AGUA FRIA

WATERCOURSE MASTER PLAN ADDENDUM

CHANNELIZATION ALTERNATIVE - VOLUME III

TECHNICAL ANALYSIS

FINAL REPORT

11-30-04

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EDAW

Flood Control District of Maricopa County

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

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DATE: November 30, 2004

TO: Doug Williams, AICP - FCDMC

FROM: Jon Fuller, PE

RE: Agua Fria Channelization Project
FCD 2002 C009 – Assignment #5

CC: Roger Baele, PE – DEA
Jay Hicks, RLA - EDAW

Registered Professional Engineer (Civil)
CERTIFICATE NO. 23846
JONATHAN E. FULLER
Date Signed 11-30-04
ARIZONA, U.S.A.

Introduction

This memorandum summarizes the results of technical analyses performed by JE Fuller/Hydrology & Geomorphology, Inc. (JEF) in support of the proposed Agua Fria River Watercourse Master Plan (AFR WCMP) amendment. The amended AFR WCMP recommends that the Agua Fria River be channelized from the Indian School Road Bridge to the Central Arizona Project (CAP) siphon crossing, as described in the *Agua Fria River Channelization Conceptual Plan* (JEF, 2002).

JEF originally prepared this memorandum for the Flood Control District of Maricopa County (District) as the deliverable for Work Assignment #5 of On-Call Contract No. FCD 2002C009. Work Assignment #5 was prepared in conjunction with work performed by David Evans & Associates (DEA) and EDAW, both of whom have prepared separate reports summarizing their contributions to the AFR WCMP amendment. This revision of the original JEF memorandum was prepared under Work Assignment #4 of On-Call Contract No. FCD 2004C025. The results of the following technical analyses performed by JEF are described in this memorandum:

- ***Channel Parameters***. The channel parameters analysis included consideration of the minimum recommended channel radius, freeboard, grade control structures, flood channel/terrace transition, transition from non-channelized to channelized reaches, tributary inflows, and potential variation in channel roughness and its impact on channel capacity.
- ***Bank Protection***. An engineered bank protection design has been proposed and described by DEA. Conceptual design details for bioengineering alternatives to traditional engineering bank protection are presented.
- ***Sedimentation Engineering***. The sedimentation engineering analysis included estimates of scour depth and toe down at bank protection, grade control structures, drop structures, and bridge piers, as well as consideration of armoring potential and sediment continuity.
- ***Erosion Hazard Zone Delineation***. Erosion hazard zones previously delineated for the AFR WCMP were revised to reflect changes resulting from implementation of the channelization alternative.

Data Sources

Data for the AFR WCMP amendment technical analyses were obtained from the following sources:

- Channel Alignment. The recommended channel alignment was prepared by DEA, in conjunction with JEF, EDAW, and District staff.
- Channel Profile. The channel profile was developed by DEA as described in the DEA deliverables for this project.
- Hydraulic Structure Design. Design details for bank protection, grade controls, drop structures, channel geometry, and terrace configurations were provided by DEA.
- Sediment Data. Sediment gradation data were obtained from the original Agua Fria River Watercourse Master Plan (KHA, 2002).
- HEC-RAS Model. A HEC-RAS model for the proposed channel alignment and profile was developed by JEF using topographic data, structure detail drawings, and plan/profile drawings provided by DEA. Preliminary HEC-RAS model results were used to refine the recommended corridor geometry to provide the required conveyance.
- Design Guidelines. Technical guidance for the analyses summarized in this memorandum was obtained from the District's *Drainage Design Manual for Maricopa County - Hydraulics* (2004).

HEC-RAS Modeling Notes

Bridge Data. Bridge data were obtained from as-built plans collected by DEA and provided to JEF. DEA surveyors provided datum adjustments for bridge elevations shown on the as-built plans to facilitate their use with topographic data in the HEC-RAS model. Where as-built plans did not provide adequate information, HEC-RAS models were based on previous HEC-RAS or HEC-2 models prepared for floodplain delineation studies that had been prepared for and approved by the District. No horizontal control data were available from which to locate bridge piers and abutments relative to the proposed channelization alignment. Therefore, horizontal position of bridge piers and abutments were estimated by comparing as-built plans and the bridge position shown on digital aerial photographs provided by the District.

Roughness Coefficients. Manning's N values were selected based on engineering judgment, previous floodplain delineation study models of the Agua Fria River, and District technical guidelines for selecting Manning's N values. It was assumed that periodic channel maintenance would be preformed to prevent vegetation or other impacts from significantly increasing channel roughness. In general, an N value of 0.030 was used for the flood channel, and 0.045 was used for the terrace. For comparison purposes, a HEC-RAS model with channel N values of 0.045 and terrace N values of 0.065 was run to simulate increased roughness to due poor maintenance and/or vegetative growth. Use of higher N values resulted in 100-year water surface elevations that averaged 1.6 feet

higher than the subcritical profile model, with a maximum increase of 2.5 feet within the channelization reach. The HEC-RAS results indicate that if roughness values increase to the level modeled, the increase in the 100-year water surface elevation would deplete the freeboard but not overtop the channel. Given the short duration of expected peak discharges, and the likely removal of roughness factors by high stage and velocity during actual floods, potential loss of some freeboard may not be a significant problem. Specific issues related to channel capacity and freeboard are discussed elsewhere in this memorandum. HEC-RAS results from the "low N" model were used for scour calculations because they are conservative with respect to velocity. A technical memorandum supporting and justifying the N values used in the HEC-RAS models is provided in the Appendix.

Flood Channel Width Transitions. The proposed channel configuration includes a contraction of the terrace to convey the entire discharge over drop structures that are narrower than the 1,000-foot corridor. At the drop structure transitions, the flood channel widens from 500 feet to 600 feet upstream of the drop structure, and then contracts to 500 feet downstream of the drop structure. At the point where the flood channel contracts to 500 feet, the floodplain terrace contraction ends and overbank flow is allowed to spread over the terrace. Additional losses in these transitions were accounted for by increasing the contraction and expansion coefficients to 0.3 and 0.5, respectively.

Ineffective Flow Areas. Ineffective flow areas were defined upstream and downstream of corridor/terrace constrictions, using the assumptions of 4:1 flow expansion and 1:1 flow contraction.

Cross Section Geometry. Cross section geometry was obtained from the AutoCAD plan and profile drawings and design details provided by DEA. Cross sections were spaced at 500-foot intervals, except where closer spacing was required to depict geometry changes at constrictions, bends, and structures. Cross sections were aligned perpendicular to the primary flow direction. The HEC-RAS model was coded assuming the cement-stabilized alluvium (CSA) bank protection would be mantled by soil material installed at 4:1 (terrace margin) and 3:1 (corridor margin) slopes, except in the channel expansion and contraction reach adjacent to the drop structures and bridges, and in the narrowed reach from downstream end of the El Mirage Landfill to the upstream end of the Vulcan Materials aggregate mine, where the limited corridor width required steeper side slopes and no mantling.

Skewed Drop Structures. Several drop structures located at bridge crossings are designed with significant skew angles to the flood channel. It is likely that the hydraulics of the skewed drop structures are more complex than depicted by the one-dimensional HEC-RAS model.[1] Therefore, further hydraulic analysis of these drop structures is recommended prior to final design.

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[1] During project team meetings, DEA presented compelling evidence that the increased costs associated with construction of non-skewed drop structures adjacent to skewed (existing) bridges were prohibitive and justified the increased hydraulic complexity and uncertainty at the structures. At these meetings, District staff approved continued use of the skewed drop structures for this level of analysis.

Mixed Profile. The HEC-RAS model indicates that supercritical flow occurs over the face of most of the 6:1 drop structures. Mixed profile HEC-RAS models were run to evaluate the results relative to the subcritical profile. In general, the mixed profile results were identical to the subcritical profile, with several notable exceptions. Table A-1 showing the comparison of water surface elevations and velocities is provided in the Appendix. The water surface elevations from the subcritical profile are conservative with respect to capacity and were therefore used to estimate water surface elevations.

Channel Parameters

Channel Radius. The *District's Drainage Design Manual – Hydraulics* lists the following equation to compute the minimum recommended radius of curvature for constructed channels with subcritical flow:

$$R_c \geq 3\,T \qquad \text{(Eq'n 6.26)}$$

Where: R_c = radius of curvature (ft)
T = channel top width (ft)

For the recommended typical channel cross section (500-foot flood channel, 1,000-foot total channel), the minimum radii of curvature computed from Equation 6.26 are 1,500 feet for the flood channel and 3,000 feet for the total channel, respectively. Inspection[1] of the channel alignment prepared by DEA indicates that the minimum radius for the flood channel is 1,500 ft,[2] but that the average radius is well above 1,500 feet. The radius of curvature for the total channel is frequently below the 3,000 feet minimum suggested by Equation 6.26. However, since the proposed low flow and corridor sinuosity is similar to the sinuosity of the pre-development natural channel and floodplain of the Agua Fria River, it is likely that the proposed alignment reflects reasonable river mechanics relationships with respect to channel radius and curvature.

