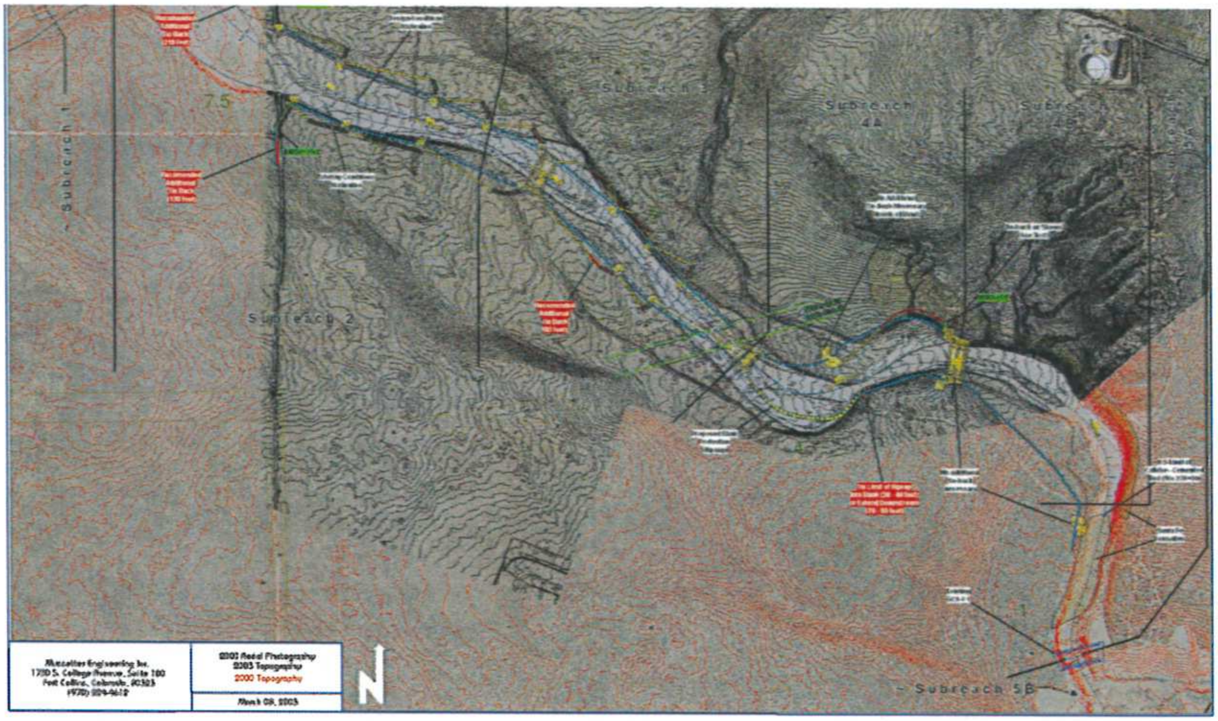
Hydraulic and Channel Stability Analysis and Designing Assistance for Main Branch Calabacillas Arroyo Upstream from Swinburne Dam



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1. INTRODUCTION

1.1. Project Objectives and Background

Mussetter Engineering, Inc. (MEI) was retained by Wilson and Company, Inc. (Wilson) to assist in designing of proposed channel protection measures in the approximately 1.2 mile reach of Calabacillas Arroyo between the existing grade-control structure (GCS#1) near the upstream limit of the pool of Swinburne Dam to the Sandoval County Line (**Figure 1.1**). The evaluation included protection measures that are being designed by Wilson in the vicinity of the proposed McMahon Boulevard Bridge and the portion of the reach downstream from the bridge, as well as the protection measures that are being designed by Mark Goodwin and Associates (Goodwin) in the portion of the reach upstream from the bridge. The primary objectives of MEI's work were as follows:

1. Evaluate the impacts of the proposed structural measures on channel capacity.
2. Evaluate the impacts of the proposed structural measures on the lateral and vertical stability of the channel.
3. Assist Wilson and Goodwin in revising the designs to better meet public safety objectives while preserving, to the extent possible, the naturalistic characteristics of the arroyo.

The analyses that were performed by MEI for this work generally followed procedures described in the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) Sediment and Erosion Design Guide (Mussetter et al., 1994), subsequently referred to as the Design Guide.

The locations and general layout of the initially-proposed protection measures that were analyzed are essentially the same as those shown on Sheet 1 of 1 in the Master Plan for the Arroyo Vista Subdivision that was prepared by Mark Goodwin and Associates (Goodwin) in June 2004 (Goodwin, 2004). The protection measures include Grade Control Structure (GCS) No. 2 at Sta 250+00, GCS#3 at Sta 263+43, GCS#4 at Sta 280+94, a series of spur dikes, and features associated with the proposed McMahon Boulevard bridge crossing. Several of the details, including the crest elevations and widths of the grade-control structures, the configuration of the McMahon Boulevard bridge opening, and widths between the spurs were modified from the Goodwin Master Plan based on discussions at a meeting that was held on October 12, 2004, and subsequent telephone conversations and email messages between Goodwin, Wilson, MEI, and AMAFCA. One important change from the Goodwin plan involved elimination of Grade Control Structure (GCS) No. 5 at the upstream end of the reach. Other specific changes included reducing the bottom widths of GCS#2 and GCS#4 to 140 feet and GCS#3 to 100 feet. The resulting features that were included in the initial evaluation are shown on **Map 1, Appendix A**. The spur dikes are labeled on the map as Spur Dikes 1 through 9 from downstream to upstream, with subscript L for the left bank spurs (looking downstream) and subscript R for the right bank spurs. The structures were designed with the intent to reduce vertical incision and lateral migration, and therefore, to reduce the width of the original prudent line corridor. The final design plan was developed by modifying the location and orientation of many of the elements of the initial plan based on results from the preliminary hydraulic, vertical stability, and lateral migration analyses (**Map 2, Appendix A**).

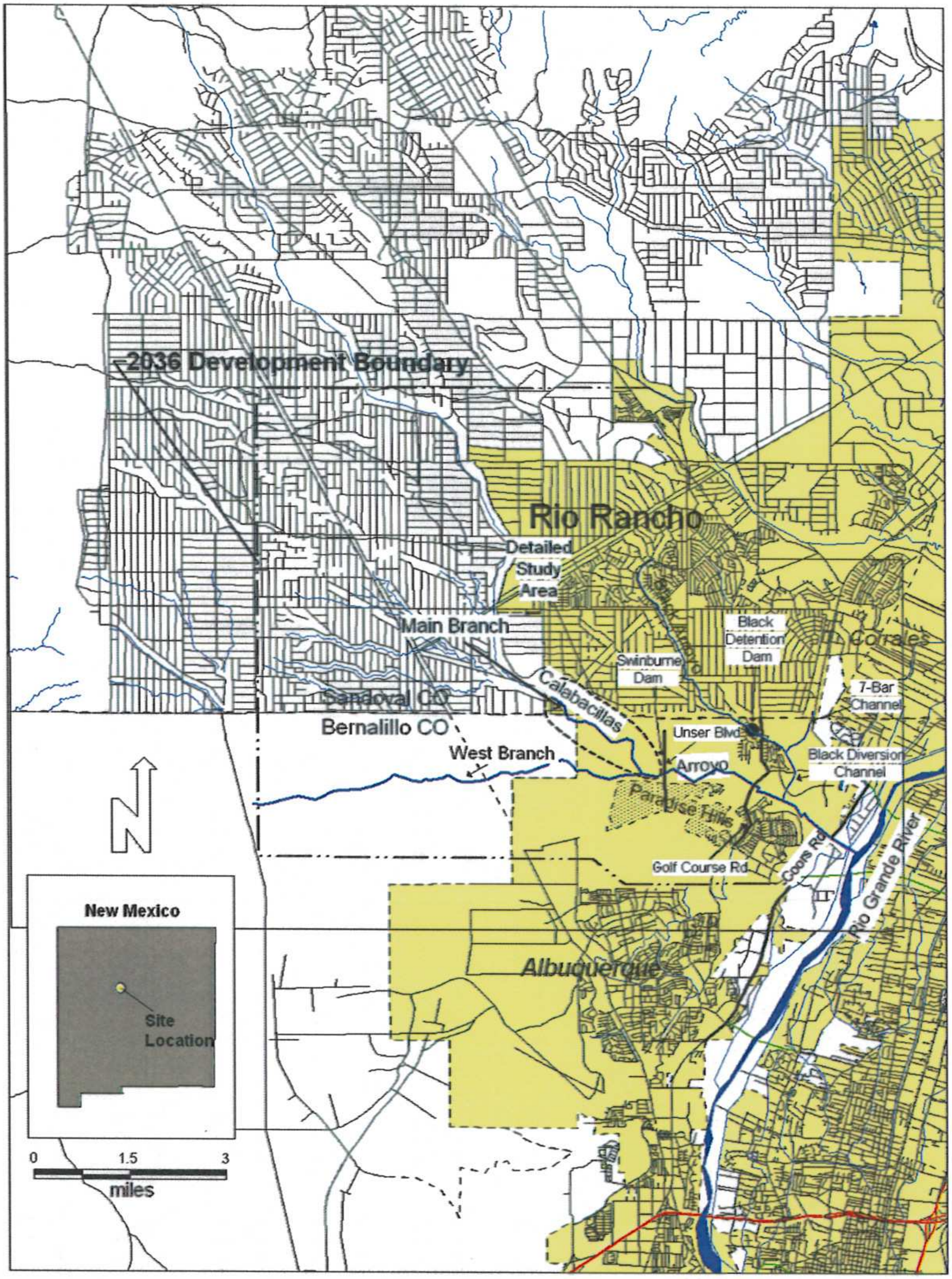

Figure 1.1. Vicinity map of the project area.

1.2. Authorization

This investigation was conducted by Mussetter Engineering, Inc. (MEI) under a contract agreement with Wilson. Goodwin's client and AMAFCA provided a portion of the funding for the work. Mr. Dan Aguirre, P.E., was the Wilson's Project Manager, and Dr. Robert Mussetter was MEI's Project Manager. Mr. John Kelley represented AMAFCA's interests in the work, and Mr. Doug Hughes was the Project Manager for Goodwin. Stuart Trabant, P.E. (Colorado) was MEI's project engineer and he performed most of the analyses that were conducted by MEI.

1.3. Description of Existing Arroyo

In 2001, AMAFCA and Bohannon-Huston, Inc. (BHI) constructed GCS #1, in the main branch of Calabacillas Arroyo approximately 0.6 miles above from Swinburne Dam to limit upstream incision associated with future flows and the relatively steep channel gradient at the upstream end of the Swinburne Pool. The structure is located at Sta 229+85 (Appendix A), and it has a crest width of 140 feet and a design drop height of 7 feet. Upstream from GCS#1, the Calabacillas channel is relatively unmodified by man's activities. An outcrop of the weakly cemented sandstones and mudstones of the Santa Fe Formation is present in the bed between GCS#1 and Sta 238+00 that also provides at least temporary baselevel control for the upstream reach (**Plate 1**). These relatively erosion-resistant materials also inhibit erosion into the toe of the left bank in this reach. The thickness of the weakly cemented sediments is unknown, and there is a possibility that without grade stabilization, the baselevel for the upstream channel could be lowered with attendant upstream incision, shifting the control to GCS#1.

The weakly cemented Santa Fe Formation materials in the left (east) bank extend upstream to about Sta 254+00. The outcrop pattern, which includes the previously discussed outcrop in the bed of the channel, is controlled by an east-dipping, high angle normal fault located to the east that parallels the channel (Kelley, 1977). From about Sta 254+00 to Sta 262+00, the arroyo is constrained on the left bank by a relatively erosion-resistant, caliche-cemented terrace that extends about 15 feet above the bed of the arroyo (**Plate 2**). The right bank through this reach is composed of an approximately 8 foot high alluvial terrace that is erodible.

Farther upstream, the valley widens and the channel is inset below widely spaced alluvial terraces from about Sta 262+00 to Sta 282+00. Where the channel bank is formed by the terrace rather than the floodplain, the terrace is generally being eroded. For the remainder of the reach along the right bank, the bank height rarely exceeds 3 feet except where the bank and the terrace coincide (**Plate 3**). In general, the left side of the channel in this reach is composed of the more erosion-resistant, caliche-cemented terrace or outcrop of the Santa Fe Formation.

Two bed material samples that were collected along the study reach in 1995 had median sizes (D_{50}) of 0.8 and 1.3 mm, indicating that the typical bed material in the reach has an average D_{50} of about 1.0 mm (**Figure 1.2**). The bed material is very similar to samples collected on Calabacillas Arroyo downstream of Swinburne Dam.

Plate 1. View upstream of weakly-cemented sandstone and siltstone in the bed of the Main Branch of Calabacillas Arroyo at about Sta 232+00. Santa Fe Formation forms the left bank of the arroyo in the background.

Plate 2. View downstream of caliche-cemented terrace on left bank of the Main Branch of Calabacillas Arroyo at about Sta 250+00.

Plate 3. View downstream of the Main Branch of Calabacillas Arroyo from about Sta 317+00. The left bank of the arroyo is composed of Santa Fe Formation. A wide floodplain is located beyond the right bank.

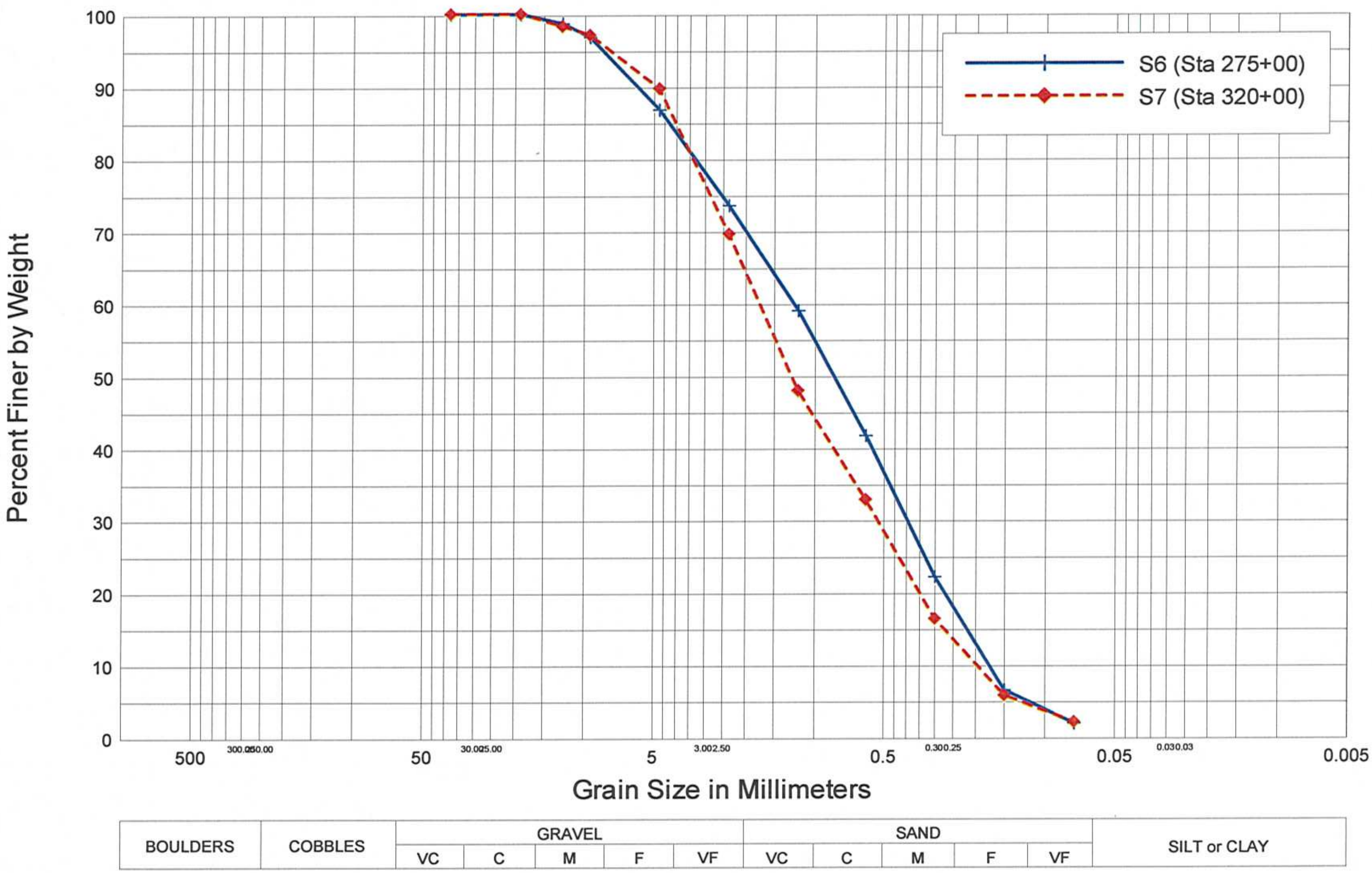

Figure 1.2. Gradation curves for bed-material samples collected from the study reach of the Main Branch of Calabacillas Arroyo upstream of the Swinburne Dam pool.

2. HYDROLOGY

The flood hydrology of Calabacillas Arroyo has previously been estimated by others [Resource Technology, Inc. (RTI, 1987); Resource Consultants, Inc. (RCI, 1989); Smith Engineering Company (SEC, 1994a and 1994b)] using the Corps of Engineers HEC-1 model (USACE, 1990a). The SEC HEC-1 models, developed for both pre-development and 2036 development conditions, were used in this study since they represent the most recent information.

Results from the SEC (1994a and 1994b) models indicate that the clear-water (i.e., unbulked) peak discharge associated with the 2-year storm under pre-development conditions ranges from about 710 cfs at the upstream end of the project reach to about 1,060 cfs at Swinburne Dam, and the 100-year peak discharge ranges from about 8,210 cfs to 12,310 cfs at these locations (**Table 2.1**). Under the projected 2036 development conditions, the 2-year peak discharges at the two locations increase to about 2,000 cfs to 3,880 cfs, respectively, and the 100-year peak discharges increase to 9,540 cfs to 16,760 cfs. Peak discharges and runoff volumes for other storm frequencies and locations along the reach for both pre-development and future (2036) conditions are summarized in Table 2.1, and typical hydrographs for the portion of the reach between the West Branch confluence and Sta 303+00 for pre-development and 2036 conditions are shown in **Figures 2.1 and 2.2**. These hydrographs indicate that the runoff response is faster under 2036 conditions than under pre-development conditions, with the time-to-peak decreasing from about 3 hours to about 2 hours. Table 2.1 also includes the bulked peak flows for each case that account for the volume of sediment being carried by the flow. The bulking factors were estimated by increasing the peak discharge by a factor that represents the average volume of sediment in flow based on previous sediment transport computations that were performed by MEI (2000), and they generally ranged from about 2 percent during the 2-year event to about 5 percent during the 100-year flow. Copies of the HEC-1 input and summary output files for the entire Calabacillas watershed, including the study reach for this work, were included in Appendix A of the report entitled, "Hydraulic Capacity and Stability Analysis for Levees Between Coors Road and the Rio Grande" (MEI, 1996a).

As has been previously discussed with AMAFCA, it is MEI's opinion that the peak flows and runoff volumes that are estimated from the models that were used for this study for the more frequent floods may be unreasonably high. This opinion is based on the lack of significant runoff events that approached the 2-year peak discharge in at least the past 10 years. The implication of over-predicting the more frequent floods on the channel stability analysis is discussed in Chapter 4.

