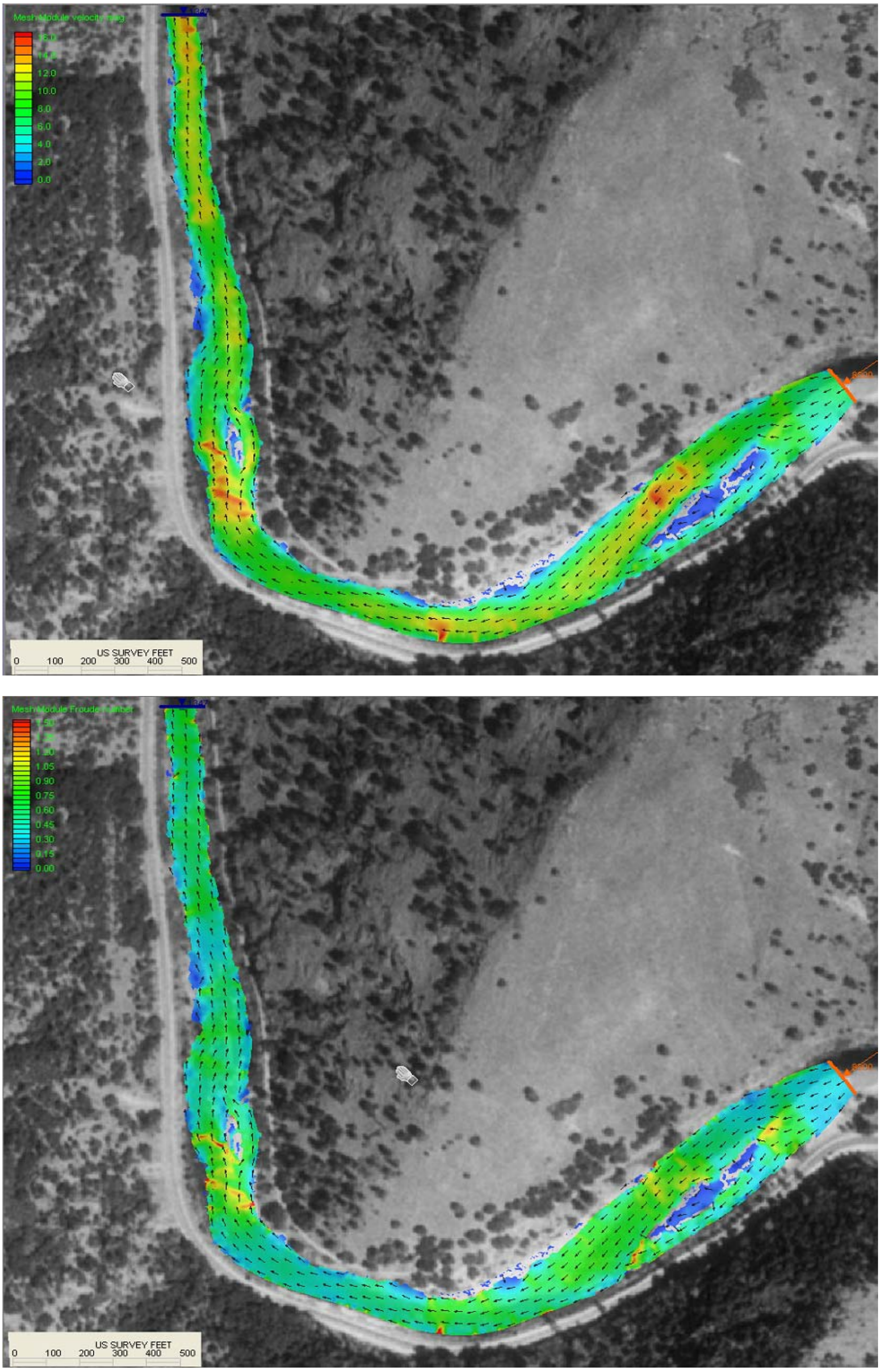




Balance Hydrologics, Inc.

Figure 8.15. Water surface elevation (upper) and depth (lower) contours for the natural channel condition at the rafting flow (8,800 cfs). The box plots represent the deviation of the water surface elevation from the value estimated in the calibrated HEC-RAS model.

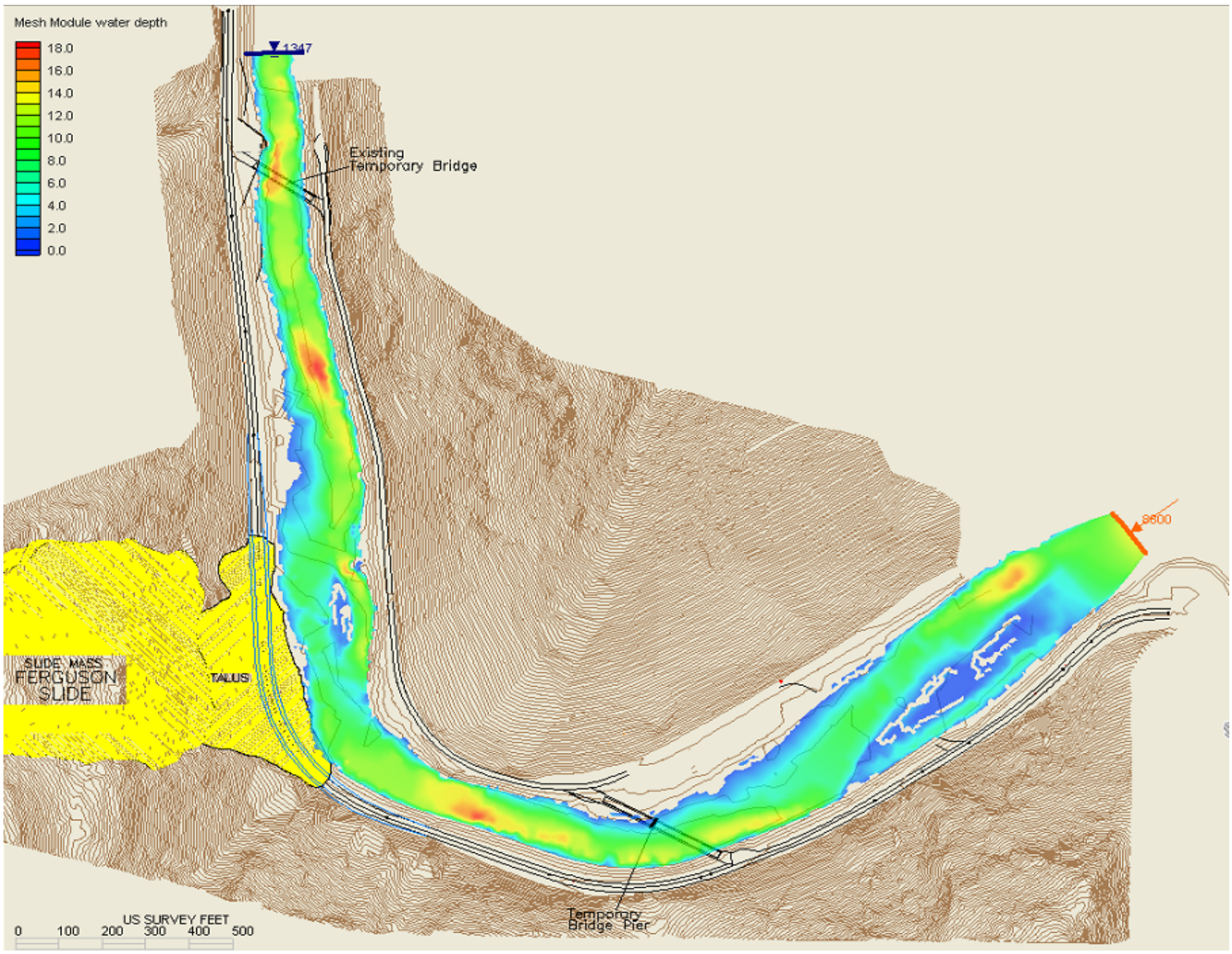
© 2008 Balance Hydrologics, Inc.

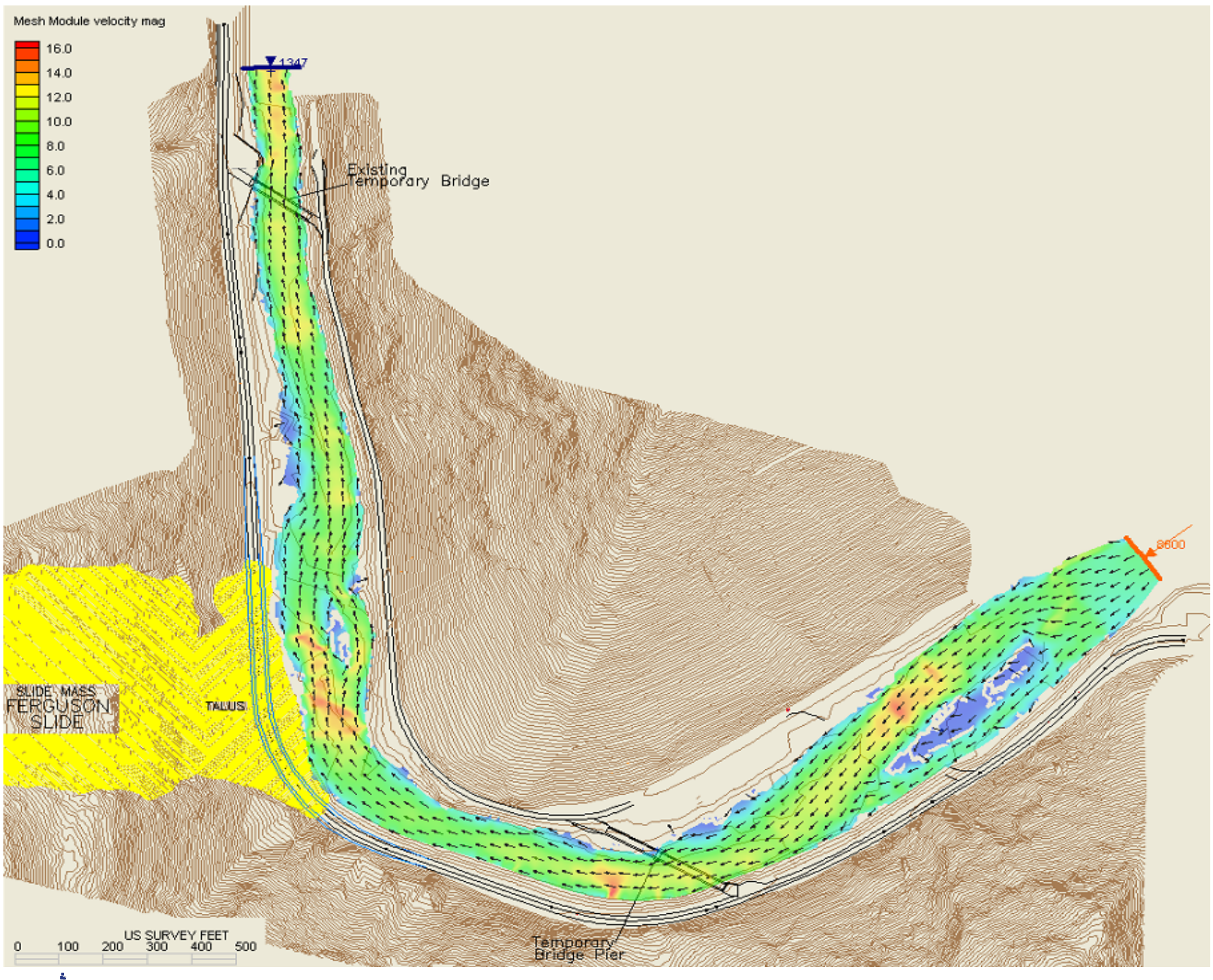


Balance Hydrologics, Inc.

Figure 8.16. Velocity (upper) and Froude # (lower) contours for the natural channel condition at the rafting flow (8,800 cfs).

© 2008 Balance Hydrologics, Inc.





Balance Hydrologics, Inc.

Figure 8.17. Depth (upper) and velocity (lower) contours for the temporary bridge conditions for Alternatives C and T at the upstream location and C, T, and S at the downstream location, for a flood flow of 8,800 cfs.

© 2008 Balance Hydrologics, Inc.

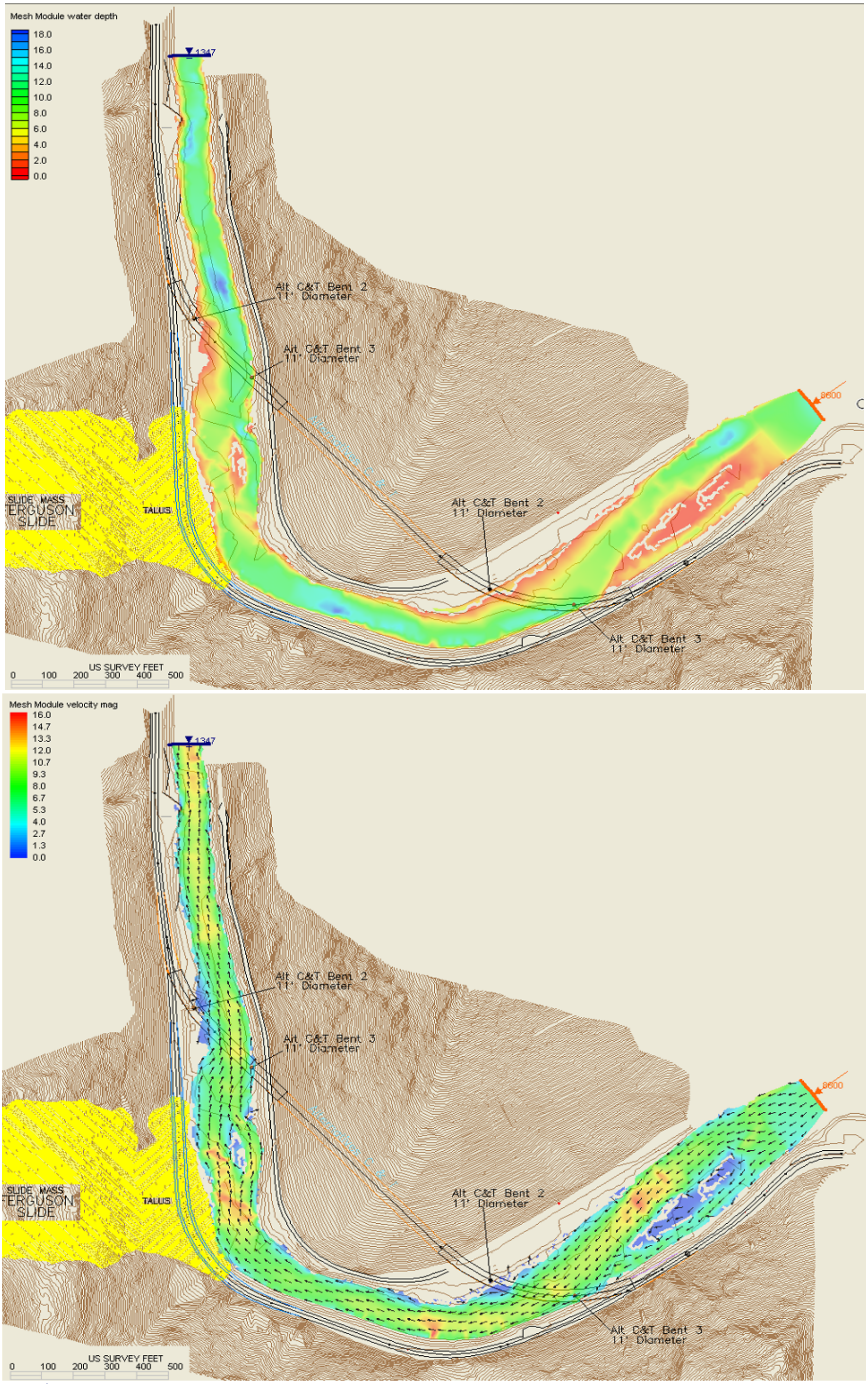
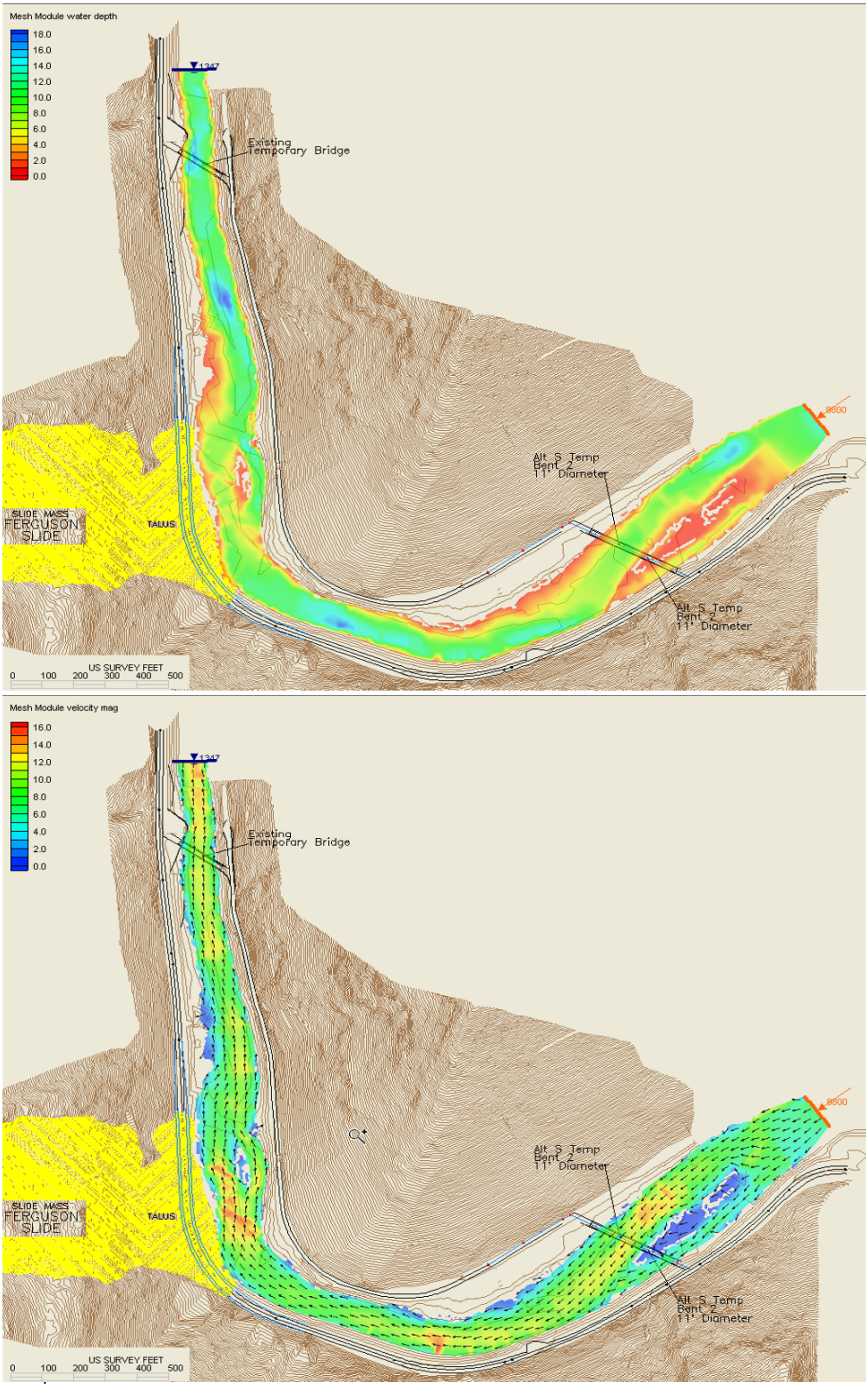


Figure 8.18. Depth (upper) and velocity (lower) contours for the C and T bridge alternatives at the rafting flow (8,800 cfs).