The channel radius analysis suggests that for flows that overtop the flood channel and inundate the terrace, helicoidal flow may occur and roughness values may be underestimated by as much as 0.003 (USACE, 1995, Hydraulic Design of Flood Control Channels EM 1110-2-1601). Potential impacts on water surface elevation caused by helicoidal flow may be addressed for by designing the channel with adequate freeboard and by accounting for superelevation in the freeboard allowance. Freeboard and superelevation were accounted for in the channel capacity analysis described below.

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[1] Curve radius on the DEA AutoCAD plan was determined by selecting individual curve segments and using the "list" command.

[2] Station 552+00 to 565+00 near the drop structure on the Bethany Home Road alignment.

Freeboard. The *District's Drainage Design Manual – Hydraulics* lists the following equation to compute the minimum recommended freeboard for constructed channels with subcritical flow:

$$FB = 0.25\ (y + V^2/2g) \qquad \text{(Eq'n 6.25)}$$

Where:
FB = freeboard (ft)
y = flow depth (ft)
V = average channel velocity (ft/sec)
g = gravitational constant, 32.2 ft/sec^2

The project design is based on providing capacity for the 100-year water surface elevation plus freeboard. HEC-RAS results suggest an average 100-year freeboard of 2.3 feet, with a maximum of 3.4 feet. Freeboard is added to the computed water surface elevation after consideration of superelevation, as shown in Table 2.

For reaches with levees,[1] the minimum freeboard is usually dictated by FEMA standards, which range from three to four feet depending on the location relative to hydraulic structures. U.S. Army Corps of Engineers (USACE) freeboard standards were also considered, given the presence of the USACE levees downstream of Camelback Road. The USACE now computes freeboard using risk-based economic analyses, rather than by hydraulic formulae. In the past, the USACE used freeboard criteria shown in Table 1. Information on USACE freeboard standards for channel with levees is summarized in EM-1110-2-1913: Design and Construction of Levees (USACE, 2000).

Table 1. Freeboard Criteria

Agency	Channel Type	Standard	Source
USACE	Open Channels	Risk-based economic analysis	EM 1110-2-1913 (2000 Version)
FCDMC	Open Channels	Formula	Drainage Design Manual: Hydraulics
USACE/ FCDMC	New River Levee	SPF: WSEL + 1 ft Q100: WSEL + 3 ft Q100: WSEL + 4 ft.	+ superelevation in curves @ Bridges
FEMA	Levee	WSEL + 3-4 ft	CFR 44: Parts 60-70

Superelevation. The *District's Drainage Design Manual – Hydraulics* lists the following equation to compute the superelevation for constructed channels with subcritical flow:

$$y = (0.5\ V^2\ T) / (g\ R_c)$$

Where:
y = flow depth (ft)
V = average channel velocity (ft/sec)
T = channel top width (ft)
R_c = radius of curvature (ft)
g = gravitational constant, 32.2 ft/sec^2

[1] Levee reaches are currently proposed upstream of Grand Avenue. Other potential levee sections may exist elsewhere within the project limits, but require new topographic mapping and design plans with identified bank elevations.

For the AFR WCMP channel, superelevation averages 0.1 foot, with a maximum computed superelevation of 1.5 foot, based on the subcritical low N HEC-RAS profile.

Channel Capacity Analysis. Locations where the HEC-RAS results indicate that the proposed channel geometry and alignment did not contain the 100-year water surface elevation plus freeboard and superelevation are listed in Tables 2 and 3, as is a recommended action to provide increased capacity. Channel capacity information for the SPF profiles is provided in the Appendix in Table A-3. Because the final design of the channelization corridor is likely to include some revision of the current design concept, and will need to address issues such as flood storage in sand and gravel mines that would affect peak discharges, it is recommended that reaches with slight capacity deficits be addressed during final design. In no case was capacity deficits computed that could not be addressed by modifying the current design with the proposed footprint. In most cases, increases in water surface elevation due to increased roughness merely diminished the freeboard available, rather than exceeding the actual capacity of the corridor. Refer to HEC-RAS model results for more detailed information. The following information regarding channel capacity was derived from the HEC-RAS modeling:

- Increased roughness due to vegetative growth or other obstructions in the channel (High N HEC-RAS model) will compromise the normal freeboard requirement, but will not cause 100-year water surface elevations to increase above the designed bank elevation.
- Additional bank height (greater than 12 feet) is recommended in the drop structure transition reaches.
- The SPF peak is contained in the proposed channel without freeboard, but the SPF with the higher N values is not contained in numerous reaches.
- Channel capacity can be increased by any of the following measures:
    - Increasing the flood channel width (e.g., 600-ft flood channel with 400-ft terrace).
    - Decreasing the terrace height (i.e., more frequent inundation of the terrace).
    - Increasing the bank height or levee height.
    - Maintaining channel roughness to design levels.
- Decreasing flood peaks by converting (or adapting) existing sand and gravel excavations to flood storage basins. Preliminary calculations indicate that the Walker Pit could store the entire 100-year design hydrograph for the Agua Fria River.

Table 2. Channel Capacity Including Freeboard, Superelevation from 100-Year HEC-RAS Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
Levee Reach Adjacent to Vulcan Aggregate Mine							
94682*	1126.56	2.8	0.1	1129.4	1129.01	0.4	Increase levee height
94582	1126.46	2.8	0.1	1129.3	1128.85	0.5	
94182	1125.41	2.9	0.1	1128.4	1128.21	0.2	
93932	1125.05	2.9	0.1	1128.0	1127.81	0.2	
93682	1124.71	2.8	0.1	1127.6	1127.41	0.2	
93432	1124.4	2.8	0.1	1127.3	1127.01	0.3	
93182	1124.04	2.9	0.1	1127.0	1126.61	0.4	
93000	1123.76	2.9	0.1	1126.7	1126.32	0.4	
92500	1122.59	2.9	0.1	1125.6	1125.52	0.1	
Downstream of Toe of Drop Structure at Cactus Road Alignment							
83488	1103.99	2.7	0.1	1106.8	1106.29	0.5	Increase slope paving along landfill
83380	1103.94	2.7	0.1	1106.7	1106.12	0.6	
Downstream Toe of Drop Structure Upstream of El Mirage Wash Confluence							
79688	1093.53	2.7	0.1	1096.3	1096.21	0.1	Increase bank height through drop structure
79588	1093.42	2.7	0.1	1096.2	1096.05	0.2	
Proposed Mining Reach Between Peoria Ave & Olive Ave Alignments							
75988	1083.89	2.8	0.1	1086.7	1086.29	0.4	Widen flood channel 100 ft or raise bank height
75888	1083.8	2.8	0.1	1086.7	1086.13	0.5	
75488	1082.78	2.9	0.1	1085.8	1085.49	0.3	
75238	1082.4	2.9	0.1	1085.4	1085.09	0.3	
74988	1082.03	2.9	0.1	1085.0	1084.69	0.3	
74738	1081.75	2.8	0.1	1084.7	1084.29	0.4	
74488	1081.21	2.9	0.1	1084.2	1083.89	0.3	
74238	1080.82	2.9	0.1	1083.8	1083.49	0.3	
74000	1080.32	2.9	0.1	1083.3	1083.11	0.2	
73500	1079.46	2.9	0.1	1082.4	1082.31	0.1	
73000	1078.74	2.8	0.1	1081.7	1081.51	0.2	
Downstream Toe of Drop Structure at Northern Ave							
66988	1062.29	2.8	0.1	1065.2	1064.42	0.8	Increase bank height through drop structure
66888	1062.2	2.8	0.1	1065.1	1064.27	0.8	
Downstream Toe of Drop Structure at Glendale Ave Bridge							
61558.5	1048.21	2.8	0.1	1051.1	1050.78	0.3	Increase bank height through drop structure
61458.5	1048.1	2.8	0.1	1050.9	1050.63	0.3	
61258.5	1047.66	2.8	0.1	1050.6	1050.33	0.2	
Downstream Toe of Drop Structure Between Glendale Ave & Bethany Home Rd							
58482	1037.48	2.7	0.1	1040.3	1040.16	0.1	Increase bank height through drop structure
58382	1037.36	2.7	0.1	1040.2	1040.01	0.2	
58182	1036.89	2.8	0.1	1039.8	1039.71	0.1	
Downstream Toe of Drop Structure at Bethany Home Rd							
55782	1027.34	2.7	0.1	1030.2	1030.11	0.0	Increase bank height through drop structure
55682	1027.23	2.7	0.1	1030.0	1029.96	0.1	

Note: 1. Required elevation is sum of 100-year WSEL, freeboard, and superelevation.