Table 2.1. Summary of peak discharges and storm runoff volumes, Main Branch, Calabacillas Arroyo upstream of Swinburne Dam.						
Recurrence Interval (years)	Pre-development Conditions			2036 Development Conditions		
	Peak Discharge (cfs)		Unbulked Runoff Volume (ac-ft)	Peak Discharge (cfs)		Unbulked Runoff Volume (ac-ft)
	Unbulked	Bulked		Unbulked	Bulked	
Node P25 - Upstream end of project reach (49.2 mi^2)						
2	709	724	169	2,003	2,054	375
5	2,279	2,353	543	3,570	3,675	772
10	3,599	3,726	859	4,953	5,099	1,114
25	5,289	5,475	1,262	6,627	6,811	1,536
50	6,721	6,945	1,605	8,063	8,276	1,896
100	8,214	8,477	1,962	9,538	9,786	2,266
Node P26 - Just downstream of west bank tributary at Station 303+00 (63.8 mi^2)						
2	925	946	219	2,890	2,967	579
5	2,971	3,068	705	5,031	5,184	1,109
10	4,692	4,860	1,113	6,927	7,148	1,570
25	6,894	7,154	1,636	9,200	9,513	2,130
50	8,761	9,107	2,080	11,124	11,523	2,604
100	10,707	11,151	2,543	13,107	13,599	3,092
Node P28 - Just downstream of north bank tributary at Station 254+00 (66.5 mi^2)						
2	963	990	229	3,173	3,277	618
5	3,096	3,215	735	5,478	5,681	1,172
10	4,889	5,099	1,161	7,527	7,809	1,656
25	7,183	7,495	1,707	9,968	10,352	2,241
50	9,128	9,531	2,170	12,025	12,508	2,737
100	11,154	11,659	2,652	14,136	14,734	3,247
Node P33 - Just downstream of the West Branch (77.4 mi^2)						
2	1,063	1,075	260	3,876	3,895	728
5	3,415	3,450	827	6,605	6,636	1,354
10	5,393	5,447	1,304	9,016	9,058	1,900
25	7,924	8,003	1,915	11,890	11,945	2,561
50	10,070	10,170	2,432	14,298	14,364	3,118
100	12,306	12,428	2,972	16,764	16,842	3,692

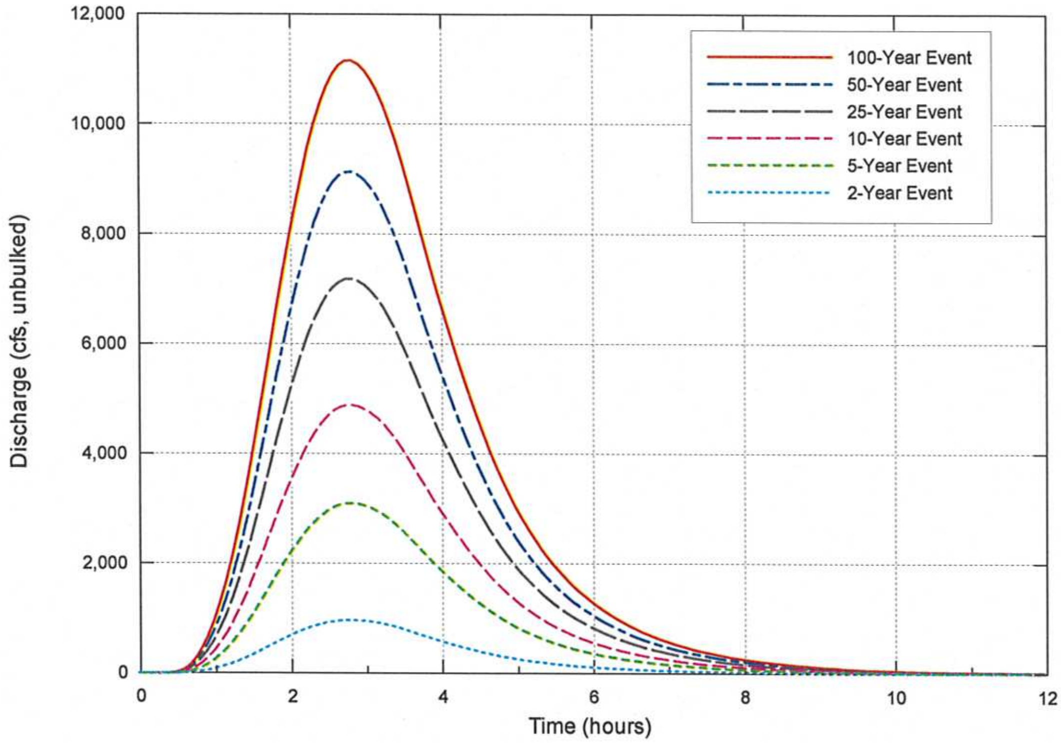

Figure 2.1. Flood Hydrographs (2-, 5-, 10-, 25-, 50-, and 100-year, unbulked) for Calabacillas Arroyo just upstream of the West Branch confluence (Node P28), pre-development conditions.

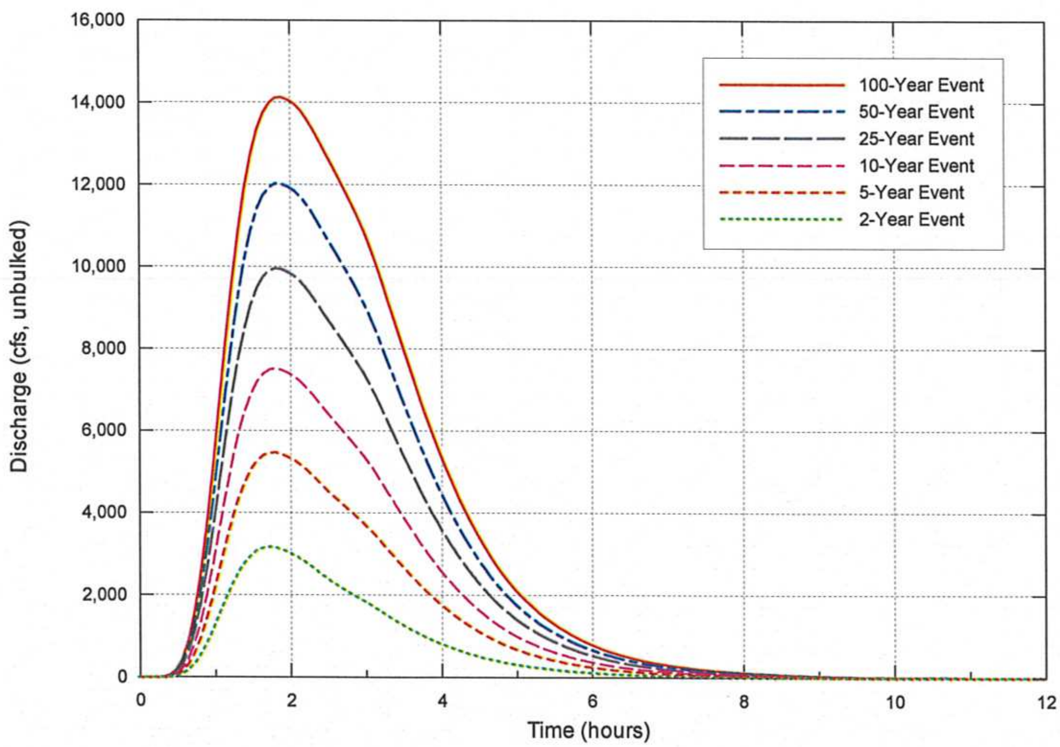

Figure 2.2. Flood Hydrographs (2-, 5-, 10-, 25-, 50-, and 100-year, unbulked) for Calabacillas Arroyo just upstream of the West Branch confluence (Node P28), 2036 development conditions.

3. HYDRAULIC ANALYSIS

3.1. Model Development

A hydraulic analysis of the main branch of Calabacillas Arroyo from Swinburne Dam upstream to approximately one-fourth mile upstream from the Sandoval County Line was performed using the U.S. Army Corps of Engineers HEC-RAS backwater model, Version 3.1.2 (USACE, 2004). The model extended downstream of the design reach to Swinburne Dam so that the as-built, stage-discharge relationship that was used in the SEC (1994a and 1994b) HEC-1 models could be used to establish the downstream boundary condition. The reach through the Swinburne Dam pool was also included in the model to facilitate preparation of the Conditional Letter of Map Revision (CLOMR), which will require the effects of Swinburne Dam on flooding to be considered. The model was used to estimate the hydraulic conditions (e.g., velocity, depth, shear stress) within the project reach under existing and design conditions for a range of discharges through the 2036 development conditions 100-year flood peak.

Cross-sectional data were developed from the available mapping that included 1-foot contour interval mapping developed by Thomas R. Mann and Associates, Inc. using aerial photography from April 2003 from Sta 245+00 to Sta 294+00, and 2-foot contour interval mapping developed by Bohannan-Huston Inc. (BHI) for the USACE using aerial photography from 2000 for the remainder of the project reach. Bentley InRoads Site 2004 Version 8.05 in conjunction with Bentley Systems MicroStation Version 8.05.01.25 was used to cut cross sections from the mapping data that was obtained by MEI in digital terrain model (DTM) format. The BHI mapping was referenced to North American Vertical Datum of 1988 (NAVD88); however, the more recent mapping by Thomas Mann was referenced to National Geodetic Vertical Datum of 1929 (NGVD29). The BHI mapping was, therefore, converted to NGVD29 to maintain consistency with the newer mapping. **All elevations reported throughout the remainder of this study are referenced to NGVD29.** In preparing the HEC-RAS model, minor adjustments to the channel invert elevations were made, where appropriate, based on a survey that was conducted by Wilson in 2004. Encroachments were added to the model at a few locations throughout the study reach to eliminate ineffective flow areas. Bank stations were established using topographic breaks identified on the cross sections and vegetation lines identified on the aerial photographic base for the 2000 mapping.

Manning's *n* roughness coefficients of 0.034 and 0.044 were used for the main channel and overbank areas, respectively. The main channel value of 0.034 is based on a base *n*-value of 0.020 for upper regime flow plus appropriate adjustments to account for the presence of obstructions, vegetation along the banks, and nonlinearity and nonuniformity of the channel. The overbank *n*-value was selected to represent the effects of the generally sparse vegetation. These values are consistent with the values used to model the arroyo downstream of Swinburne Dam (MEI, 1996a and 1998).

The HEC-RAS model was initially run in both subcritical and supercritical modes which indicated that critical or near critical flow conditions will occur through most of the study reach upstream from GCS#1. The subcritical results were used in the subsequent analyses because supercritical flow conditions are not sustained in sand-bed channels except for very short durations over very short distances (Trieste, 1992; Mussetter et al., 1994).

To evaluate the effects of the design, the hydraulic analysis was performed for each of the following scenarios:

Scenario 1. Existing channel geometry under existing conditions hydrology.
Scenario 2. Existing channel geometry under 2036 development conditions hydrology.
Scenario 3. Design channel geometry at the existing grade under 2036 development conditions hydrology.
Scenario 4. Design channel geometry at the equilibrium slope under 2036 development conditions hydrology.

Scenarios 1 and 2 were selected to establish a baseline with which to compare the results under design conditions. Scenario 3 was analyzed to evaluate the effects of the protective measures soon after completion of the project, which represents worst-case conditions with respect to flooding. Scenario 4 was evaluated to determine the hydraulic characteristics with the protective measures in place at the ultimate gradient to which the channel is expected to develop, based on the equilibrium slope that was estimated during the prudent line study (MEI, 2000).

Scenario 3 was initially modeled based on the design features shown on Map 1 (Appendix A), and results from the initial model and stability analysis were used by Goodwin and Wilson to develop the final plan (Map 2, Appendix A). The Scenario 3 model was subsequently modified to reflect the modified design, which includes a trapezoidal channel in the reach between the upstream face of McMahon Boulevard bridge and the upstream project limit. Because the alignment of the channel under the modified design is somewhat different from existing conditions, a revised station line was developed to determine the distance between cross-sections, evaluate the required drop heights at the grade-control structures, and locate the design features. A comparison of the equivalent stationing under existing and final design conditions is provided in **Table 3.1**, and **Figure 3.1** shows the longitudinal profile of the design reach based on the revised stationing. The following discussion of the hydraulic models and computed hydraulic conditions through the design reach refer to the final design plan.

Table 3.1. Summary of longitudinal stationing under existing and design conditions.

Existing Conditions Stationing	Design Conditions Stationing
198+00	198+00
255+00	255+00
263+17	262+82
264+10	263+51
265+30	264+71
267+97	267+65
268+70	268+38
270+72	270+57
273+36	273+11
278+00	277+71
281+18	280+90
284+17	283+85
288+24	287+84
289+83	289+38
291+40	290+92
293+89	293+30
305+00	304+18
400+00	399+18

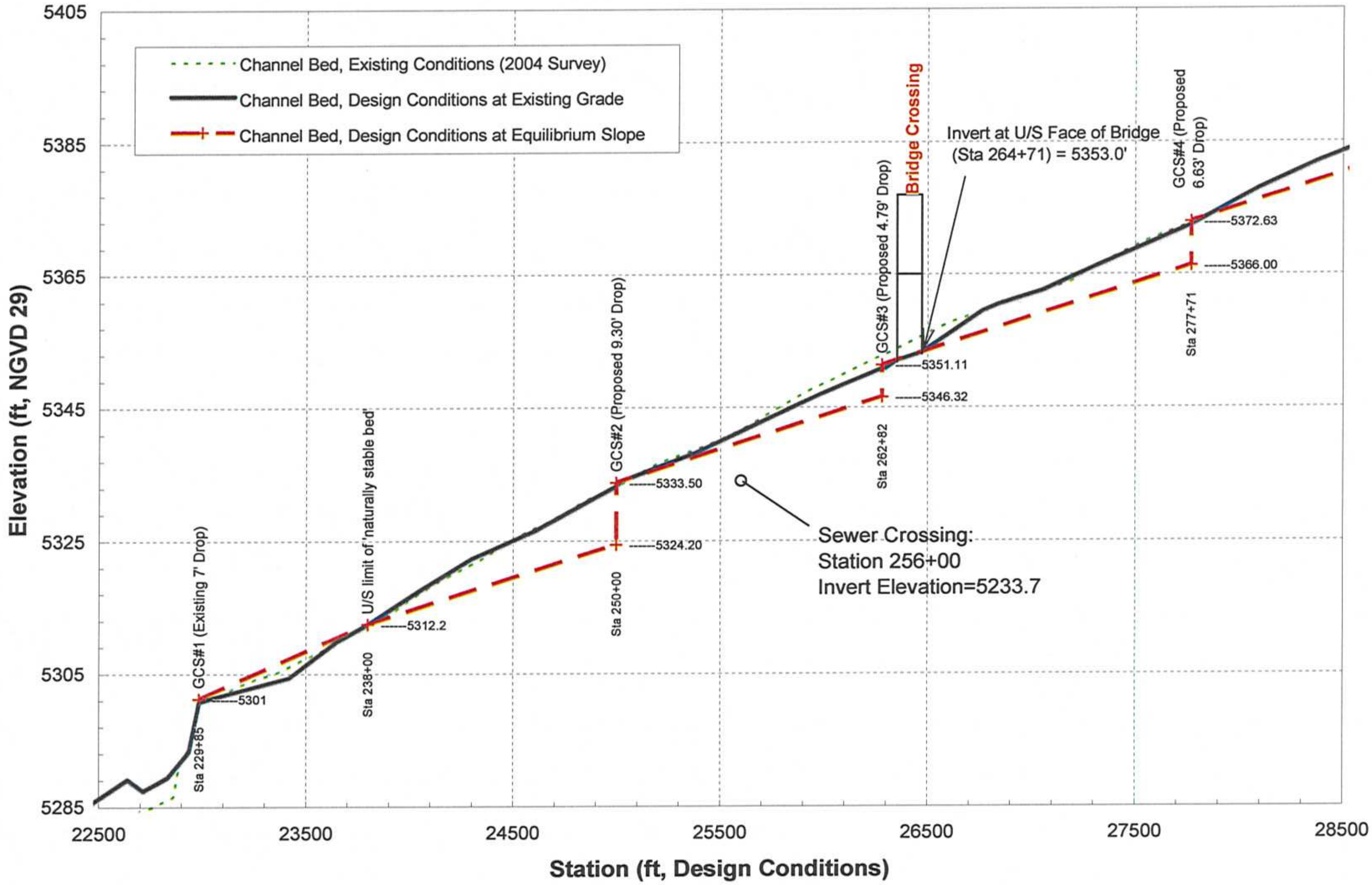

Figure 3.1. Longitudinal profiles of the existing channel bed, the channel bed under design conditions at the existing grade, and the channel bed under design conditions at the equilibrium slope. The location and elevation of the design grade control structures, as well as McMahon Boulevard Bridge is also shown.

In the portion of the project reach upstream from McMahon Boulevard, the top-of-bank elevation was set at two feet above the maximum 100-year water-surface elevation (Scenario 3), and the channel top width was established based on the top-of-bank alignment shown in Map 2 (Appendix A). The cross-sectional geometry of the spur sections in this reach included a 2H:1V sloping nose with a top-of-nose elevation coincident with the design top-of-bank. With this configuration, the spurs do not project into the channel. In-channel Manning's *n*-values for design conditions were reduced to 0.025 in this portion of the project reach to account for removal of the channel irregularities under the design channel alignment. The design features in the portion of the project reach downstream from McMahon Boulevard Bridge that were incorporated into the model consisted of the right bank riprap between Sta 256+60 and Sta 262+82, GCS#2, and Spurs 1R and 3L. Since Spurs 1R and 3L projected from the existing bank to the specified nose location (unlike the spurs upstream of the bridge crossing), the existing cross-sectional geometry was modified to include the spur geometry. For the portions of the hydraulic model that extended up- or downstream of the project reach, the existing channel geometry was used.

The modeled geometry of the grade-control structures included a 30-foot wide, 6-inch deep notch below the crest elevation with a crest thickness (parallel with the direction of flow) of 15 feet. The overall crest width of the structures was 140 feet, except at GCS#3, where a 100-foot crest width was used to coincide with the bottom width through the bridge opening. The geometry through the bridge is a 7-foot deep, rectangular channel with a bottom width of 100 feet (perpendicular to the direction of flow) that was sized to convey the 10-year peak flow (**Figure 3.2**). The rectangular channel through the bridge is bound by a 15-foot wide bench along the left bank, a 12-foot wide bench along the right bank, and 2H:1V sloping abutments. The invert elevation at the upstream face of the bridge is 5353.0 feet, with a 1 percent slope extending downstream to the crest of GCS#3.

The model for Scenario 4 was developed by lowering the bed elevations to the estimated equilibrium slope and assuming that the incised channel will have 2H:1V sideslopes.

3.2. Hydraulic Model Results

The design channel profile (i.e., pre-incision) under 2036 development conditions hydrology represents worst-case conditions with respect to flooding along the reach. Predicted water-surface elevations from the Scenario 2 (existing conditions topography, 2036 hydrology) and Scenario 3 (design conditions topography, 2036 hydrology) models were compared to evaluate the potential effect of the project on flood elevations during the 10-year and 100-year peak flows (**Figures 3.3a and 3.3b**, respectively). These results indicate that the 10-year water-surface elevations would be about 1.0 feet higher in the vicinity of Sta 245+00, about 0.8 feet higher at GCS#2 and about 0.5 feet higher near Sta 257+00, about 700 feet upstream from GCS#2 under design conditions (Figure 3.3a). At all other locations, the 10-year water-surface profile is at or below the existing conditions profile. Similar changes differences occur for the 100-year peak flow in the reach downstream from McMahon Bridge, with the water-surface elevations about 1.0 foot higher between Sta 243+00 and Sta 245+00, about 1.4 feet higher at GCS#2, and about 0.8 feet higher near Sta 257+00. At flows greater than the 10-year event, McMahon Bridge also causes upstream backwater effects, with a maximum increase in water-surface elevation over existing conditions of about 4.0 feet at the upstream face of the bridge during the 100-year peak flow (Figure 3.3b). The backwater effect under these conditions extends about 370 feet upstream from the bridge. The design 100-year water-surface elevation is also about 1.0 foot higher just upstream from GCS#4 than it is under existing conditions. At all other locations, the design 100-year water-surface elevation is at or below the existing conditions elevations.

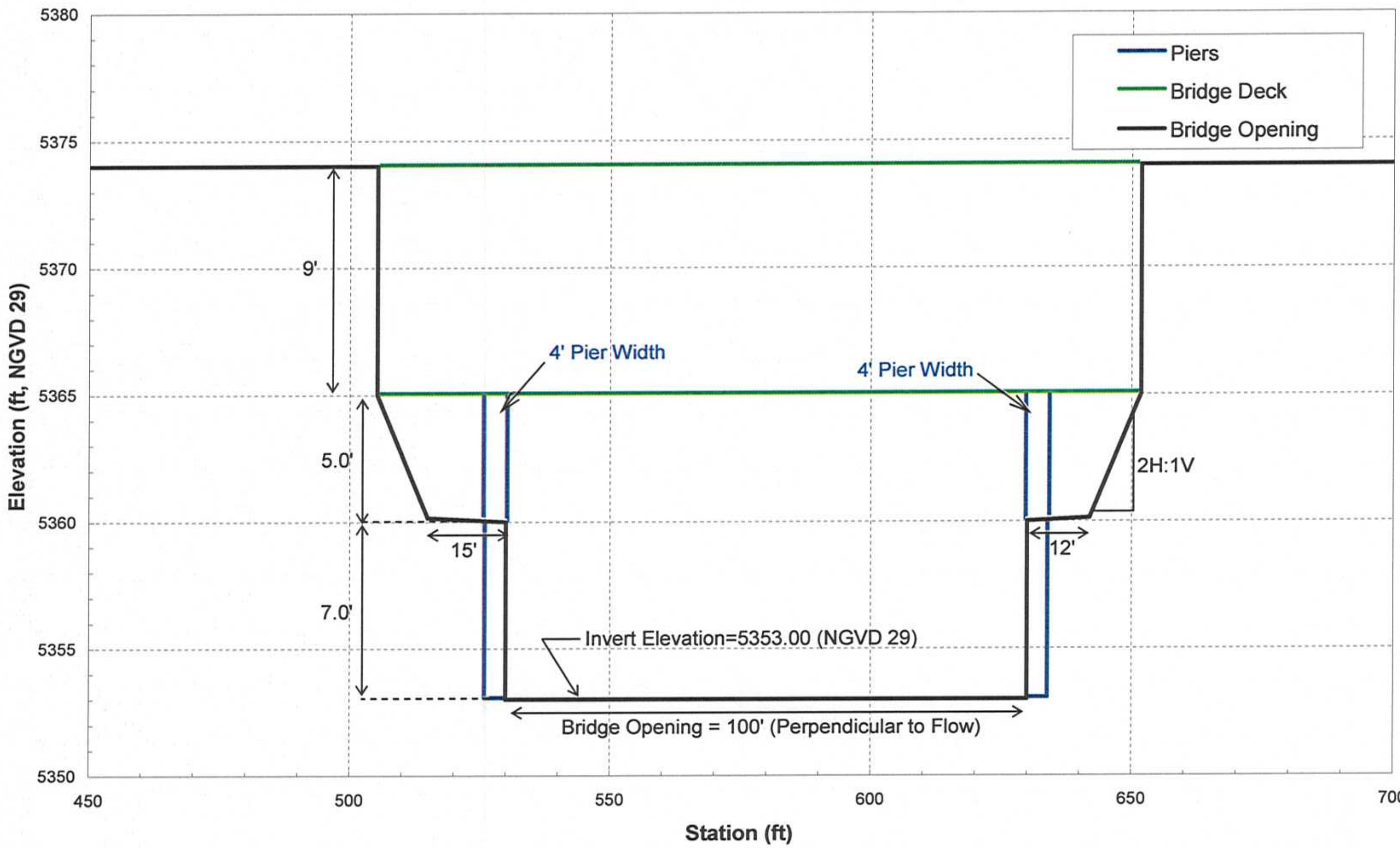

Figure 3.2. Cross-sectional geometry at the upstream face of the proposed McMahon Boulevard bridge opening.