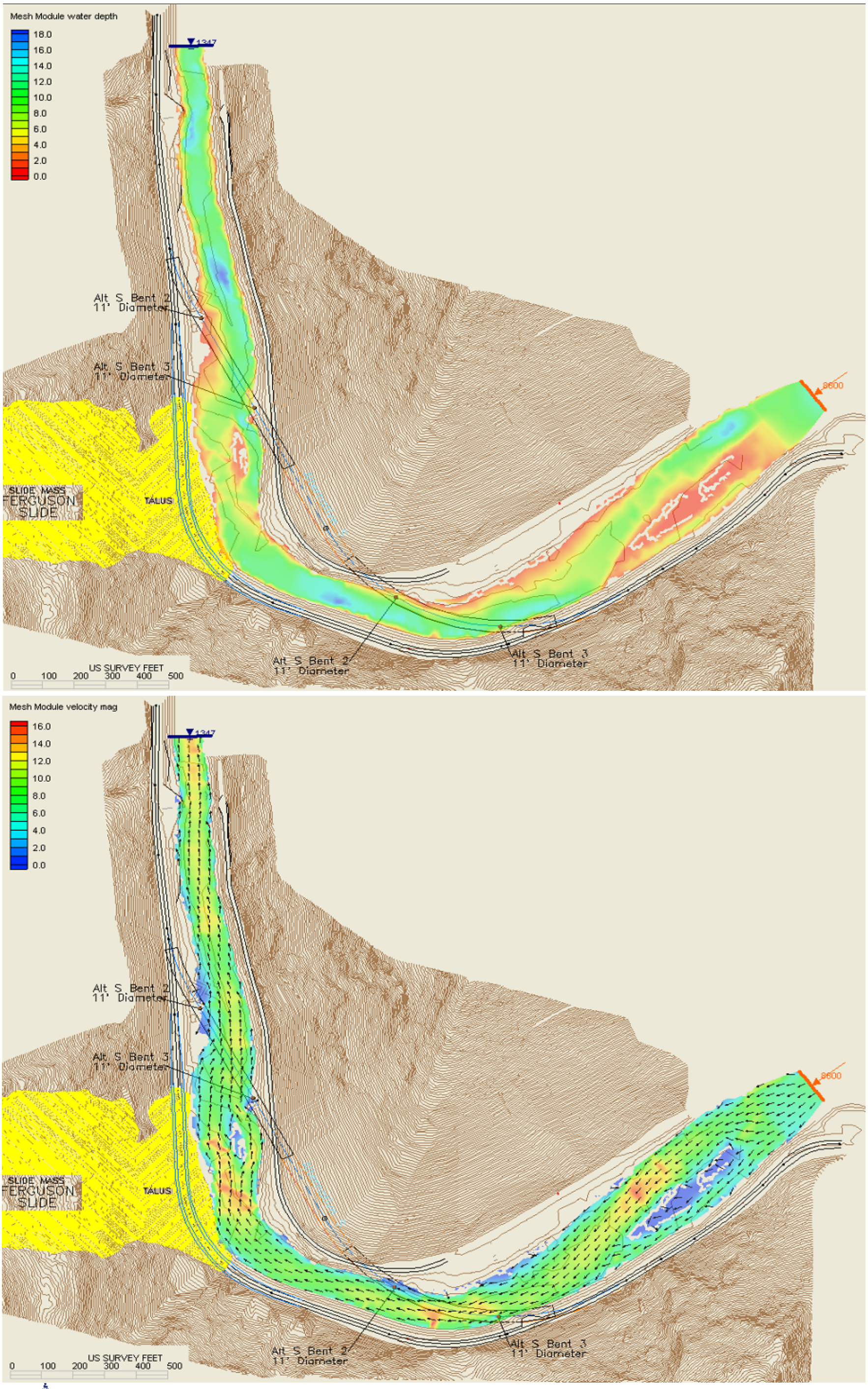
© 2008 Balance Hydrologics, Inc.



Balance Hydrologics, Inc.

Figure 8.19. Depth (upper) and velocity (lower) contours for the temporary bridges proposed during the constructin of Alternative S at the rafting flow (8,800 cfs).

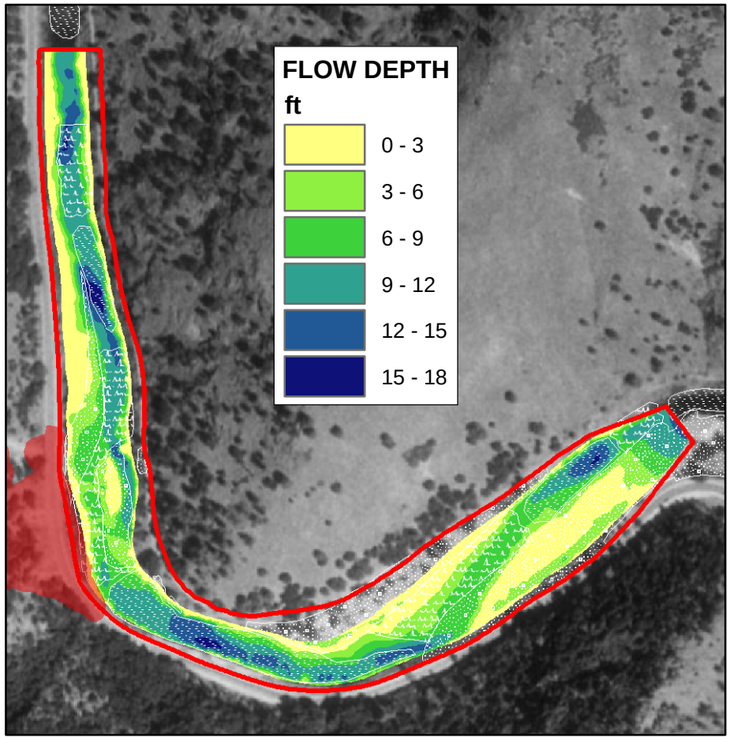
© 2008 Balance Hydrologics, Inc.

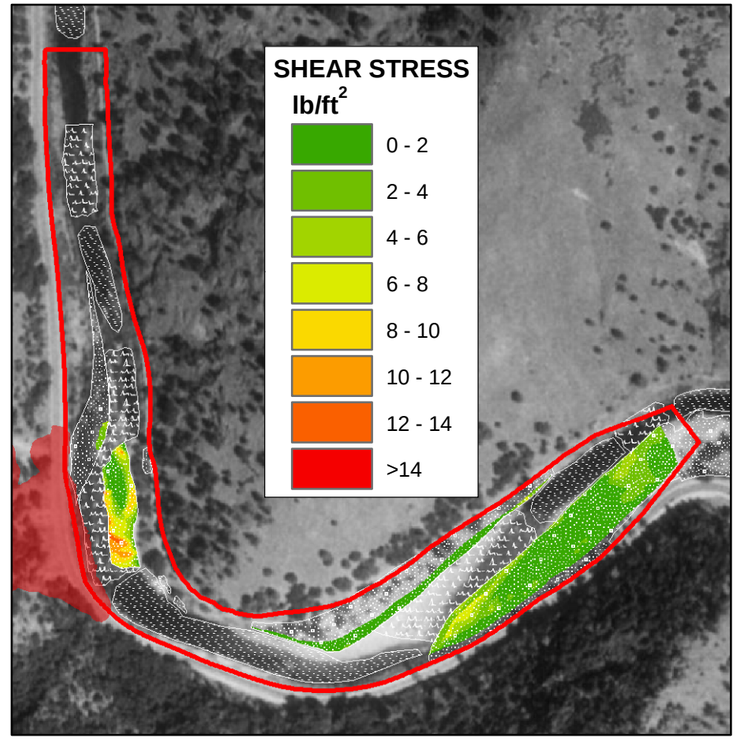


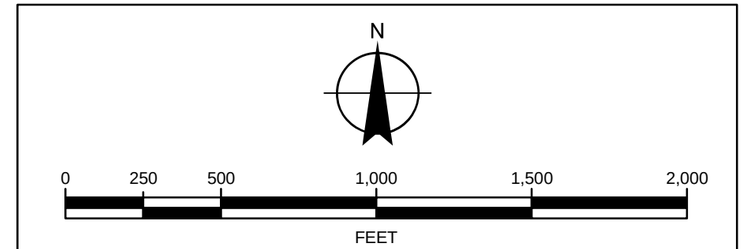
Balance Hydrologics, Inc.

Figure 8.20. Depth (upper) and velocity (lower) contours for the Alternative S at the rafting flow (8,800 cfs).

© 2008 Balance Hydrologics, Inc.







LEGEND

- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

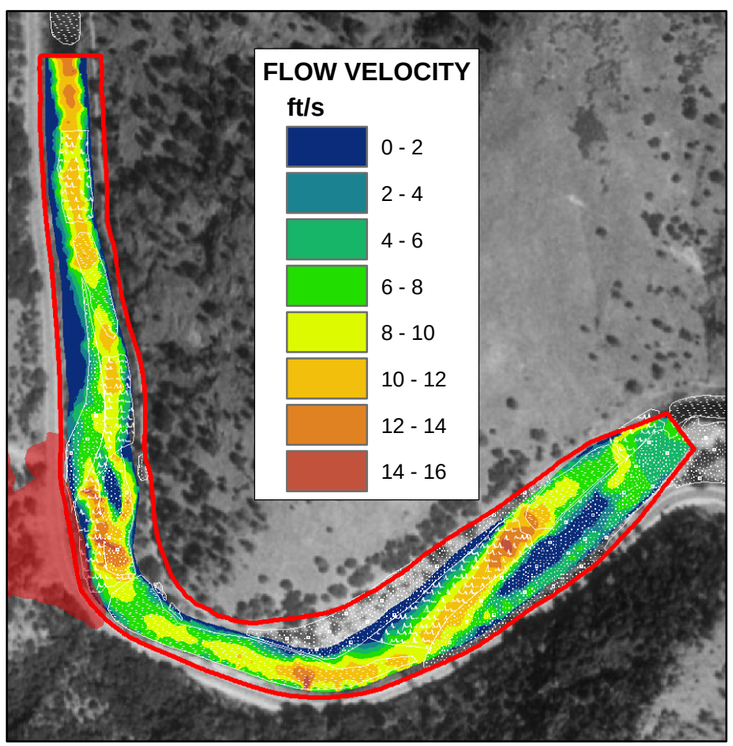
- Bars
- Pool
- Riffle

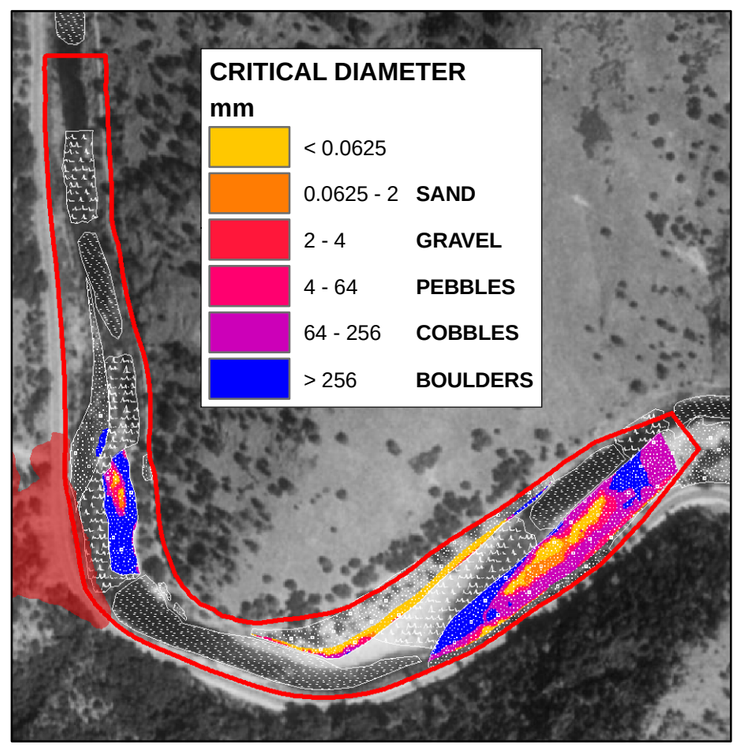
NOTES:

1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed, dependent upon local bed slope and grain size distribution.

3. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





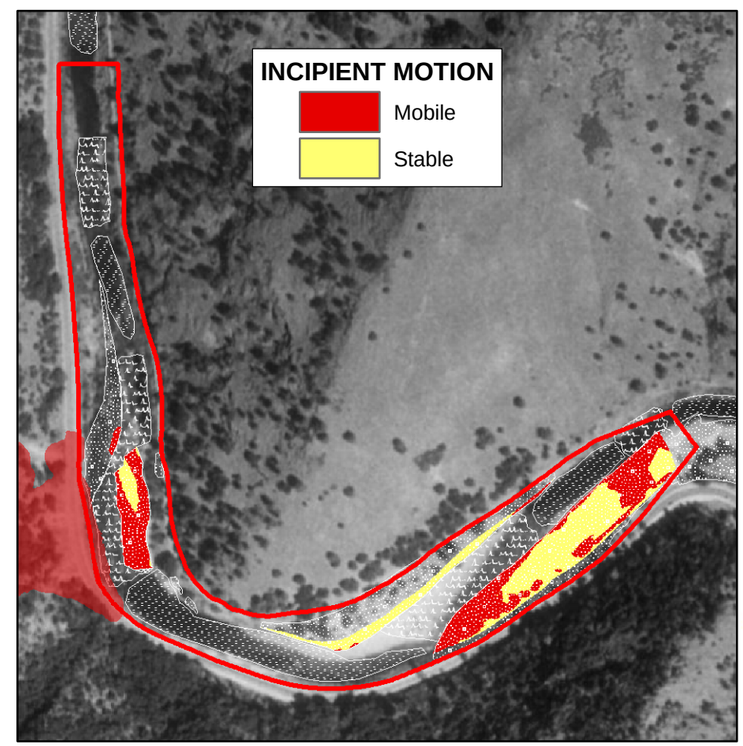
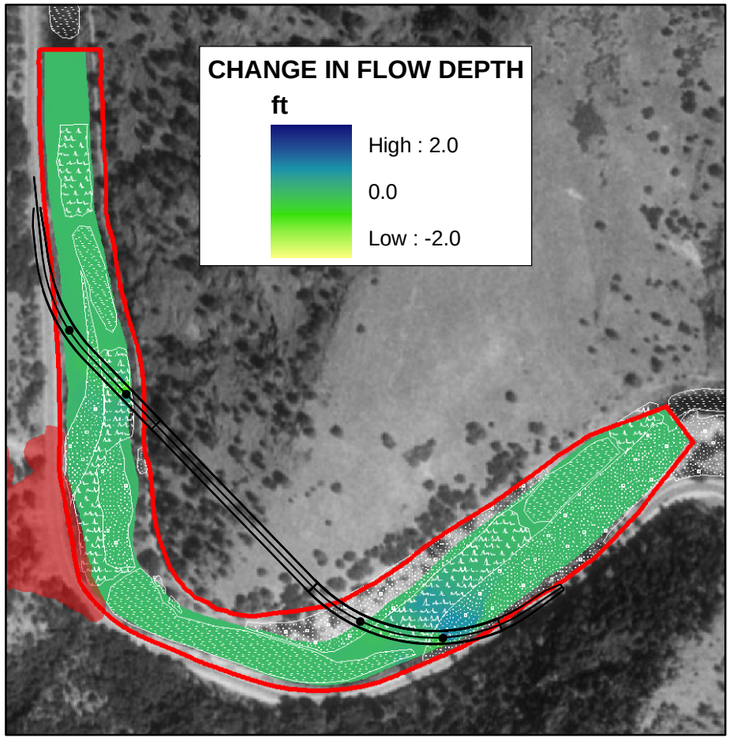


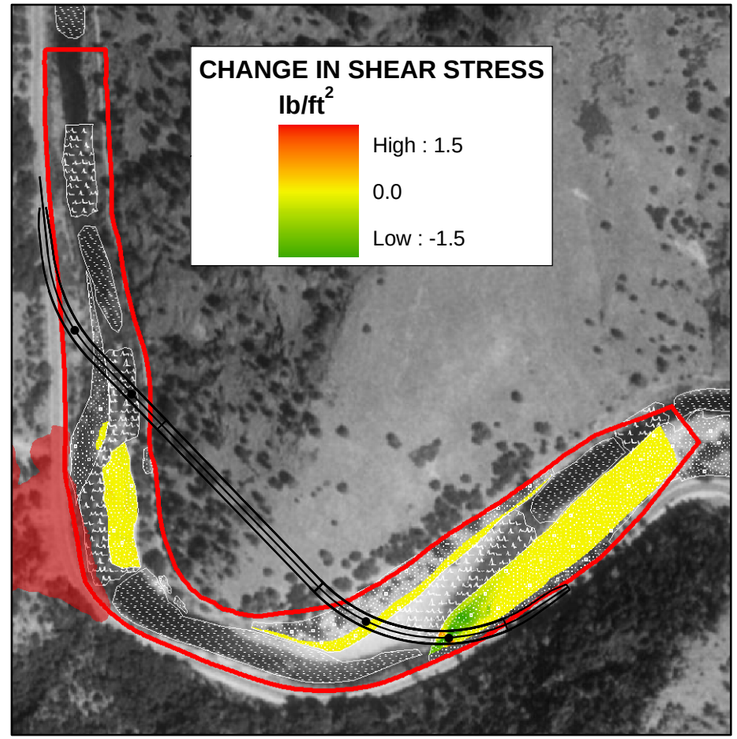
Figure 9.1 Depth, velocity, shear stress, critical sediment diameter, and incipient motion potential under natural conditions at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_NaturalLo_110708.mxd





N

0 250 500 1,000 1,500 2,000

FEET

LEGEND

- Alternative C & T
- Alternative C & T piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

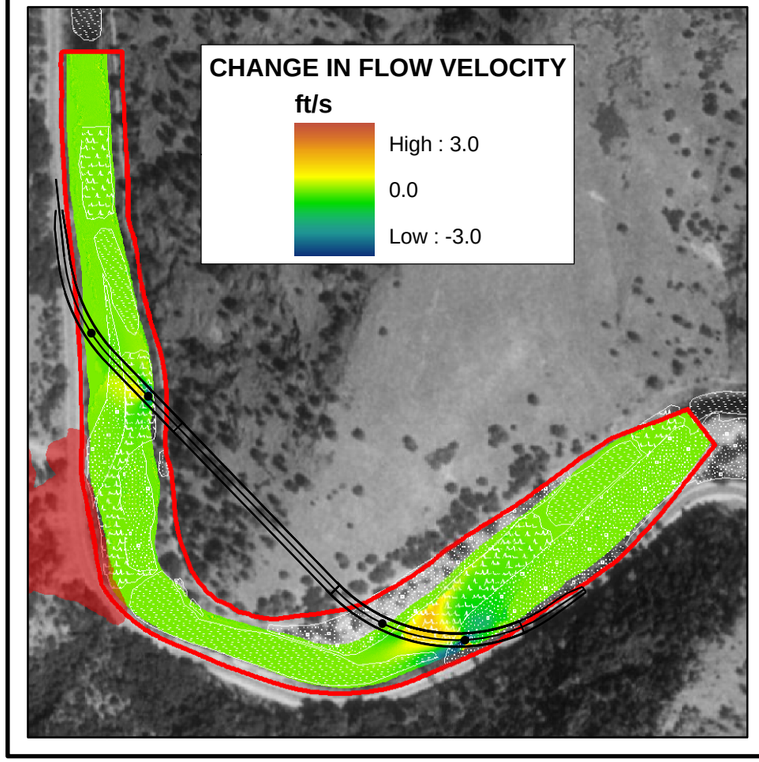
- Bars
- Pool
- Riffle

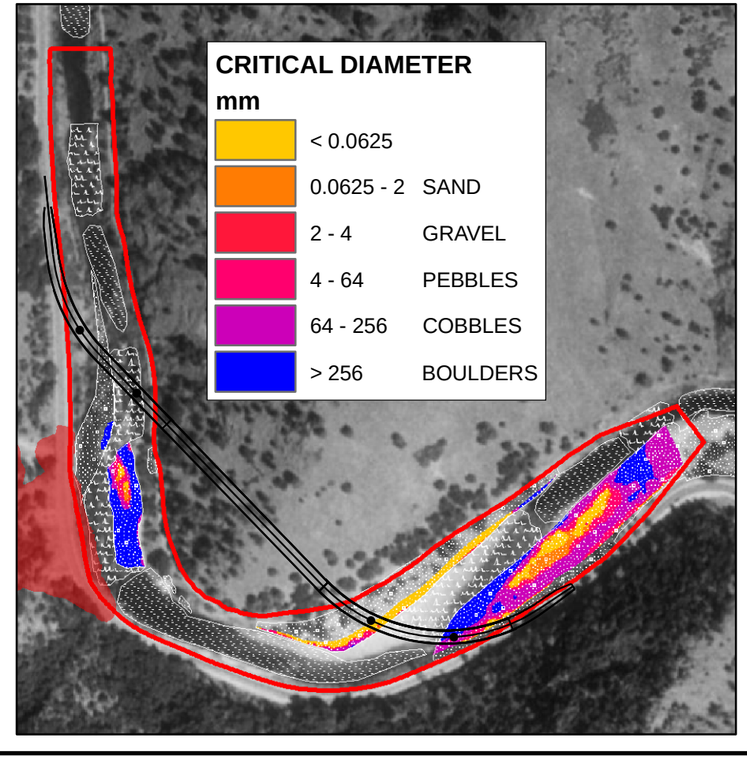
NOTES:

1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





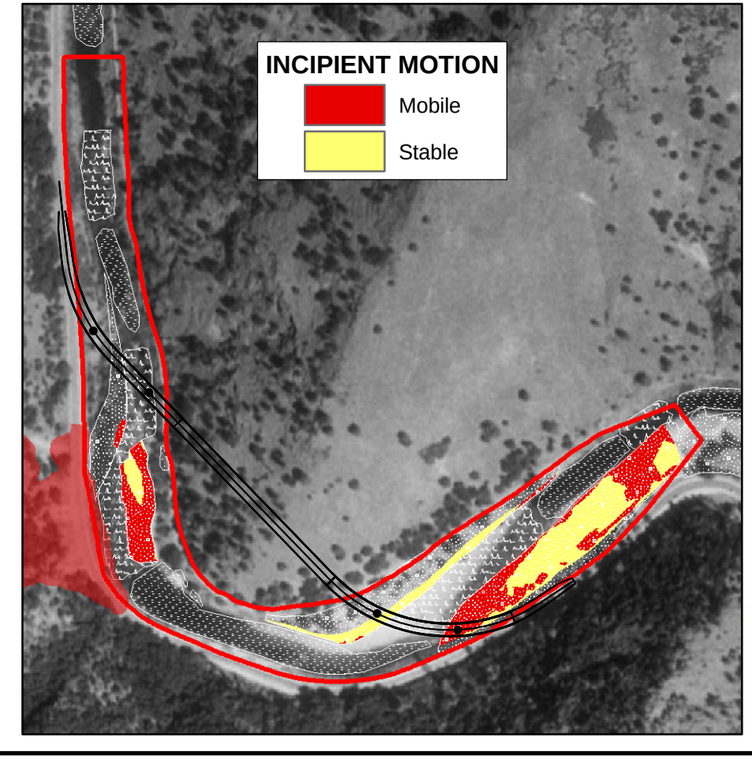
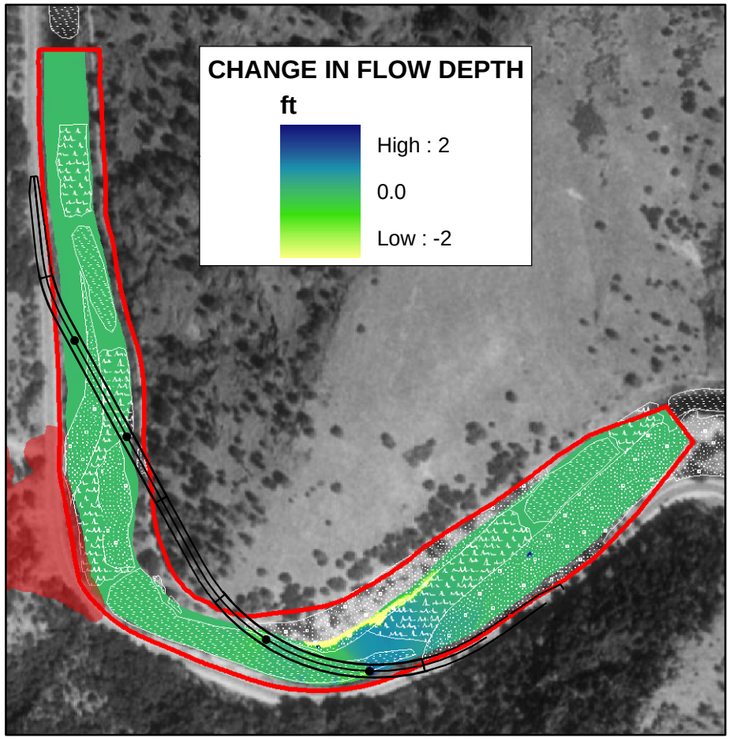


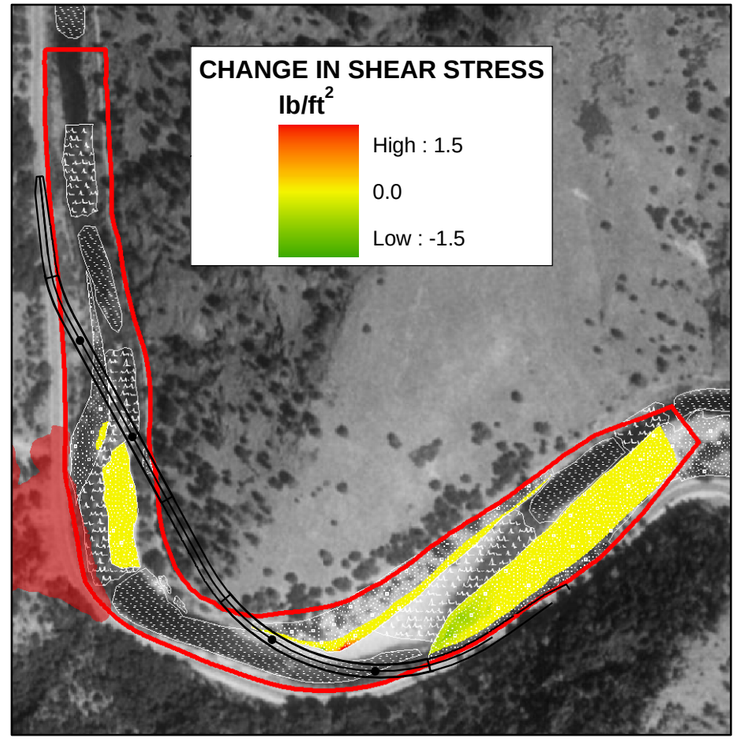
Figure 9.2 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternatives C & T at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltCTLo_010609.mxd





N

0 250 500 1,000 1,500 2,000

FEET

LEGEND

- Alternative S
- Alternative S piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

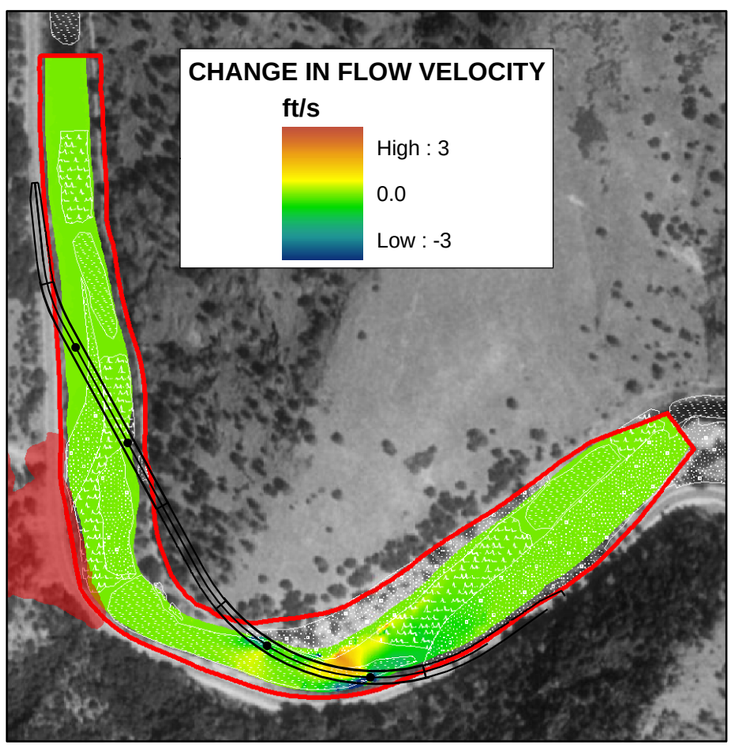
- Bars
- Pool
- Riffle

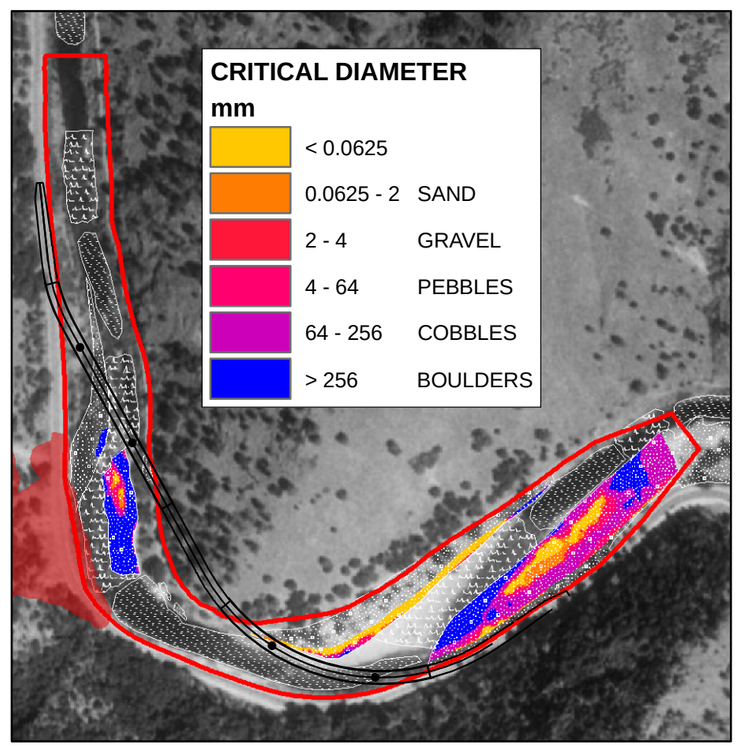
NOTES:

1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent on local bed slope and grain size distribution.

3. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





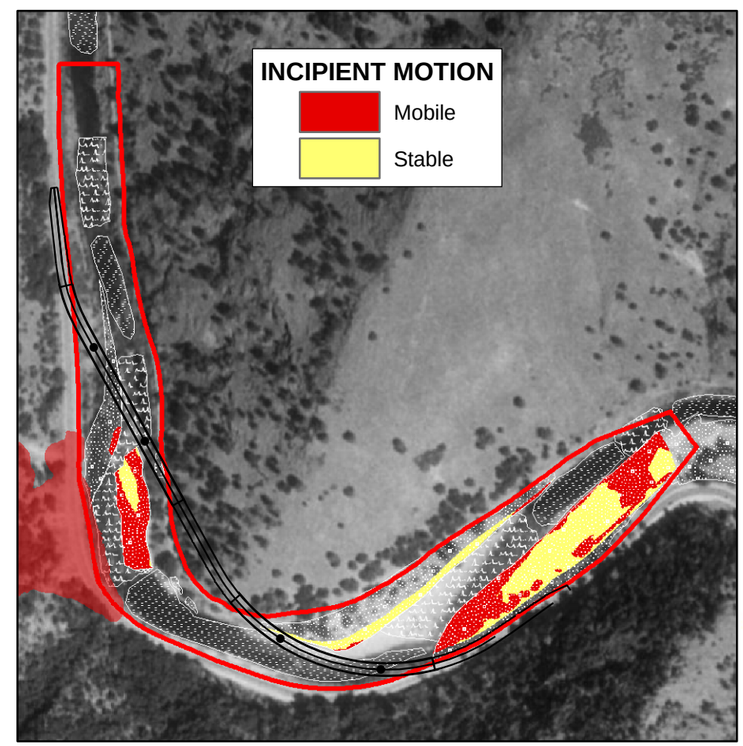
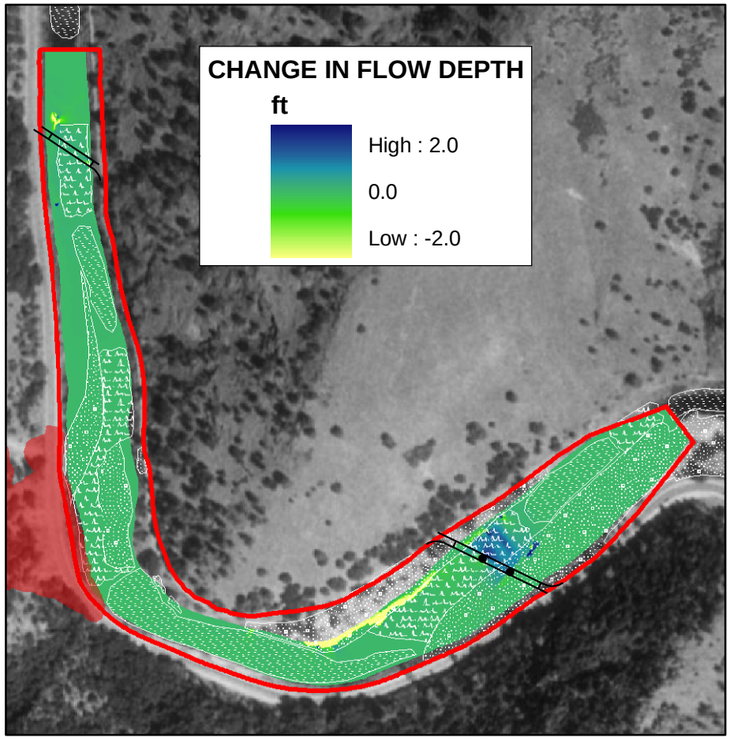


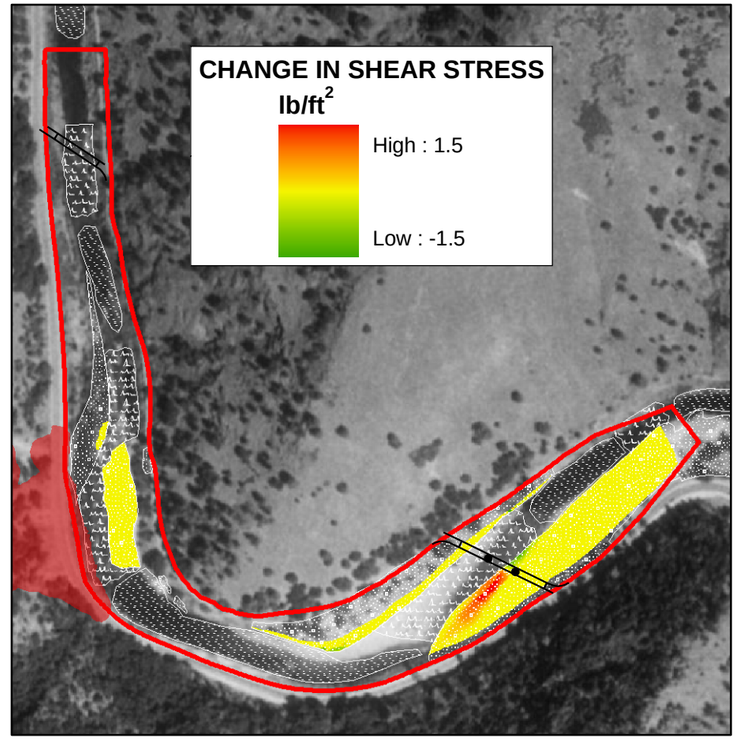
Figure 9.3 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternative S at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California.

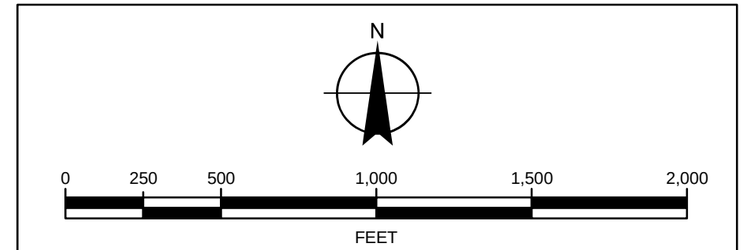
Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltSLo_010709.mxd







LEGEND

- Alternative S Temporary Bridges
- Alternative S Temporary Piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

- Bars
- Pool
- Riffle

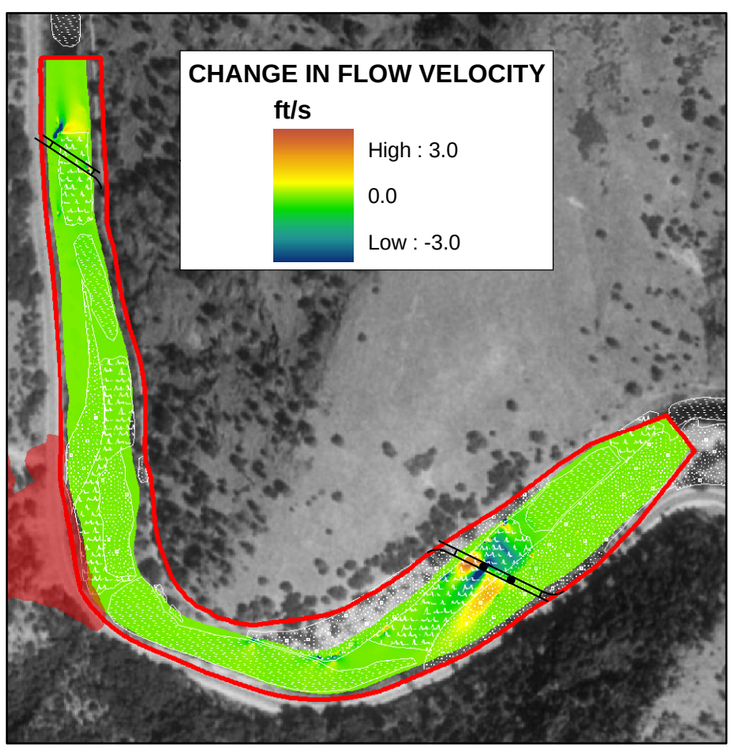
NOTES:

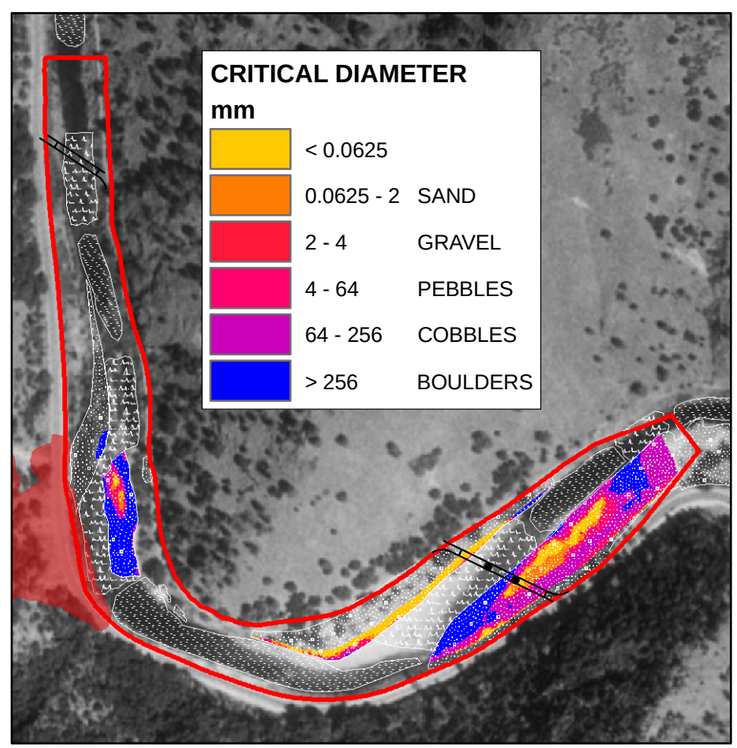
1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. Permanent restoration Alternative S bridges shown for reference only; model results only reflect the effects of the Alternative S temporary bridge.

4. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





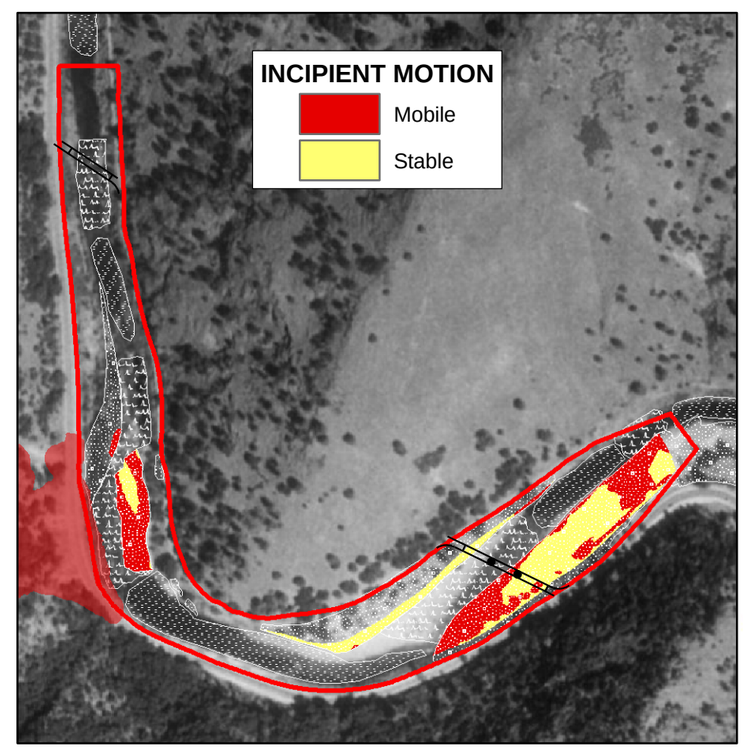
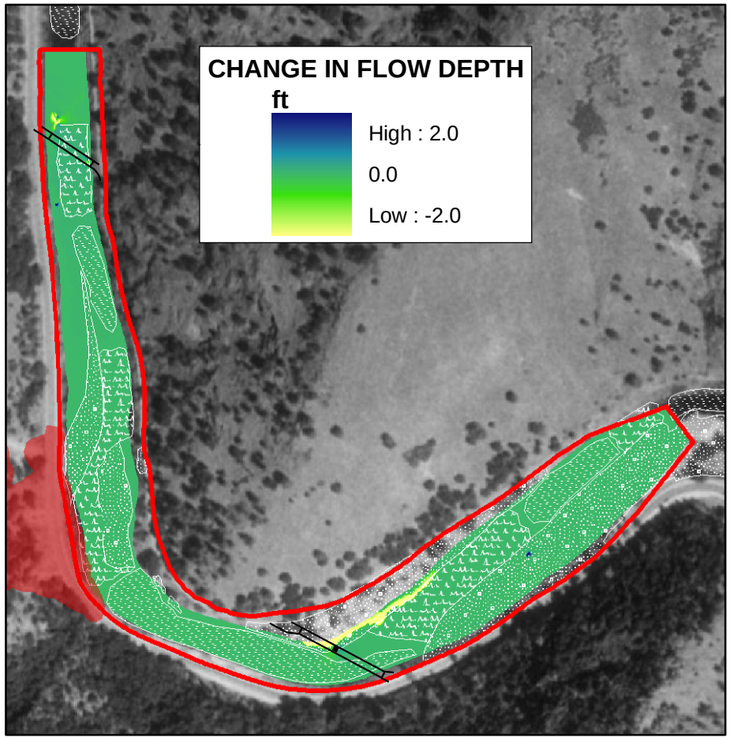


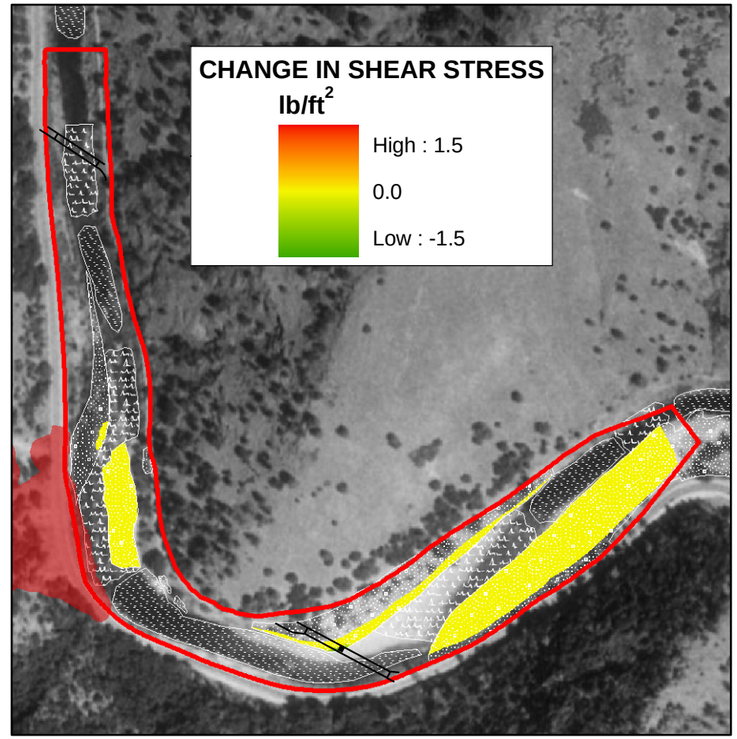
Figure 9.4 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternative S Temporary Bridges at low flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.

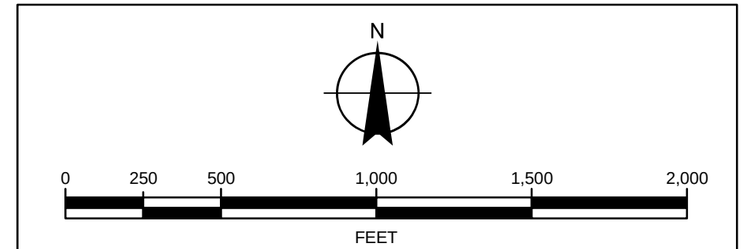
Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltStempLo_010709.mxd







LEGEND

- Temporary Bridges
- Temporary Bridge Pier
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

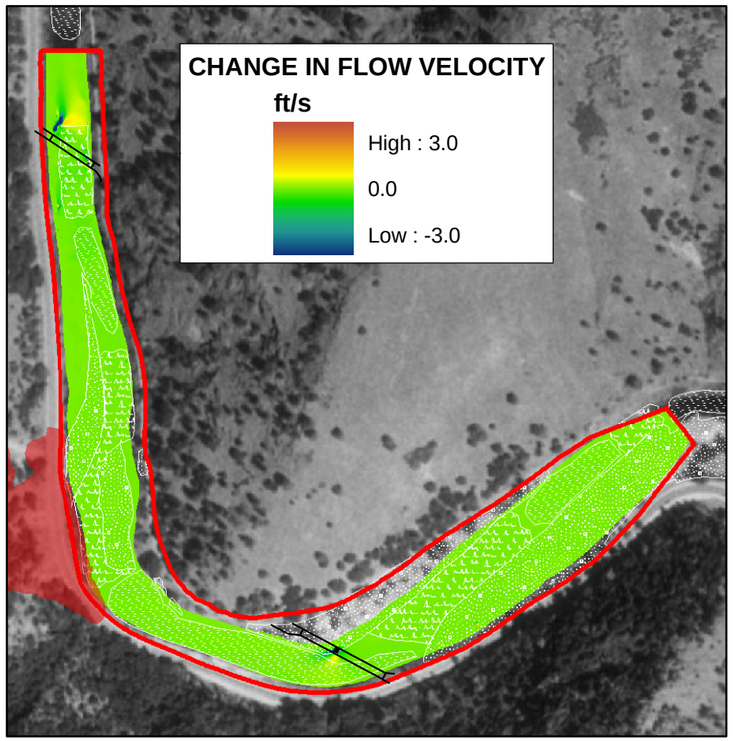
- Bars
- Pool
- Riffle

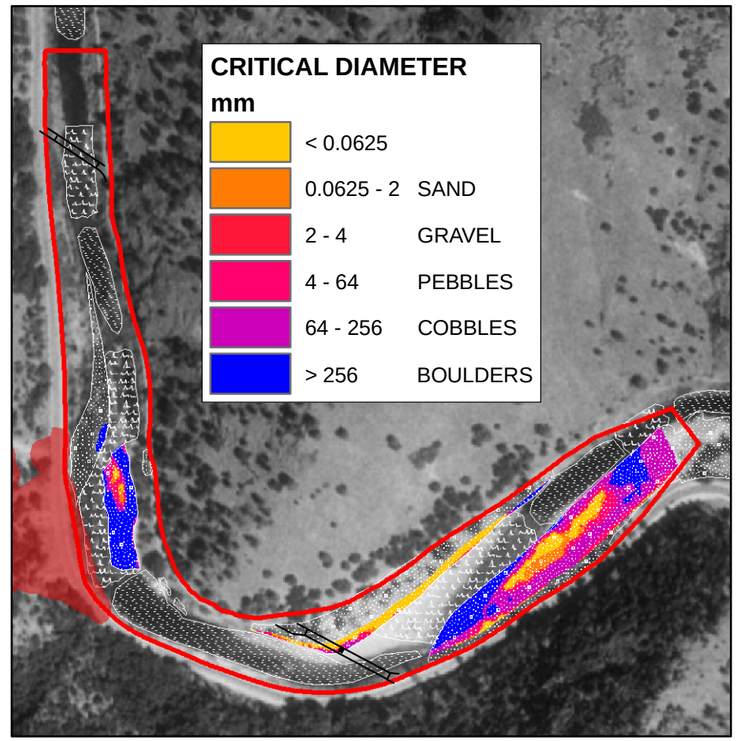
NOTES:

1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





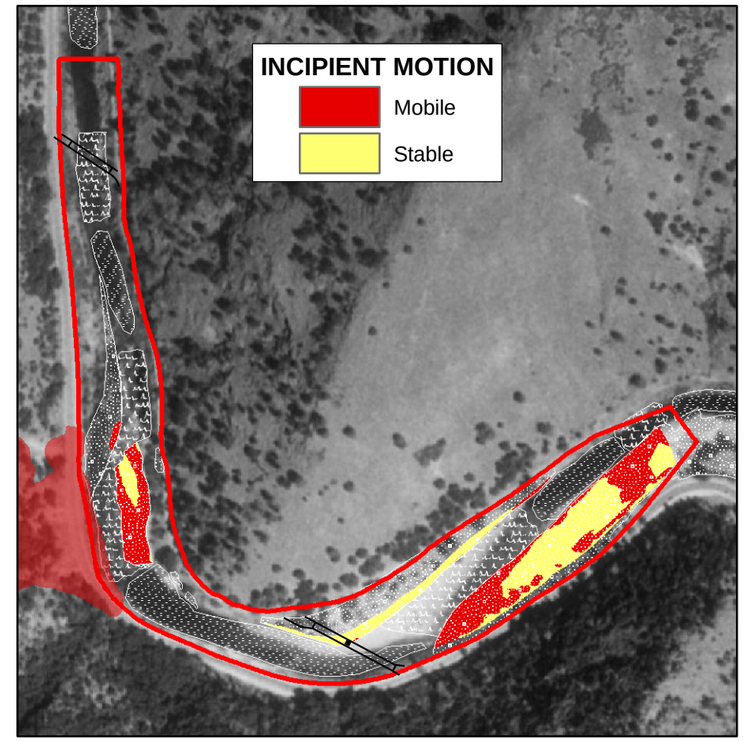
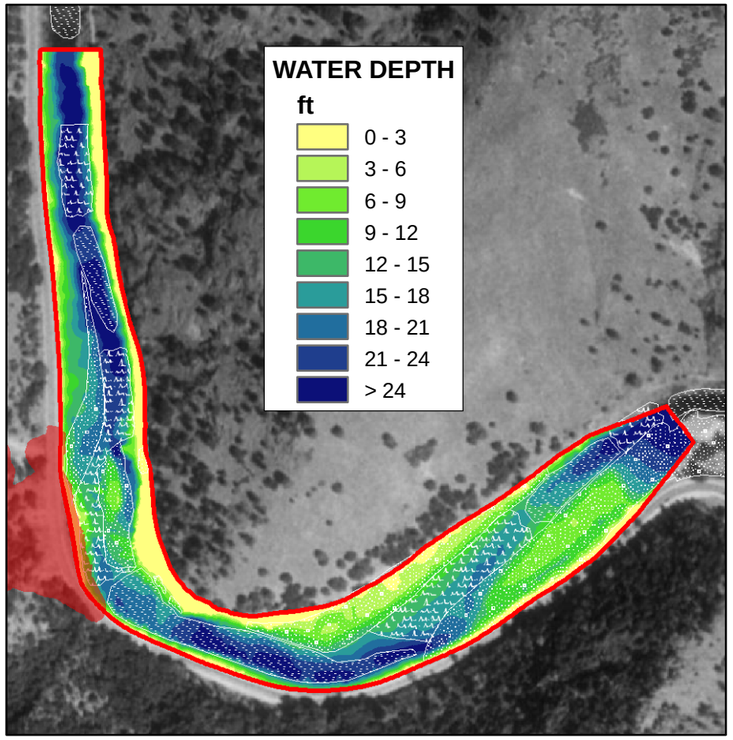


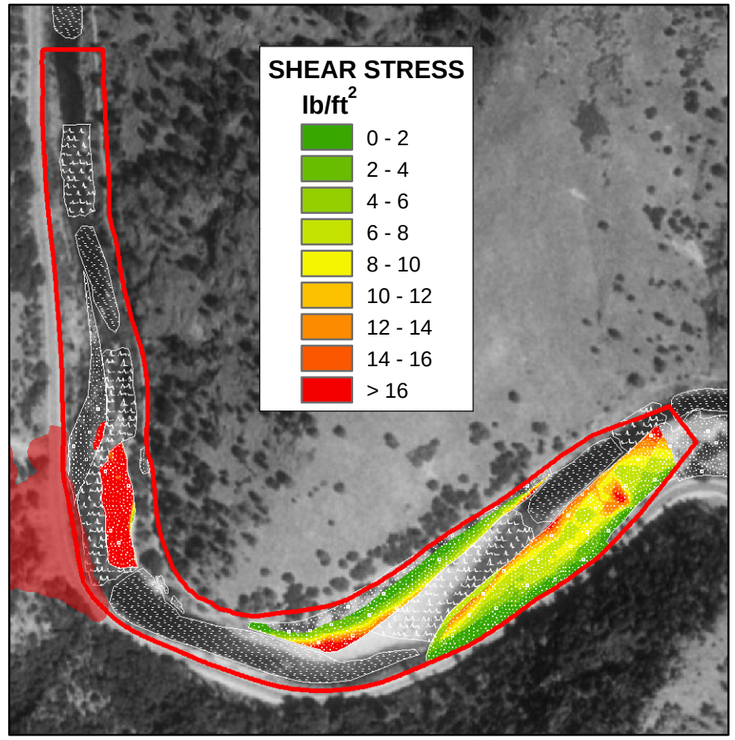
Figure 9.5 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for the No Build Alternative at low flow (8,800 cfs), Merced River at Ferguson Slide, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_NoBuildLo_010709.mxd





N

0 250 500 1,000 1,500 2,000

FEET

LEGEND

Ferguson Rock Slide Extents

2D Model Domain

Geomorphic Features

Bars

Pool

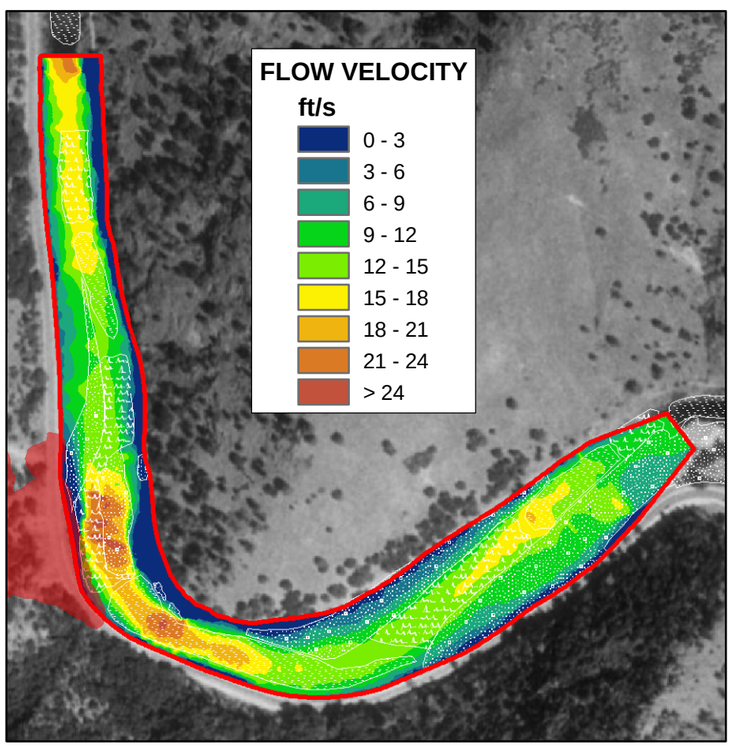
Riffle

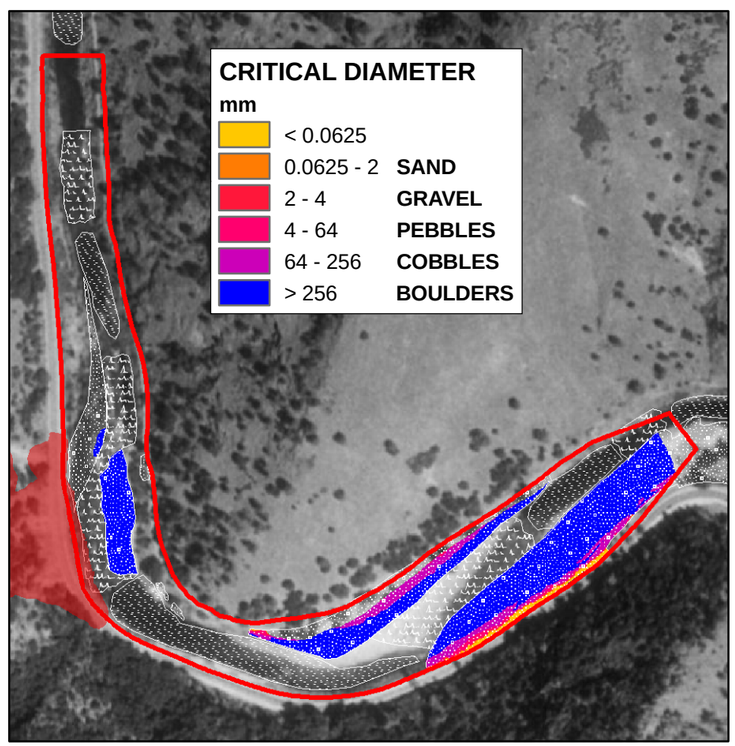
NOTES:

1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





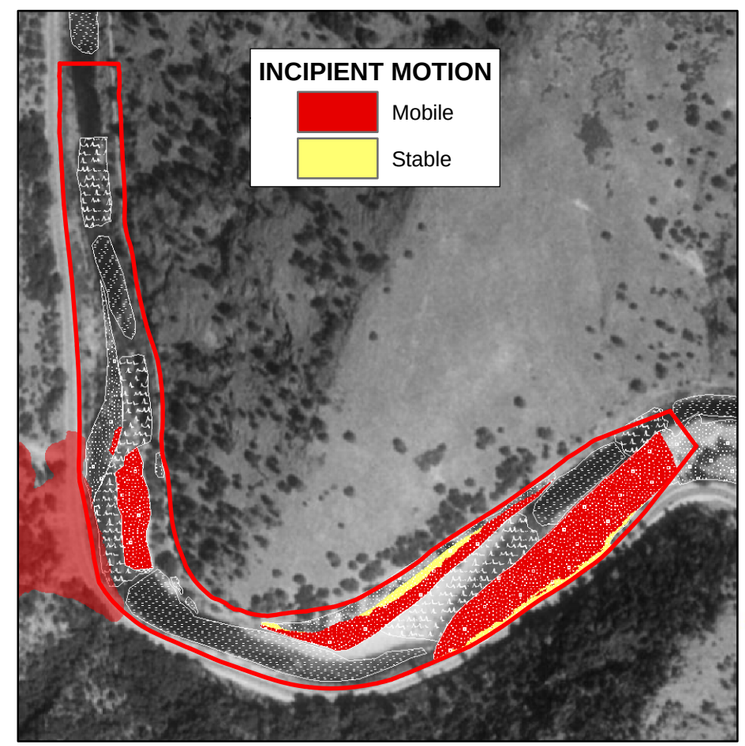
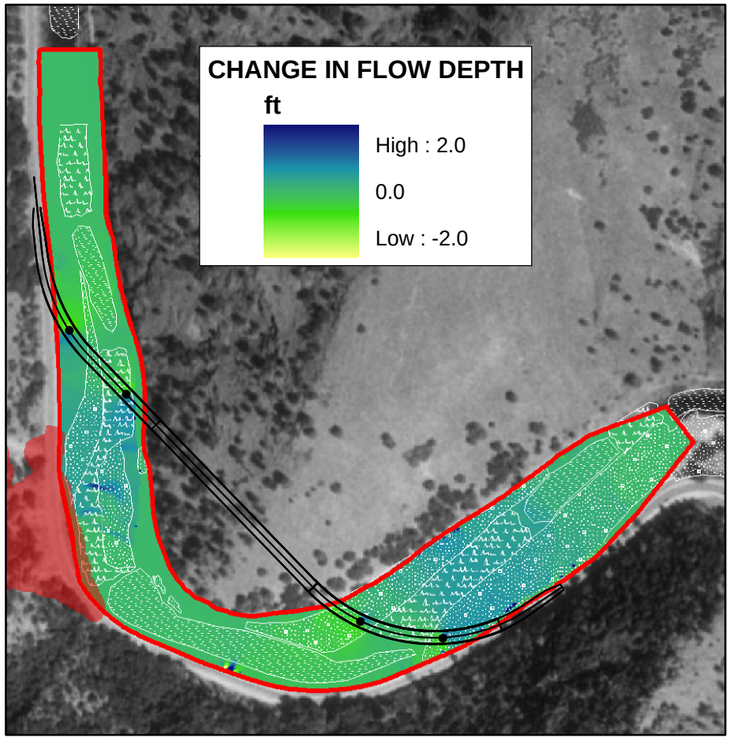


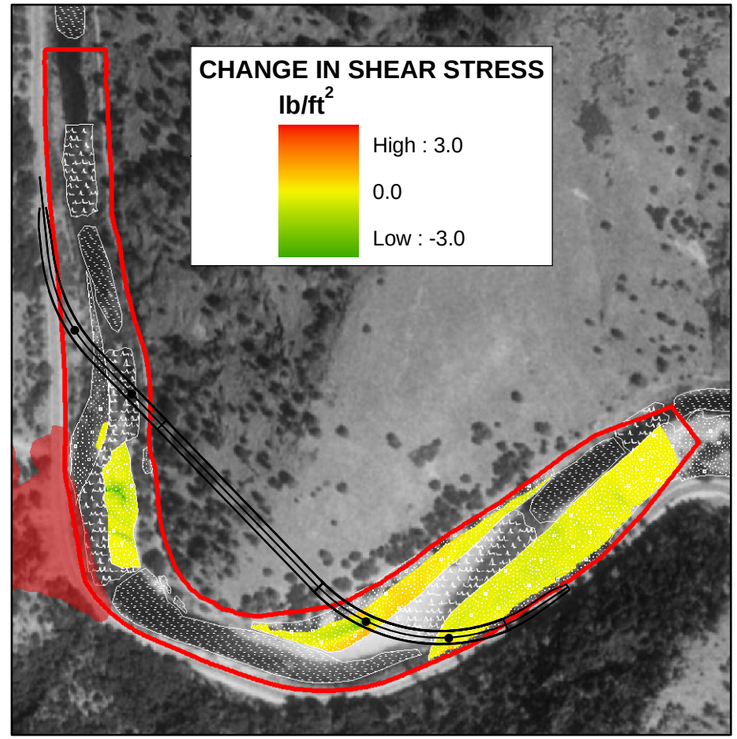
Figure 9.6 **Depth, velocity, shear stress, critical sediment diameter, and incipient motion potential under natural channel conditions at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.**

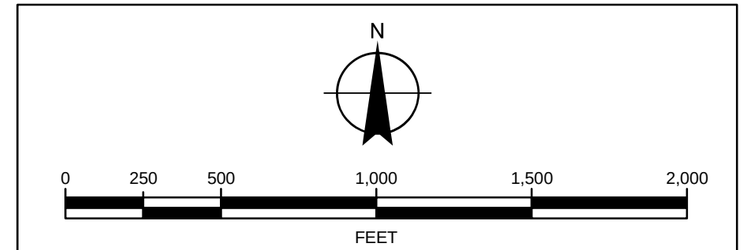
Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_NatConditions_110708.mxd







LEGEND

- Alternative C & T
- Alternative C & T piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

- Bars
- Pool
- Riffle

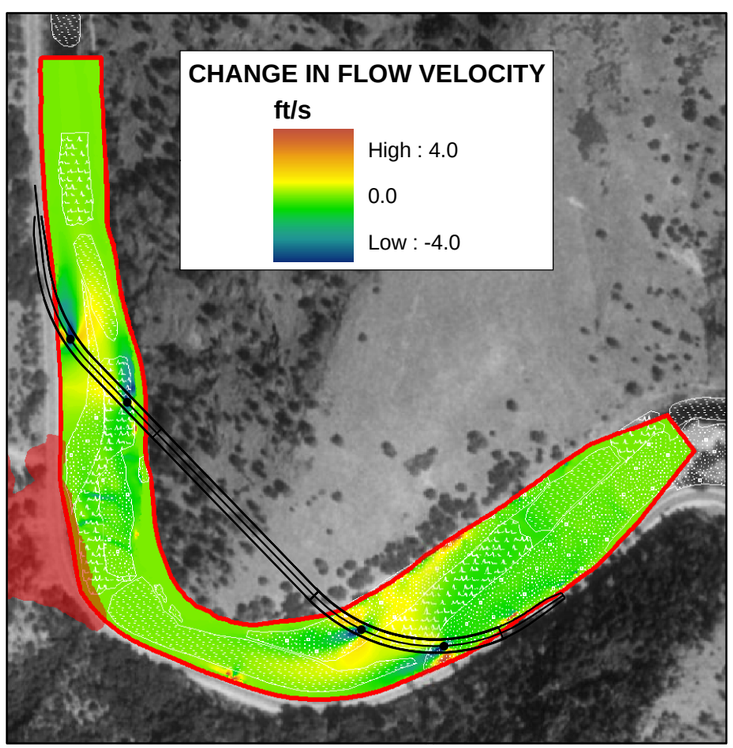
NOTES:

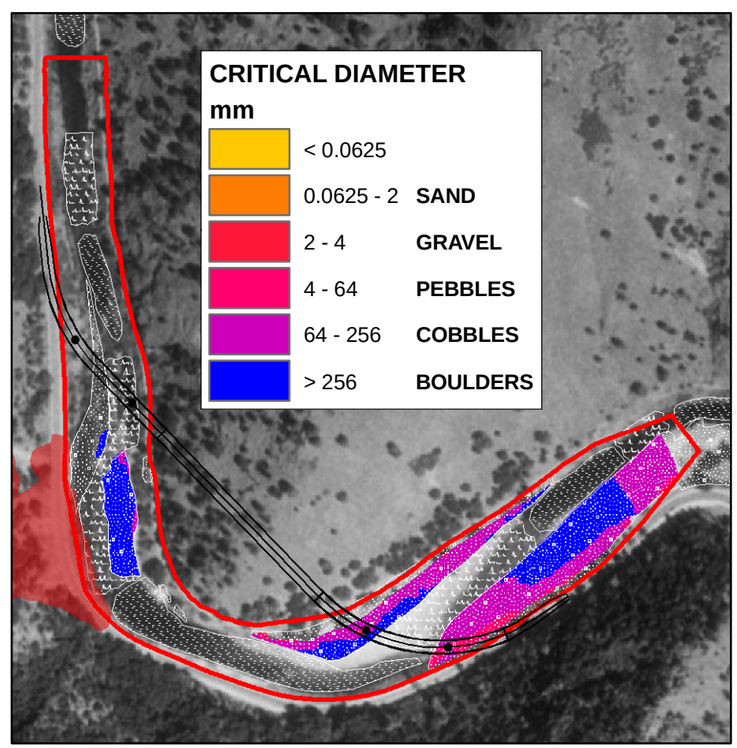
1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed, dependent upon local bed slopes and grain size distribution.

3. See Table 1 for comparisons of discharge during measured historical high flow events.

4. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





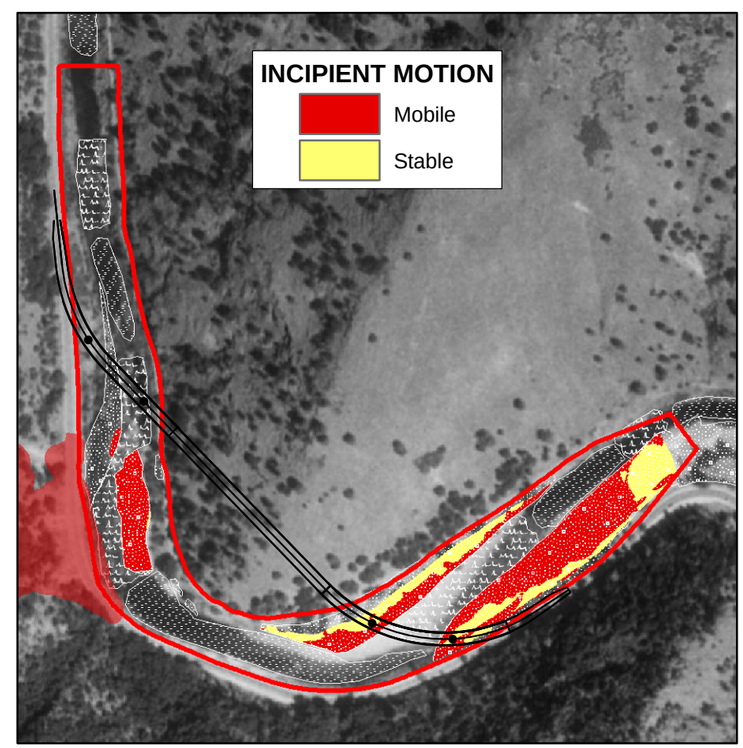
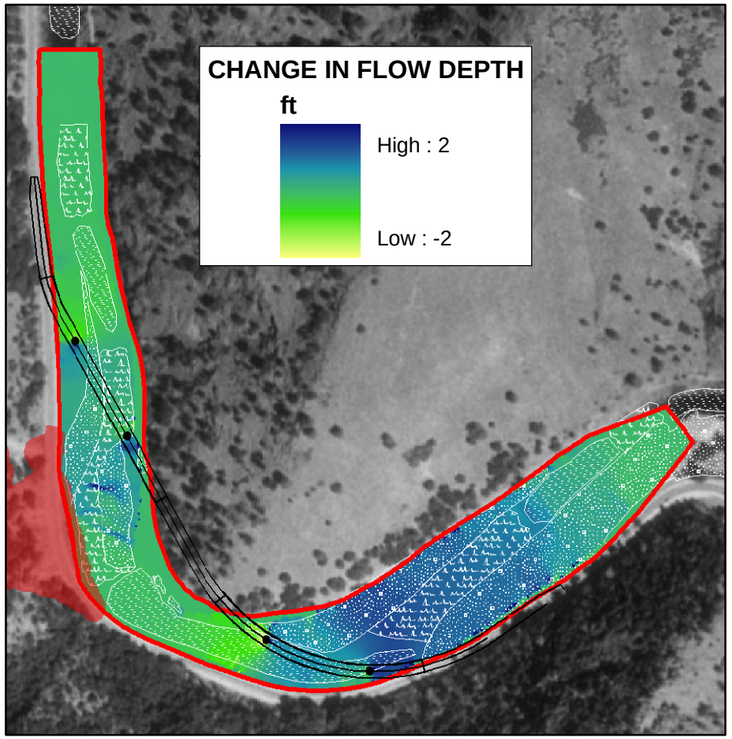


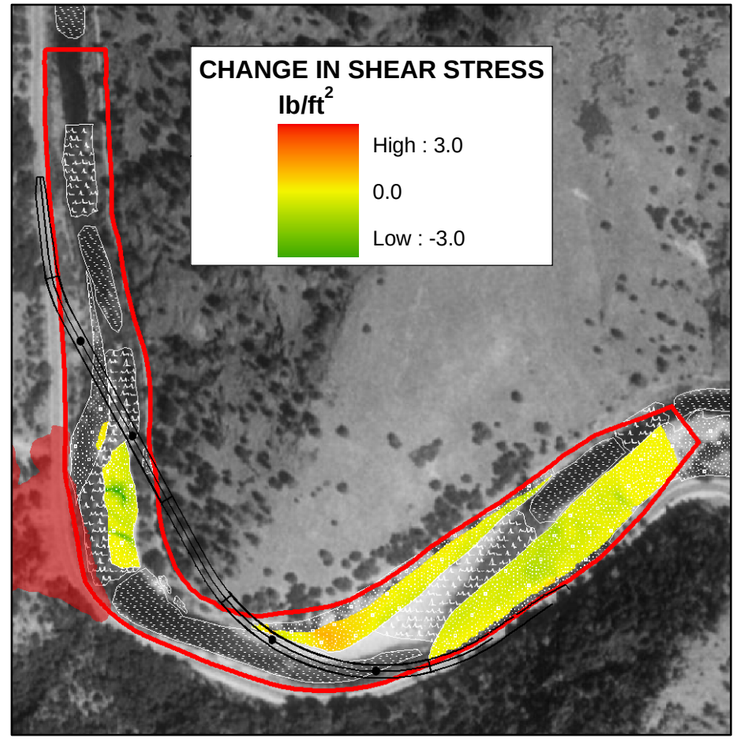
Figure 9.7 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternatives C & T at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltCT_010609.mxd





N

0 250 500 1,000 1,500 2,000

FEET

LEGEND

- Alternative S
- Alternative S piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

- Bars
- Pool
- Riffle

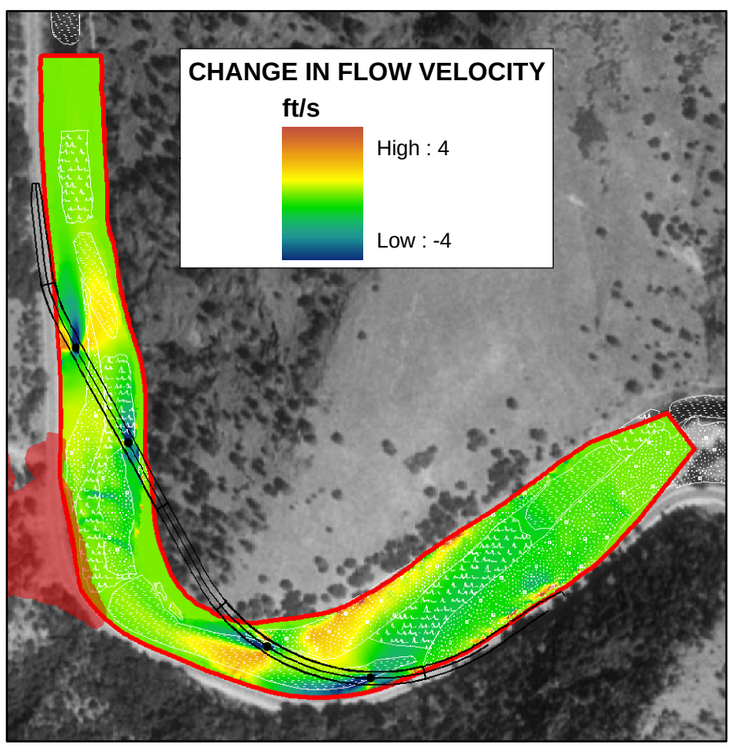
NOTES:

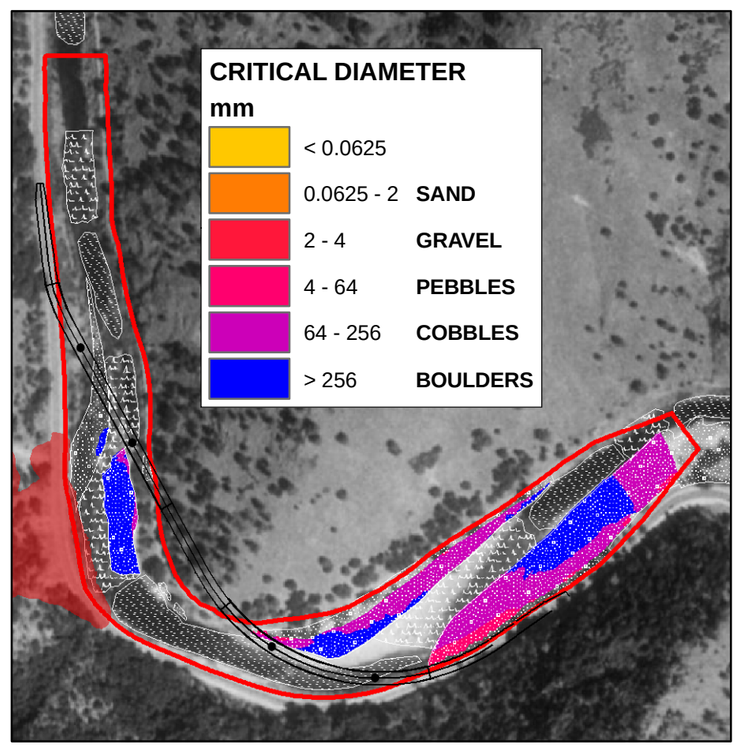
1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. See Table 1 for comparisons of discharge during measured historical high flow events.

4. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





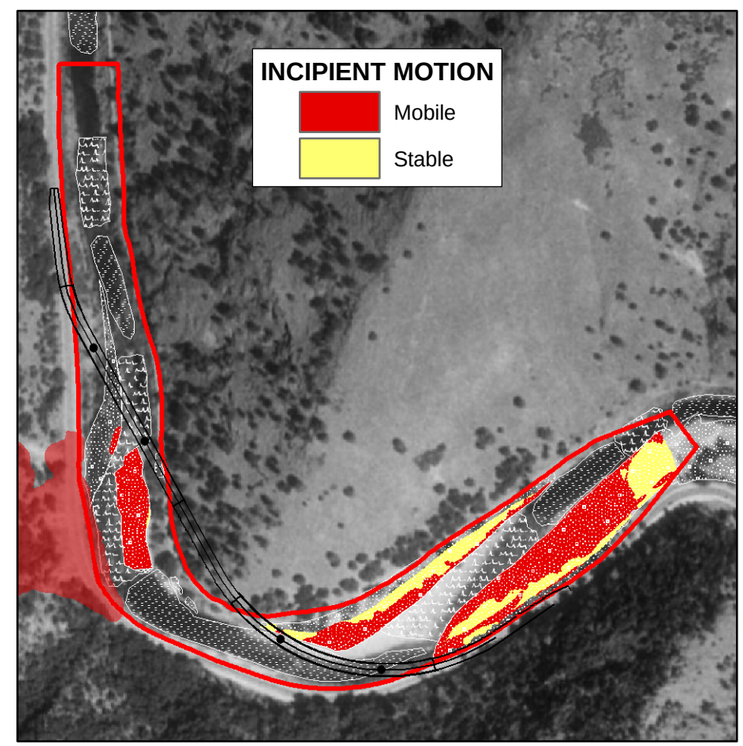
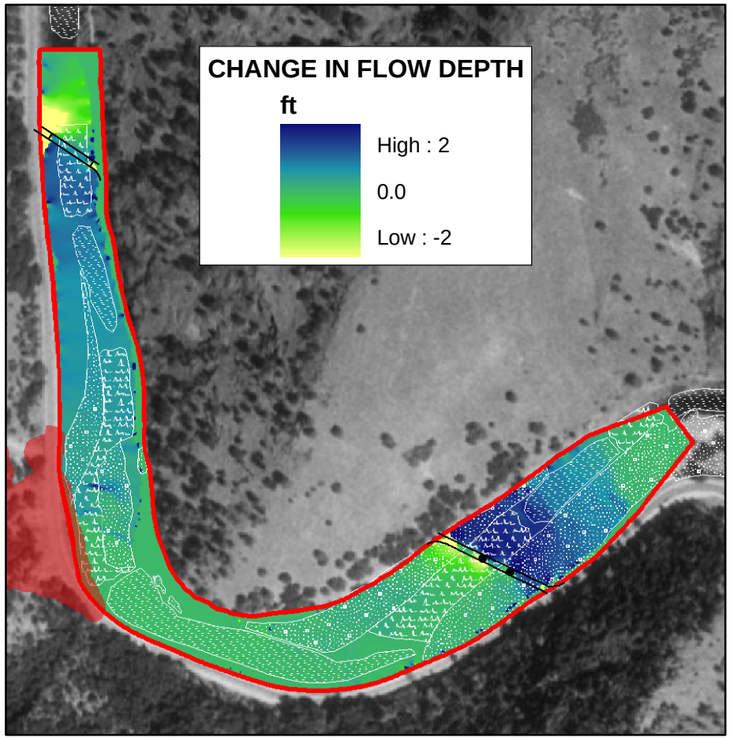


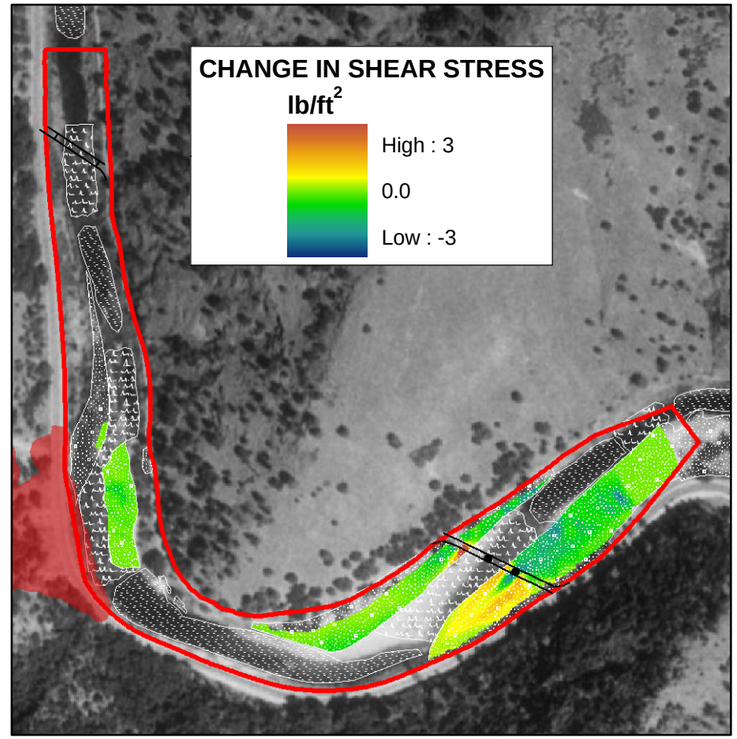
Figure 9.8 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternative S at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.

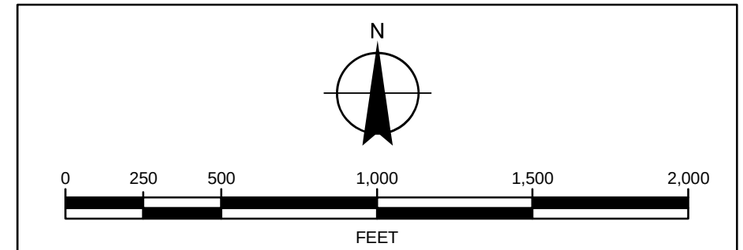
Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltS_010709.mxd







LEGEND

- Alternative S Temporary Bridges
- Alternative S Temporary Piers
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

- Bars
- Pool
- Riffle

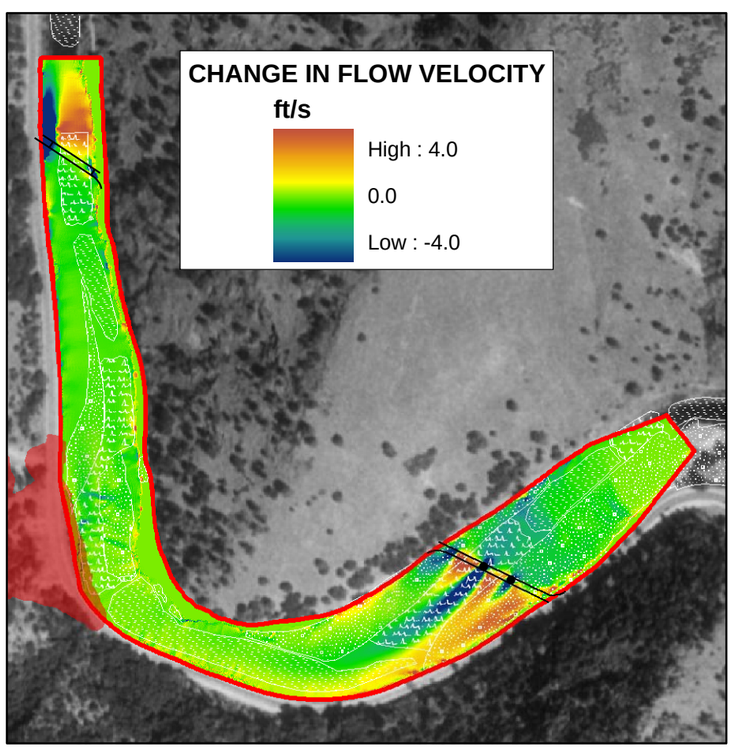
NOTES:

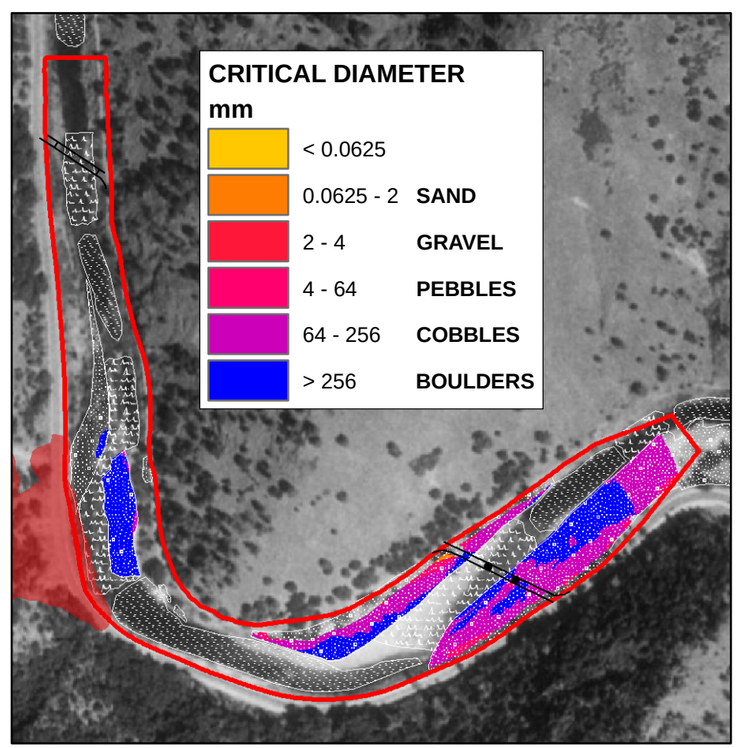
1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. See Table 1 for comparisons of discharge during measured historical high flow events.

4. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





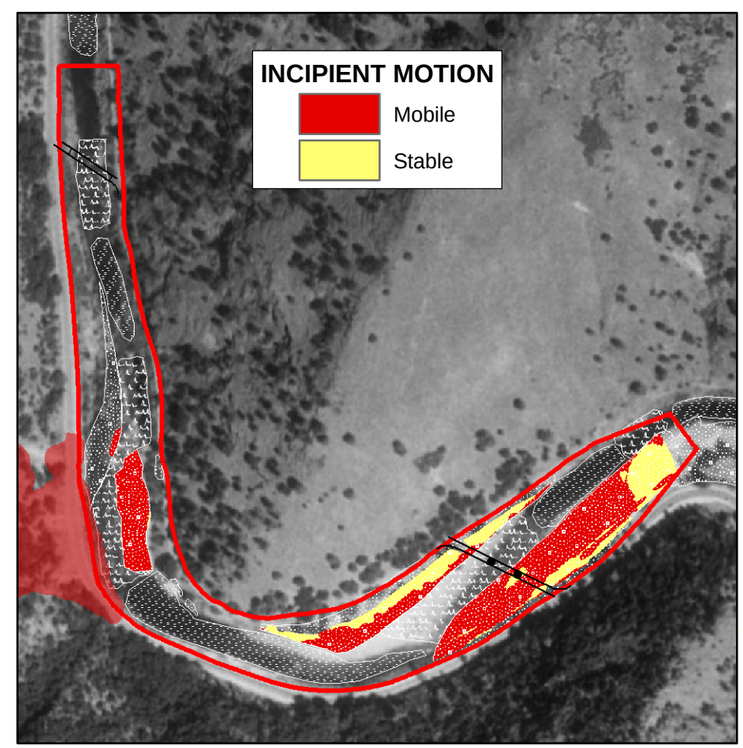
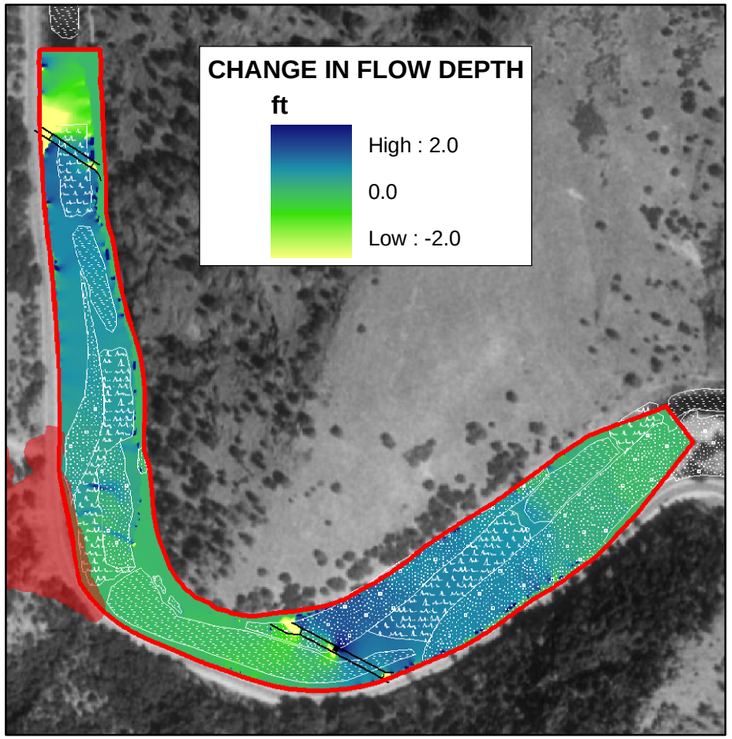


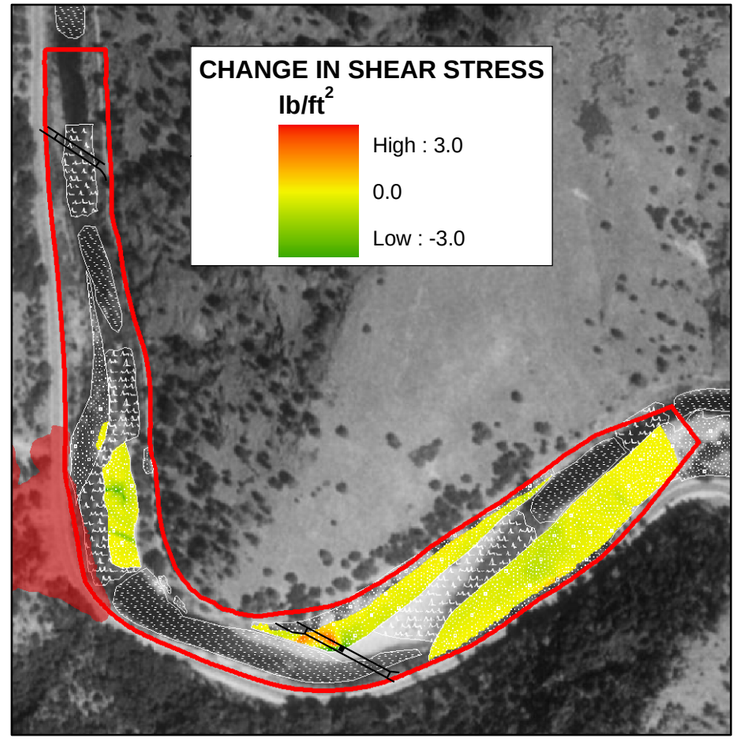
Figure 9.9 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for Alternative S Temporary Bridges at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.

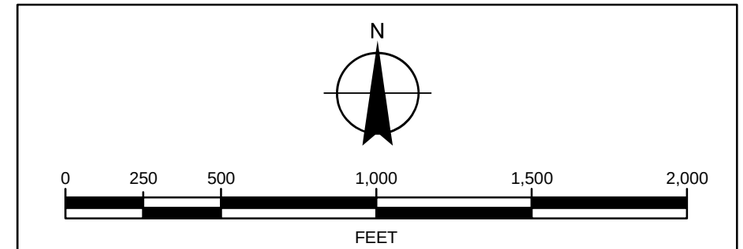
Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_AltStemp_010709.mxd







LEGEND

- Temporary Bridges
- Temporary Bridge Pier
- Ferguson Rock Slide Extents
- 2D Model Domain

Geomorphic Features

- Bars
- Pool
- Riffle

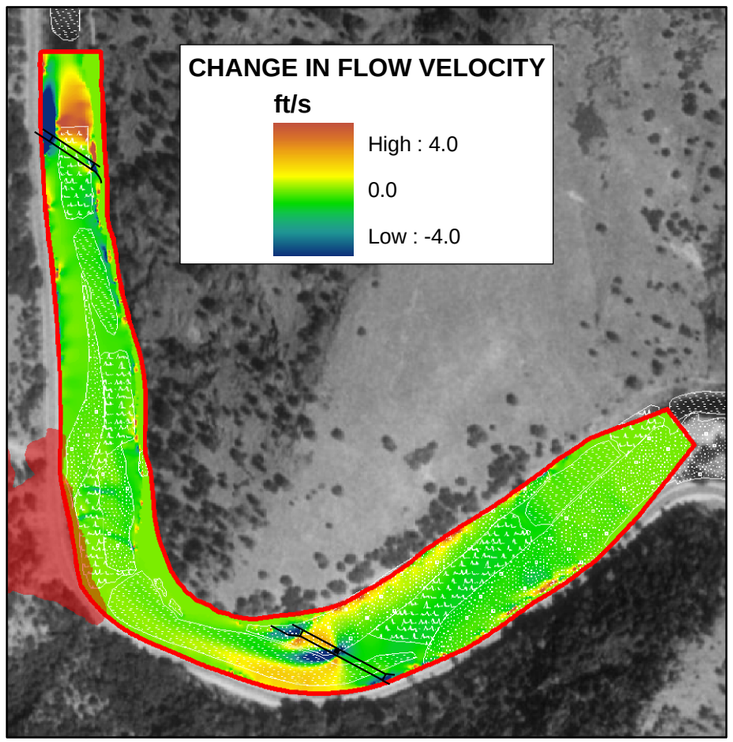
NOTES:

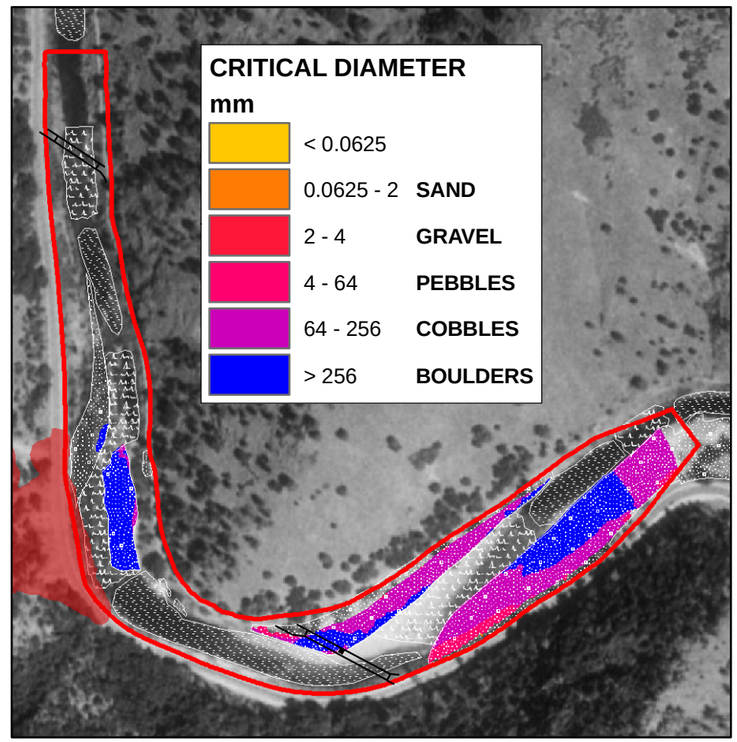
1. Depictions of shear stress, critical diameter, and incipient motion that are restricted to channel margin bars reflects grain size measurements performed on exposed surfaces in the field. Grain size measurements were not possible on the bed due to flow conditions.

2. Where calculations are absent on the channel bed, sediment transport behavior is likely to be similar to adjacent bars where calculations have been performed dependent upon local bed slope and grain size distribution.