Table 3. Channel Capacity Including Freeboard, Superelevation
100-Year HEC-RAS High N Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
Downstream of Drop Structure at Agua Fria Blvd							
130876	1231.89	2.7	0.1	1234.7	1234.31	0.3	Widen flood channel & increase bank height through drop structure transition reach
130664	1231.50	2.7	0.1	1234.3	1233.82	0.5	
130264	1230.20	2.8	0.1	1233.1	1232.88	0.2	
130014	1229.60	2.7	0.1	1232.4	1232.30	0.1	
129764	1229.01	2.7	0.1	1231.8	1231.71	0.1	
129500	1228.37	2.7	0.1	1231.2	1231.10	0.1	
127000	1222.55	2.7	0.1	1225.3	1225.25	0.0	
126500	1221.52	2.7	0.1	1224.3	1224.08	0.2	
Downstream of Drop Structure Above Pinnacle Peak Road							
125488	1215.29	2.7	0.1	1218.1	1217.76	0.3	Widen flood channel & increase bank height through drop structure transition reach
125388	1215.09	2.7	0.1	1217.9	1217.53	0.3	
124988	1213.82	2.8	0.1	1216.7	1216.61	0.1	
122500	1208.16	2.7	0.1	1210.9	1210.89	0.0	
122000	1207.08	2.7	0.1	1209.8	1209.74	0.1	
121500	1206.05	2.7	0.1	1208.8	1208.59	0.2	
Downstream of Drop Structure Above Walker Pit							
120488	1200.41	2.8	0.1	1203.3	1202.32	1.0	Widen flood channel & increase bank height through drop structure transition reach
120388	1200.25	2.8	0.1	1203.1	1202.11	1.0	
119988	1199.00	2.9	0.1	1202.0	1201.27	0.7	
119500	1197.72	2.8	0.1	1200.6	1200.25	0.4	
119000	1196.51	2.7	0.1	1199.3	1199.20	0.1	
118500	1195.48	2.7	0.1	1198.2	1198.15	0.1	
118000	1194.53	2.7	0.1	1197.3	1197.10	0.2	
117500	1193.53	2.7	0.1	1196.3	1196.05	0.2	
117000	1192.53	2.7	0.1	1195.3	1195.00	0.3	
116715	1191.98	2.7	0.1	1194.8	1194.40	0.4	
116465	1191.14	2.7	0.1	1193.9	1193.88	0.1	
Downstream of Drop Structure Below Walker Pit							
114982	1177.71	2.8	0.1	1180.5	1179.90	0.6	Widen flood channel & increase bank height through drop structure transition reach
114882	1177.53	2.8	0.1	1180.4	1179.69	0.7	
114482	1176.85	2.8	0.1	1179.7	1178.85	0.8	
114185	1175.92	2.8	0.1	1178.9	1178.23	0.6	
113935	1175.30	2.8	0.1	1178.2	1177.70	0.5	
113685	1174.70	2.8	0.1	1177.6	1177.18	0.4	
113435	1174.11	2.7	0.1	1176.9	1176.65	0.3	
113185	1173.54	2.7	0.1	1176.3	1176.13	0.2	
113000	1173.13	2.7	0.1	1175.9	1175.75	0.2	
112500	1172.07	2.7	0.1	1174.8	1174.70	0.1	
112400	1171.86	2.7	0.1	1174.6	1174.49	0.1	
112300	1171.64	2.7	0.1	1174.4	1174.28	0.1	
112200	1171.45	2.7	0.1	1174.2	1174.07	0.1	

Table 3. Channel Capacity Including Freeboard, Superelevation
100-Year HEC-RAS High N Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
112100	1171.25	2.7	0.1	1174.0	1173.86	0.1	
112000	1171.06	2.7	0.1	1173.8	1173.65	0.2	
111900	1170.86	2.7	0.1	1173.6	1173.44	0.2	
111800	1170.66	2.7	0.1	1173.4	1173.23	0.2	
111700	1170.46	2.7	0.1	1173.2	1173.02	0.2	
111600	1170.26	2.7	0.1	1173.0	1172.81	0.2	
111500	1170.06	2.7	0.1	1172.8	1172.60	0.2	
111043	1169.16	2.7	0.1	1171.9	1171.63	0.3	
110793	1168.37	2.7	0.1	1171.2	1171.11	0.1	
Downstream of Drop Structure Above McMicken Dam Outfall Confluence							
110007	1162.43	3.0	0.0	1165.5	1163.53	1.9	Widen flood channel & increase bank height through drop structure transition reach
109907	1162.31	3.0	0.0	1165.3	1163.37	2.0	
109507	1161.34	3.0	0.1	1164.4	1162.73	1.7	
109257	1160.87	3.0	0.1	1163.9	1162.33	1.6	
109007	1160.42	3.0	0.1	1163.4	1161.93	1.5	
108757	1159.99	2.9	0.1	1163.0	1161.53	1.5	
108507	1159.44	2.9	0.1	1162.4	1161.13	1.3	
108257	1159.05	2.9	0.1	1162.0	1160.73	1.3	
108007	1158.67	2.9	0.1	1161.6	1160.33	1.3	
107507	1157.89	2.9	0.1	1160.8	1159.53	1.3	
107000	1157.12	2.9	0.1	1160.1	1158.72	1.3	
106500	1156.35	2.9	0.1	1159.3	1157.92	1.4	
106000	1155.53	2.9	0.1	1158.5	1157.12	1.4	
105500	1154.71	2.9	0.1	1157.7	1156.32	1.3	
105000	1153.87	2.9	0.1	1156.8	1155.52	1.3	
104500	1153.04	2.9	0.1	1156.0	1154.72	1.3	
104000	1152.20	2.9	0.1	1155.1	1153.92	1.2	
103500	1151.35	2.9	0.1	1154.3	1153.12	1.2	
103000	1150.48	2.9	0.1	1153.4	1152.32	1.1	
102500	1149.62	2.8	0.1	1152.5	1151.52	1.0	
102180	1148.85	2.8	0.1	1151.7	1151.01	0.7	
101930	1147.84	2.8	0.1	1150.7	1150.61	0.1	
Downstream of Drop Structure at Bell Rd							
101238	1144.32	3.0	0.1	1147.4	1145.50	1.9	Widen flood channel & increase bank height through drop structure transition reach
101138	1144.16	3.0	0.1	1147.2	1145.34	1.9	
100738	1143.13	3.0	0.1	1146.2	1144.70	1.5	
100488	1142.64	3.0	0.1	1145.7	1144.30	1.4	
100238	1142.19	2.9	0.1	1145.2	1143.90	1.3	
99988	1141.77	2.9	0.1	1144.7	1143.50	1.2	
99738	1141.38	2.9	0.1	1144.3	1143.10	1.2	
99488	1140.98	2.9	0.1	1143.9	1142.70	1.2	
99000	1140.21	2.9	0.1	1143.1	1141.92	1.2	
98500	1139.41	2.9	0.1	1142.3	1141.12	1.2	

Table 3. Channel Capacity Including Freeboard, Superelevation
100-Year HEC-RAS High N Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
98000	1138.62	2.9	0.1	1141.5	1140.32	1.2	
97500	1137.87	2.9	0.1	1140.8	1139.52	1.3	
97000	1137.04	2.9	0.1	1140.0	1138.72	1.3	
96500	1136.18	2.9	0.1	1139.1	1137.92	1.2	
96000	1135.30	2.9	0.1	1138.2	1137.12	1.1	
95718	1134.79	2.8	0.1	1137.7	1136.67	1.0	
95468	1134.02	2.9	0.1	1137.0	1136.27	0.7	
Levee Reach Adjacent to Vulcan Aggregate Mine							
94682	1128.64	3.2	0.0	1131.8	1129.01	2.8	Widen flood channel & increase bank height through drop structure transition reach
94582	1128.52	3.2	0.0	1131.7	1128.85	2.9	
94182	1127.63	3.2	0.1	1130.9	1128.21	2.7	
93932	1127.23	3.2	0.1	1130.5	1127.81	2.7	
93682	1126.85	3.2	0.1	1130.1	1127.41	2.7	
93432	1126.51	3.2	0.1	1129.7	1127.01	2.7	
93182	1126.15	3.2	0.1	1129.4	1126.61	2.8	
93000	1125.88	3.2	0.1	1129.1	1126.32	2.8	
92500	1124.94	3.2	0.1	1128.2	1125.52	2.7	
92000	1124.24	3.2	0.1	1127.5	1126.72	0.7	
91600	1123.49	3.2	0.1	1126.8	1126.08	0.7	
91500	1123.33	3.2	0.1	1126.6	1125.92	0.7	
91000	1122.50	3.2	0.1	1125.8	1125.12	0.6	
90500	1121.66	3.2	0.1	1124.9	1124.32	0.6	
90000	1120.81	3.2	0.1	1124.0	1123.52	0.5	
89500	1119.94	3.2	0.1	1123.2	1122.72	0.4	
89000	1119.05	3.1	0.1	1122.3	1121.92	0.3	
88500	1117.89	3.1	0.1	1121.1	1121.00	0.1	
Downstream of Toe of Drop Structure at Cactus Road Alignment							
83488	1105.81	3.1	0.0	1109.0	1106.29	2.7	Widen flood channel & increase bank height through drop structure transition reach
83380	1105.72	3.1	0.0	1108.9	1106.12	2.7	
83280	1104.98	3.2	0.1	1108.3	1105.96	2.4	
83000	1104.49	3.1	0.1	1107.7	1105.51	2.2	
82500	1103.52	3.1	0.1	1106.7	1104.71	2.0	
82000	1102.56	3.0	0.1	1105.7	1103.91	1.7	
81500	1101.62	3.0	0.1	1104.7	1103.11	1.6	
81000	1100.56	2.9	0.1	1103.6	1102.31	1.3	
80592	1099.64	2.9	0.1	1102.6	1101.66	0.9	
80462	1099.30	2.9	0.1	1102.2	1101.45	0.8	
Downstream Toe of Drop Structure Upstream of El Mirage Wash Confluence							
79688	1095.42	3.1	0.1	1098.5	1096.21	2.3	Widen flood channel & increase bank height
79588	1095.29	3.1	0.1	1098.4	1096.05	2.4	
79188	1094.24	3.1	0.1	1097.4	1095.41	2.0	
78938	1093.74	3.1	0.1	1096.9	1095.01	1.9	
78688	1093.26	3.0	0.1	1096.4	1094.61	1.8	