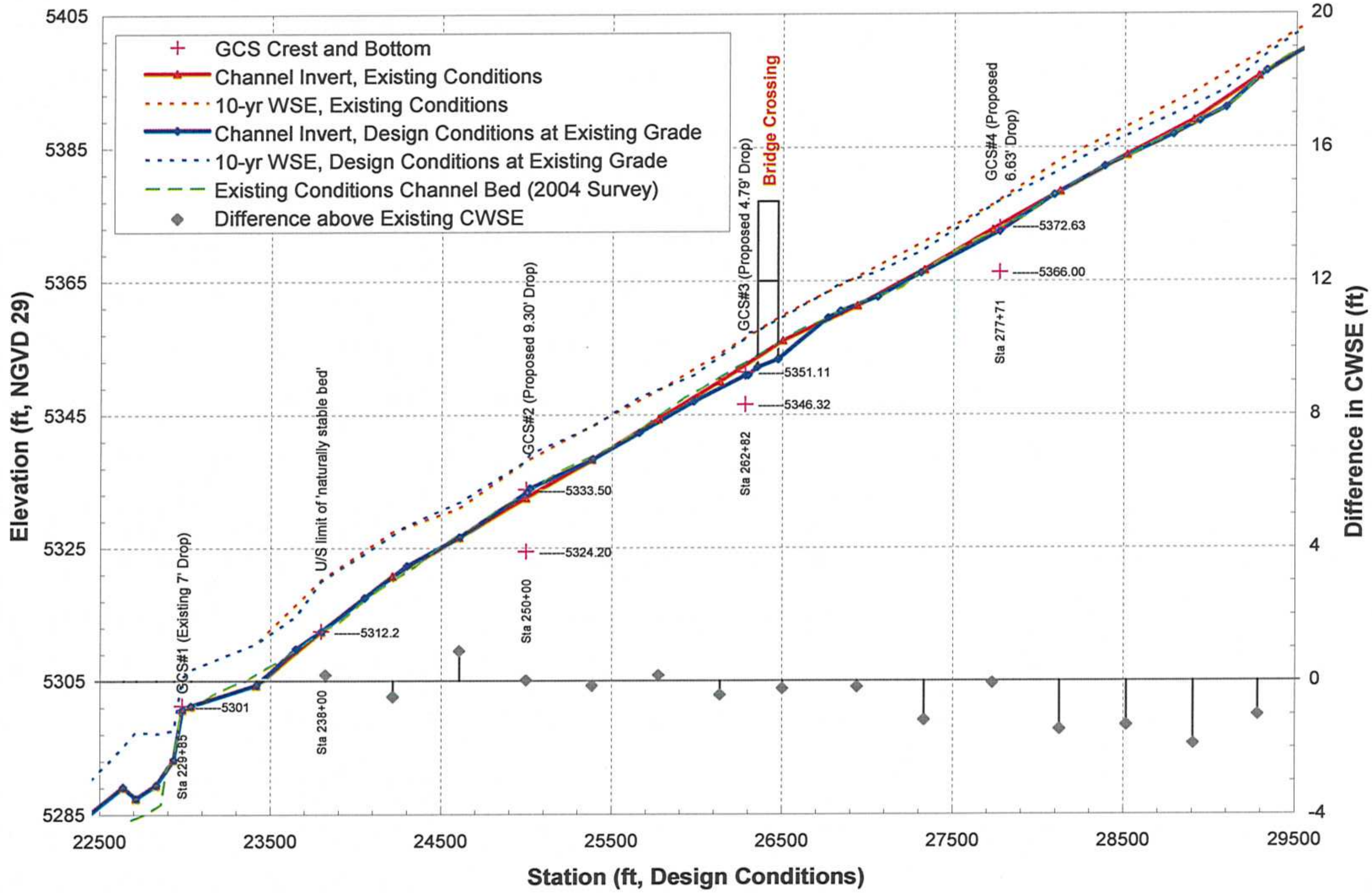

Figure 3.3a. Channel bed and computed water-surface profiles through the project reach for existing conditions and design conditions at existing grade for the 10-year (unbulked) 2036 development conditions peak.

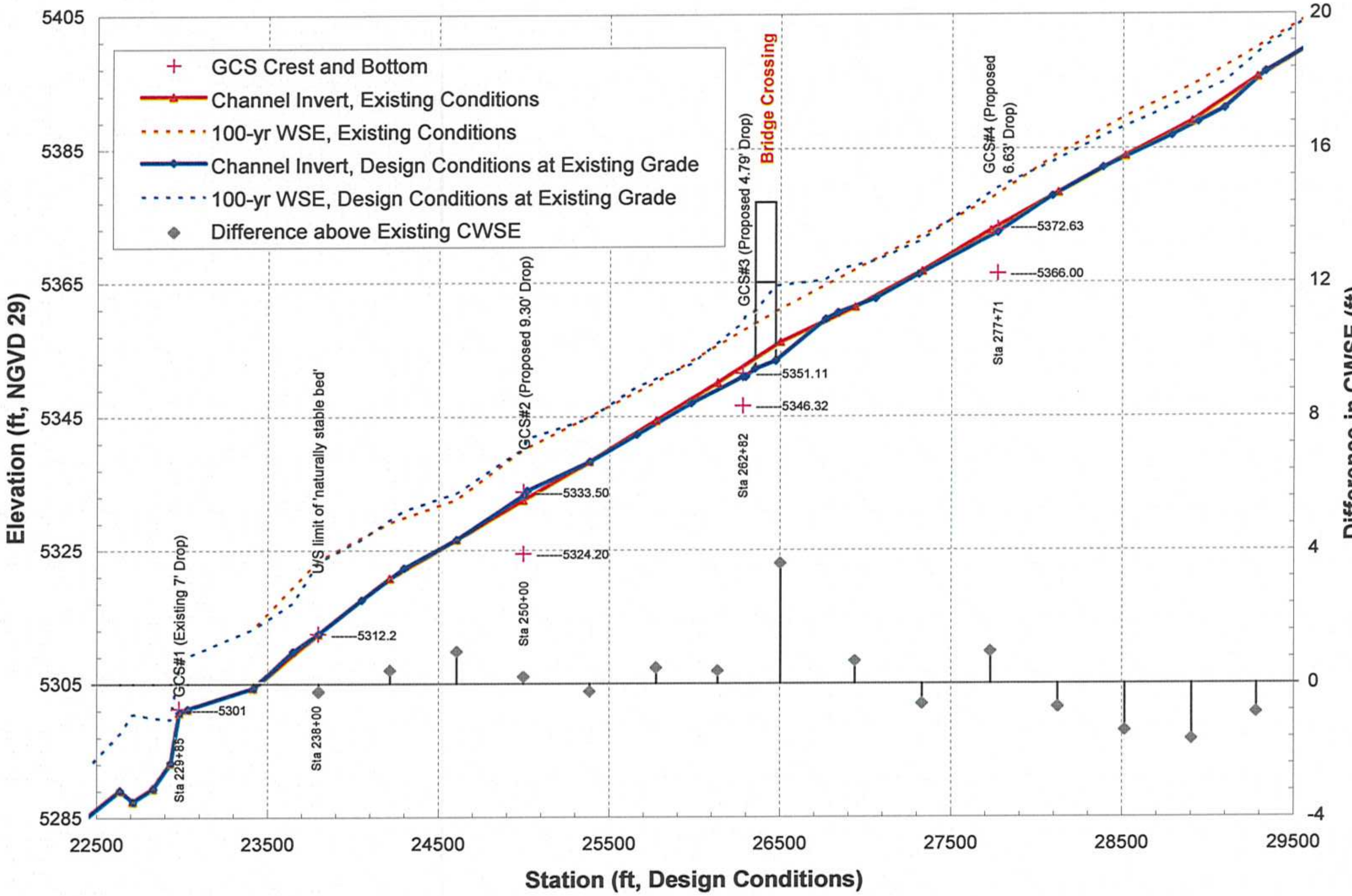

Figure 3.3b. Channel bed and computed water-surface profiles through the project reach for existing conditions and design conditions at existing grade for the 100-year (unbulked) 2036 development conditions peak.

Figure 3.4 shows computed main channel velocity profiles for existing conditions (Scenario 2), design conditions at existing grade (Scenario 3), and design conditions at the equilibrium slope (Scenario 4) at the 100-year (unbulked), 2036 development conditions peak. The profiles indicate that the main channel velocity at each of the grade-control structures is higher under design conditions, even with the design profile where there is no drop across the structure, because the channel is constricted at these locations. Upstream from McMahon Bridge (Sta 264+72), the velocities under the design conditions are also generally higher than existing conditions due to the narrower channel (**Figure 3.5**). Downstream of the bridge crossing, where the channel is naturally constricted by the terraces on both banks, the impact of the spurs on velocity is less significant.

To facilitate analysis of the vertical and lateral stability of the channel under existing and design conditions, the study reach was subdivided into eight computational subreaches based on the location of the proposed grade-control structures, similarity of geomorphic characteristics, and the location of significant tributaries (**Table 3.2**). The subreach limits are also shown on Map 2 (Appendix A). Hydraulic results for the portion of the model within each subreach were used to estimate reach-averaged hydraulic conditions over the range of modeled flows for use in the sediment- continuity analysis (Chapter 4). For each of the model scenarios, average values of main channel velocity, hydraulic depth, effective width, and energy slope were computed for each hydraulic subreach over a range of discharges through the peak of the 2036 development conditions 100-year event (14,140 cfs at existing GCS#1) (**Tables 3.3 through 3.5**). Under the design conditions scenarios, the cross sections at the crest of the grade-control structures (and at the base of the grade-control structures in the equilibrium slope [Scenario 4] model) and through the bridge were not included in the averaging because they do not represent typical conditions in erodible portions of the reach.

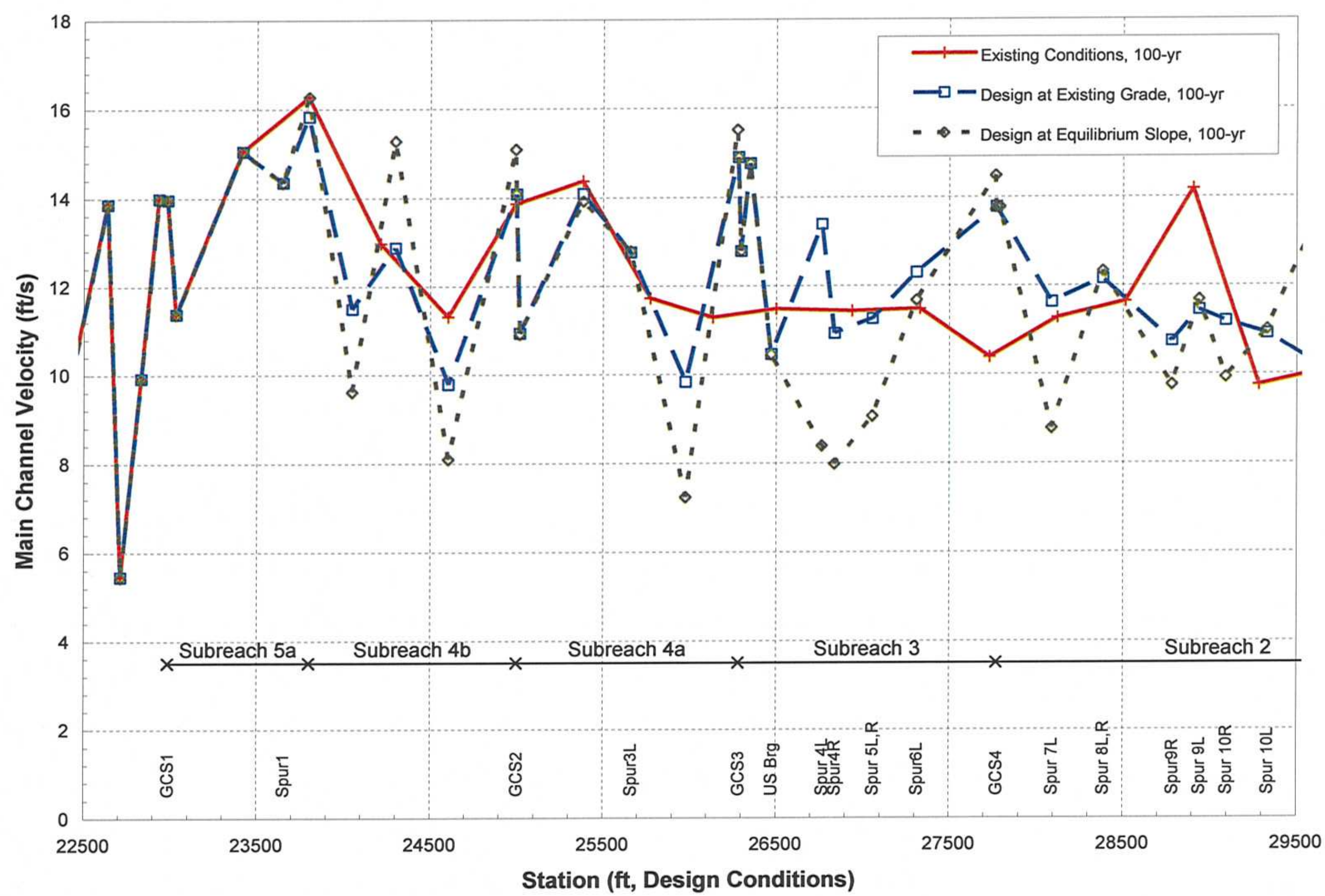

Figure 3.4. Main channel velocity profiles for existing conditions, design conditions at existing grade, and design conditions at the equilibrium slope for the 100-year (unbulked) 2036 development conditions peak.

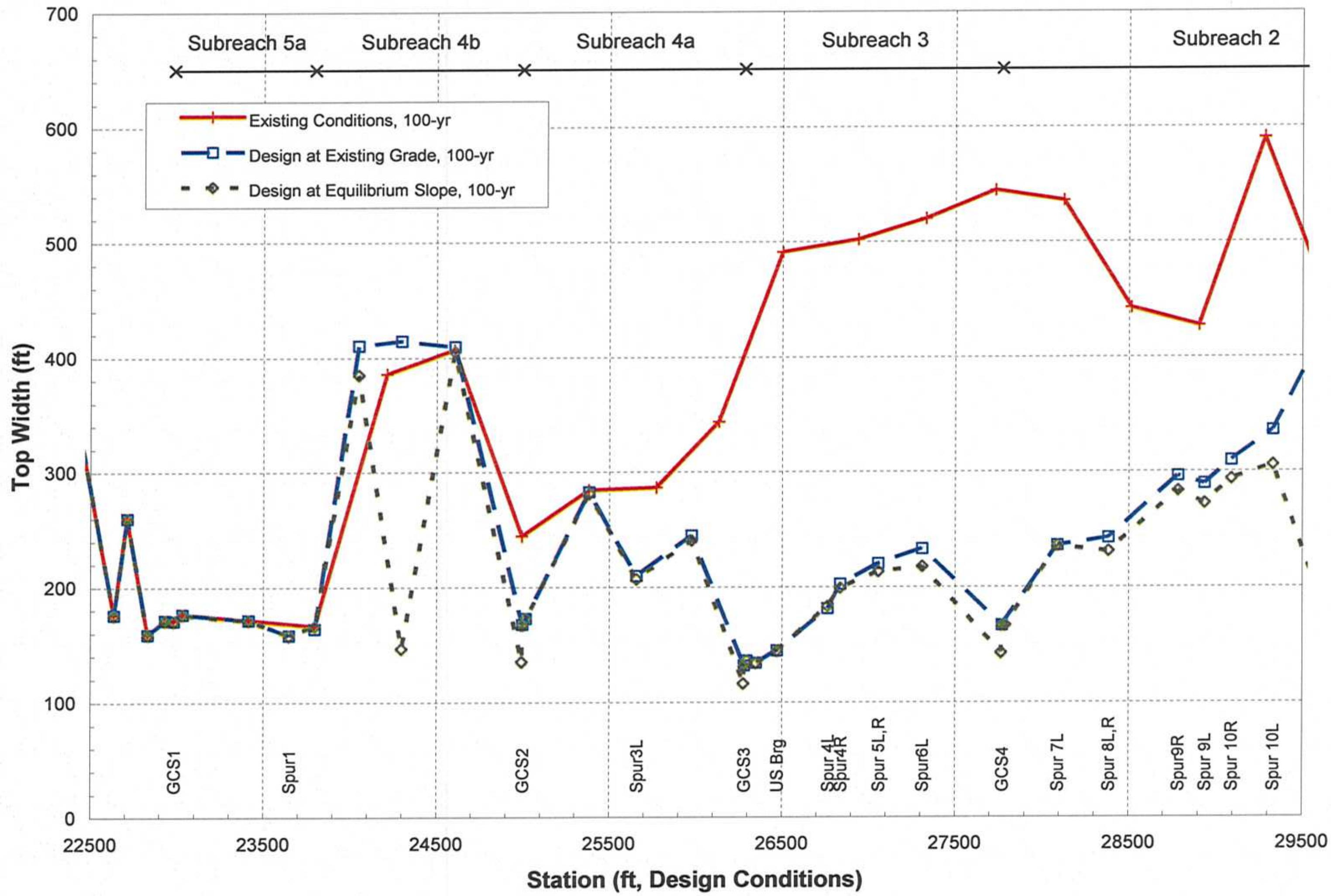

Figure 3.5. Topwidth profiles for existing conditions, design conditions at existing grade, and design conditions at the equilibrium slope for the 100-year (unbulked) 2036 development conditions peak.

Table 3.2. Summary of subreach limits for reach-averaged hydraulic conditions for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam.

Subreach	Downstream Limit	Downstream Station (ft)*	Upstream Limit	Upstream Station (ft)*	Length (ft)*
1	West bank tributary at Sta 302+50	30,250	0.25 miles upstream of Sandoval County Line	32,444	2,194
2	GCS#4 Crest	27,771	West bank tributary at Sta 302+50	30,250	2,479
3	GCS#3 Crest	26,282	GCS#4 Crest	27,771	1,489
4a	GCS#2 Crest	25,000	GCS#3 Crest	26,282	1,282
4b	Upstream limit of naturally stable (caliche) bed	23,800	GCS#2 Crest	25,000	1,200
5a	GCS#1 Crest	22,985	Upstream limit of naturally stable (caliche) bed	23,800	815
5b	Confluence with West Branch	22,150	GCS#1 Crest	22,985	835
6	Swinburne Dam	19,845	Confluence with West Branch	22,150	2,305

*From design conditions station line

Table 3.3. Summary of existing reach-averaged hydraulic conditions for the project reach of the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam.

Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
1	57	2.4	0.3	78	0.0153
	114	2.9	0.4	94	0.0147
	285	3.7	0.6	125	0.0141
	570	4.8	0.9	132	0.0138
	1140	6.2	1.3	137	0.0136
	2850	8.3	2.4	140	0.0114
	5710	9.9	3.6	142	0.0094
	7420	10.1	4.2	142	0.0079
	9700	11.2	4.7	142	0.0082
2	78	2.4	0.3	107	0.0151
	156	3.0	0.4	120	0.0150
	390	4.1	0.7	143	0.0148
	780	5.0	0.9	175	0.0149
	1560	6.2	1.2	202	0.0150
	3900	7.9	2.0	236	0.0130
	7800	9.4	3.0	245	0.0108
	10140	10.2	3.5	245	0.0103
	13260	11.3	4.0	245	0.0105
3	78	2.7	0.3	91	0.0166
	156	3.3	0.5	103	0.0155
	390	4.3	0.7	123	0.0143
	780	5.2	1.0	144	0.0134
	1560	6.3	1.4	161	0.0127
	3900	7.7	2.3	183	0.0104
	7800	9.6	3.2	184	0.0103
	10140	10.3	3.6	184	0.0098
	13260	11.2	4.2	184	0.0097
4a	80	2.8	0.4	73	0.0138
	160	3.4	0.5	86	0.0138
	401	4.4	0.7	126	0.0154
	802	5.2	0.9	166	0.0158
	1603	6.3	1.3	189	0.0141
	4008	8.4	2.3	200	0.0123
	8017	10.7	3.5	200	0.0113
	10422	11.5	4.1	200	0.0104
	13628	12.5	4.8	200	0.0099

Table 3.3. Summary of existing reach-averaged hydraulic conditions for the project reach of the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam (continued).					
Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
4b	85	2.9	0.3	87	0.0180
	169	3.5	0.5	104	0.0186
	423	4.8	0.8	115	0.0170
	845	5.8	1.1	134	0.0157
	1690	7.2	1.6	142	0.0142
	4225	9.4	2.9	149	0.0112
	8450	10.9	4.5	153	0.0082
	10985	11.9	5.2	153	0.0083
	14365	12.7	6.1	153	0.0075
5a	85	2.3	0.3	112	0.0117
	169	2.9	0.5	114	0.0107
	423	4.0	0.9	118	0.0099
	845	5.1	1.4	122	0.0092
	1690	6.5	2.0	127	0.0086
	4225	8.9	3.5	134	0.0078
	8450	11.1	5.3	142	0.0069
	10985	12.2	6.3	143	0.0067
	14365	13.2	7.4	146	0.0063
5b	85	1.8	0.4	127	0.0059
	169	2.2	0.6	131	0.0051
	423	2.9	1.0	144	0.0045
	845	3.9	1.5	150	0.0047
	1690	5.1	2.1	160	0.0051
	4225	6.6	2.9	223	0.0055
	8450	8.3	4.2	241	0.0052
	10985	9.0	5.0	245	0.0050
	14365	9.1	6.2	254	0.0038
6	100	1.9	0.2	285	0.0166
	200	2.3	0.3	305	0.0144
	500	2.8	0.5	328	0.0090
	1000	3.2	0.8	370	0.0066
	2000	3.0	1.5	437	0.0025
	5000	2.9	3.2	536	0.0009
	10000	1.9	8.6	602	0.0001
	13000	1.7	12.1	630	0.0001
	17000	1.7	15.6	638	0.0000

Table 3.4. Summary of reach-averaged hydraulic conditions for design conditions at the existing grade scenario (Scenario 3).

Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
1	57	2.5	0.3	71	0.0148
	114	2.9	0.4	94	0.0144
	285	3.7	0.6	125	0.0141
	570	4.8	0.9	132	0.0139
	1140	6.2	1.3	137	0.0136
	2850	8.3	2.4	140	0.0114
	5710	9.9	3.6	142	0.0094
	7420	10.1	4.2	142	0.0079
	9700	11.2	4.7	142	0.0082
2	78	1.9	0.2	226	0.0187
	156	2.5	0.3	231	0.0175
	390	3.4	0.5	243	0.0157
	780	4.3	0.7	253	0.0147
	1560	5.4	1.1	267	0.0137
	3900	7.4	1.8	287	0.0129
	7800	9.4	2.8	297	0.0115
	10140	10.2	3.3	298	0.0109
	13260	11.1	4.0	300	0.0102
3	78	2.0	0.2	186	0.0156
	156	2.6	0.3	186	0.0154
	390	3.6	0.6	187	0.0146
	780	4.7	0.9	188	0.0132
	1560	6.0	1.4	190	0.0121
	3900	8.2	2.5	195	0.0105
	7800	10.2	3.8	201	0.0091
	10140	11.1	4.5	204	0.0086
	13260	12.0	5.3	208	0.0080
4a	83	2.4	0.3	130	0.0166
	166	3.1	0.4	131	0.0161
	415	4.4	0.7	133	0.0153
	829	5.6	1.1	134	0.0142
	1658	6.9	1.8	137	0.0117
	4144	8.6	2.9	164	0.0092
	8288	10.8	4.5	171	0.0082
	10774	11.7	5.3	174	0.0078
	14089	12.6	6.3	177	0.0071

Table 3.4. Summary of reach-averaged hydraulic conditions for design conditions at the existing grade scenario (Scenario 3) (continued).

Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
4b	85	2.7	0.3	95	0.0174
	169	3.5	0.5	106	0.0175
	423	4.7	0.8	116	0.0165
	845	5.8	1.1	127	0.0150
	1690	7.2	1.7	134	0.0130
	4225	9.4	3.0	148	0.0104
	8450	10.3	5.1	151	0.0063
	10985	11.0	5.9	151	0.0059
	14365	11.4	7.1	151	0.0049
5a	85	2.5	0.3	106	0.0141
	169	3.2	0.5	110	0.0139
	423	4.3	0.8	119	0.0124
	845	5.4	1.3	124	0.0113
	1690	6.8	1.9	129	0.0103
	4225	9.3	3.3	136	0.0089
	8450	11.5	5.1	144	0.0079
	10985	12.5	6.0	146	0.0076
	14365	13.6	7.0	149	0.0071
5b	85	1.8	0.4	127	0.0059
	169	2.2	0.6	131	0.0051
	423	2.9	1.0	144	0.0045
	845	3.9	1.5	150	0.0047
	1690	5.1	2.1	160	0.0051
	4225	6.6	2.9	223	0.0055
	8450	8.3	4.2	241	0.0052
	10985	9.0	5.0	245	0.0050
	14365	9.1	6.2	254	0.0038
6	100	1.9	0.2	285	0.0166
	200	2.3	0.3	305	0.0144
	500	2.8	0.5	328	0.0090
	1000	3.2	0.8	370	0.0066
	2000	3.0	1.5	437	0.0025
	5000	2.9	3.2	536	0.0009
	10000	1.9	8.6	602	0.0001
	13000	1.7	12.1	630	0.0001
	17000	1.7	15.6	638	0.0000

Table 3.5. Summary of reach-averaged hydraulic conditions for design conditions at the equilibrium slope scenario (Scenario 4).

Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
1	57	2.4	0.3	78	0.0153
	114	2.9	0.4	94	0.0147
	285	3.7	0.6	125	0.0141
	570	4.8	0.9	132	0.0138
	1140	6.2	1.3	137	0.0136
	2850	8.3	2.4	140	0.0114
	5710	9.9	3.6	142	0.0094
	7420	10.1	4.2	142	0.0079
	9700	11.2	4.7	142	0.0082
2	78	2.4	0.3	107	0.0151
	156	3.0	0.4	120	0.0150
	390	4.1	0.7	143	0.0148
	780	5.0	0.9	175	0.0149
	1560	6.2	1.2	202	0.0150
	3900	7.9	2.0	236	0.0130
	7800	9.4	3.0	245	0.0108
	10140	10.2	3.5	245	0.0103
	13260	11.3	4.0	245	0.0105
3	78	2.7	0.3	91	0.0166
	156	3.3	0.5	103	0.0155
	390	4.3	0.7	123	0.0143
	780	5.2	1.0	144	0.0134
	1560	6.3	1.4	161	0.0127
	3900	7.7	2.3	183	0.0104
	7800	9.6	3.2	184	0.0103
	10140	10.3	3.6	184	0.0098
	13260	11.2	4.2	184	0.0097
4a	80	2.8	0.4	73	0.0138
	160	3.4	0.5	86	0.0138
	401	4.4	0.7	126	0.0154
	802	5.2	0.9	166	0.0158
	1603	6.3	1.3	189	0.0141
	4008	8.4	2.3	200	0.0123
	8017	10.7	3.5	200	0.0113
	10422	11.5	4.1	200	0.0104
	13628	12.5	4.8	200	0.0099

Table 3.5. Summary of reach-averaged hydraulic conditions for design conditions at the equilibrium slope scenario (Scenario 4) (continued).

Subreach	Discharge (cfs)	Main Channel Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)
4b	85	2.9	0.3	87	0.0180
	169	3.5	0.5	104	0.0186
	423	4.8	0.8	115	0.0170
	845	5.8	1.1	134	0.0157
	1690	7.2	1.6	142	0.0142
	4225	9.4	2.9	149	0.0112
	8450	10.9	4.5	153	0.0082
	10985	11.9	5.2	153	0.0083
	14365	12.7	6.1	153	0.0075
5a	85	2.3	0.3	112	0.0117
	169	2.9	0.5	114	0.0107
	423	4.0	0.9	118	0.0099
	845	5.1	1.4	122	0.0092
	1690	6.5	2.0	127	0.0086
	4225	8.9	3.5	134	0.0078
	8450	11.1	5.3	142	0.0069
	10985	12.2	6.3	143	0.0067
	14365	13.2	7.4	146	0.0063
5b	85	1.8	0.4	127	0.0059
	169	2.2	0.6	131	0.0051
	423	2.9	1.0	144	0.0045
	845	3.9	1.5	150	0.0047
	1690	5.1	2.1	160	0.0051
	4225	6.6	2.9	223	0.0055
	8450	8.3	4.2	241	0.0052
	10985	9.0	5.0	245	0.0050
	14365	9.1	6.2	254	0.0038
6	100	1.9	0.2	285	0.0166
	200	2.3	0.3	305	0.0144
	500	2.8	0.5	328	0.0090
	1000	3.2	0.8	370	0.0066
	2000	3.0	1.5	437	0.0025
	5000	2.9	3.2	536	0.0009
	10000	1.9	8.6	602	0.0001
	13000	1.7	12.1	630	0.0001
	17000	1.7	15.6	638	0.0000

4. SEDIMENT-CONTINUITY AND VERTICAL STABILITY ANALYSES

A sediment-continuity analysis was performed to determine the potential effects of the design on the vertical stability of the project reach, and to provide input to the lateral stability evaluation (Chapter 5). The analysis was conducted by developing bed material transport capacity rating curves for each of the individual hydraulic subreaches, estimating the bed material supply from upstream and local tributaries, and computing the aggradation/degradation potential of each subreach based on the difference between the supply and transport capacity.

Based on field observations (MEI, 1996b), the arroyo appears to be vertically stable over most of the study reach under existing conditions, although some degradation has occurred downstream from the existing GCS#1 (Figure 3.3a). Future development in the basin will increase the magnitude and frequency of runoff-producing events and reduce the sediment supply to the reach, resulting in a significant increase in the potential for degradation and associated lateral instability.

4.1. Bed-material Sediment-transport Relationships

The bed-material rating curves (i.e., relationships between bed-material transport capacity and discharge) for each hydraulic subreach were developed using the reach-averaged hydraulics predicted by the HEC-RAS models, as discussed in the previous chapter, and the MPM-Woo sediment-transport equation, as presented in the Design Guide (Mussetter et al., 1994). The representative bed material gradation that was used in the analysis was taken from the composite gradation that was previously developed by MEI (1996a and 1998). This gradation has a median (D_{50}) size of 1.05 mm (Figure 1.2).

Using the representative gradation, the bed material transport capacity relationship for the project reach is:

$$q_s = 7.8x10^{-6} V^{5.39} D^{-0.35} (1 - C_f)^{-2.43} \tag{4.1}$$

where q_s = bed-material transport capacity per unit width in cfs/ft
V = flow velocity in fps
D = hydraulic depth in feet
C_f = fine sediment concentration by weight.

Since the bed-material sediment-transport capacity predicted by Equation 4.1 depends on the fine sediment concentration, estimates of fine sediment yields were made for each storm event under both pre-development and 2036 development conditions. The estimates for pre-development conditions were made using the Modified Universal Soil Loss Equation (MUSLE), as adapted to the Albuquerque area in the Design Guide. Details of the calculations are summarized in **Appendix B**. To establish the fine sediment concentrations for 2036 development conditions, it was assumed that fine sediment yields would decrease from the existing conditions values in proportion to the relative increase in impervious area associated with urbanization. Based on the projected basin development from the Calabacillas Arroyo Drainage Management Plan (RTI, 1987), the impervious area in the portion of the Main Branch basin upstream from the West Branch was assumed to increase from zero percent under pre-development conditions to 12.5 percent under 2036 conditions. This implies that the fine sediment yields would decrease by 12.5 percent under 2036 development conditions. Since the computed fine sediment concentrations do not vary significantly between events for a particular condition, and since Equation 4.1 is relatively insensitive to C_f, average annual values of C_f were used in Equation 4.1 to compute the bed-material transport rates over the range of flows that were evaluated. The resulting fine sediment concentrations used in the analysis were 26,560 ppm for pre-development conditions, and 13,160 ppm for 2036 development conditions.

The bed-material sediment rating curves for each hydraulic subreach and each scenario were developed by substituting the reach-averaged hydraulics into Equation 4.1 (**Figures 4.1 through 4.5**). As illustrated in Figures 4.1 through 4.5, the rating curves are relatively insensitive to the predicted change in fine sediment yield associated with future development. In Subreaches 2 and 3, the transport capacity is lower under Scenario 3 (design conditions at existing grade) than under Scenarios 1 and 2 (existing conditions) at discharges below 2,000 to 4,000 cfs because the natural low-flow channel was removed in the design. At higher discharges, the transport capacity is higher under Scenario 3 because the channel is somewhat narrower. In Subreaches 4a and 4b, the transport capacities are similar for flows up to 3,000 to 4,000 cfs, but somewhat lower at higher flows. The decrease in channel gradient under Scenario 4 (design conditions at the equilibrium slope) causes a significant decrease in transport capacities in Subreaches 2 through 4b over the entire range of flows that were evaluated. In Subreach 5a, the transport capacities are higher under Scenario 3 than under Scenarios 1 and 2 due to the presence of spurs that constrict the channel, and the transport capacities for Scenario 4 are very similar to those under Scenario 3 since the vertical incision is limited by the erosion-resistant bed in this reach.

4.2. Sediment Continuity

The sediment continuity analysis was performed by integrating the bed-material rating curves for each subreach over the appropriate hydrograph to obtain a transport volume. The resulting volume was then compared on a subreach-by-subreach basis with the upstream supply. If the bed-material transport capacity in a particular subreach for a given storm exceeds the supply, degradation is indicated; if the supply exceeds the capacity, aggradation is indicated. The total supply to a given subreach includes the supply from the upstream channel and from any tributaries along the subreach. There are several tributaries along the study reach that deliver sediment to the mainstem. Ideally, bed-material sediment supply would be estimated by computing the transport capacity for an adjacent upstream reach that is judged to be in equilibrium. Adequate information was not available to use this approach, and an alternative approach that is described in the following sections was, therefore, taken.

4.2.1. Pre-development Conditions Hydrology (Scenario 1)

As discussed in Section 1.3, with the exception of degradation just upstream of the Swinburne Dam pool, most of the study reach appears to be in equilibrium under existing conditions. The hydraulic analysis and preliminary sediment-transport calculations for Subreaches 2 and 3 under existing conditions support this observation. Assuming these reaches to be in equilibrium, the bed-material sediment yield per unit area to the upstream end of Subreach 2 was computed by dividing the average bed material transport capacity of Subreaches 2 and 3 by the total drainage area. The existing conditions unit bed-material sediment yields at the upstream end of the project reach and for the tributaries were set equal to the computed yield at the upstream end of Subreach 2. The existing conditions total bed material sediment supplies from upstream and tributary sources were then estimated by multiplying the unit yields by the respective drainage areas.

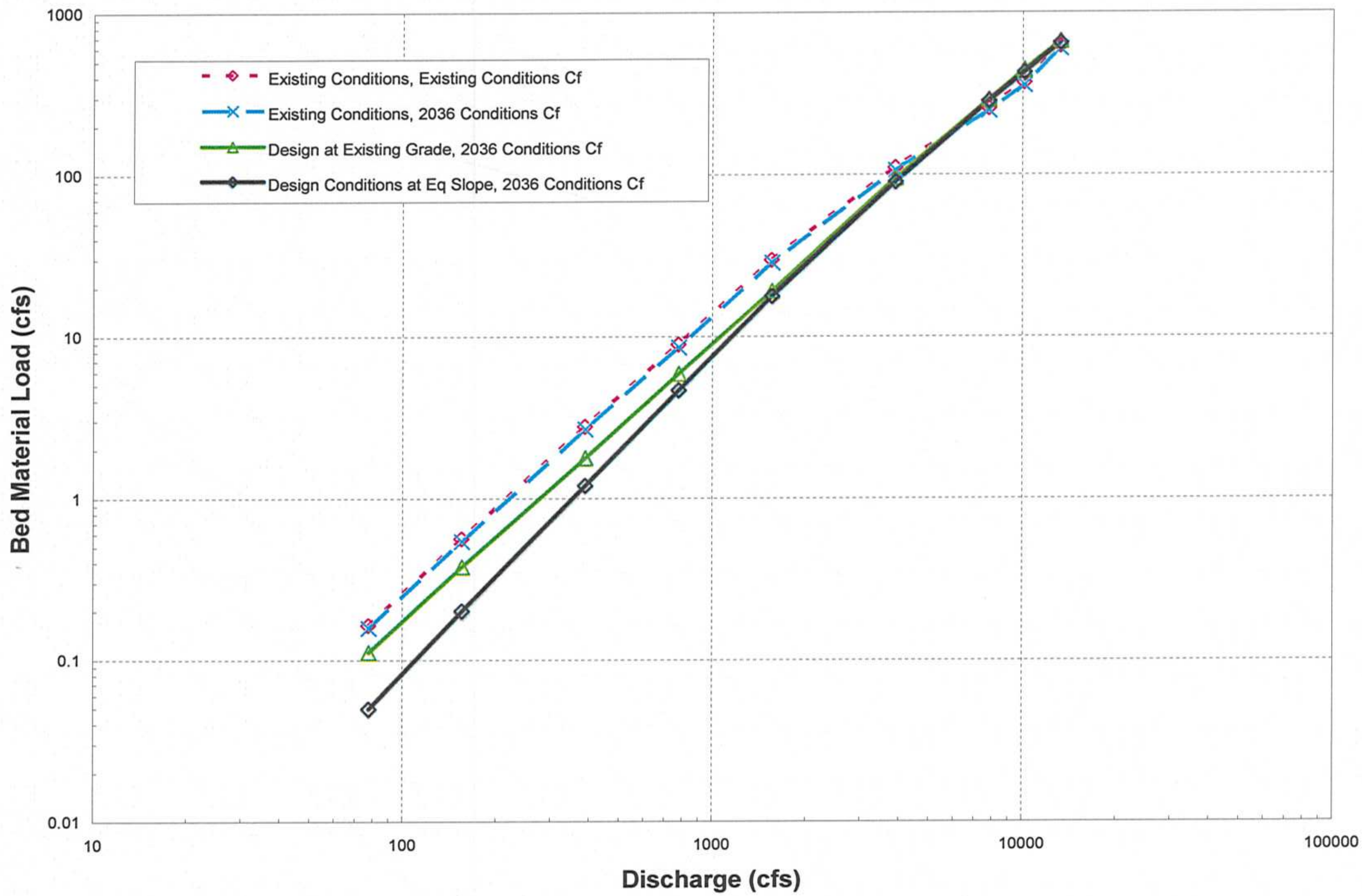

Figure 4.1. Bed-material sediment-rating curves for Subreach 2 under Scenarios 1 through 4.