3. See Table 1 for comparisons of discharge during measured historical high flow events.

4. Values predicted with results from 2D hydrodynamic modeling using FESWMS-2D.





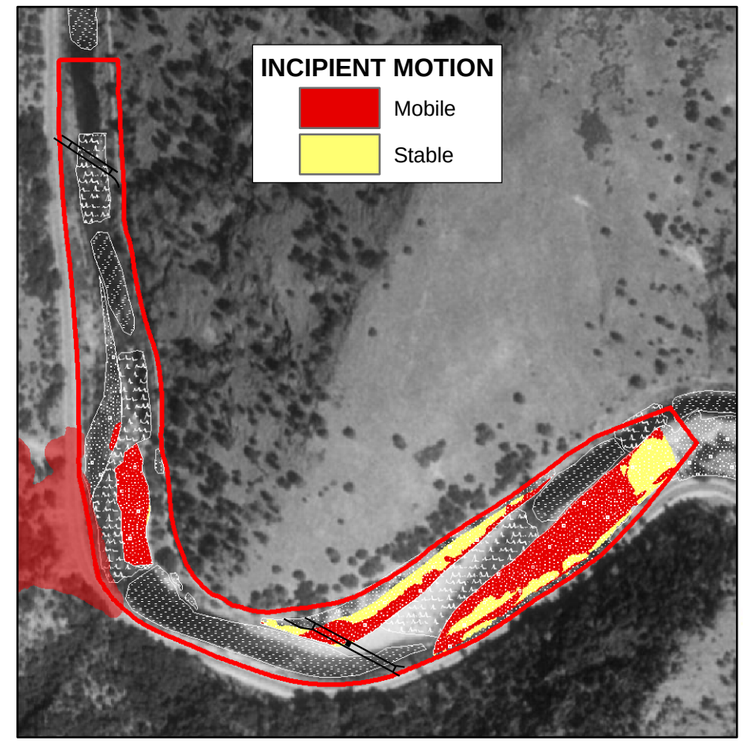
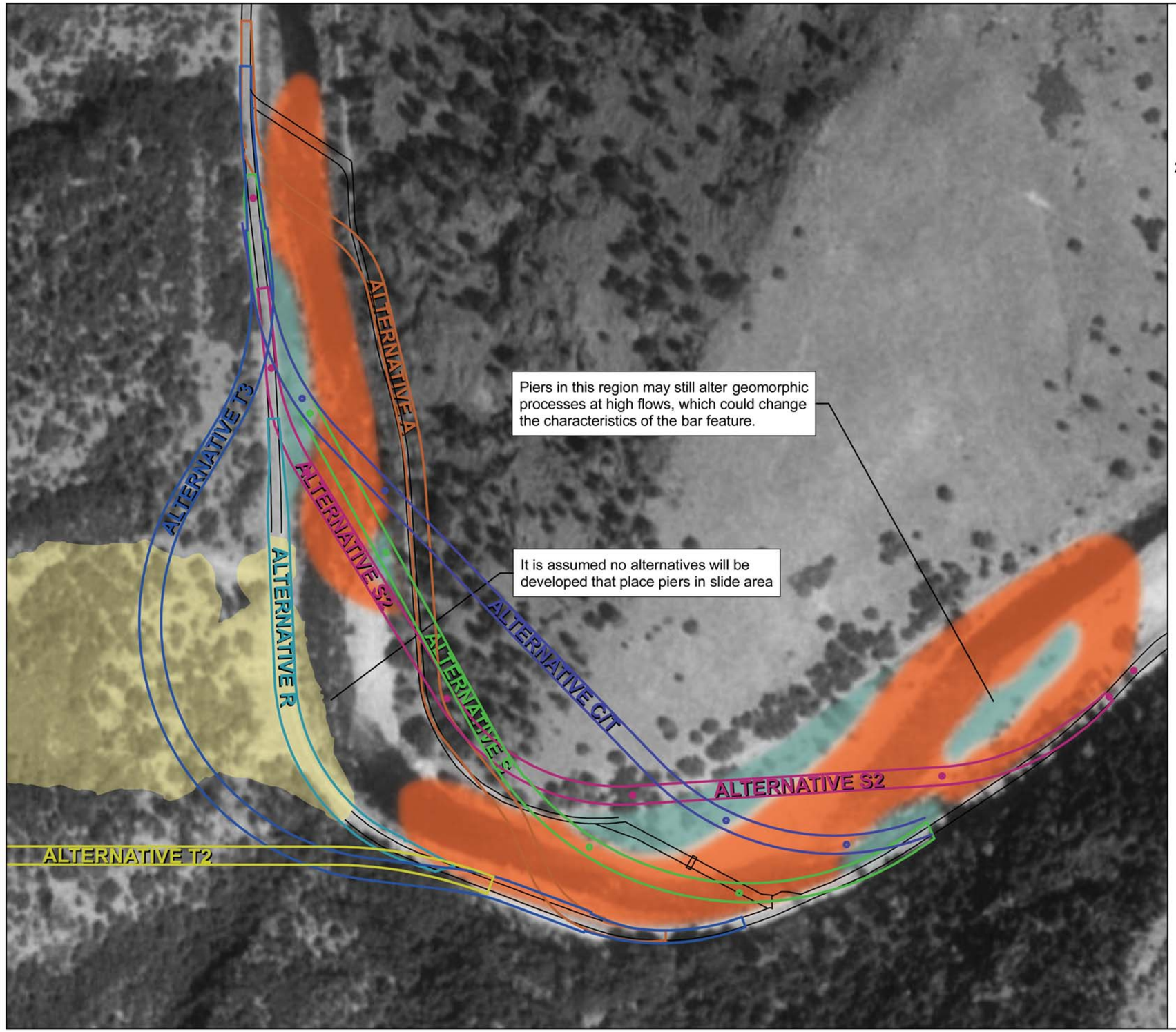


Figure 9.10 Contour plots showing the depth, velocity and shear stress relative to natural conditions; critical sediment diameter; and incipient motion for the No Build Alternative at high flow (42,000 cfs), Merced River at Ferguson Slide, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_NoBuild_010709.mxd



N

250 125 0 250 500

FEET

LEGEND

Suggested Pier Feasibility Zones

- Preferred
- Not recommended

Ferguson Rock Slide Extents

Caltrans Transportation Alternatives

- Alternative A
- Alternatives C and T
- Alternative S
- Alternative R
- Alternative S2
- Alternative T2
- Alternative T3
- Existing Alignment

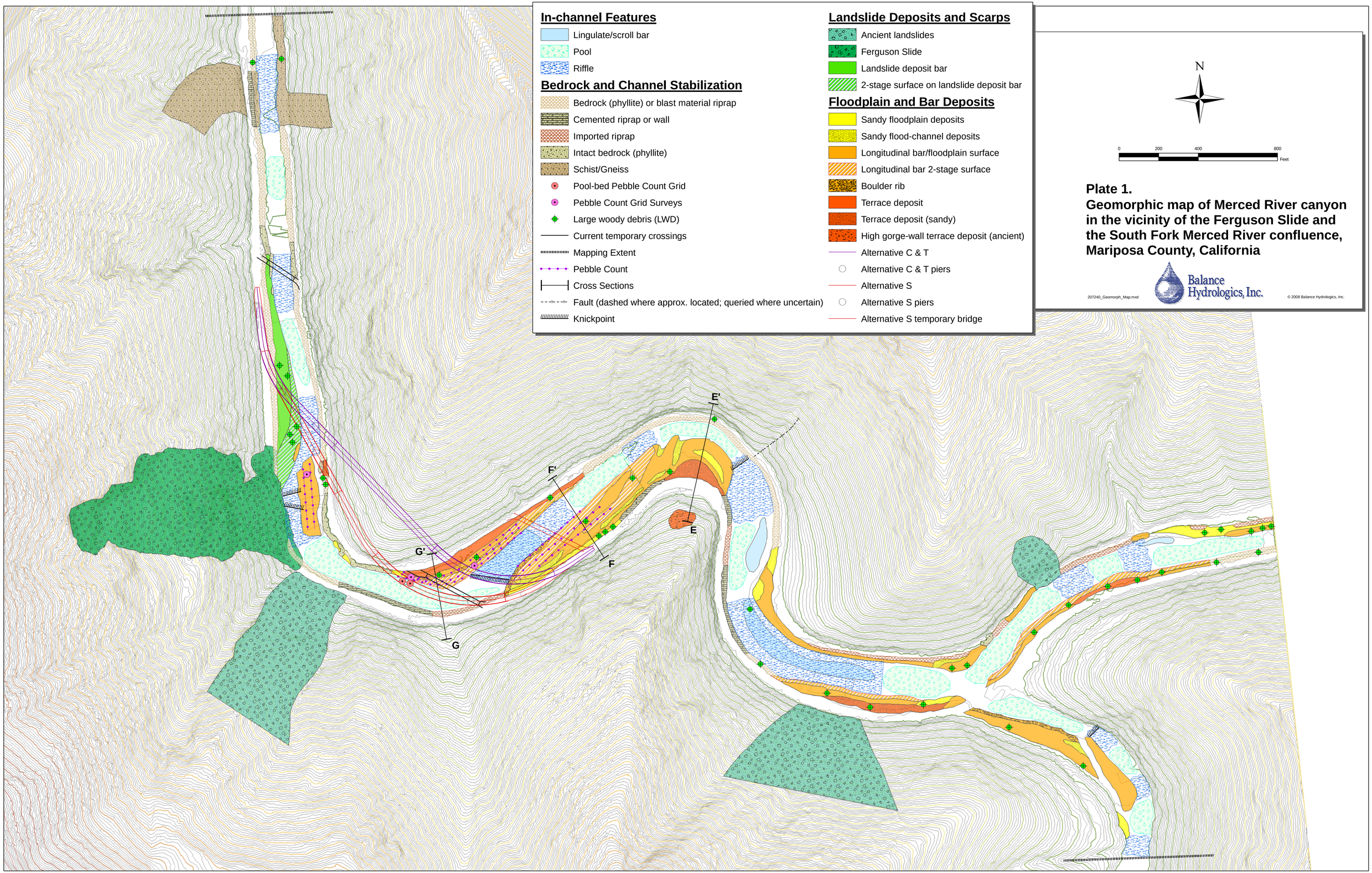
Figure 10.1 Suggested pier feasibility zones for continued planning and review of Ferguson Rock Slide Transportation Restoration Alternatives, Merced River, Mariposa County, California.

Balance Hydrologics, Inc.

©2008 Balance Hydrologics, Inc.

W:\Projects\207240 Merced River\207240_Figure_PierLocation_012709.mxd

PLATES



In-channel Features

- Lingulate/scroll bar
- Pool
- Riffle

Bedrock and Channel Stabilization

- Bedrock (phyllite) or blast material riprap
- Cemented riprap or wall
- Imported riprap
- Intact bedrock (phyllite)
- Schist/Gneiss
- Pool-bed Pebble Count Grid
- Pebble Count Grid Surveys
- Large woody debris (LWD)
- Current temporary crossings
- Mapping Extent
- Pebble Count
- Cross Sections
- Fault (dashed where approx. located; queried where uncertain)
- Knickpoint

Landslide Deposits and Scarps

- Ancient landslides
- Ferguson Slide
- Landslide deposit bar
- 2-stage surface on landslide deposit bar

Floodplain and Bar Deposits

- Sandy floodplain deposits
- Sandy flood-channel deposits
- Longitudinal bar/floodplain surface
- Longitudinal bar 2-stage surface
- Boulder rib
- Terrace deposit
- Terrace deposit (sandy)
- High gorge-wall terrace deposit (ancient)
- Alternative C & T
- Alternative C & T piers
- Alternative S
- Alternative S piers
- Alternative S temporary bridge

Plate 1.
Geomorphic map of Merced River canyon in the vicinity of the Ferguson Slide and the South Fork Merced River confluence, Mariposa County, California

207240_Geomorph_Map.mxd

Balance Hydrologics, Inc.

© 2008 Balance Hydrologics, Inc.

APPENDICES

APPENDIX A

ASCE Scour Equations

APPENDIX A
ASCE Scour Equations

The following narrative briefly defines the scour equations, along with their input parameters, used to supplement our scour analysis based on standard-of-practice guidance contained in the Army Corps of Engineers circular HEC-18 (Richardson and Davis, 2001) and other related documents (HEC-20 and HEC-23). References for each method are given in the report body, section 8.

Blodgett (1986):

d_s = 12 ft. for D_{50} < 0.005 ft.,
$d_s = 6.5 D_{50}^{-0.11}$ for D_{50} > 0.005 ft.,

where d_s is the scour depth [ft] and D_{50} is the median particle diameter of bed sediment [ft].

Lacey (1930):

$$d_s = 0.47\left(\frac{Q}{f}\right)^{1/3} * Z ,$$

where d_s is the scour depth [ft], Q is the design discharge [cfs], f is Lacey's silt factor, equal to $1.76(D_m)^{1/2}$, D_m is the mean grain size diameter of bed material [mm], and Z = 0.25 – 0.5 (for straight to moderate bends).

Blench (1969):

$$d_s = \frac{q_f^{2/3}}{F_{b0}^{1/3}} * Z ,$$

where d_s is the scour depth [ft], q_f is the design discharge per unit width [cfs/ft], F_{b0} is Blench's "zero bed factor" [ft/s^2], and Z = 0.6 (for straight to severe bends).

Competent/Limiting Velocity Control (Pemberton and Lara, 1984):

$$d_s = d_m \left(\frac{V_m}{V_c} - 1 \right),$$

where d_s is the scour depth [ft], d_m is the mean water depth [ft], V_m is the mean flow velocity [ft/s], and V_c is the competent mean velocity [ft/s] (from Pemberton and Lara, 1984; after Neill, 1973).

APPENDIX B

Aerial photographs consulted

Aerial photogrammetric APSRS database of Merced River at Ferguson Slide

Quad = Kinsley

SE corner of quad	LATITUDE	37°37'30" N
	LONGITUDE	119°52'30" W

207240 Merced River Geomorphology Study (CalTrans/Parsons)

APSRS search results

AGENCY	DATE	PROJECT	SCALE	FOCAL LENGTH	FILMTYPE	SENSORCLAS	CLOUD COVER	QUAD COVER
NASA, Ames	1994 Mar 30	04708	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1994 Mar 30	04709	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1994 Mar 30	04707	00063000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	90
U.S. Geological Survey	1993	NP9316	00040000	6.00in or 152mm	Black and White	Vertical Carto (Implies Stereo)	0	100
California Dept. of Water Resources	1992 Jul 22	WR-BAR	00012000	6.00in or 152mm	Black and White	Vertical Carto (Implies Stereo)	0	20
U.S. Forest Service, ASCS, APFO	1992 Jul	615160	00015840	8.25in or 210mm	Color	Vertical Carto (Implies Stereo)	0	90
NASA, Ames	1991 Sep 18	04311	00031000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	70
NASA, Ames	1991 Sep 18	04311	00031000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	70
NASA, Ames	1991 Sep 18	04309	00067000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1991 Aug 26	04286	00031000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1991 Aug 26	04286	00031000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1991 Aug 26	04287	00061000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1991 Aug 26	04287	00061000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1990 Aug 14	04095	00055000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	60	20
NASA, Ames	1990 Aug 14	04095	00055000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	30	70
NASA, Ames	1990 Aug 14	04095	00054000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1990 Jun 11	04034	00031000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1990 Jun 08	04032	00030000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	60
NASA, Ames	1990 Jun 08	04032	00030000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	80
NASA, Ames	1990 Jun 08	04032	00030000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1990 Apr 18	04023	00126000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	70	50
NASA, Ames	1990 Apr 18	04023	00128000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	60	60
NASA, Ames	1990 Apr 18	04023	00130000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	60	30
U.S. Forest Service, ASCS, APFO	1990	615160	00080000	6.00in or 152mm	Black and White	Vertical Carto (Implies Stereo)	0	100
NASA, Ames	1988 Sep 09	03800	00130764	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1988 Sep 09	03800	00130000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1988 Aug 17	03798	00122000	1.97in or 50mm	Black and White	Vertical Reconnaissance	0	70
NASA, Ames	1988 Aug 17	03797	00064999	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03739	00065000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1988 Jul 14	03739	00065000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03739	00065000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03740	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03740	00032642	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03740	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1988 Jul 14	03740	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 14	03740	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1988 Jul 14	03740	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1988 Jul 09	03737	00065500	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1988 Jul 09	03737	00061727	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1988 Jul 08	03735	00062058	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1988 May 12	03708	00131000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1988 May 12	03708	00130833	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1987 May 04	03627	00065000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	30

© 2008 Balance Hydrologics, Inc.

Aerial photogrammetric APSRS database of Merced River at Ferguson Slide

Quad = Kinsley

SE corner of quad	LATITUDE	37°37'30" N
	LONGITUDE	119°52'30" W

207240 Merced River Geomorphology Study (CalTrans/Parsons)

APSRS search results

AGENCY	DATE	PROJECT	SCALE	FOCAL LENGTH	FILMTYPE	SENSORCLAS	CLOUD COVER	QUAD COVER
NASA, Ames	1987 May 04	03627	00064000	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1987 May 04	03628	00033800	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, Ames	1987 May 04	03628	00034500	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	40
U.S. Geological Survey	1987	NP8720	00040000	6.00in or 152mm	Color Infrared	Vertical Carto (Implies Stereo)	0	100
U.S. Geological Survey	1986 Sep	RADMAR	00250000	Other	Black and White	Side-Looking Airborne Radar		100
NASA, Ames	1986 Aug 07	03580	00063500	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	20
WAC Corp.	1986 Jul	WAC-86	00031680	6.00in or 152mm	Black and White	Vertical Carto (Implies Stereo)	0	100
Mars Associates, Inc.	1985	SO-CAL	00500000	Other	Black and White	Side-Looking Airborne Radar	0	100
Bureau of Land Management, Denver	1985	CA85AC	00012000	12.0in or 305mm	Color	Vertical Carto (Implies Stereo)	10	40
NASA, Ames	1984 Sep 07	03405	00129636	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	50
NASA, Ames	1982 Jul 14	03086	00031133	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	60
U.S. Forest Service, Region 5	1982	615160	00024000	8.25in or 210mm	Color	Vertical Carto (Implies Stereo)	0	90
NASA, Ames	1980 Jun 23	02893	00119090	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	80
NASA, Ames	1980 Jun 23	02893	00127111	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	40
NASA, Ames	1980 Jun 23	02894	00030800	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	30
U.S. Geological Survey	1979 Nov 07	VERF	00080000	Other	Black and White	Vertical Carto (Implies Stereo)	0	100
Bureau of Land Management, Denver	1978	CA0478	00012000	6.00in or 152mm	Color	Vertical Carto (Implies Stereo)	10	30
U.S. Forest Service, Region 5	1977 Sep 23	615160	00024000	8.25in or 210mm	Color	Vertical Carto (Implies Stereo)	0	100
U.S. Geological Survey	1976 Oct 10	VDYM	00080000	Other	Black and White	Vertical Carto (Implies Stereo)	0	100
NASA, Ames	1974 Jul 17	01863	00020725	1.97in or 50mm	Color	Oblique	0	70
NASA, Ames	1974 May 09	01749	00126000	6.00in or 152mm	Black and White	Vertical Reconnaissance	0	30
NASA, Ames	1973 Dec 18	01584	00130006	6.00in or 152mm	Black and White	Oblique	20	100
NASA, Ames	1973 Dec 18	01584	00130006	6.00in or 152mm	Black and White	Oblique	20	100
NASA, Ames	1973 Dec 18	01584	00130006	6.00in or 152mm	Black and White	Oblique	20	100
NASA, Ames	1973 Nov 19	01550	00129000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	20	90
NASA, Ames	1973 Nov 19	01550	00124000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	10	70
NASA, Ames	1973 Aug 24	01397	00123000	6.00in or 152mm	Color	Vertical Reconnaissance	0	100
NASA, Ames	1973 Jun 29	01299	00117000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1973 Jun 29	01299	00123000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	50
NASA, Ames	1973 Jun 29	01299	00127000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1973 Jun 29	01299	00118000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1973 Jun 29	01300	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	70
NASA, Ames	1973 Jun 29	01300	00032000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	70
NASA, Ames	1973 Jun 29	01300	00030000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1973 Jun 29	01300	00033000	1.97in or 50mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1973 Apr 02	01054	00123000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1973 Feb 16	00957	00124000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
U.S. Forest Service, Region 5	1973	HAP-2	00064000	6.00in or 152mm	Black and White	Vertical Carto (Implies Stereo)		100
NASA, Ames	1972 Oct 24	00776	00120000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	10	100
NASA, Ames	1972 Oct 03	00714	00127000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	10	100
NASA, Ames	1972 Oct 03	00714	00129000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	20	100
NASA, Ames	1972 Jul 27	00565	00118000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1972 Jul 07	00493	00130006	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	30	40

© 2008 Balance Hydrologics, Inc.

Aerial photogrammetric APSRS database of Merced River at Ferguson Slide

Quad = Kinsley

SE corner of quad	LATITUDE	37°37'30" N
	LONGITUDE	119°52'30" W

207240 Merced River Geomorphology Study (CalTrans/Parsons)

APSRS search results

AGENCY	DATE	PROJECT	SCALE	FOCAL LENGTH	FILMTYPE	SENSORCLAS	CLOUD COVER	QUAD COVER
NASA, Ames	1972 Jul 07	00493	00125000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	10	20
NASA, Ames	1972 Jul 07	00493	00128000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	10	100
NASA, Ames	1972 Jun 19	00465	00130006	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	20
NASA, Ames	1972 Jun 19	00465	00123000	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, Ames	1972 May 12	00346	00125000	6.00in or 152mm	Black and White	Vertical Reconnaissance	10	50
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	30
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	50
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	40
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100

© 2008 Balance Hydrologics, Inc.