Table 3. Channel Capacity Including Freeboard, Superelevation
100-Year HEC-RAS High N Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
78438	1092.79	3.0	0.1	1095.9	1094.21	1.6	through drop structure transition reach
78188	1092.35	3.0	0.1	1095.4	1093.81	1.6	
77938	1091.90	3.0	0.1	1094.9	1093.41	1.5	
77688	1091.44	2.9	0.1	1094.4	1093.01	1.4	
77500	1091.08	2.9	0.1	1094.1	1092.71	1.4	
77000	1090.07	2.9	0.1	1093.0	1091.91	1.1	
76872	1089.81	2.9	0.1	1092.8	1091.71	1.1	
Proposed Mining Reach Between Peoria Ave & Olive Ave Alignments							
75988	1085.92	3.2	0.0	1089.1	1086.29	2.8	Widen flood channel & increase bank height through drop structure transition reach
75888	1085.80	3.2	0.0	1089.0	1086.13	2.9	
75488	1084.91	3.2	0.1	1088.2	1085.49	2.7	
75238	1084.48	3.2	0.1	1087.8	1085.09	2.7	
74988	1084.05	3.2	0.1	1087.3	1084.69	2.6	
74738	1083.70	3.2	0.1	1086.9	1084.29	2.6	
74488	1083.15	3.2	0.1	1086.4	1083.89	2.5	
74238	1082.67	3.2	0.1	1085.9	1083.49	2.4	
74000	1082.10	3.1	0.1	1085.3	1083.11	2.2	
73500	1080.95	3.1	0.1	1084.1	1082.31	1.8	
73000	1079.85	3.0	0.1	1082.9	1081.51	1.4	
72892	1079.32	3.0	0.1	1082.4	1081.34	1.1	
72692	1078.86	2.9	0.1	1081.8	1081.02	0.8	
72492	1078.45	2.8	0.1	1081.3	1080.70	0.6	
72392	1078.11	2.8	0.1	1081.0	1080.54	0.4	
72288	1075.08	3.0	0.1	1078.1	1076.37	1.8	
72188	1074.93	3.0	0.1	1078.0	1076.22	1.8	
71988	1074.62	3.0	0.1	1077.7	1075.92	1.7	
71788	1074.04	2.9	0.1	1077.0	1075.62	1.4	
71500	1073.56	2.9	0.1	1076.5	1075.19	1.4	
71000	1072.79	2.9	0.1	1075.7	1074.44	1.3	
70500	1072.09	2.9	0.1	1075.0	1073.69	1.3	
70000	1071.43	2.9	0.0	1074.3	1072.94	1.4	
69500	1070.71	2.9	0.1	1073.7	1072.19	1.5	
69000	1069.96	2.9	0.1	1072.9	1071.44	1.5	
68500	1069.17	2.9	0.1	1072.1	1070.69	1.4	
68000	1068.34	2.9	0.1	1071.3	1069.94	1.3	
Downstream Toe of Drop Structure at Northern Ave							
66988	1063.91	3.1	0.1	1067.1	1064.42	2.7	Widen flood channel & increase bank height through drop structure transition reach
66888	1063.78	3.1	0.1	1067.0	1064.27	2.7	
66688	1062.68	3.2	0.1	1066.0	1063.97	2.0	
66488	1062.27	3.1	0.1	1065.4	1063.67	1.8	
66000	1061.14	3.0	0.1	1064.2	1062.94	1.3	
65500	1060.39	3.0	0.1	1063.5	1062.19	1.3	
65000	1059.73	2.8	0.0	1062.6	1061.44	1.2	

Table 3. Channel Capacity Including Freeboard, Superelevation
100-Year HEC-RAS High N Model

River Station	WSEL	Free-board (ft)	Super-elevation (ft)	Required Elevation[1]	Top of Designed Channel	Additional Capacity Needed (ft)	Recommended Action to Provide Capacity
64500	1059.21	2.8	0.0	1062.1	1060.69	1.4	
64000	1058.45	2.9	0.1	1061.4	1059.94	1.5	
63500	1057.67	2.9	0.1	1060.6	1059.19	1.4	
63000	1056.88	2.9	0.1	1059.8	1058.43	1.4	
62500	1055.82	2.9	0.1	1058.8	1057.68	1.1	
Downstream Toe of Drop Structure at Glendale Ave Bridge							
61558.5	1050.07	3.1	0.1	1053.2	1050.78	2.4	Widen flood channel & increase bank height through drop structure transition reach
61458.5	1049.93	3.1	0.1	1053.1	1050.63	2.5	
61258.5	1049.50	3.1	0.1	1052.7	1050.33	2.4	
61058.5	1048.91	3.1	0.1	1052.1	1050.03	2.1	
61000	1048.80	3.1	0.1	1052.0	1049.94	2.1	
60500	1047.82	3.0	0.1	1050.9	1049.19	1.7	
60000	1046.89	3.0	0.1	1049.9	1048.44	1.5	
59500	1046.00	2.9	0.1	1049.0	1047.69	1.3	
Downstream Toe of Drop Structure Between Glendale Ave & Bethany Home Rd							
58482	1039.24	3.1	0.1	1042.4	1040.16	2.2	Widen flood channel
58382	1039.09	3.1	0.1	1042.2	1040.01	2.2	
58182	1038.62	3.1	0.1	1041.8	1039.71	2.0	
57982	1037.97	3.1	0.1	1041.1	1039.41	1.7	
57500	1036.99	2.9	0.1	1040.0	1038.69	1.3	
57000	1036.11	2.9	0.1	1039.0	1037.94	1.1	
56500	1035.16	2.8	0.1	1038.0	1037.19	0.9	
Downstream Toe of Drop Structure at Bethany Home Rd							
55782	1029.13	3.0	0.1	1032.2	1030.11	2.1	Widen flood channel
55682	1028.98	3.0	0.1	1032.1	1029.96	2.1	
55482	1028.51	3.1	0.1	1031.6	1029.66	2.0	
55282	1027.84	3.1	0.1	1031.0	1029.36	1.6	
55000	1027.16	3.0	0.1	1030.2	1028.94	1.3	
54500	1025.92	2.9	0.1	1028.9	1028.19	0.7	
54000	1024.84	2.7	0.1	1027.6	1027.44	0.2	
Downstream of Camelback Road & New River Confluence							
49709.6	1015.19	2.7	0.0	1017.9	1017.14	0.8	Widen flood channel
49500	1014.99	2.7	0.0	1017.7	1016.95	0.8	
49000	1014.53	2.7	0.0	1017.2	1016.50	0.7	
48500	1014.05	2.7	0.0	1016.8	1016.05	0.7	
48000	1013.48	2.7	0.0	1016.2	1015.60	0.6	
47500	1012.75	2.6	0.0	1015.4	1015.15	0.3	
47000	1012.15	2.6	0.0	1014.7	1014.70	0.0	

Note: 1. Required elevation is sum of 100-year WSEL, freeboard, and superelevation.