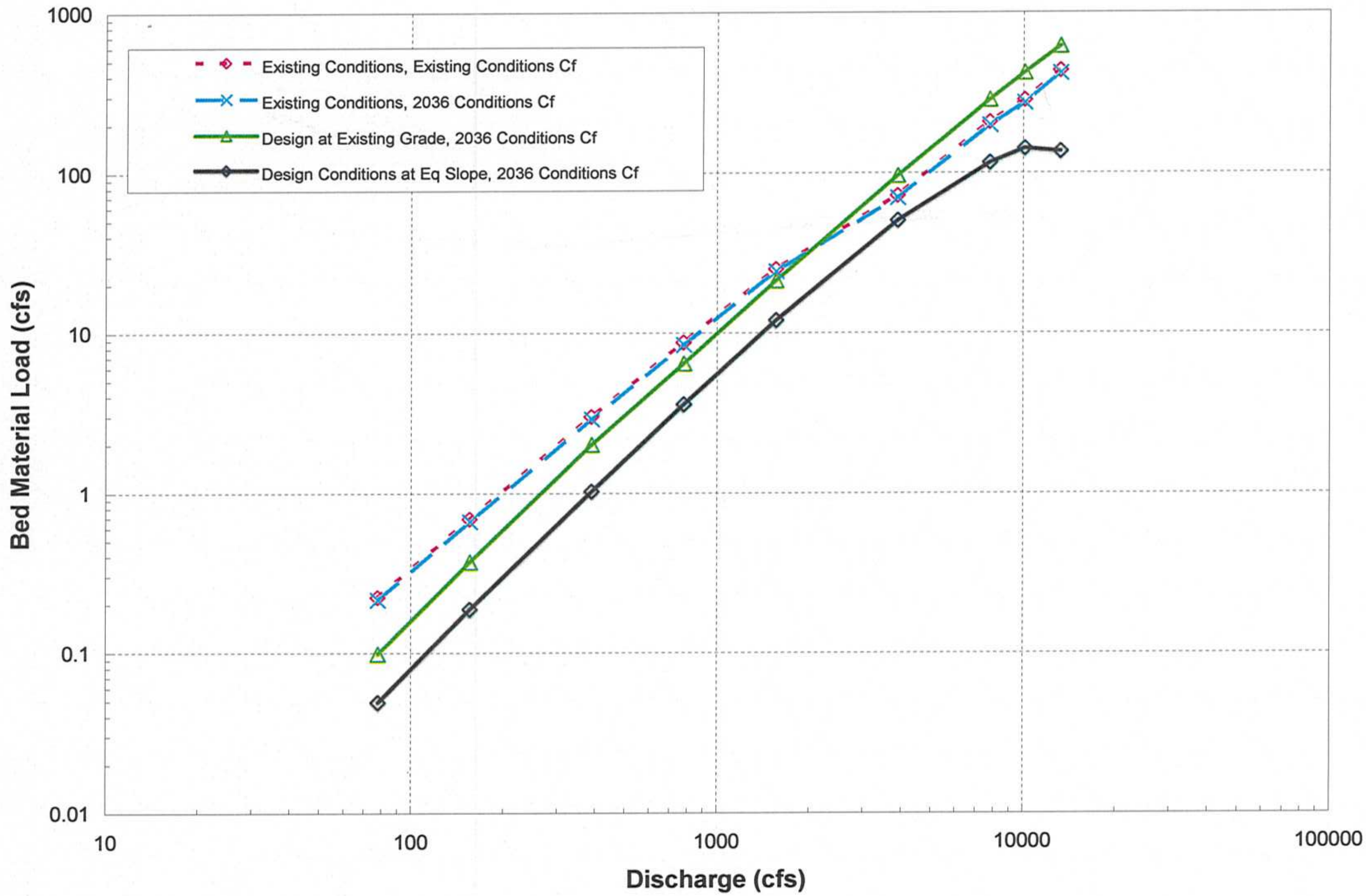

Figure 4.2. Bed-material sediment-rating curves for Subreach 3 under Scenarios 1 through 4.

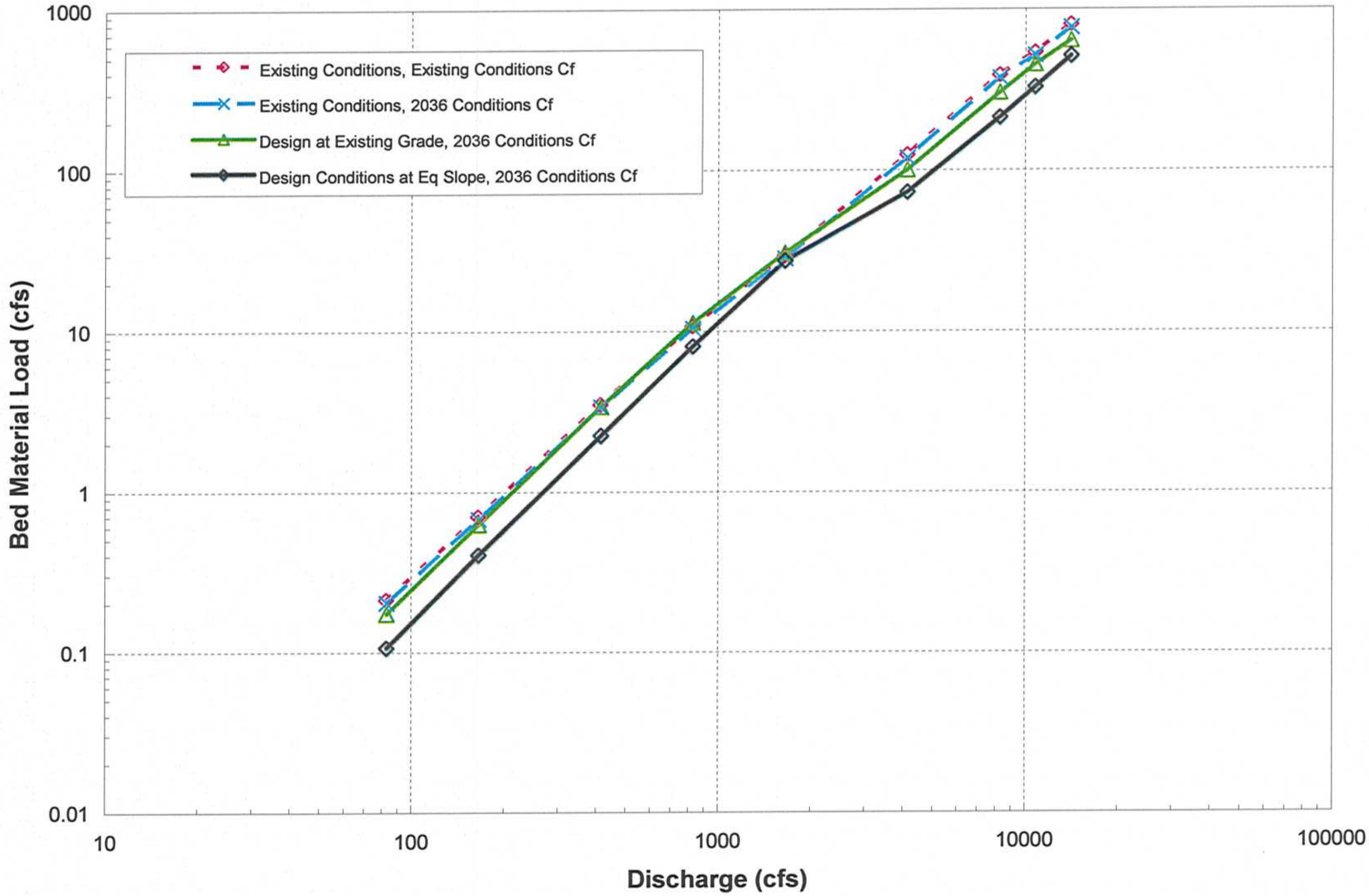

Figure 4.3. Bed-material sediment-rating curves for Subreach 4a under Scenarios 1 through 4.

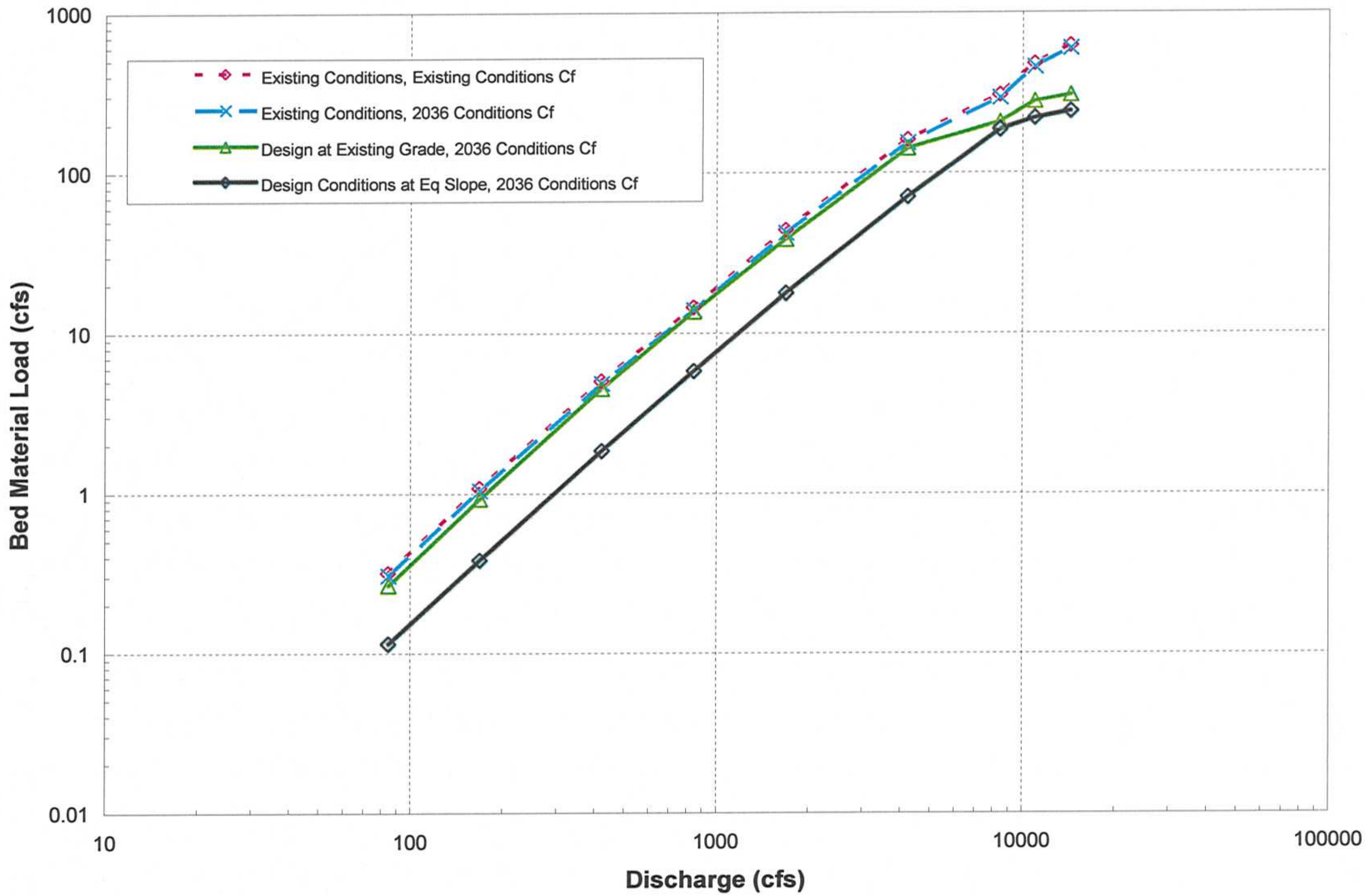

Figure 4.4. Bed-material sediment-rating curves for Subreach 4b under Scenarios 1 through 4.

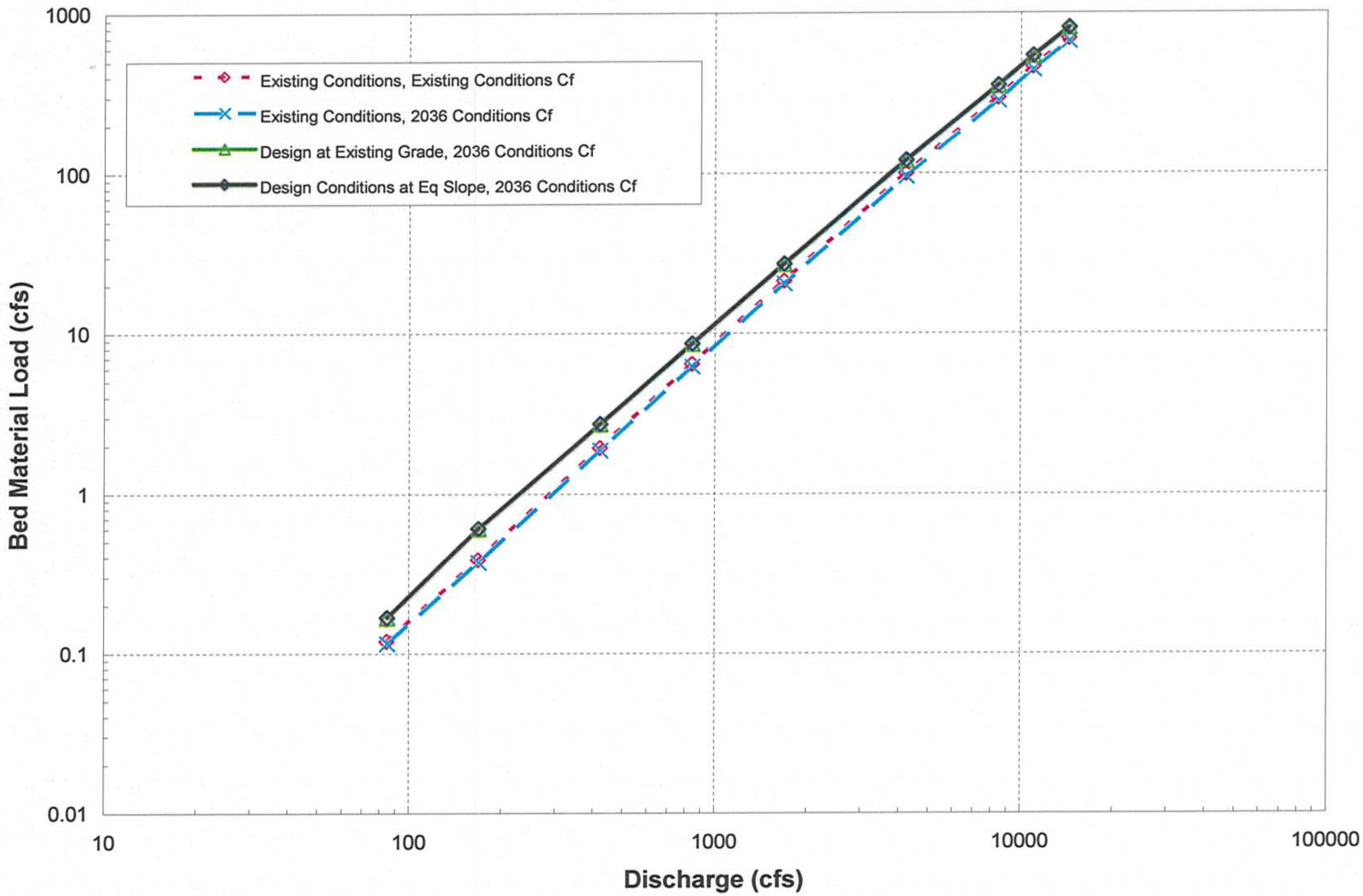

Figure 4.5. Bed-material sediment-rating curves for Subreach 5a under Scenarios 1 through 4.

The estimated bed material transport volumes for the supply reach and each of the computation subreaches associated with the 2-, 5-, 10-, 25-, 50- and 100-year events under Scenario 1 are summarized in **Table 4.1** and the differences from subreach-to-subreach are shown in **Figure 4.6**. To evaluate long-term trends, the average annual sediment volumes in each subreach were also estimated using the following equation that was taken from the Design Guide:

$$Y_m = 0.015Y_{100}+0.015Y_{50}+0.04Y_{25}+0.08Y_{10}+0.2Y_5+0.4Y_2 \quad (4.2)$$

where Y_m = magnitude of the average annual event
Y_i = magnitude of the event for the 2-, 5-, 10-, 25-, 50-, and 100-year storm.

The results in Figure 4.6 indicate that Subreaches 1 through 3 are in relative equilibrium under existing conditions, which is consistent with field observations, Subreach 4a is degradational, and Subreach 4b is degradational events up to the 25-year storm, but aggradational for larger storms. This is consistent with the steeper slopes and the narrower channel in this area (Figures 3.3 and 3.5). The aggradation indicated in Subreach 5a is the result of the flatter slopes upstream of GCS#1, while the aggradation in Subreaches 5b and 6 is caused by backwater effects from Swinburne Dam.

4.2.2. 2036 Development Conditions Hydrology (Scenario 2)

Future development in the watershed will, over the long-term, reduce the sediment supply to the study reach from the upstream channel and tributaries due to the increase in impervious area and adjustments in the upstream channel and tributaries. Ultimately, the sediment supply to the project reach will adjust to the watershed sediment yield. Future bed-material sediment supplies were, therefore, estimated by decreasing the pre-development conditions bed material sediment supplies by the assumed increase in impervious area. As previously described, the Calabacillas DMP (RTI, 1987) estimated that about 12.5 percent of the Main Branch watershed upstream of the West Branch will be impervious under 2036 conditions. As a result, the 2036 development conditions bed-material sediment supplies were assumed to be 87.5 percent of the existing conditions supplies.

The bed material transport volumes that were obtained for Scenario 2 (existing conditions topography, 2036 conditions hydrology) are summarized in **Table 4.2** and the aggradation/degradation volumes resulting from differences in these volumes are summarized in **Figure 4.7**. Under these conditions, all of the subreaches upstream of GCS#1 (Subreaches 1 through 5a) are degradational due to the combined effects of the reduced sediment supply and increased sediment-transport capacities. The largest degradation tendency occurs in Subreaches 4a and 4b because these reaches are the steepest under existing conditions. Despite the reduction in long-term sediment supply from the watershed associated with the increased impervious area and adjustments in the upstream channel and tributaries, the results indicate that the subreaches in the backwater zone of Swinburne Dam (Subreaches 5b and 6) are aggradational. It should be noted that the sediment-continuity results for 2036 conditions are based on existing arroyo hydraulics. The channel will adjust to the developed watershed conditions by flattening its gradient and widening; thus, the estimated degradation volumes represent worst-case conditions that would occur if the indicated storms occurred soon after construction of the project.

Table 4.1. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, existing conditions under pre-development conditions hydrology (Scenario 1).

Subreach	Bed Material Supply (yd^3, bulked)	Bed Material Transport Capacity (yd^3, bulked)	Aggradation/Degradation Volume (yd^3, bulked)
2-year Event			
1	4,180	3,660	520
2	4,900	5,420	-520
3	5,410	5,410	0
4a	5,650	6,250	-600
4b	6,250	8,490	-2,240
5a	8,490	3,630	4,860
5b	3,630	920	2,710
6	1,580	1,120	460
5-year Event			
1	24,820	26,920	-2,100
2	34,270	36,270	-2,000
3	36,260	28,080	8,180
4a	29,460	37,300	-7,840
4b	37,300	51,070	-13,770
5a	51,070	26,850	24,220
5b	26,840	8,720	18,120
6	12,690	1,430	11,260
10-year Event			
1	46,320	53,150	-6,830
2	66,870	70,030	-3,160
3	70,030	50,070	19,960
4a	52,650	76,570	-23,920
4b	76,580	96,200	-19,620
5a	96,200	57,820	38,380
5b	57,820	19,380	38,440
6	26,770	1,500	25,270

Table 4.1. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, existing conditions under pre-development conditions hydrology (Scenario 1) (continued).