Aerial photogrammetric APSRS database of Merced River at Ferguson Slide

Quad = Kinsley

SE corner of quad	LATITUDE	37°37'30" N
	LONGITUDE	119°52'30" W

207240 Merced River Geomorphology Study (CalTrans/Parsons)

APSRS search results

AGENCY	DATE	PROJECT	SCALE	FOCAL LENGTH	FILMTYPE	SENSORCLAS	CLOUD COVER	QUAD COVER
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00281	00130006	6.00in or 152mm	Color Infrared	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	10	100
NASA, Ames	1972 Apr 14	00280	00130006	6.00in or 152mm	Color	Oblique	0	60
NASA, Ames	1972 Apr 07	00273	00130006	6.00in or 152mm	Color Infrared	Oblique	50	100
NASA, Ames	1972 Apr 07	00273	00130006	6.00in or 152mm	Color Infrared	Oblique	50	20
NASA, Ames	1972 Apr 07	00273	00130006	6.00in or 152mm	Color Infrared	Oblique	40	100
NASA, Ames	1972 Apr 07	00273	00130006	6.00in or 152mm	Color Infrared	Oblique	40	100
NASA, Ames	1972 Apr 07	00273	00130006	6.00in or 152mm	Color Infrared	Oblique	40	100
NASA, Ames	1972 Apr 07	00274	00130006	6.00in or 152mm	Color Infrared	Oblique	70	80
NASA, Ames	1972 Apr 07	00274	00130006	6.00in or 152mm	Color Infrared	Oblique	70	70
NASA, Ames	1972 Apr 07	00274	00130006	6.00in or 152mm	Color Infrared	Oblique	60	40
NASA, JS	1969 Jul 17	1000	00063281	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	30
NASA, JS	1969 Jul 17	1000	00062490	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	40
NASA, JS	1969 Jul 17	1000	00119723	6.00in or 152mm	Color	Vertical Reconnaissance	0	90
NASA, JS	1969 Jul 17	1000	00120073	6.00in or 152mm	Color	Vertical Reconnaissance	0	60
Agr. Stabilization and Conserv. Service	1961 Jul 27	EJC	00020000	8.25in or 210mm	Black and White	Vertical Carto (Implies Stereo)	0	100
U.S. Forest Service, Region 5	1971 Jul	EXV	00015840	8.25in or 210mm	Color	Vertical Carto (Implies Stereo)	0	100
NASA, JS	1970 Jul 24	1390	00060495	12.0in or 305mm	Color Infrared	Vertical Reconnaissance	0	60
NASA, JS	1970 Jul 24	1390	00116849	6.00in or 152mm	Color Infrared	Vertical Reconnaissance	0	100
NASA, JS	1970 Jul 24	1390	00121183	6.00in or 152mm	Color	Vertical Reconnaissance	0	100
U.S. Forest Service, Region 5	1970 Jul	EWA	00015840	8.25in or 210mm	Color	Vertical Carto (Implies Stereo)	0	40
U.S. Army	1955 Sep 09	001	00063000		Black and White	Vertical Carto (Implies Stereo)	0	100
Whittier College	1954	C20500	00039996		Black and White	Vertical Carto (Implies Stereo)		100
U.S. Geological Survey	1945 Dec 15	BS	00039200	Other	Black and White		0	100
Whittier College	1944	C9120	00007200	12.0in or 305mm	Black and White	Vertical Carto (Implies Stereo)	0	100
Whittier College	1931	C01840	00018000	8.25in or 210mm	Black and White	Vertical Carto (Implies Stereo)	0	90

© 2008 Balance Hydrologics, Inc.

Aerial photogrammetric USGS EROS (Earth Explorer) database of Merced River at Ferguson Slide

207240 Merced River Geomorphology Study (CalTrans/Parsons)

USGS EROS search results, orthophotoquads (yellow were utilized):

MAP NAME	QUADRANT	STATE	PRODUCT GROUP	ACQUISITION DATE	VERSION	STATUS	BAND TYPE	SE CORNER	RESOLUTION	ENTITY ID
Kinsley	SE	CA	3.75-Min B/W	7/16/1993	1	H	BW	37°37'30.00"N 119°52'30.00"W	1	DI00000000051743
Kinsley	SE	CA	3.75-Min B/W	8/20/1998	2	A	BW	37°37'30.00"N 119°52'30.00"W	1	DI00000001131640

USGS EROS search results, aerial photo single frames (yellow were utilized):

AGENCY	QUALITY	CLOUD COVER	DATA TYPE	ACQUISITION DATE	MICROFRAME LOCATION	FOCAL LENGTH (MM)	IMAGE TYPE	SE CORNER	STEREO OVERLAP %	ENTITY ID	ROLL NBR	FRAME NBR	RECORDING TECHNIQUE	FILM LENGTH x WIDTH (MM)	SCALE	FLYING HEIGHT (FT)
NASA-Ames Research Center	8	0	Individua	6/29/1973	01940464	610	CIR	37°35'18.00"N 119°48'10.12"W	60	ARS7300013000160	001300	160	Vertical Reconnaissance	457 x 229	33000	65003
NASA-Ames Research Center	8	0	Individua	6/29/1973	01940465	610	CIR	37°37'12.00"N 119°48'24.01"W	70	ARS7300013000161	001300	161	Vertical Reconnaissance	457 x 229	32000	65003
NASA-Ames Research Center	8	0	Individua	6/29/1973	01940466	610	CIR	37°38'50.00"N 119°48'18.72"W	70	ARS7300013000162	001300	162	Vertical Reconnaissance	457 x 229	32000	65003
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680131	609.6	CIR	37°38'06.00"N 119°50'01.28"W	70	ARS5880037400130	003740	130	Vertical Reconnaissance	457 x 229	32000	64600
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680132	609.6	CIR	37°37'00.00"N 119°49'59.99"W	70	ARS5880037400131	003740	131	Vertical Reconnaissance	457 x 229	32000	64600
NASA-Ames Research Center	8	0	Individua	9/18/1991	04320928	609.6	CIR	37°38'08.01"N 119°51'09.07"W	50	ARS5910043110061	004311	61	Vertical Reconnaissance	457 x 229	31000	66800
NASA-Ames Research Center	8	0	Individua	9/19/1991	04320929	609.6	CIR	37°36'30.67"N 119°51'13.57"W	50	ARS5910043110062	004311	62	Vertical Reconnaissance	457 x 229	31000	66800
NASA-Ames Research Center	8	0	Individua	6/8/1990	04040499	609.6	CIR	37°35'44.00"N 119°52'41.70"W	50	ARS5900040320007	004032	7	Vertical Reconnaissance	457 x 229	30000	30000
NASA-Ames Research Center	8	0	Individua	6/8/1990	04040500	609.6	CIR	37°37'22.00"N 119°52'44.62"W	50	ARS5900040320008	004032	8	Vertical Reconnaissance	457 x 229	30000	30000
NASA-Ames Research Center	8	0	Individua	6/8/1990	04040501	609.6	CIR	37°38'54.00"N 119°52'47.39"W	50	ARS5900040320009	004032	9	Vertical Reconnaissance	457 x 229	30000	30000
NASA-Ames Research Center	8	0	Individua	3/30/1994	04730254	609.6	CIR	37°38'05.55"N 119°52'01.16"W	50	ARS40047080068	004708	68	Vertical Reconnaissance	457 x 229	32000	64100
NASA-Ames Research Center	8	0	Individua	3/30/1994	04730255	609.6	CIR	37°36'24.93"N 119°51'42.84"W	50	ARS40047080069	004708	69	Vertical Reconnaissance	457 x 229	32000	64100
NASA-Ames Research Center	8	0	Individua	3/30/1994	04730348	609.6	CIR	37°38'05.55"N 119°52'01.16"W	50	ARS40047090068	004709	68	Vertical Reconnaissance	457 x 229	32000	64100
NASA-Ames Research Center	8	0	Individua	3/30/1994	04730349	609.6	CIR	37°36'24.93"N 119°51'42.84"W	50	ARS40047090069	004709	69	Vertical Reconnaissance	457 x 229	32000	64100
NASA-Ames Research Center	8	0	Individua	6/23/1980	02260454	609.6	CIR	37°39'58.43"N 119°44'56.98"W	60	ARS5800028940076	002894	76	Vertical Reconnaissance	457 x 229	30800	65003
NASA-Ames Research Center	8	0	Individua	6/23/1980	02260455	609.6	CIR	37°38'15.80"N 119°44'45.96"W	60	ARS5800028940077	002894	77	Vertical Reconnaissance	457 x 229	30800	65003
NASA-Ames Research Center	8	0	Individua	6/23/1980	02260456	609.6	CIR	37°36'31.90"N 119°44'34.84"W	60	ARS5800028940078	002894	78	Vertical Reconnaissance	457 x 229	30800	65003
NASA-Ames Research Center	8	0	Individua	7/14/1982	02330402	609.6	CIR	37°38'55.71"N 119°47'02.18"W	60	ARS5820030860134	003086	134	Vertical Reconnaissance	457 x 229	31133	65003
NASA-Ames Research Center	8	0	Individua	7/14/1982	02330403	609.6	CIR	37°37'10.50"N 119°47'00.24"W	60	ARS5820030860135	003086	135	Vertical Reconnaissance	457 x 229	31133	65003
NASA-Ames Research Center	8	0	Individua	5/4/1987	03630715	609.6	CIR	37°33'14.58"N 120°00'59.26"W	60	ARS5870036280013	003628	13	Vertical Reconnaissance	457 x 229	33800	65003
NASA-Ames Research Center	8	0	Individua	5/4/1987	03630716	609.6	CIR	37°33'27.49"N 120°01'12.00"W	60	ARS870036280014	003628	14	Vertical Reconnaissance	457 x 229	33800	65003
NASA-Ames Research Center	8	0	Individua	6/11/1990	04050662	609.6	CIR	37°35'30.00"N 119°44'30.01"W	60	ARS900040340019	004034	19	Vertical Reconnaissance	457 x 229	31000	67900
NASA-Ames Research Center	8	0	Individua	6/11/1990	04050663	609.6	CIR	37°36'58.50"N 119°44'28.72"W	60	ARS900040340020	004034	20	Vertical Reconnaissance	457 x 229	31000	67900
NASA-Ames Research Center	8	0	Individua	6/11/1990	04050664	609.6	CIR	37°38'32.00"N 119°44'27.35"W	60	ARS900040340021	004034	21	Vertical Reconnaissance	457 x 229	31000	67900
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680284	609.6	CIR	37°36'17.78"N 119°44'37.03"W	60	ARS880037400283	003740	283	Vertical Reconnaissance	457 x 229	32000	64800
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680285	609.6	CIR	37°37'36.00"N 119°44'37.39"W	60	ARS880037400284	003740	284	Vertical Reconnaissance	457 x 229	32000	64800
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680286	609.6	CIR	37°38'54.36"N 119°44'37.75"W	60	ARS880037400285	003740	285	Vertical Reconnaissance	457 x 229	32000	64800
NASA-Ames Research Center	8	0	Individua	7/14/1988	03680130	609.6	CIR	37°39'18.00"N 119°49'59.99"W	60	ARS880037400129	003740	129	Vertical Reconnaissance	457 x 229	32000	64800
NASA-Ames Research Center	8	0	Individua	7/9/1988	03671164	609.6	CIR	37°35'54.00"N 119°44'14.03"W	60	ARS880037380801	003738	801	Vertical Reconnaissance	457 x 229	32000	65000
U.S. Geological Survey	8	0	Individua	12/15/1945	00010164	132.08	BW	37°36'47.16"N 119°53'02.31"W	60	ARS1BS0000020028	000002	28	Low Oblique	229 x 229	39200	17000
U.S. Geological Survey	8	0	Individua	12/15/1945	00010164	132.08	BW	37°36'53.86"N 119°50'12.998"W	60	ARS1BS0000020029	000002	29	Low Oblique	229 x 229	39200	17000
U.S. Geological Survey	8	0	Individua	9/1/1955	00360361	153	BW	37°39'12.38"N 119°49'58.14"W	60	ARS1VJS0000010003	000001	3	Vertical Cartographic	229 x 229	47000	23500
U.S. Geological Survey	8	0	Individua	9/1/1955	00360363	152	BW	37°37'26.27"N 119°47'57.63"W	60	ARS1VJS0000020062	000002	62	Vertical Cartographic	229 x 229	47000	23500
U.S. Geological Survey	8	0	Individua	9/1/1955	00360361	153	BW	37°36'47.49"N 119°49'50.32"W	60	ARS1VJS0000010002	000001	2	Vertical Cartographic	229 x 229	47000	23500

© 2008 Balance Hydrologics, Inc.

APPENDIX C

Merced River at the Ferguson Slide Reach, Condition Bathymetric and Topographic Surveys: Field Data Collection Procedures

Merced River at the Fergusson Slide Reach
Condition Bathymetric and Topographic Surveys:

Field Data Collection Procedures

July 2008

Prepared For:
Mark W. Strudley, PhD.
Hydrologist/Geologist
Balance Hydrologics, Inc.
800 Bancroft Way, Suite #101
Berkeley, CA 94710-2227

Prepared By:
Environmental Data Solutions
818 5th Ave., Suite 208
San Rafael, CA 94901

Environmental
Data Solutions

Services provided pursuant to this agreement are intended solely for the use and benefit of Balance Hydrologics.

No other person or entity shall be entitled to rely on the services, opinions, recommendations, plans or specifications provided pursuant to this agreement without the express written consent of Environmental Data Solutions.

DRAFT DOCUMENT
NOT FOR PUBLIC RELEASE OR CITATION

TABLE OF CONTENTS

		Page No.
1. INTRODUCTION		4
1.1	Project Description	4
2. METHODS		4
2.1	Bathymetric Surveys	4
2.2	Topographic surveys	5
2.3	Survey Control	5
3. QA / QC and RESULTS		6
3.1	Weather Conditions	6
3.2	Fathometer Barcheck Results	6
4. REFERENCES		7

1. INTRODUCTION

1.1 PROJECT DESCRIPTION

On July 22, 23 and 24 2008, Environmental Data Solutions (EDS) conducted a condition Class 1 hydrographic survey of a 2,654 ft reach of the Merced River at the Fergusson Slide. The survey also included an extensive topographic survey of the channel banks and the channel areas which were inaccessible by survey vessel.

Raw bathymetric survey data were collected in four pools, and processed using *Hypack 2008* hydrographic data collection and processing software. Topographic data were collected using a *Leica System 1200* Real-Time Kinematic GPS (RTK-GPS) base and rover unit system in conjunction with a *Topcon* electronic theodoloite (total station)

This field data collection report outlines survey methodologies and quality assurance and quality control protocols utilized to achieve a Class 1 survey.

2. METHODS

2.1 BATHYMETRIC SURVEYS

The survey crew utilized a 14' rigid Zodiac survey vessel, compliant with all U.S. Coast Guard safety regulations, to perform the survey. A graph displaying the results from a squat correction test conducted for the vessel in March 2004 is aboard the vessel (per U.S. Army Corps of Engineers' specifications).

The hydrographic surveys utilized Class 1 methods and accuracies as outlined in the Army Corps of Engineers' January 2002 *Hydrographic Surveying Manual* (EM 1110-2-1003). Bathymetric data were collected using two *Odom Hydrotrac* survey-grade fathometers coupled with 3-degree 200-kHz transducers. The transducers were mounted on each gunwale of the vessel resulting in two lines of survey data per survey transect,

Position data (geographic coordinates) were collected using a second *Leica* RTK GPS rover unit mounted above the fathometers. Survey vessel motion (heave) was measured using a *TSS CMS-25* motion sensor (the motion sensor was also attached to the top of the fathometer). Vessel motion data were applied to the raw sounding data before being logged to the computer. The data stream was collected using a *Pentium*-based CPU running *Hypack Max* (2008) survey planning, data collection and reduction software. Squat corrections were added automatically to the raw sounding data depending on vessel speed, fuel consumption and resulting squat.

Horizontal position control data were collected in geographic coordinates (latitude and longitude in decimal degrees) based on the North American Datum of 1983 (NAD83) using the *RTK GPS* system. *Corpscon* Version 6.0 was used to convert the geographic coordinates to California State Plane Zone 3 (NAD83, feet) coordinates.

The 2-person field crew included Mr. James Kulpa (Hydrographer - EDS), and Darren Gewant (Surveyor-EDS).

2.2 TOPOGRAPHIC SURVEYS

EDS field personnel used RTK GPS technology in order to collect topographic data on the surrounding river banks, cobble bars and channel bottoms which were inaccessible by survey vessel. RTK surveying requires dual frequency LI/L2 GPS receivers: One receiver is set over a known point and the rover receiver collects XYZ data as well as point meta-data as the field surveyor travels from point to point along a pre-determined survey transect.

The survey team utilized a *Leica System 1200* RTK base station and rover unit coupled with a *Pacific Crest* UHF radio modem. The RTK base station was set over a known CALTRANS control point (Meta data supplied by Balance Hydrologics) with the rover unit programmed to collect data in California State Plane Coordinates, Zone 3 (NAD 83 Datum) and ft NAVD 88.

2.3 SURVEY CONTROL

The bathymetric and topographic surveys tied into two CALTRANS control points and held to the points listed in Table 1.

Table 1 - Control Point Meta-Data

CALTRANS Pt ID	Northing (SPCZ3 ft)	Easting (SPCZ3 ft)	Orthometric Ht (ft NGVD)
FERG5	2061911.6140	6736182.8630	1385.857
CCL196	2062137.7840	6736517.4130	1389.829

3. QA / QC AND RESULTS

In order to achieve Class 1 accuracy requirements, meta-data describing various hydrographic survey variables were measured, recorded and utilized before, during and after each survey day. QA / QC protocols and meta-data are presented in this section.

3.1 WEATHER CONDITIONS

Table 2 presents all environmental meta-data.

Table 2 - Environmental Meta-Data

Date	22 – July - 08	23 – July - 08	24 – July - 08
Weather	Clear	Clear	Clear
Wind	Morning – 0 knts Afternoon – 10 knts	Morning – 0 knts Afternoon – 10 knts	Morning – 0 knts Afternoon – 10 knts
Water Surface Conditions	Smooth	Smooth	Smooth
Stage during hydro (ft NGVD)	1372.44	1360.76	1344.33 and 1340.10
Survey Time Period (pst)	12:00 – 16:30	9:00 – 14:00	13:00 – 15:00
Bottom Type	Cobble	cobble	cobble

3.2 FATHOMETER BARCHECK RESULTS

There are two standard procedures used to ensure fathometer data quality: 1) speed of sound calculations and 2) fathometer barcheck calibration. Fathometers calculate water depth by using algorithms based on the speed of sound through the water column. Since the speed of sound is affected by water temperature and salinity, these values were recorded each day before the bathymetric surveys began (utilizing an *Odom Digi-Bar Pro*). An accurate speed of sound measurement was then programmed into the fathometer. The average water column temperature and salinity was 18° C and 0 ppt, resulting in a speed of sound constant of 4803 ft/sec.

The second protocol is a barcheck calibration which is performed on the fathometer before and after each survey. This procedure consists of lowering a 12 inch diameter, weighted steel plate below the fathometer transducer and recording the actual depth of the disc (via markings on a cable) and the fathometer output (output was corrected for the transducer depth offset). All barchecks corresponded with fathometer output

4. REFERENCES

U.S. Army Corps of Engineers. 2002. *Hydrographic Survey Manual*, Engineering and Design Manual No. EM-1110-2-1003, Washington D.C.

APPENDIX D

Individuals contacted

Individuals Contacted

Name	Position	Phone No.	Email
Zephyr Whitewater Expeditions			
Bob Ferguson[1]	Owner	1-800-431-3636	---
Jimmy Gado[1]	Guide	1-800-431-3636	---
		1-209-379-2007	---
		1-209-768-4239	---
U.S. Bureau of Land Management			
James Eicher[1]	Associate Field Manager	1-916-985-4474	James_Eicher@ca.blm.gov
USDA Forest Service/Dept. of Earth and Env. Sciences, California State University, Fresno			
Jerry DeGraff[2]	On Scene Coordinator	1-559-297-0706 ext. 4932	jdegraff@fs.fed.us
	Adjunct Professor	---	---
Miwok and Paiute Native American Tribes			
Bill Tucker[2]	Construction Monitor	---	---

Notes:

[1] Consulted via telephone conversation.

[2] Consulted on-site during Bathymetric Survey reconnaissance visit of July 9, 2008.

© 2008 Balance Hydrologics, Inc.