Low Flow Channel. The *District's Drainage Design Manual – Hydraulics* lists the following equation to determine whether a low flow channel should be designed for a constructed channel:

$$b / (V\,y) \geq 1.4 \qquad \text{(Eq'n 6.24)}$$

Where: b = channel bottom width (ft)
V = average channel velocity (ft/sec)
y = flow depth (ft)

In general, the recommended AFR WCMP channel has a bottom width of 444 ft (except in constricted reaches adjacent to drop structures). Average 100-year velocities in the flood channel ranges from 4.0 to 13.4 ft/sec, with an average of 8.5 ft/sec. 100-year maximum flow depths range from 4.1 to 13.3 ft, with an average of 7.9 ft. Therefore, given the computed flow characteristics, the District design guidelines indicate that a low flow channel should be constructed in the bed of the flood channel. The drop structures designed by DEA included a 50-foot wide, 2-foot deep notch in the upstream face to accommodate a low flow channel.

As noted in the AFR WCMP Lateral Stability Report (JEF, 2001) and the Agua Fria River Conceptual Channelization Plan (JEF, 2002), the Agua Fria River is a braided stream. Therefore, it should be expected that a low flow channel constructed within the flood channel will be destroyed or significantly altered by floods, and is likely to require periodic maintenance to preserve the designed single-channel configuration. It may be prudent to evaluate more stable, natural low flow channel designs during final design of the corridor alignment. Design of the low flow channel should consider the presence of nuisance flows, recharge delivery paths, storm drain outfalls, local tributary inflows, the desired aesthetic values for the channel, channel maintenance needs, and vegetation control or enhancement goals.

Flood Channel/Terrace Transition. The proposed corridor geometry includes a flood channel that conveys flows up to and including the 10-year event, and a floodplain terrace that will be inundated and convey flows during larger floods.[1] The terrace alternates from the left to right side of the corridor over the length of the channelization project in a similar manner to that of a natural floodplain which may alternate sides of a meandering stream. During flows that exceed the flood channel capacity, flow will exit the flood channel and inundate the terrace. At the point where a terrace is pinched out by the flood channel, flows from the terrace will re-enter the main channel area.

The processes of flood flow leaving and entering the flood channel should be no different than the processes that occur in analogous situations on natural channels with sinuous main channels and alternating floodplains. Therefore, no additional scour or unusual hydraulics is expected in these situations. Furthermore, because the water surface elevation during inundation of the terrace exceeds the flood channel/terrace bank

[1] Refer to project deliverables prepared by DEA and EDAW for more detailed descriptions of the channel corridor cross section and alignment.

elevation, no free overfall or excess turbulence is expected that can be modeled using a one-dimension computer model like HEC-RAS. No guidance for modeling this type of hydraulic situation was found in the District's *Drainage Design Manual – Hydraulics*, nor is design for such transitions part of the standard engineering design procedures. The three-dimensional modeling or physical modeling of the terrace/flood channel transition required to evaluate the hydraulics of this transition is beyond the scope of this study.

A second type of flood channel/terrace transition will occur near the proposed drop structures when flow rates exceed the terrace elevation. The channel configuration proposed by DEA contracts the corridor width over the drop structures to shorten the required length of the drop structures, and thus reduce construction costs. This constriction of the corridor width will result in contraction scour in the main channel near the approach to the drop structure and abutment scour as overbank flow accelerates around the raised terrace into the flood channel. Scour calculations using the FHWA HEC-18 Manual equation predict scour depths shown in Table 4. Toe down for bank protection from the approach section upstream the drop structure constriction to the drop structure face (or channel bed paving under bridge sections) should be based on the maximum contraction and abutment scour depths shown in Table 3.

Table 4. Contraction & Abutment Scour Depths at Drop Structure Constrictions (ft)

Estimated Scour	Contraction	Abutment
Average	0.9	8.2
Maximum	1.4	10.9
Minimum	0.4	6.7

Finally, the upstream slopes of the raised terrace areas at the drop structures should be protected from erosion using CSA, rip rap or some other form of bank stabilization, in a manner similar to that used for abutment slope protection at bridges.

Channelized/Non-Channelized Transition. There is only one transition from non-channelized to channelized flow within the proposed channelization plan. At the upstream end of the proposed channelization project, downstream of the CAP flume crossing, a transition from the unchannelized "natural" floodplain of the Agua Fria River to the channel corridor should be constructed to direct runoff into the corridor. The challenges to containing flood flow and directing it to the channelized cross section at this location include the following:

- CAP Recharge Canal. The CAP releases flow from a recently constructed structure at the siphon crossing. Water released from the siphon outlet flows in an earthen canal to the recharge facility located upstream of the Agua Fria Road crossing. The transition structure at the upstream end of the channelization project must accommodate delivery of water from the siphon outlet to the recharge canal.
- Drop Structure. The proposed 17-foot high, 700-foot wide drop structure will force strongly supercritical flow to occur. HEC-RAS modeling results indicate that flow at the toe of the drop structure will have velocities exceeding 30 feet per

second and a Froude number of greater than seven. An energy dissipater or alternative design will be required for this structure.

- Beardsley Canal Flume Crossing. The flume crossing of the Beardsley Canal crossing the Agua Fria River immediately upstream of the transition point. The transition must accommodate continued operation of the flume. No as-built information was available from which to accurately model the Beardsley Canal Flume in the HEC-RAS model.

The proposed conceptual design of the transition from the non-channelized reach to the channelized corridor consists of a levee to direct flow toward the proposed drop structure. The levee should extend through the Beardsley Canal flume crossing, past the CAP recharge canal outlet structure, and tie into the west bank immediately downstream of the small tributary that enters the Agua Fria River from the west upstream of the CAP right-of-way. The levee should extend downstream of the top of the drop structure far enough to assure containment of the SPF discharge. The levee may be constructed of any one of several types of materials, ranging from cement-stabilized alluvium (CSA) to gabions to rip-rap protected earthen material. Certainly, use of CSA throughout the rest of the channelization project tends to favor use of CSA for the proposed levee. Erosion protection of the levee face should be toed-down below the expected scour depth, or a minimum of 10 feet, as discussed below. Conceptual design sketches of the proposed transition are shown in Figure 1.

Grade Control Structures. The *Agua Fria River Channelization Conceptual Plan* (JEF, 2002) envisioned placement of grade control structures at one mile intervals and at each bridge crossing. The proposed channelization plan prepared by DEA for this project replaces most of the grade control structures with CSA drop structures. Design details for the currently proposed drop and grade control structures are provided in the report and design drawings prepared by DEA. DEA design details for the drop structures are also provided in the appendix to this memorandum. The drop structures proposed by DEA consist of 6:1 CSA slope paving. The proposed design intends to minimize hazards to pedestrian, ATV, and equestrian traffic, facilitate use of the river bed as an access road for construction and mining activities, and minimize some of the hydraulic hazards associated with vertical drop structures. The scope of services for this project indicates that structural analyses of grade control structures will be provided by the District. Evaluation of the durability of CSA drop structures and channel bottom paving during design flow conditions should be included with the structural analysis. The results of the scour analyses performed by JEF for the proposed drop structures are provided later in this memorandum.

Tributary Outfalls. Nineteen tributary outfalls that contribute to the Agua Fria River in the project reach were identified for consideration of conceptual design alternatives. Figure 2 shows the location, identifying code (ID), tributary name and 100-year peak flow rate for each tributary. Outfall design conceptual plans were prepared for each tributary and are included in Figures 3 through 20. Note that the scope of services called for a conceptual design level of effort and a generic design for the tributary confluences, and that the proposed designs are non-generic and site-specific. This low level of design

detail is appropriate, as opposed to final or preliminary design, given that local property owners will almost certainly modify the proposed tributary confluence design to suit their individual development plans.

The proposed conceptual design for the majority of the tributary outfalls is for the tributary runoff to spill down the bank of the corridor channel, either directly into the flood channel or onto the floodplain terrace. Some tributary outfalls will require constructed channels to route the tributary wash across the natural floodplain abandoned by the channelization to the constructed channel bank.

The following special considerations for specific tributaries are noted:

- ID 5 – Unnamed Wash (CP S706 in the North Peoria ADMP HEC-1 Model). This tributary currently outfalls into the Bard Ranch Property, which is currently a tangerine orchard, and has no defined channel leading to the Agua Fria River. It is our understanding that the Bard Ranch Property will be converted to residential or commercial subdivisions in the future and drainage plans for the wash may significantly differ from the concept shown in Figure 7. Therefore, the proposed design should be expected to be revised by the local property owners.

- ID 8 – Unnamed Wash (CP S707 in the North Peoria ADMP HEC-1 Model; Figure 10). This wash currently flows into an existing sand and gravel excavation and does not reach the Agua Fria River. Drop structures and/or grade control structures should have been designed as part of the sand and gravel mining floodplain use permit to mitigate headcut erosion hazards on the tributary.