Subreach	Bed Material Supply (yd^3, bulked)	Bed Material Transport Capacity (yd^3, bulked)	Aggradation/Degradation Volume (yd^3, bulked)
25-year Event			
1	78,130	87,530	-9,400
2	110,680	116,120	-5,440
3	116,120	86,470	29,650
4a	90,830	142,880	-52,050
4b	142,880	148,400	-5,520
5a	148,410	107,560	40,850
5b	107,560	37,590	69,970
6	50,060	1,370	48,690
50-year Event			
1	108,000	112,810	-4,810
2	144,820	158,520	-13,700
3	158,520	121,530	36,990
4a	127,550	209,260	-81,710
4b	209,260	193,310	15,950
5a	193,310	157,950	35,360
5b	157,950	56,680	101,270
6	73,930	1,310	72,620
100-year Event			
1	141,800	136,290	5,510
2	178,310	208,830	-30,520
3	208,830	158,860	49,970
4a	166,760	278,260	-111,500
4b	278,260	254,510	23,750
5a	254,500	219,700	34,800
5b	219,710	79,770	139,940
6	102,420	1,270	101,150
Average Annual			
1	17,220	18,340	-1,120
2	23,440	25,180	-1,740
3	25,180	19,450	5,730
4a	20,410	29,110	-8,700
4b	29,120	33,960	-4,840
5a	33,950	21,410	12,540
5b	21,410	7,210	14,200
6	9,960	950	9,010

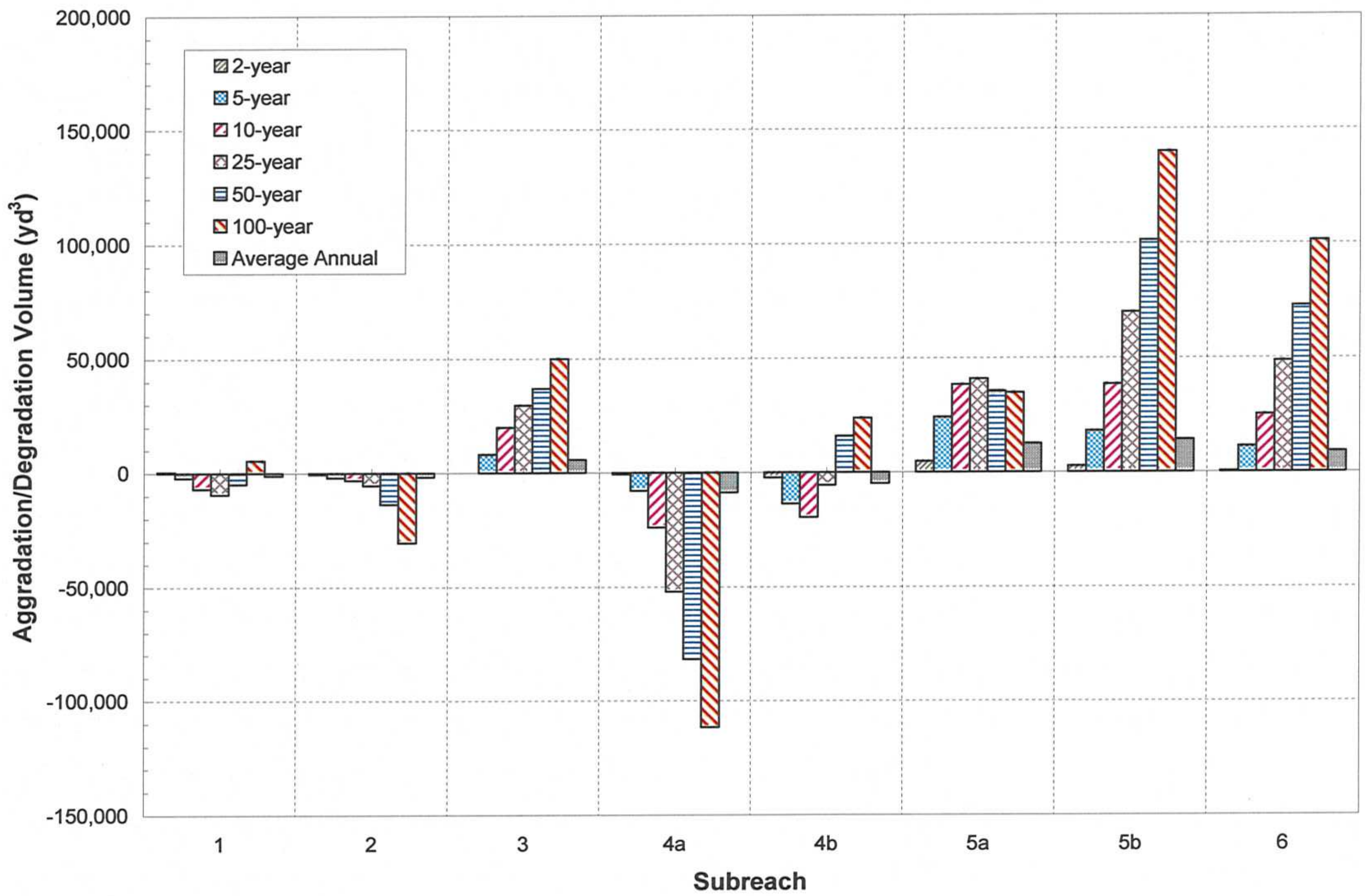

Figure 4.6. Results from the sediment-continuity analysis (aggradation/degradation volumes) for Scenario 1 (existing conditions under pre-development conditions hydrology).

Table 4.2. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, existing conditions under 2036 development conditions hydrology (Scenario 2).

Subreach	Bed Material Supply (yd^3, bulked)	Bed Material Transport Capacity (yd^3, bulked)	Aggradation/Degradation Volume (yd^3, bulked)
2-year Event			
1	3,660	16,440	-12,780
2	4,740	28,550	-23,810
3	4,730	22,290	-17,560
4a	4,940	29,230	-24,290
4b	4,940	41,810	-36,870
5a	4,950	22,010	-17,060
5b	4,940	7,160	-2,220
6	5,330	1,110	4,220
5-year Event			
1	21,720	47,040	-25,320
2	28,150	70,510	-42,360
3	28,150	50,200	-22,050
4a	29,360	78,620	-49,260
4b	29,370	97,140	-67,770
5a	29,360	61,460	-32,100
5b	29,360	20,850	8,510
6	31,660	1,140	30,520
10-year Event			
1	40,520	75,230	-34,710
2	52,540	110,220	-57,680
3	52,540	82,070	-29,530
4a	54,800	137,000	-82,200
4b	54,800	141,390	-86,590
5a	54,800	105,880	-51,080
5b	54,800	37,260	17,540
6	59,090	1,050	58,040

Table 4.2. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, existing conditions under 2036 development conditions hydrology (Scenario 2) (continued).			
Subreach	Bed Material Supply (yd^3, bulked)	Bed Material Transport Capacity (yd^3, bulked)	Aggradation/Degradation Volume (yd^3, bulked)
25-year Event			
1	68,370	106,850	-38,480
2	88,620	163,220	-74,600
3	88,630	125,540	-36,910
4a	92,430	219,550	-127,120
4b	92,430	200,640	-108,210
5a	92,430	171,250	-78,820
5b	92,440	62,070	30,370
6	99,670	990	98,680
50-year Event			
1	94,500	128,100	-33,600
2	122,510	216,830	-94,320
3	122,510	164,220	-41,710
4a	127,780	290,250	-162,470
4b	127,780	262,780	-135,000
5a	127,780	236,410	-108,630
5b	127,770	83,540	44,230
6	137,780	970	136,810
100-year Event			
1	124,080	162,100	-38,020
2	160,850	287,540	-126,690
3	160,850	210,910	-50,060
4a	167,760	371,480	-203,720
4b	167,760	321,540	-153,780
5a	167,760	308,730	-140,970
5b	167,760	97,020	70,740
6	180,890	950	179,940
Average Annual			
1	15,060	30,630	-15,570
2	19,520	48,430	-28,910
3	19,520	36,170	-16,650
4a	20,360	57,080	-36,720
4b	20,360	64,250	-43,890
5a	20,360	44,590	-24,230
5b	20,370	15,210	5,160
6	21,950	820	21,130

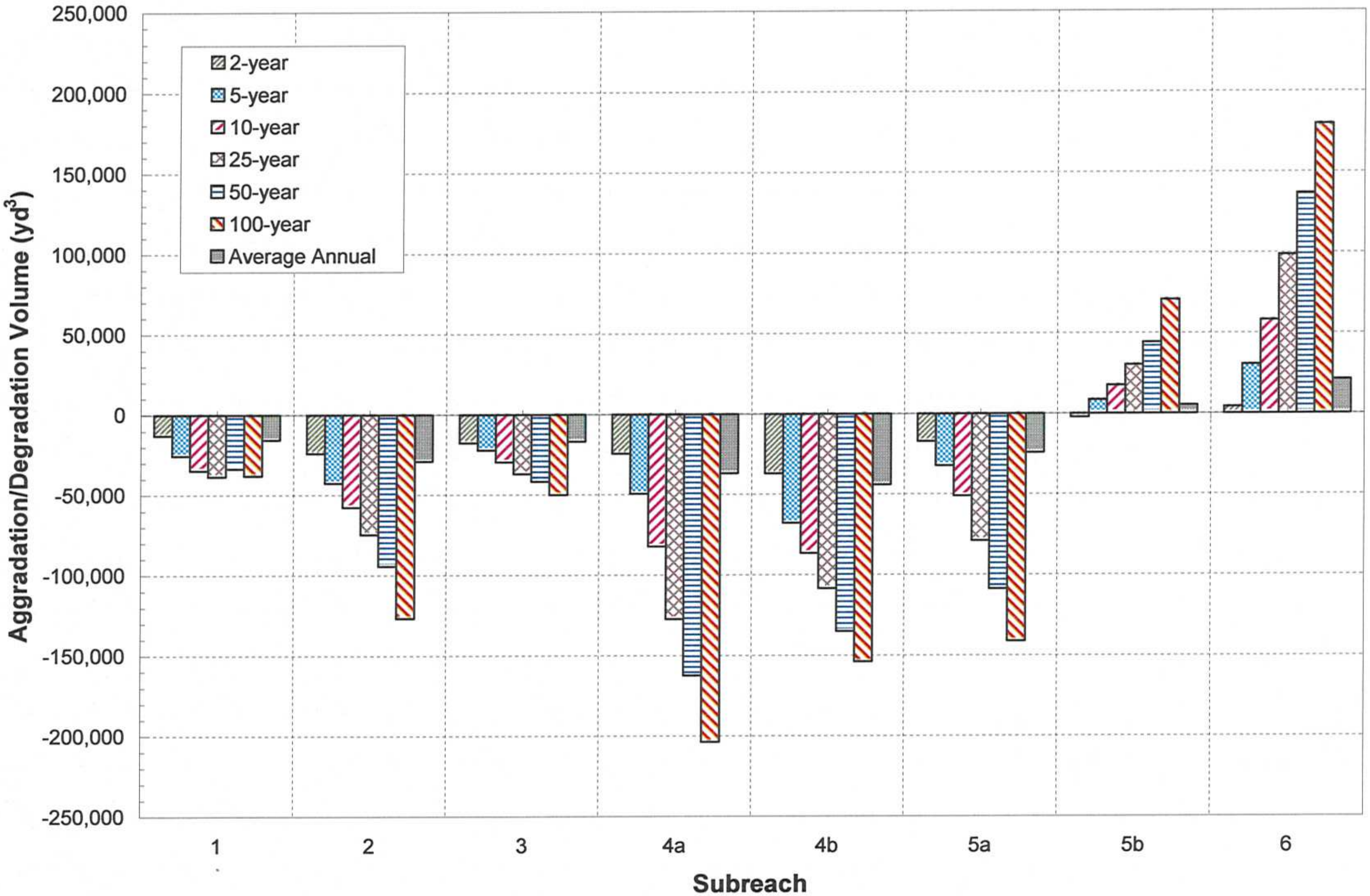

Figure 4.7. Results from the sediment continuity analysis (aggradation/degradation volumes) for Scenario 2 (existing conditions under 2036 development conditions hydrology).

As discussed in Chapter 2, it is MEI's opinion that the peak flows and runoff volumes for the more frequent events that were with the existing HEC-1 model that was used for this project may be unreasonably high. If this is correct, the sediment-transport rates and aggradation/degradation volumes for these events are also unreasonably high. Based on Equation 4.2, the 2- and 5-year events contribute 52 to 54 percent of the average annual sediment volumes for the subreaches upstream of the existing GCS#1 (Subreaches 1 through 5a). Lowering the sediment volumes for these events would result in significantly lower average annual sediment volumes

4.2.3. Sediment-Continuity Analysis for Design Conditions (Scenarios 3 and 4)

The effects of the design features on sediment transport through the reach (Scenarios 3 and 4) were evaluated using the same procedures that were described in the previous section with the reach-averaged hydraulics from the design conditions models. The results for Scenario 3 are summarized in **Table 4.3** and **Figure 4.8**, and the results for Scenario 4 are summarized in **Table 4.4** and **Figure 4.9**. As expected, a significant degradational tendency will occur throughout the reach upstream from the backwater effects of the Swinburne pool under Scenario 3.

At the equilibrium gradient that was used in the design (Scenario 4), Subreach 3 is approximately in balance with the upstream sediment supply under average annual conditions, but this subreach is aggradational during the floods exceeding the 10-year event due to the backwater effects that are created by McMahon Bridge. Subreaches 4a and 4b are slightly degradational under average annual conditions, due in part to the sediment trapping effects of the bridge. The ultimate equilibrium gradient in these two subreaches will probably be about 70 percent the gradient used in the design, which implies that an additional grade-control structure could be required in the future if the watershed develops to the condition that was assumed in the 2036 conditions model, and the entire upstream reach adjusts to the higher flows and diminished sediment supply. Because this will likely take at least a few to several decades to occur, the equilibrium slope used in the design is believed to provide a reasonable level of protection for the foreseeable future. Subreach 5a is controlled, at least temporarily, by the caliche-cemented material that corps out in the bed. So long as this material is not cut through, significant degradation is not anticipated.

For Scenario 4 (design conditions at the equilibrium slope), the results in Figure 4.8 indicate decreased degradation compared to Scenario 3 throughout the majority of the project reach (Subreaches 2 through 4b) due to flattened slopes and subsequent decreased velocities. The degradation in Subreach 5a is identical to Scenario 3 since the slope was not flattened to the equilibrium slope under the assumption that the caliche-hardened bed in the subreach will control vertical incision. The subreach upstream of the project reach (Subreach 1) indicates increased degradation volumes compared to Scenario 3 due to increased velocities associated with the incised channel geometry of the subreach.

Table 4.3. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, modified design conditions at the existing channel grade under 2036 development conditions hydrology (Scenario 3).

Subreach	Bed Material Supply (yd3, bulked)	Bed Material Transport Capacity (yd3, bulked)	Aggradation/Degradation Volume (yd3, bulked)
2-year Event			
1	3,650	16,500	-12,850
2	4,740	21,550	-16,810
3	4,740	27,600	-22,860
4a	4,950	27,600	-22,650
4b	4,940	38,770	-33,830
5a	4,940	28,830	-23,890
5b	4,940	7,160	-2,220
6	5,330	1,110	4,220
5-year Event			
1	21,720	47,090	-25,370
2	28,150	63,160	-35,010
3	28,150	64,210	-36,060
4a	29,360	65,650	-36,290
4b	29,360	86,960	-57,600
5a	29,370	78,190	-48,820
5b	29,360	20,850	8,510
6	31,660	1,140	30,520
10-year Event			
1	40,530	75,280	-34,750
2	52,540	110,910	-58,370
3	52,540	110,730	-58,190
4a	54,800	110,390	-55,590
4b	54,800	116,950	-62,150
5a	54,800	133,400	-78,600
5b	54,800	37,260	17,540
6	59,090	1,050	58,040

Table 4.3. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, modified design conditions at the existing channel grade under 2036 development conditions hydrology (Scenario 3) (continued).			
Subreach	Bed Material Supply (yd3, bulked)	Bed Material Transport Capacity (yd3, bulked)	Aggradation/Degradation Volume (yd3, bulked)
25-year Event			
1	68,370	106,890	-38,520
2	88,620	180,930	-92,310
3	88,620	177,190	-88,570
4a	92,430	176,060	-83,630
4b	92,440	150,800	-58,360
5a	92,430	213,530	-121,100
5b	92,440	62,070	30,370
6	99,670	990	98,680
50-year Event			
1	94,500	128,140	-33,640
2	122,500	248,700	-126,200
3	122,500	239,330	-116,830
4a	127,770	238,540	-110,770
4b	127,770	181,160	-53,390
5a	127,770	291,960	-164,190
5b	127,770	83,540	44,230
6	137,780	970	136,810
100-year Event			
1	124,070	162,140	-38,070
2	160,840	324,020	-163,180
3	160,840	307,560	-146,720
4a	167,760	303,630	-135,870
4b	167,760	204,940	-37,180
5a	167,760	379,380	-211,620
5b	167,760	97,020	70,740
6	180,890	950	179,940
Average Annual			
1	15,060	30,670	-15,610
2	19,520	45,950	-26,430
3	19,520	48,030	-28,510
4a	20,360	48,170	-27,810
4b	20,370	54,080	-33,710
5a	20,360	56,450	-36,090
5b	20,370	15,210	5,160
6	21,950	820	21,130

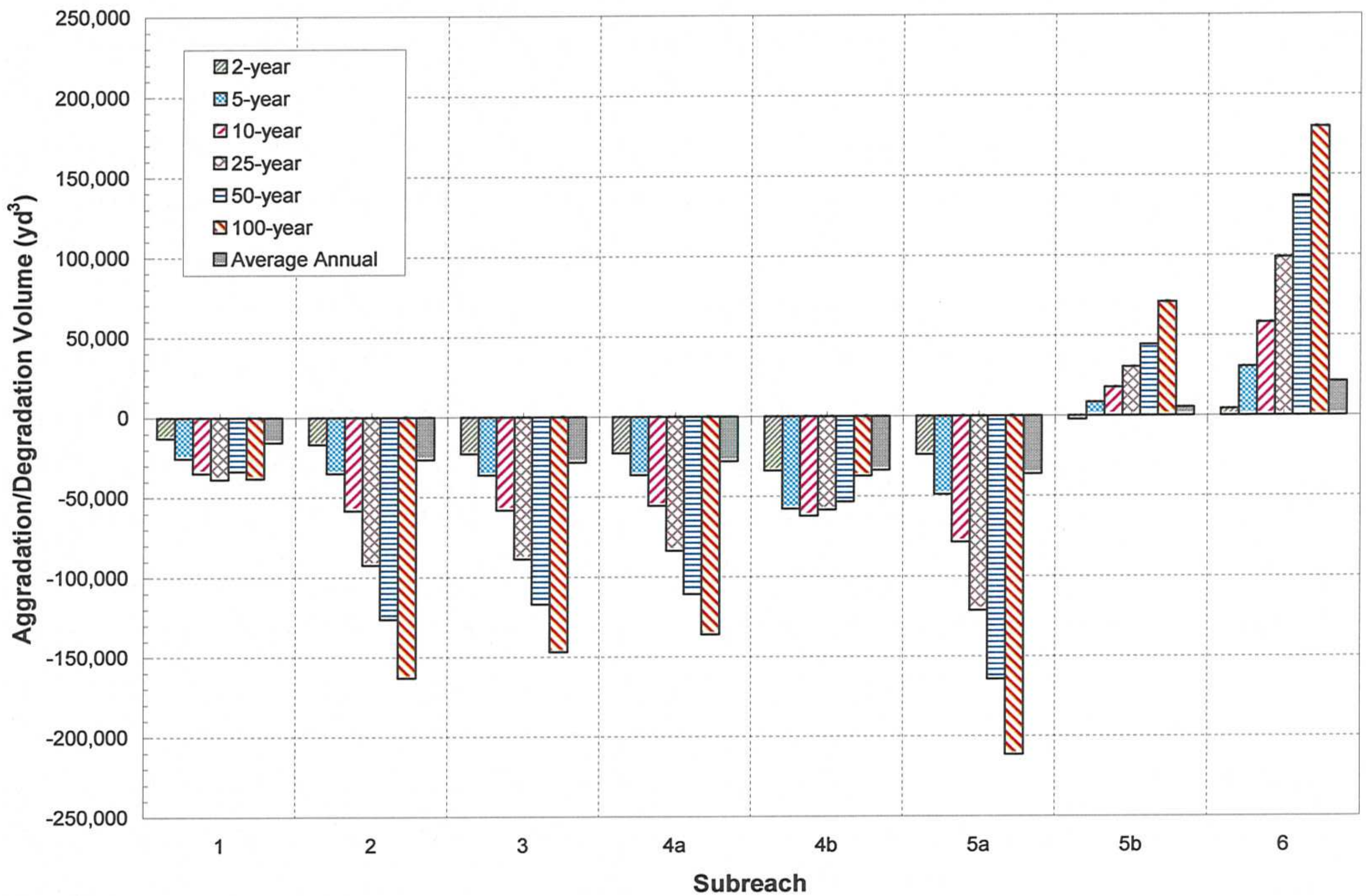

Figure 4.8. Estimated aggradation/degradation volumes for Scenario 3 (design conditions topography, existing channel gradient, 2036 hydrology) for the 2-, 5-, 10- 25-, 50- and 100-year storms. Also shown are the average annual aggradation/degradation volumes.

Table 4.4. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, modified design conditions at the anticipated equilibrium slope under 2036 development conditions hydrology (Scenario 4).

Subreach	Bed Material Supply (yd3, bulked)	Bed Material Transport Capacity (yd3, bulked)	Aggradation/Degradation Volume (yd3, bulked)
2-year Event			
1	3,660	16,600	-12,940
2	4,740	19,800	-15,060
3	4,740	12,490	-7,750
4a	4,950	22,340	-17,390
4b	4,940	18,160	-13,220
5a	4,940	28,830	-23,890
5b	4,940	7,160	-2,220
6	5,330	1,110	4,220
5-year Event			
1	21,710	51,990	-30,280
2	28,150	59,510	-31,360
3	28,150	32,530	-4,380
4a	29,360	48,930	-19,570
4b	29,360	46,110	-16,750
5a	29,370	78,190	-48,820
5b	29,360	20,850	8,510
6	31,660	1,140	30,520
10-year Event			
1	40,530	90,370	-49,840
2	52,540	105,860	-53,320
3	52,540	51,110	1,430
4a	54,800	79,900	-25,100
4b	54,800	74,980	-20,180
5a	54,800	133,400	-78,600
5b	54,800	37,260	17,540
6	59,090	1,050	58,040
25-year Event			
1	68,360	144,480	-76,120
2	88,620	173,790	-85,170
3	88,630	73,140	15,490
4a	92,440	125,510	-33,070
4b	92,430	109,760	-17,330
5a	92,430	213,530	-121,100
5b	92,440	62,070	30,370
6	99,670	990	98,680

Table 4.4. Summary of sediment continuity results for the Main Branch of Calabacillas Arroyo upstream of Swinburne Dam, modified design conditions at the anticipated equilibrium slope under 2036 development conditions hydrology (Scenario 4) (continued).

Subreach	Bed Material Supply (yd3, bulked)	Bed Material Transport Capacity (yd3, bulked)	Aggradation/Degradation Volume (yd3, bulked)
50-year Event			
1	94,500	194,780	-100,280
2	122,500	238,780	-116,280
3	122,510	87,320	35,190
4a	127,770	171,730	-43,960
4b	127,770	132,380	-4,610
5a	127,770	291,960	-164,190
5b	127,770	83,530	44,240
6	137,780	960	136,820
100-year Event			
1	124,070	250,990	-126,920
2	160,850	311,960	-151,110
3	160,850	95,520	65,330
4a	167,760	225,850	-58,090
4b	167,760	150,680	17,080
5a	167,760	379,380	-211,620
5b	167,760	96,960	70,800
6	180,890	950	179,940
Average Annual			
1	15,060	36,730	-21,670
2	19,530	43,510	-23,980
3	19,530	21,260	-1,730
4a	20,370	36,100	-15,730
4b	20,360	31,120	-10,760
5a	20,360	56,450	-36,090
5b	20,370	15,210	5,160
6	21,960	820	21,140

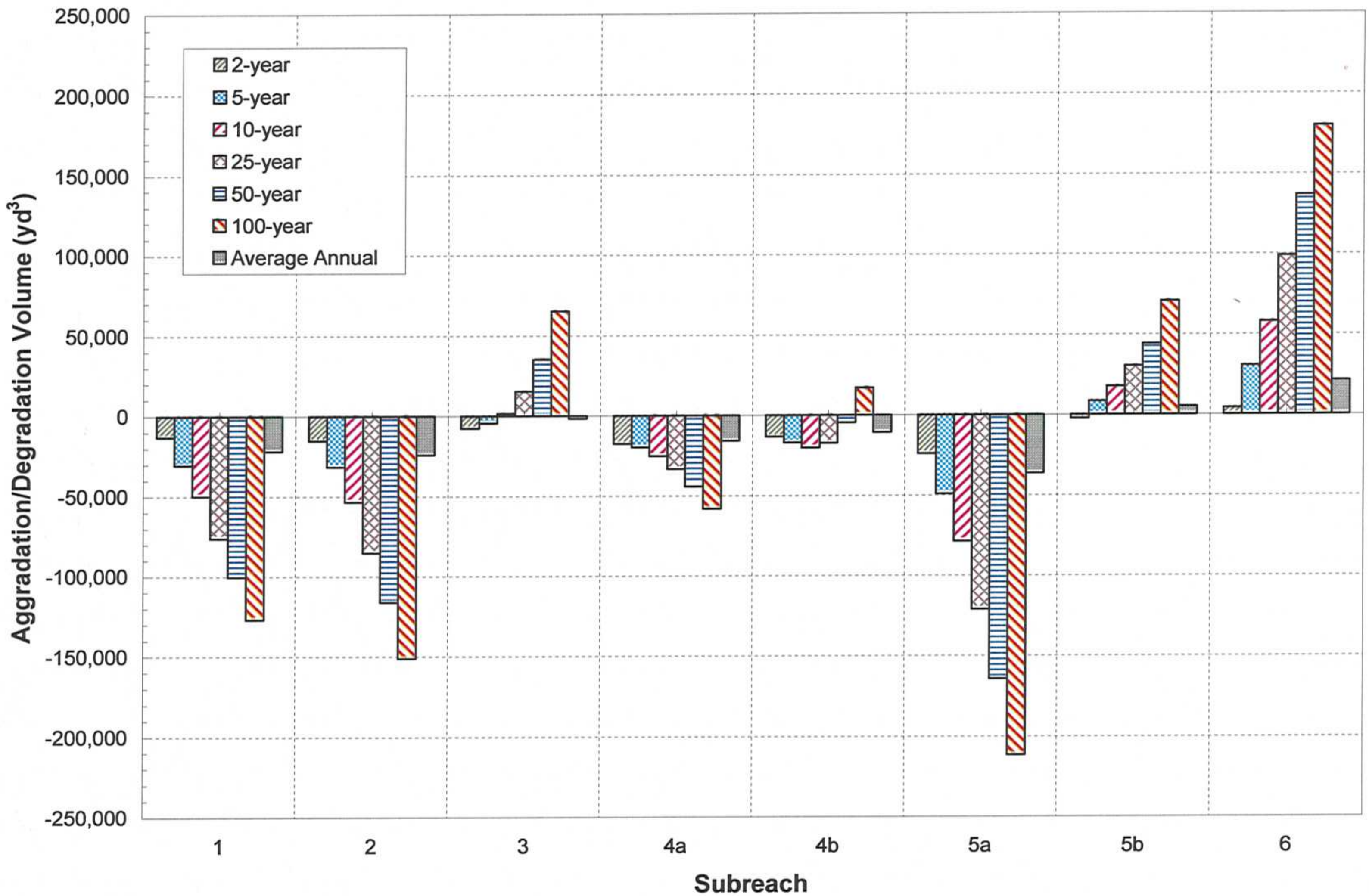

Figure 4.9. Estimated aggradation/degradation volumes for Scenario 4 (design conditions topography, equilibrium gradient, 2036 hydrology) for the 2-, 5-, 10- 25-, 50- and 100-year storms. Also shown are the average annual aggradation/degradation volumes.

5. RECOMMENDATIONS FOR DESIGN AND LATERAL STABILITY ANALYSIS

The primary objectives of this study were to assist Wilson and Goodwin in designing the channel and other features to provide adequate protection for properties adjacent to the project reach from flooding and erosion during future large storm events. The results from the hydraulic and sediment continuity analyses presented in the previous chapters were used to evaluate the potential for lateral migration and to develop recommendations for designing channel protection measures that are included in the final design configuration (Map 2, Appendix A).

5.1. Analysis of the Initially-Proposed Design

The elements of the design that was initially proposed by Goodwin were described in the introduction to this report, and they included four new grade-control structures to provide vertical stability, and a series of spur dikes and other bank protection measures to provide lateral stability (Appendix A, Map 1). The key downstream baselevel (vertical) control for the project reach is the existing GCS#1 that is located at Station 229+85. Based on the historic behavior of the channel through the caliche-cemented reach upstream from GCS#1, however, it was assumed that the channel will not degrade between the structure and about Station 238+00 for purposes of designing the elements of this project. **It is important to note that the caliche will eventually erode, and, since its thickness and lateral extent are not known, there is reasonable potential for additional degradation in this reach that could affect the stability of upstream project elements.** A grade-control structure (GCS#3) is proposed for the downstream side of McMahon Bridge, which is located at about the mid-point of the project reach, to protect the channel invert through the bridge opening. The estimated long-term equilibrium slope for the reach that was estimated by MEI (1999) to be about 1 percent would result in an excessive drop height across this structure if additional protection is not provided between the GCS#1 and the bridge. As a result, GCS#2 was proposed at Sta 250+00, about midway between the downstream baselevel control and the bridge crossing. Using the caliche-cemented reach as the downstream control with the 1 percent equilibrium slope, the design drop-heights across GCS#2 and GCS#3 would be 9.3 feet and 4.3 feet, respectively (**Table 5.1**). (If the caliche is breached, the drop height at GCS#2 could be as much as 12.5 feet.) The potential degradation between the control at McMahon Bridge and the upstream project limit would also be excessive in the absence of additional grade-control. GCS#4 was, therefore, proposed at Sta 280+94, which would result in a potential drop height of 8.9 feet. Note that the drop heights in Table 5.1 represent the overall drop from the primary crest of the structure to the downstream equilibrium channel elevation (i.e., the low flow notch in the structure was neglected in establishing the gradient.)

Table 5.1. Summary of drop heights for the grade-control structures for **initially-proposed** design.

Grade-Control Structure	Station (ft)	Drop Height (ft)
GCS#2	250+00	9.3
GCS#3	263+43	4.3
GCS#4	280+94	8.9

To provide protection against lateral migration, a series of spurs were proposed for the reach, at an average spacing of about 400 feet upstream from McMahon Bridge and about 550 feet downstream from the bridge. The top width of the design channel ranged from about 240 feet at spur constrictions to about 320 feet midway between the spurs.

The original Prudent Line for the project reach that was presented in MEI (2000) was developed using the "CURVCALC" software and the existing arroyo geometry with no lateral controls. The structures that are proposed as part of this project will limit lateral erosion to an area that is well within the original Prudent Line corridor. The potential for lateral migration and possible flanking of the spurs and grade control structures under the proposed design will be controlled by a combination of the flow expansion angle downstream from each structure and the bend geometry that could develop in areas that are not limited by the flow expansion angle. Based on guidelines in Federal Highway Administration HEC-23 (Lagasse et al., 1997) for essentially impermeable dikes that are oriented perpendicular to the flow, the expansion angle would be about 18 degrees, and significant bank erosion on the outside of the expansion zone is very unlikely. (In fact, this area is a separation zone that will most likely be depositional.) The erosion envelope that is created by the flow expansion zone is shown on Map 1 (Appendix A).

The lateral erosion potential associated with bend development in areas within zone created by the expansion angle was evaluated using the methods described in the Design Guide, as follows:

1. The dominant discharge was estimated by determining the peak discharge of the flood hydrograph that would transport the average annual bed material load with the existing channel topography and 2036 hydrology (Scenario 2, Table 4.2). The resulting dominant discharge for the reach is about 4,300 cfs.

2. The dominant channel width was estimated using Equation 3.79 from the *Design Guide*, ($W_D = 4.6Q_D^{0.4}$) to be about 130 feet.

3. The potential maximum lateral migration distance under unconstrained conditions was estimated using Equation 3.79 from the *Design Guide* ($\Delta_{max}=16.1Q_D^{0.4}$) to be about 460 feet.

4. The potential maximum lateral migration distance between each of the proposed structures that will provide lateral control was estimated using Figure 3.24 from the *Design Guide* based on the spacing between the structures.

The resulting potential migration distances are indicated by the lines labeled "DMAX" on Map 1 (Appendix A) at the upstream side of each of the structures that will provide lateral control. (Note that the reach upstream from Spurs 9L and 9R would be unconstrained by this project; thus, the lateral migration distances shown on Map 1 are based on results from the CURVCALC analysis.)

The erosion envelope between the structures consists of the smaller erosion distance indicated by either the flow expansion zone or the maximum lateral migration distance. The resulting envelope is indicated by the red line labeled "Potential Bank Erosion Envelope" on Map 1. (This line indicates areas where there is potential for erosion beyond the top-of-bank that was proposed under the initial design alternative, shown by the blue line on Map 1.) The results of this analysis indicate that erosion distances between the structures could range up to about 140 feet beyond the proposed top-of-bank, which could result in flanking of several of the structures. In addition, the alignment of the spurs and channel immediately upstream from McMahon

Bridge could result in flow impingement on the right bridge abutment and an undesirable flow transition into the bridge opening. The orientation and location of GCS#4 also appeared to be problematic because it directed the flow exiting the structure toward the right bank upstream of Spur 6L. The results from this analysis were provided to Goodwin and Wilson who made adjustments to the proposed design to address the potential erosion issues.

5.2. Final Design

The modified design incorporates several changes to the original design, as shown in **Appendix A, Map 2**. These changes included realignment of the opening through McMahon Bridge, relocation of GCS#4 about 300 feet downstream from its original location, re-alignment of the channel between GCS#4 and the bridge, and changes in the number and location of the spur dikes. Additional bank protection is also provided upstream from the spur dikes and other features to prevent flanking. GCS#2 and GCS#3 are essentially the same as in the initial design. The design drop heights for the three structures under the modified design were determined based on the 1 percent equilibrium slope and a new station line that corresponds to the revised channel alignment (**Table 5.2**, Figure 3.1). Consistent with the assumption used in evaluating the initial design, the design drop height at GCS#2 was established using the upstream limit of the caliche-cemented bed at Sta 238+00 as the control. **As noted above, it is recommended that this reach be monitored and corrective action taken, if necessary, because incision into the caliche-cemented bed in this area could threaten the stability of GCS#2.**

Table 5.2. Summary of drop heights for the grade-control structures for **modified** design.

Grade-Control Structure	Design Station (ft)	Drop Height (ft)
GCS#2	250+00	9.30
GCS#3	262+82	4.79
GCS#4	277+71	6.63

The placement of the spurs upstream from McMahon Bridge was established to better coincide with the existing bankline and to make more efficient use of the flow expansion limits from the next upstream structures. To reduce the amount of earthwork, a smooth channel alignment was developed between the spurs and grade control structures, and the nose of the spurs was set to coincide with the design 2H:1V bank slopes to limit the amount of projection into the channel. (Note that the footprint of the spurs shown on Map 2 includes the buried portion of the spurs tied into the bank and the portion of the nose that will be buried below the existing channel grade.) The previously proposed Spur 3R on the right bank downstream from McMahon Bridge was removed and the riprap was extended downstream from GCS#3 through the bend. In addition, Spur 2R was removed because it is located on the inside of the bend and; thus, does not recover any additional overbank area for development.

In the reach upstream from McMahon Bridge, those portions of the bankline upstream from the structures that are within the flow expansion angle and the maximum potential erosion limit will be protected using a buried trenchfill revetment that will be designed to self-launch, preventing further migration, as the bank erodes into the trench. The extent of the protection in these areas that was proposed by Goodwin are indicated by the red hatched zones on Map 2. Based on MEI's evaluation of the modified design, five areas were identified where additional protection may be necessary, as follows:

Locations 1 and 2. Because the upstream channel is not laterally constrained, the banks could eventually erode by as much as 200 feet beyond the proposed protection on the left bank and as much as 140 feet beyond the proposed protection on the right bank, which could result in flanking of Spurs 10L and 10R. Extending the protection to the limits indicated by the solid red lines on Map 2 would protect against flanking. The substantial length of protection that was proposed at these locations by Goodwin should, however, be adequate to prevent flanking during a single, large storm, or a series of several small to intermediate storms. It may be most cost-effective to install the protection as proposed by Goodwin, and then closely monitor the area after high flows. If future erosion begins to endanger the structure on one or both sides of the channel, additional protection measures could be installed at that time.

Location 3. Although the proposed trenchfill riprap upstream of Spur 5R extends to the flow expansion limits from GCS#4, the location and orientation of Spur 6L could deflect flows into the right bank upstream of the Spur 5R protection. As a result, it is recommended that the trenchfill be extended an additional 80 feet upstream as indicated by the solid red line on Map 2.

Location 4. If the riprap protection on the right bank downstream from GCS#3 is terminated at the proposed location, a scallop will probably develop in the bank immediately downstream that could eventually cause the riprap to unravel from the downstream end. While this may simply be a maintenance issue that would not endanger the installation during a single event, it may be safer and more cost-effective to either tie the riprap back into the bank for a distance of 30 to 40 feet or, preferably, extend it 75 to 100 feet farther downstream so that the flow separation point occurs along the riprap rather than an unprotected area of the bank.

Location 5. The proposed tie-back for the left end of GCS#2 does not extend to the maximum lateral erosion limit. Because this bank cuts into the erosion-resistant Santa Fe Formation in this area, the proposed tie-back may be adequate; however, sufficient information about the exact location, depth and erosion-resistance of the materials in this bank are not available at this time to be certain. It is recommended that this area be closely evaluated by a competent geotechnical engineer at the time of construction, and the tie-back extended to either competent materials or the maximum erosion limit, if necessary.

5.3. Design Top of Bank Profiles

The design top-of-bank profiles along the reach were established using the hydraulic model that was developed for this evaluation based on the water-surface elevation associated with the bulked 100-year peak flow under 2036 conditions, plus 2 feet of freeboard, with the pre-incision channel grade (**Table 5.3**).

Super-elevation of the water surface will occur at high flows on the outside of the bend (right bank) downstream from GCS#3 and the left bank upstream from GCS#2. The potential magnitude of the superelevation was estimated using the following equation (USACE, 1970):

$$\Delta Z = C\frac{V^2W}{gR_c} \tag{5.1}$$

Table 5.3. Summary of the existing channel invert elevations, equilibrium slope channel invert elevations, and design profiles including computed bulked 100-year water-surface elevation for the modified design at existing grade, computed water-surface elevation plus 2 feet of freeboard, the super-elevation of the water surface due to channel bends, and recommended riprap toe-down elevations.

Design Station (ft)	Design Feature	Existing Channel Invert Elevation (ft)	Equilibrium Slope Channel Invert Elevation (ft)	Computed Water-Surface Elevation* (ft)	Water-surface+2' Freeboard (ft)	Super-elevation (ft)	Water-surface+ Super-elevation (ft)	Left Bank Riprap Toe-down Elevation (ft)	Right Bank Riprap Toe-down Elevation (ft)
19845		5259.2	5259.2	5288.4	5290.4	NA	5288.4	NA	NA
20245		5257.2	5257.2	5288.4	5290.4	NA	5288.4	NA	NA
20695		5261.2	5261.2	5288.4	5290.4	NA	5288.4	NA	NA
21105		5270.2	5270.2	5288.4	5290.4	NA	5288.4	NA	NA
21525		5273.2	5273.2	5288.4	5290.4	NA	5288.4	NA	NA
21945		5277.2	5277.2	5288.4	5290.4	NA	5288.4	NA	NA
22295		5281.8	5281.8	5288.0	5290.0	NA	5288.0	NA	NA
22639		5288.9	5288.9	5297.6	5299.6	NA	5297.6	NA	NA
22715		5287.2	5287.2	5300.5	5302.5	NA	5300.5	NA	NA
22833		5289.2	5289.2	5299.9	5301.9	NA	5299.9	NA	NA
22935		5293.0	5293.0	5299.7	5301.7	NA	5299.7	NA	NA
22985	Existing GCS1 Crest	5300.5	5300.5	5307.5	5309.5	NA	5307.5	NA	NA
23035		5301.0	5301.0	5309.0	5311.0	NA	5309.0	NA	NA
23418		5304.1	5304.1	5313.1	5315.1	NA	5313.1	NA	NA
23650	Spur1	5309.6	5309.6	5317.0	5319.0	NA	5317.0	NA	NA
23800	Upstream limit of caliche	5312.2	5312.2	5323.1	5325.1	NA	5323.1	NA	NA
24051		5317.3	5314.7	5326.6	5328.6	NA	5326.6	NA	NA
24300		5322.0	5317.2	5330.8	5332.8	NA	5330.8	NA	NA
24608		5326.3	5320.3	5333.4	5335.4	NA	5333.4	NA	NA
25000	GCS2 Crest	5333.0	5324.2	5340.1	5342.1	1.8	5342.0	NA	NA
25020		5333.7	5333.7	5341.7	5343.7	1.1	5342.8	NA	NA
25385		5338.0	5337.4	5344.6	5346.6	0.9	5345.5	NA	NA
25660	Spur 3L	5342.0	5340.1	5349.2	5351.2	2.9	5352.1	NA	5336.6
25979		5346.7	5343.3	5352.6	5354.6	2.9	5355.5	NA	5339.8
26202	U/S Limit of RB Riprap	5349.6	5345.5	5357.2	5359.2	0.8	5357.9	NA	5342.0

Table 5.3. Summary of the existing channel invert elevations, equilibrium slope channel invert elevations, and design profiles including computed bulked 100-year water-surface elevation for the modified design at existing grade, computed water-surface elevation plus 2 feet of freeboard, the super-elevation of the water surface due to channel bends, and recommended riprap toe-down elevations (continued).