- ID 9 – Caterpillar Tank Wash (CTW) has been re-aligned by a Central Arizona Project (CAP) ditch that supplies flow to the CAP recharge facility located north of Jomax Road. The CTW is routed around CAP recharge basins before it flows through culverts under the recently constructed Agua Fria Blvd/Happy Valley Road. The conceptual plan for this tributary is to construct a channel from the road crossing to the AFR channel north of the Twin Buttes Wash outfall as shown in Figure 13. The plan for the CTW also includes increased bank stabilization toe-down in the vicinity of the historic wash confluence in the event of CAP ditch embankment failure, as shown in Figure 11.

- ID 14 – Lizard Acres Wash (Figure 16). This wash currently flows into an existing Vulcan sand and gravel excavation and does not reach the Agua Fria River. Drop structures and/or grade control structures should have been designed as part of the sand and gravel mining floodplain use permit to mitigate headcut erosion hazards on the tributary.

- ID 15 - El Mirage Wash. El Mirage Wash is not contained within a well-defined channel upstream of existing development on the west bank. It is expected that floodwater would spread out over a few hundred feet wide area above the west bank of the channelization corridor. This would cause wide shallow flow and low

velocities. Therefore the main channel bank protection will likely be sufficient to prevent erosion damage of the bank from the tributary. Additional bank stabilization toe-down is recommended for scour at the bottom of the confluence as shown in Figure 17. In response to District review comments, dikes were added along the 4:1 flow expansion lines.

- ID 17 – New River. Design of the New River confluence is outside the scope of work for this project. Currently, the proposed alignment indicates that the New River will meet the Agua Fria River channelization at grade.

Tributary channels should be designed to start at a point that will fully contain the discharge at the existing mouth of the tributary. This can be accomplished by constructing channels and/or dikes to contain the flow across the floodplain. Channel design should conform to the *District's Drainage Design Manual – Hydraulics* and the following conceptual performance specifications and recommendations are provided:

- *Freeboard* – The minimum freeboard requirements for channels and bank protection should be met including additional depth required by water surface superelevation around bends. Channels constructed with levees or dikes shall conform to FEMA freeboard requirements.

- *Channel Curvature* – Channels should be designed with a minimum radius of curvature as outlined in the *District's Drainage Design Manual – Hydraulics.*

- *Scour and Toe-Down* – Short and long term scour should be estimated to determine required toe-down depths for bank protection. Grade control structures may be practical in some channels to reduce toe-down depths required for long term scour. It may be practical for smaller or narrow channels to be designed with an armored invert to eliminate the need for toe-down protection.

- *Erosion Protection* – Lateral erosion protection shall consist of traditional armoring such as rip-rap, gabions or CSA. Traditional bank protection shall be designed according to the requirements in the *District's Drainage Design Manual.* Bioengineered bank protection is an alternative depending on hydraulic conditions and a number of other considerations as discussed later in this memorandum.

- *Sedimentation* – Sediment transport should be considered to ensure that excess erosion or deposition of sediment doesn't occur, thus reducing channel capacity. Debris basins at the tributary mouth may be appropriate for some applications to store the sediment inflow. Alternatively, channels should maintain design velocities capable of transporting the sediment inflow.

The AFR channel bank stabilization will be notched at tributary outfalls to concentrate inflow at the confluences. Figure 21 provides conceptual details for a proposed outfall spillway design. In addition, the soil mantle over the CSA bank slope should be

channelized to create a contained spillway. Stabilization at outflow spillways should consist of traditional engineered revetments. Selection of the material should be based on the AFR channel bank protection and connectivity of the spillway material. Gabion baskets mantled in a similar manner as the CSA bank protection may be a viable spillway configuration.

Stilling basins are recommended at the bottom of the outfall spillways to prevent local scour. The outfall spillways should be designed as drop structures to estimate scour at the bottom of the drop. The spillways should be designed to accommodate AFR channel scour and the additional vertical drop as a result. Erosive forces from AFR channel flooding may damage outfall stilling basins. Therefore, increasing the AFR channel bank protection toe-down should be considered as an alternative to construction of stilling basins.

Outfall spillways down the bioengineered terrace banks should be designed with traditional engineered materials. A break in the bioengineered bank protection mantle is recommended at these locations to provide more erosion resistant material down the spillway. Stilling basins will be required at the bottom of spillways on the terrace to prevent local scour.

Bank Protection

Flood Channel. A variety of bank protection alternatives were proposed in the *Agua Fria River Channelization Conceptual Plan* (JEF, 2002). For this project, DEA proposed using CSA bank protection for both banks of the flood channel and using bioengineering techniques for the bank of the floodplain terrace, as shown in Figure 22. CSA bank protection was one of the alternatives previously recommended for consideration in the *Agua Fria River Channelization Conceptual Plan* (JEF, 2002). Design details prepared by DEA for the CSA bank protection are provided in the appendix to this memorandum.

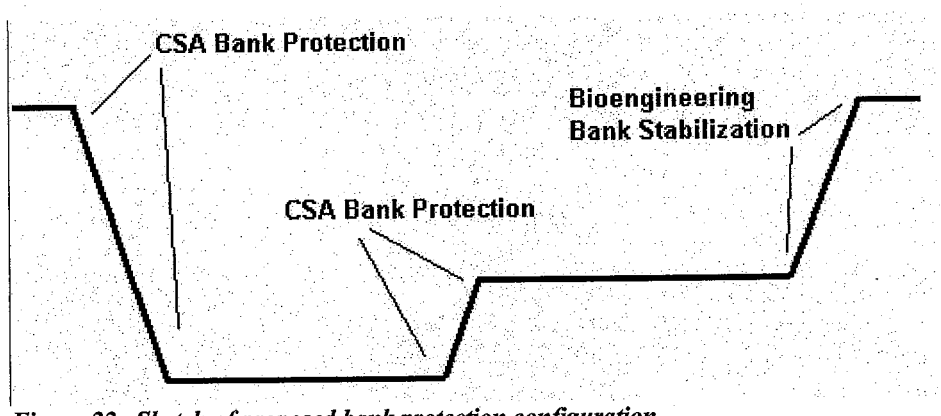

Figure 22. Sketch of proposed bank protection configuration.

Floodplain Terrace. A variety of bank protection alternatives were proposed in the *Agua Fria River Channelization Conceptual Plan* (JEF, 2002). For this project, DEA selected a bioengineered bank stabilization plan for the bank of the floodplain terrace (Figure 22).

Performance specifications and a design detail for a bioengineered bank stabilization scheme are provided below.

It is important to note the following with respect to bioengineering bank stabilization techniques:

- Standard of Practice. There is no established standard of practice for design of bioengineered bank stabilization.
- District Design Standards. The District's *Drainage Design Manual for Maricopa County – Hydraulics* does not even use the word "bioengineering," although it does note that, for subcritical flow, natural channel materials are preferred over channel lining with rip rap or concrete, and that if earthen channels are used, an armored low flow channel is recommended.
- Past District Channelization Projects. I am not aware of any bioengineered bank stabilization measures designed or constructed by the District on major river systems in Maricopa County. Thorough review of the proposed design by District engineers is recommended.
- Design Velocities. HEC-RAS modeling results indicate that velocities in the floodplain terrace will be non-erosive in the 100-year event, as shown in Table 5. Where unprotected by vegetation, velocities in the floodplain terrace will be marginally erosive at the peak of the standard project flood (SPF).
- Woody vegetation eroded from bioengineered bank slopes can accumulate on bridge piers, reducing capacity and increasing flood stages at hydraulic structures. Increased scour due to debris accumulation on bridge piers is not likely to occur since the proposed design includes paving the channel bottom through the existing bridge sections.
- Increased roughness associated with some bioengineering plans can result in reduced conveyance capacity and increased flood stages. It is assumed that minimal woody vegetation will be used in the revegetation of the mantled flood channel banks so that the impact on the assumed Manning's n values will be negligible.

Table 5. HEC-RAS Floodplain Terrace Velocities

		Left Overbank	Channel	Right Overbank
Q100	Average	2.8	8.5	2.7
	Maximum	3.5	13.4	3.6
SPF	Average	3.9	9.8	3.9
	Maximum	5.5	14.9	4.8

Bioengineered Bank Stabilization Alternative. The bank protection design proposed by DEA for this project includes a mantle of soil material constructed at a 3:1 or 4:1 slope over the CSA bank protection. The CSA bank protection will be constructed at a 1.5:1 or 2:1 slope. The soil mantle over the CSA will then be vegetated to improve the natural character of the corridor and to provide habitat and recreation opportunities. DEA's proposed mantled and planted CSA design achieves the primary advantage of a pure

bioengineering alternative. Disadvantages of bioengineered bank protection include the higher probability of failure during extreme flooding, susceptibility to failure by undercutting, reduced effectiveness due to drought, poor maintenance, or irrigation problems, and damage by fire or vandalism.