Design Station (ft)	Design Feature	Existing Channel Invert Elevation (ft)	Equilibrium Slope Channel Invert Elevation (ft)	Computed Water-Surface Elevation* (ft)	Water-surface+2' Freeboard (ft)	Super-elevation (ft)	Water-surface+ Super-elevation (ft)	Left Bank Riprap Toe-down Elevation (ft)	Right Bank Riprap Toe-down Elevation (ft)
26282	GCS3 Crest	5350.6	5346.3	5358.8	5360.8	NA	5358.8	NA	NA
26297		5350.6	5351.3	5359.9	5361.9	NA	5359.9	NA	NA
26350	Downstream Bridge Face	5351.8	5351.8	5360.5	5362.5	NA	5360.5	NA	NA
26472	Upstream Bridge Face	5353.0	5353.0	5364.4	5366.4	NA	5364.4	5348.6	5348.6
26765	Spur 4L	5359.2	5355.9	5365.2	5367.2	NA	5365.2	5349.1	5350.3
26838	Spur 4R	5360.2	5356.7	5365.8	5367.8	NA	5365.8	NA	5350.8
27057	Spur 5L,R	5362.3	5358.9	5367.5	5369.5	NA	5367.5	NA	NA
27311	Spur 6L	5365.9	5361.4	5370.8	5372.8	NA	5370.8	NA	NA
27771	GCS4 Crest	5372.1	5366.0	5378.9	5380.9	NA	5378.9	NA	NA
28090	Spur 7L	5377.6	5375.8	5382.5	5384.5	NA	5382.5	NA	NA
28385	Spur 8L,R	5381.8	5378.8	5386.6	5388.6	NA	5386.6	NA	NA
28784	Spur 9R	5386.6	5382.8	5390.8	5392.8	NA	5390.8	NA	NA
28938	Spur 9L	5388.6	5384.3	5392.8	5394.8	NA	5392.8	NA	NA
29092	Spur 10R	5390.6	5385.8	5394.6	5396.6	NA	5394.6	NA	NA
29330	Spur 10L	5396.1	5388.2	5399.9	5401.9	NA	5399.9	NA	NA
29678		5401.3	5391.7	5405.8	5407.8	NA	5405.8	NA	NA
30059		5406.4	5395.5	5411.6	5413.6	NA	5411.6	NA	NA
30451		5412.3	5399.4	5417.3	5419.3	NA	5417.3	NA	NA
30848		5417.4	5403.4	5423.4	5425.4	NA	5423.4	NA	NA
31247		5422.6	5407.4	5428.9	5430.9	NA	5428.9	NA	NA
31639		5428.6	5411.3	5433.8	5435.8	NA	5433.8	NA	NA
32045		5433.8	5415.4	5439.6	5441.6	NA	5439.6	NA	NA
32444		5439.5	5419.4	5444.5	5446.5	NA	5444.5	NA	NA

*Bulked 100-year Peak Discharge

where ΔZ = superelevation of water surface associated with bends
C = coefficient generally taken as 1 for a trapezoidal cross section in rapid flow
V = channel velocity
W = flow topwidth
g = acceleration due to gravity
R_c = bend radius of curvature

The bend downstream from GCS#3 has a radius of about 360 feet and the topwidth at this location varies from 210 feet to 240 feet, indicating about 2.9 feet of superelevation during the design flow. The bend upstream from GCS#2 has a radius of about 560 feet and the channel topwidth is about 170 feet, indicating superelevation varying up to about 1.8 feet. The top of the riprap in the bend downstream from GCS#3 should, therefore, be at least 2.9 feet above the bulked 100-year water-surface elevation, and the recommended 2-foot freeboard should be adequate at GCS#2 since it exceeds the superelevation.

5.4. Spur Scour and Toe-Down Recommendations

The potential depth of local scour that could occur at the nose of the proposed spurs was estimated using results from the hydraulic model for equilibrium slope conditions and the recommended scour equation from Federal Highway Administration HEC-20 (Lagasse et al., 1995) that is given by:

$$y_s = 1.1 y_1 \left(\frac{a}{y_1} \right)^{0.4} Fr^{0.33} \tag{5.2}$$

where y_s = predicted scour depth
a = length of the spur extending perpendicular to the direction of flow
y_1 and Fr_1 = upstream flow depth and Froude number, respectively

(Note that Equation 5.1 is valid only for conditions when the ratio of spur length to upstream flow depth is less than 25.) The projected length of the spurs was estimated to be the distance from the nose of the spur to the inside edge of the trenchfill, revetment, perpendicular to the direction of flow, resulting in lengths ranging from 14 feet at Spur 10L to 68 feet at Spur 6L. The estimated scour depths using this assumption varied from 6.1 feet at Spur 10L to 11.5 feet at Spur 6L, and total scour depth below existing grade, including degradation to the equilibrium slope ranging from 8.9 feet at Spur 7L to 16 feet at Spur 6L (**Table 5.4**). The spurs should either be toed-down below the indicated total scour depth, or sufficient material should be provided at the nose to self-launch into the scour hole in a manner that will prevent undercutting.

5.2 Riprap Design Recommendations

Recommendations for rock sizes and toe-down were developed for the riprap protection that is proposed for the right bank downstream from GCS#3 and both banks upstream from the McMahon Bridge. The recommended riprap size was calculated using methods presented in the Federal Highway Administration's HEC-11 (FHWA, 1989) that consider the local hydraulic conditions and the geometry of the bank to determine the minimum stable stone size. The hydraulic conditions used in the calculations were obtained from model results for design conditions at the existing grade (Scenario 3) for the 100-year peak flow. The velocities that will impact the proposed revetment on the outside of the bend downstream from GCS#3 (Sta 256+60 to Sta 262+00) will be considerably higher than the cross-sectionally averaged velocities from the hydraulic model. The increase in velocity was estimated using procedures outlined in the Corps of Engineers *Hydraulic Design of Flood Control Channels* (USACE, 1991).

Table 5.4. Summary of computed scour depths at the spur noses.						
Location	Design Station (ft)	Length of Spur Perpendicular to Flow (ft)	Existing Invert Elevation (ft)	Equilibrium Slope Elevation (ft)	Scour Depth Below Equilibrium Slope (ft)	Total Scour Depth (ft)*
Spur 10L	29330	14	5396.1	5388.2	6.1	13.9
Spur 10R	29092	23	5390.6	5385.8	6.6	11.4
Spur 9L	28938	26	5388.6	5384.3	6.7	11.0
Spur 9R	28784	26	5386.6	5382.8	7.0	10.9
Spur 8L,R	28385	25	5381.8	5378.8	6.6	9.6
Spur 7L	28090	25	5377.6	5375.8	7.1	8.9
Spur 6L	27311	68	5365.9	5361.4	11.5	16.0
Spur 5L,R	27057	35	5362.3	5358.9	8.1	11.5
Spur 4R	26838	19	5360.2	5356.7	5.9	9.4
Spur 4L	26765	30	5359.2	5355.9	6.8	10.1
Spur 3L	25660	40	5342.0	5340.1	7.4	9.3
Spur 1R	23650	40	5309.6	5309.6	9.7	9.7

In sizing the riprap, 2H:1V sideslopes were assumed, consistent with the design channel banks, and it was also assumed that angular riprap would be used. For the riprap upstream from the bridge crossing, the calculations indicate rock with median diameter of 1.7 feet, which corresponds to a ¼ ton gradation class as defined by the FHWA (1989), is necessary for both banks. The riprap on the right bank downstream from GCS#3 will require rock with a median diameter of about 3.0 feet (FHWA [1989] 2 ton class). The riprap should be placed on a suitable filter fabric or granular filter to a minimum thickness that is equivalent to the greater of the maximum stone size or 1.5 times the median diameter.

For the riprap-protected banks upstream from McMahon Bridge, recommended toe-down depths were developed based on the estimated contraction scour associated with the bridge constriction and the degradation associated with the equilibrium slope. Contraction scour was computed assuming live-bed scour conditions using the following equation from FHWA HEC-18 (Richardson and Davis, 2001):

$$y_s = y_1 \left(\frac{Q_2}{Q}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} - y_o \tag{5.2}$$

where y_1 = average depth in the upstream main channel
y_o = existing depth in the contracted section before scour
Q_1 = flow in the upstream channel,
Q_2 = flow in the contracted channel,
W_1 = bottom width of the upstream main channel,
W_2 = bottom width of the main channel in the contracted section
k_1 = exponent representing the mode of bed-material transport

The depth in the contracted section (y_o) was estimated as the average depth in the main channel upstream from the bridge, and the exponent k_1 was assigned a value of 0.69 for conditions with mostly suspended bed material discharge. The estimated contraction scour depth during the 100-year peak flow when the channel has degraded to the equilibrium slope is about 4.4 feet. Because the riprap will tie into Spur 4L on the left bank and Spur 4R on the right bank, it is recommended that the profile of the bottom of the riprap transition from the contraction scour depth just upstream from the bridge to the local scour depth at the spurs (**Figure 5.1,** Table 5.3).

For the right bank riprap located downstream from GCS#3, the recommended toe-down depth for the riprap were determined by estimating the bend scour, and subtracting the resulting scour depths from the computed channel invert after incision to the equilibrium slope. The bend scour depths were estimated by determining the scour necessary to reduce the differential shear stress on the outside of a bend to the average shear stress in the cross section. The resulting scour depth during the 100-year peak flow under equilibrium slope conditions range from about 3.0 to 3.4 feet, indicating that the riprap should be toed-down at least 3.5 feet below the equilibrium slope profile (**Figure 5.2**, Table 5.3).

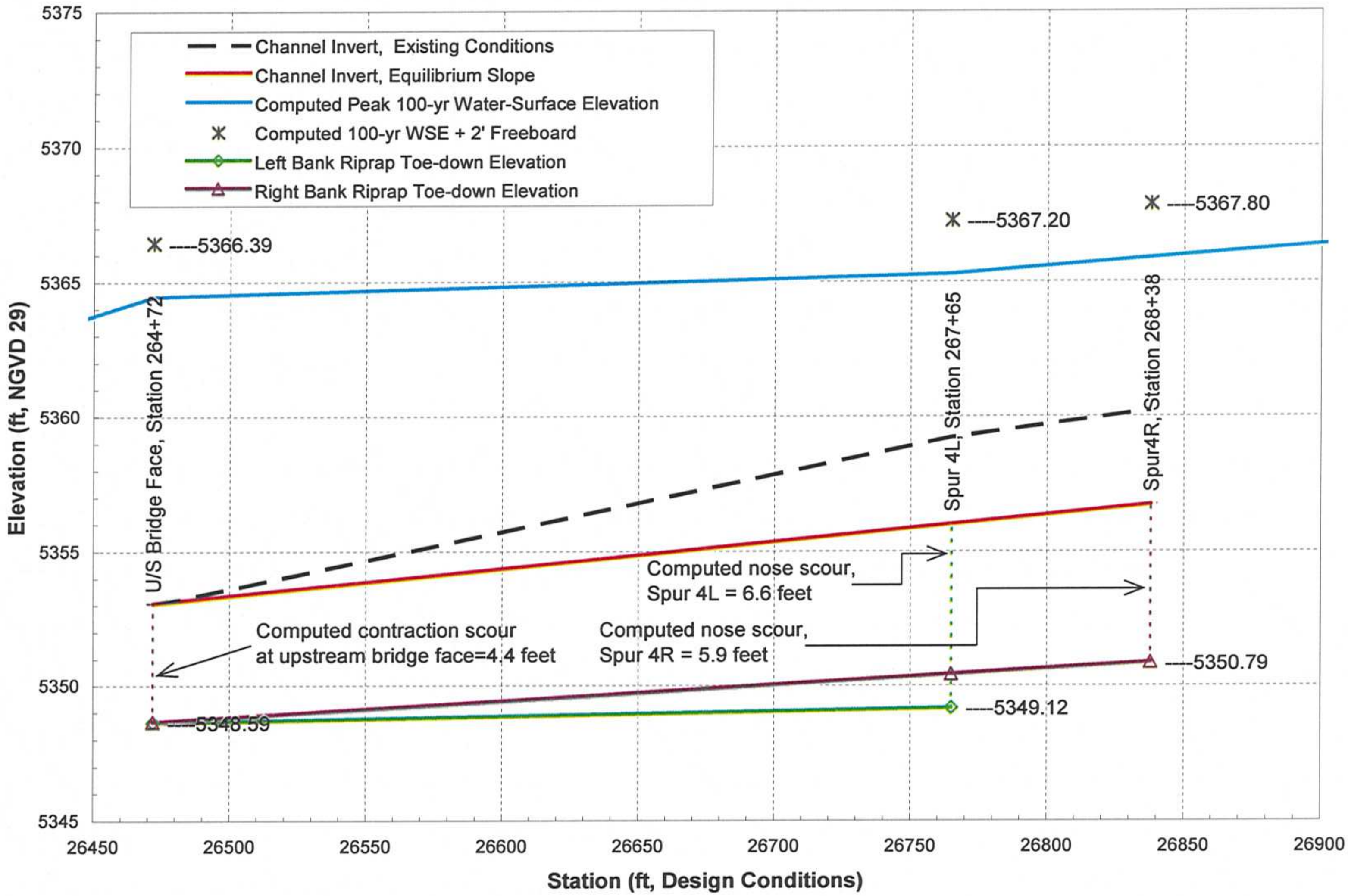

Figure 5.1. Longitudinal profile for the left and right bank riprap located upstream of the McMahon Boulevard bridge crossing.

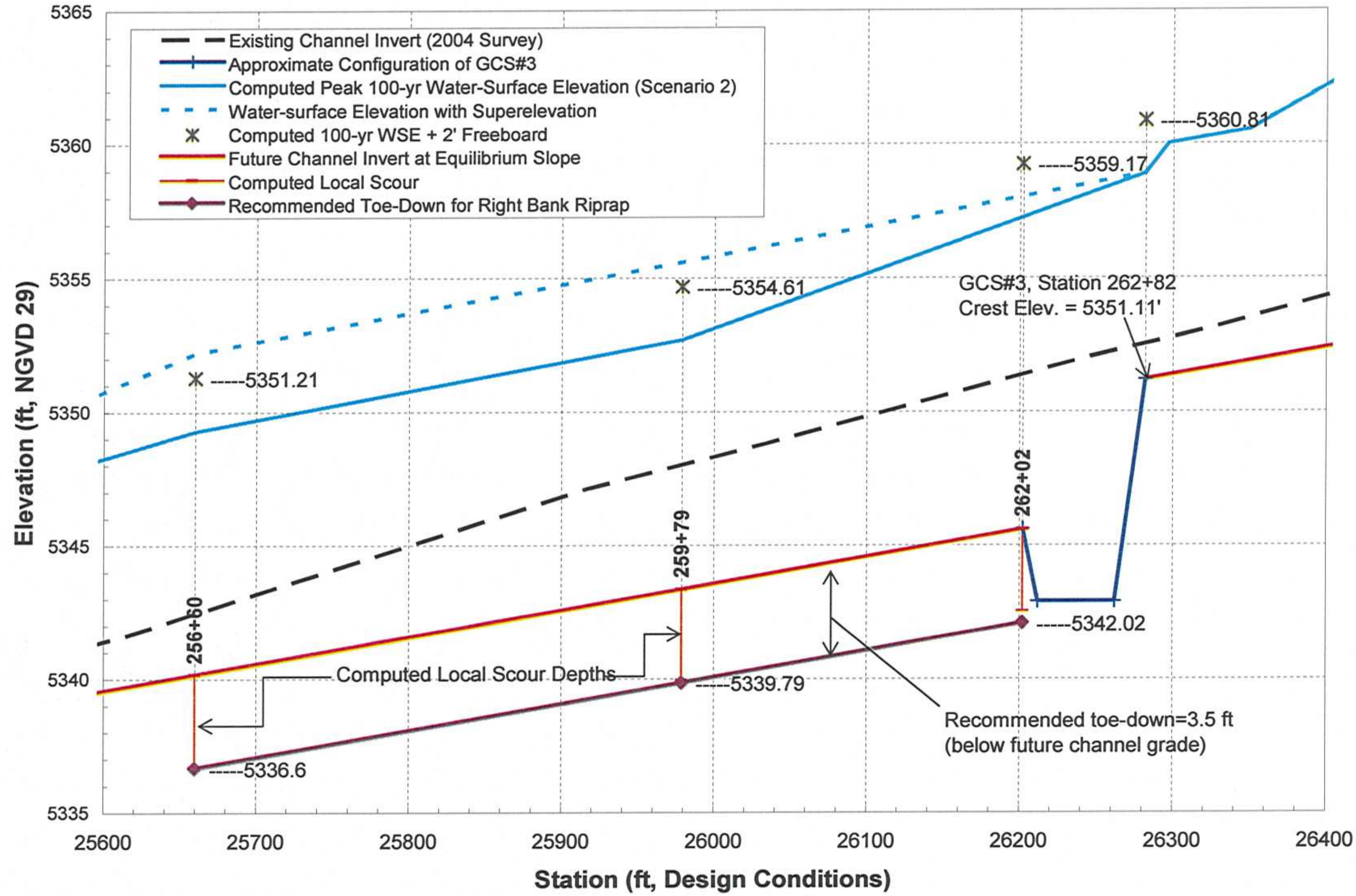

Figure 5.2. Longitudinal profile for the right bank riprap located downstream of the McMahon Boulevard bridge crossing.

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APPENDIX A
Map and Project Reach Showing Initially Proposed and Modified Design

APPENDIX B
Summary of Fine Sediment Yield Calculations

LS Factors

Basin	Lavg(ft)	Savg(ft/ft)	n	LS
w.branch	400	0.056	0.50	1.23
n. branch	400	0.043	0.40	0.75
cliffs	400	0.026	0.30	0.38

K Factors

	AmB	Bb	BCC	BKD	LtB	MaB	MWA	PAC	Total (frac)	Avg K	Area (Ac)	% Area	Basin K*%
Basin / K:	0.24	0.17	0.17	0.17	0.24	0.20	0.28	0.17					
A1		0.03	0.07	0.03	0.05	0.48		0.34	1.00	0.19	1696	0.30	0.06
A2			0.04	0.70	0.22	0.05			1.00	0.19	377	0.07	0.01
B			0.13	0.38	0.37	0.12			1.00	0.20	310	0.05	0.01
C	0.12		0.20	0.06		0.30	0.05	0.26	1.00	0.19	1907	0.34	0.06
D	0.06		0.66			0.25	0.03		1.00	0.18	1395	0.25	0.05
										Total Basin	5685	1.00	0.19

Total Fines (low estimate)

	AmB	Bb	BCC	BKD	LtB	MaB	MWA	PAC		Avg	Area	% of Total	
Basin / %Fines	30	5	5	3	25	30	38	30	Total (frac)	%Fines	Acres	Area	
A1		0.03	0.07	0.03	0.05	0.48		0.34	1.00	26.27	1696	0.30	7.84
A2			0.04	0.70	0.22	0.05			1.00	9.19	377	0.07	0.61
B			0.13	0.38	0.37	0.12			1.00	14.69	310	0.05	0.80
C	0.12		0.20	0.06		0.30	0.05	0.26	1.00	23.67	1907	0.34	7.94
D	0.06		0.66			0.25	0.03		1.00	13.68	1395	0.25	3.36
W. Branch											5685	1.00	20.55

Total Fines (high estimate)

	AmB	Bb	BCC	BKD	LtB	MaB	MWA	PAC		Avg	Area	% of Total	
Basin / %Fines	50	20	20	20	50	50	50	60	Total (frac)	%Fines	Acres	Area	
A1		0.03	0.07	0.03	0.05	0.48		0.34	1.00	49.27	1696	0.30	14.70
A2			0.04	0.70	0.22	0.05			1.00	28.02	377	0.07	1.86
B			0.13	0.38	0.37	0.12			1.00	34.78	310	0.05	1.90
C	0.12		0.20	0.06		0.30	0.05	0.26	1.00	44.61	1907	0.34	14.96
D	0.06		0.66			0.25	0.03		1.00	30.18	1395	0.25	7.40
W. Branch											5685	1.00	40.82

2-year Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	135	31	0.189	1.231	0.4	2830	5031	0.56	20.5	40.8	581	0.1	13609	1155	0.2	26687	868	0.2	20191
N. Branch	962	229	0.28	0.749	0.4	23476	42586	0.55	20.5	40.8	4822	0.1	15257	9583	0.2	29871	7203	0.2	22619

5-yr Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	411	92	0.189	1.231	0.4	9707	5031	1.93	20.5	40.8	1994	0.4	15696	3963	0.8	30718	2978	0.6	23265
N. Branch	3091	735	0.28	0.749	0.4	86720	42586	2.04	20.5	40.8	17812	0.4	17519	35399	0.8	34225	26606	0.6	25944

10-yr Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	648	143	0.189	1.231	0.4	16036	5031	3.19	20.5	40.8	3294	0.7	16665	6546	1.3	32584	4920	1.0	24690
N. Branch	4881	1161	0.28	0.749	0.4	144682	42586	3.40	20.5	40.8	29718	0.7	18486	59059	1.4	36079	44389	1.0	27362

25-yr Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	952	208	0.189	1.231	0.4	24534	5031	4.88	20.5	40.8	5039	1.0	17514	10015	2.0	34216	7527	1.5	25937
N. Branch	7171	1707	0.28	0.749	0.4	222700	42586	5.23	20.5	40.8	45743	1.1	19336	90906	2.1	37707	68324	1.6	28608

50-yr Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	1209	262	0.189	1.231	0.4	31918	5031	6.34	20.5	40.8	6556	1.3	18079	13029	2.6	35298	9792	1.9	26765
N. Branch	9115	2170	0.28	0.749	0.4	291357	42586	6.84	20.5	40.8	59845	1.4	19888	118932	2.8	38764	89388	2.1	29418

100-yr Cf

									min	max	min			max			Average		
Basin	Qp (cfs)	Vw (ac-ft)	K	LS	CP	Ys (tons)	A(acres)	Ys(t/acre)	% Fines	% Fines	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)	Ysf(tons)	Ysf(t/acre)	Cf(ppm)
W. Branch	1478	320	0.189	1.231	0.4	39951	5031	7.94	20.5	40.8	8206	1.6	18519	16308	3.2	36143	12257	2.4	27411
N. Branch	11139	2652	0.28	0.749	0.4	364737	42586	8.56	20.5	40.8	74917	1.8	20362	148886	3.5	39670	111901	2.6	30112

Avg Annual Cf

Basin	Ysf(tons)	A(acres)	Vw(ac-ft)	Ysf(t/ac)	Cf(ppm)	Future Cf
W. Branch	1968.339	5031	59.3	0.39	23845	6669
N. Branch	17505.58	42586	472.1	0.41	26559	13157