100-year channel velocities estimated from the HEC-RAS model indicate that average channel velocities are approximately 8.5 feet per second, with maximum velocities exceeding 13 feet per second in reaches with soil-mantled CSA. These velocities are likely to erode bank vegetation from the mantled bank slopes adjacent to the flood channel or cause significant local erosion, particularly if the vegetative cover is damaged by drought or human activities. Therefore, a strict bioengineering alternative could not be recommended for the flood channel banks unless an erosion buffer outside the corridor limits were delineated using the techniques outlined in the *Draft Erosion Hazard Zone Delineation and Development Guidelines* (JEF, 2003). Requirement for an erosion hazard zone boundary outside the corridor right-of-way would defeat one of the main objectives of the channelization project. Therefore, bioengineered bank stabilization is not recommended for the banks of the flood channel.

As shown in Table 5, estimated maximum 100-year velocities on the floodplain terrace are less than four feet per second, and average 100-year velocities on the floodplain terrace are less than three feet per second. According to Tables 6.3 and 6.4 of the District's *Drainage Design Manual – Hydraulics*, the maximum permissible velocity is 2.5 ft/sec for unvegetated sandy loams and 5.0 ft/sec for fine gravel, with maximum permissible velocity for vegetated banks ranging from 3.5 to 6.0 ft/sec. Given the short duration of flow on the floodplain terrace predicted from the design hydrograph for the Agua Fria River, the risk of erosion of the bioengineered floodplain terrace bank is minimal. However, because a small risk of erosion of the bioengineered floodplain terrace bank exists, an erosion hazard buffer will be defined at the outer limit of the corridor adjacent to the floodplain terrace bank, as described later in this memorandum.

Bioengineered bank stabilization is recommended for the bank of the floodplain terrace (Figure 22), pending approval of the proposed design by District staff. Vegetation of the floodplain terrace banks will not only help mitigate visual impacts of the constructed channel, but it will also provide habitat and recreation opportunities, and achieve a more natural character for the channelization corridor. Bank vegetation provides soil stability by minimizing the exposure of bare, unprotected soils to flood waters, by the binding effect of roots on the soil matrix, and by lowering flow velocities through increased roughness. Vegetated banks also tend to be less saturated and have better internal drainage than non-vegetated banks. Although plant-specific detailed design specifications for vegetation or revegetation are beyond the scope of this analysis, the following recommendations for bioengineered revegetation are provided:

- Plant Species. Use of native vegetation is encouraged to assure high survival rates and to minimize environmental impacts. Plants should be selected using the following criteria:

  - Flood tolerance vs. planting zone. Only flood tolerant plants should be planted in areas likely to be or remain flooded.
  - Drought tolerance. Drought tolerant plants are more likely to survive over the long-term.
  - Deep rooting. Deep rooting plants withstand erosion better, and are more likely to find a natural, sustained water supply.
  - Habitat value. Use of plant species with high habitat value is encouraged.
  - Ground cover. True ground cover species are generally not found in natural, non-irrigated settings in central Arizona. Plants with hanging branches may provide the same effect as low growing ground cover for the purposes of erosion protection and resistance to flood velocities.
  - Native species. Use of plants native to central Arizona is encouraged.
  - Vertical complexity. Design of a plant community with under story and over story species is encouraged.
- Toe of Slope. Deep rooting, long-lived, woody species should be planted at the toe of bank slopes and along the bank slope up to the 10-year water surface elevation to minimize the potential for undercutting, to provide the greatest resistance to higher velocities, and to mimic natural riparian plant density and distribution. Planting of riparian vegetation at the toe of the bank is encouraged for the following reasons:
  - Toe protection. The root mass, trunk, and leaf canopy provide protection from erosion at the critical toe area of the bank.
  - Irrigation. Irrigation is easier to accomplish at the toe of the bank than on the bank slope.
  - Water table. Roots from species placed at the bank toe are more likely to reach the water table than those placed on the bank slope.
  - Undercutting. Plants at the bank toe are less likely to be undercut than plants on the bank slope.
  - Aesthetics. Use of larger plants at the floodplain elevation, with smaller upland species on the bank slope mimics the natural environment.
  - Water quality. Design of denser swath of vegetation at the bank slope provides barrier, conduit, filter, and riparian sink functions for the stream corridor.
- Bank Slope. Use of ground cover species is encouraged from the toe of slope to the 100-year water surface elevation or top of bank.
- Top of Slope. Use of drought-tolerant desert species is recommended above the 100-year water surface elevation. Planting should mimic natural upland plant density and distribution.
- Irrigation. Irrigation may be required to assure plant survival, especially immediately after planting and for planting on upland slopes above the floodplain.
- Monitoring/Maintenance. A regular monitoring and maintenance program should be established to assure plant survival and assure that project goals are met. Monitoring should be conducted prior to the growing and planting seasons.
- Undercutting. Where the potential for long-term degradation to undercut bank vegetation is high, the recommended grade control structures will minimize the potential for undercutting of vegetated bank slopes.

- Landscape Character. Consideration of viewsheds and natural landscape character is recommended in design of revegetation.

More detailed information on use of vegetation in channel restoration and design is provided in the following references:

- Briggs, M., 1996, Riparian Ecosystem Recovery in Arid Lands – Strategies and References. University of Arizona Press, Tucson, Arizona.

- Federal Interagency Stream Restoration Working Group, 1998, *Stream Corridor Restoration – Principles, Processes, and Practices.*

The 4:1 vegetated mantled slopes may be subject to erosion at the transitions to and from the non-mantled 2:1 CSA bank protection in the channel reaches adjacent to the drop structures, and at tributary confluences that overfall the bank slopes. A groin or jetty-type structure may be required to protect the mantle from erosion, particularly given the likelihood of turbulent flow and cross waves in the expansions and contractions near the drop structures. Design of the jetty/groin feature is deferred until final design of the corridor.

Sedimentation Engineering

Bank Protection Toe-Down. Bank protection should be toed-down below the expected 100-year total scour depth in the flood channel. The recommended design scour depth for the flood channel bank protection is the sum of the general scour and long-term scour in the reaches between drop structures, and the sum of the general scour, long-term scour, contraction scour and abutment scour in reaches hydraulically impacted by the contraction and expansion near the drop structures. Locally bank protection may require additional toe down where tributary outfalls enter the flood channel.

General scour was estimated using the City of Tucson Drainage Design Manual (SLA, 1989) equations. Contraction and abutment scour were estimated using live-bed scour equations from the FWHA HEC-18 bridge scour manual. General, contraction, and abutment scour occur during the passage of a flood and represent the depth of the bed material moved at the peak sediment transport flow rate. Long-term scour was estimated from the equilibrium slope analysis described below. Long-term scour is the net change in bed elevation caused by a net sediment deficit or falling base level. Hydraulic data for the scour equations were obtained from the HEC-RAS model. Geometric data required for scour analyses were obtained from the DEA plan and profile drawings and engineering details. Predicted scour depths are shown in Tables 6 to 9.

Table 6. General and Long-Term Scour Estimates for the 2-Year Event

Reach	River Stationing	Proposed Slope (ft/ft)	Avg. Equilibrium Slope (ft/ft)	General Scour Depth (ft)	Long Term Scour Depth (ft)
1	1570+00 -1520+12	0.0024	0.0029	-1.7	2.4
2	1519+88 -1450+18	0.0024	0.0028	-1.6	3.1
3	1449+82 -1409+15	0.0024	0.0028	-1.6	1.8
4	1408+85 -1359+27	0.0024	0.0028	-1.6	2.2
5	1358+73 -1309+24	0.0024	0.0027	-1.6	1.5
6	1308+76 -1255+12	0.0023	0.0026	-1.6	1.6
7	1254+88 -1205+12	0.0023	0.0026	-1.6	1.6
8	1204+88 -1157+15	0.0021	0.0026	-1.6	2.2
9	1156+85 -1150+18	0.0021	0.0026	-1.6	0.4
10	1149+82 -1100+43	0.0021	0.0026	-1.6	2.3
11	1100+07 -1012+62	0.0016	0.0024	-1.7	6.9
12	1012+38 - 947+18	0.0016	0.0017	-1.7	0.8
13	946+82 - 881+70	0.0016	0.0017	-1.9	0.8
14	881+50 – 850+00	0.0019	0.0018	-1.6	-0.1
15	850+00 – 835+12	0.0016	0.0017	-1.6	0.2
16	834+88 – 797+12	0.0016	0.0017	-1.7	0.4
17	796+88 – 760+12	0.0016	0.0017	-1.7	0.4
18	759+88 – 723+12	0.0016	0.0017	-1.7	0.5
19	722+88 – 670+12	0.0015	0.0017	-1.7	1.3
20	669+88 – 615+91.5	0.0015	0.0018	-1.6	1.7
21	615+58.5 - 585+18	0.0015	0.0017	-1.7	0.6
22	584+82 – 558+18	0.0015	0.0017	-1.7	0.6
23	557+82 – 498+84.8	0.0015	0.0014	-2.0	-0.6
24	498+60.68 - 445+00	0.0009	0.0007	-1.8	-1.3

Notes:
1. A positive value for the long-term scour estimate indicates aggradation.
2. Scour computations are based on the City of Tucson Drainage Design Manual, Chapter 6 methods.

The data shown in Table 6 indicate that minor general scour occurs during the 2-year event, but that most reaches of the proposed channel will aggrade. Aggradation during small floods, followed by degradation during large floods is a common process in many natural streams in the Southwest. It is important to note that long-term scour was predicted based on equilibrium slope equations that are subject to considerable error, rather than by direct measurement or by modeling of sediment supply and transport integrated over a flow duration curve. Therefore, the results should be interpreted as order-of-magnitude estimates of the expected direction of change rather than precise indications of actual change. Given the potential for aggradation during the more frequent floods, a program of regular inspection and sediment maintenance is recommended to assure that adequate channel capacity is provided.

Table 7. General and Long-Term Scour Estimates for the 10-Year Event

Reach	River Stationing	Proposed Slope (ft/ft)	Avg. Equilibrium Slope (ft/ft)	General Scour Depth (ft)	Long Term Scour Depth (ft)
1	1570+00 -1520+12	0.0024	0.0022	-2.8	-0.9
2	1519+88 -1450+18	0.0024	0.0022	-2.4	-1.3
3	1449+82 -1409+15	0.0024	0.0022	-2.4	-0.7
4	1408+85 -1359+27	0.0024	0.0022	-2.4	-0.8
5	1358+73 -1309+24	0.0024	0.0021	-2.5	-1.6
6	1308+76 -1255+12	0.0023	0.0021	-2.6	-1.5
7	1254+88 -1205+12	0.0023	0.0020	-2.6	-1.2
8	1204+88 -1157+15	0.0021	0.0020	-2.6	-0.4
9	1156+85 -1150+18	0.0021	0.0021	-2.4	0.0
10	1149+82 -1100+43	0.0021	0.0020	-2.6	-0.4
11	1100+07 -1012+62	0.0016	0.0019	-2.7	2.3
12	1012+38 - 947+18	0.0016	0.0014	-2.7	-1.4
13	946+82 - 881+70	0.0016	0.0014	-3.5	-1.6
14	881+50 - 850+00	0.0019	0.0016	-2.5	-1.0
15	850+00 - 835+12	0.0016	0.0013	-2.5	-0.4
16	834+88 - 797+12	0.0016	0.0014	-2.7	-0.8
17	796+88 - 760+12	0.0016	0.0014	-2.7	-0.8
18	759+88 - 723+12	0.0016	0.0014	-2.7	-0.8
19	722+88 - 670+12	0.0015	0.0014	-2.7	-0.4
20	669+88 - 615+91.5	0.0015	0.0014	-2.7	-0.4
21	615+58.5 - 585+18	0.0015	0.0014	-2.7	-0.3
22	584+82 - 558+18	0.0015	0.0014	-2.7	-0.2
23	557+82 - 498+84.8	0.0015	0.0012	-3.4	-1.6
24	498+60.68 - 445+00	0.0009	0.0006	-2.4	-1.5

Notes:
1. A positive value for the long-term scour estimate indicates aggradation.
2. Scour computations are based on the City of Tucson Drainage Design Manual, Chapter 6 methods.

The results shown in Tables 7 and 8 indicate that the 10- and 100-year floods will be generally degradational, in contrast to the 2-year results shown in Table 6. A notable exception is Reach 11, for which long-term aggradation was predicted, even during a 100-year event. The results for Reach 11, which extends from Bell Road to the McMicken Outfall Channel, clearly indicates that the slope is too flat. The slope of Reach 11 can be steepened by removing the fall at the drop structure at the upstream end and/or dropping the lip elevation of the drop structure at the downstream end. The amount of aggradation predicted would be significant, and would likely result in deposition that would diminish the conveyance capacity of the corridor below design levels without extensive maintenance activity.

Table 8. General and Long-Term Scour Estimates for the 100-Year Event					
Reach	**River Stationing**	**Proposed Slope (ft/ft)**	**Avg. Equilibrium Slope (ft/ft)**	**General Scour Depth (ft)**	**Long Term Scour Depth (ft)**
1	1570+00 -1520+12	0.0024	0.0021	-5.2	-1.6
2	1519+88 -1450+18	0.0024	0.0021	-4.3	-2.0
3	1449+82 -1409+15	0.0024	0.0021	-4.3	-1.2
4	1408+85 -1359+27	0.0024	0.0021	-4.2	-1.3
5	1358+73 -1309+24	0.0024	0.0019	-4.5	-2.3
6	1308+76 -1255+12	0.0023	0.0020	-4.7	-2.0
7	1254+88 -1205+12	0.0023	0.0019	-4.6	-1.8
8	1204+88 -1157+15	0.0021	0.0019	-4.7	-1.0
9	1156+85 -1150+18	0.0021	0.0020	-4.5	-0.1
10	1149+82 -1100+43	0.0021	0.0019	-4.7	-0.9
11	1100+07 -1012+62	0.0016	0.0018	-4.9	1.5
12	1012+38 - 947+18	0.0016	0.0013	-4.9	-1.8
13	946+82 - 881+70	0.0016	0.0013	-6.9	-2.0
14	881+50 - 850+00	0.0019	0.0015	-4.7	-1.0
15	850+00 - 835+12	0.0016	0.0013	-4.3	-0.5
16	834+88 - 797+12	0.0016	0.0013	-5.1	-1.0
17	796+88 - 760+12	0.0016	0.0013	-5.0	-1.0
18	759+88 - 723+12	0.0016	0.0013	-5.2	-1.1
19	722+88 - 670+12	0.0015	0.0013	-4.9	-0.7
20	669+88 - 615+91.5	0.0015	0.0014	-4.9	-0.6
21	615+58.5 - 585+18	0.0015	0.0014	-5.0	-0.3
22	584+82 - 558+18	0.0015	0.0014	-5.0	-0.2
23	557+82 - 498+84.8	0.0015	0.0012	-6.5	-1.7
24	498+60.68 - 445+00	0.0009	0.0006	-4.1	-1.5

Notes:
1. A positive value for the long-term scour estimate indicates aggradation.
2. Scour computations are based on the City of Tucson Drainage Design Manual, Chapter 6 methods.

Table 9. Contraction & Abutment Scour Depths Near Drop Structure Constrictions (ft)		
Estimated Scour	**Contraction**	**Abutment**
Average	0.9	8.2
Maximum	1.4	10.9
Minimum	0.4	6.7

The recommended toe-down for the CSA bank protection, except at the drop structures (discussed below) is the sum of the 100-year general scour depth, the long-term scour depth based on 100-year equilibrium slope, and the maximum computed 100-year local scour (contraction/abutment scour, where applicable). District review staff report that the District prefers to use a minimum toe-down of 10 feet on major watercourses with CSA bank protection (Personal communication from M. Lopez, PE and E. Raleigh, PE on July 8, 2004). Except in Reach 6 (Stn 1308+76 to 1210+00), the total computed scour was less than 10 feet. It is our understanding that DEA is modifying the proposed channel

plan and profile to include additional drop structures in Reach 6, which will reduce the expected long-term scour, thus making the 10-foot minimum toe-down acceptable.

Therefore, the District's 10-foot minimum toe down is recommended for the entire project reach, except in the following locations: (1) where abutment scour is predicted at the entrance to the drop structure reaches, (2) in the scour area downstream of the drop structures, and (3) at tributary outfalls. In the short reach at the inlet to the drop structure channelization where abutment scour is expected, a toe-down of 20 feet is recommended. Alternatively, the entire channel could be lined within the contracted drop structure sections. Toe down for drop structures and tributary outfalls are discussed elsewhere in this memorandum. Note that the toe-down recommended in this memorandum relates to toe-down for scour protection only. Any additional toe down required to assure structural, geotechnical, or hydrodynamic stability, or other construction-related factors is outside the scope of the JEF analyses.

Scour at Tributary Inlets. As discussed elsewhere in this memorandum, tributary inflows will be routed to the channelized corridor via constructed channels. Where these channel overfall the CSA bank protection into the flood channel of the Agua Fria corridor, scour is likely and will require additional toe down or scour protection for both the channelization CSA bank protection as well as the vegetated mantle slope.

Grade Control & Drop Structures. The depth of scour at a drop structure was calculated using three local scour equations, two of which were for submerged flow conditions (the most likely condition for the Agua Fria River channelization at most drop structures), and one which was developed for unsubmerged conditions (likely at the largest drop structure). The equations are formulized as follows:

Submerged Drop Structure with a Vertical Wall (SLA, 1989)[1]:

$$Z_{lss} = 0.581q^{0.667}(h/Y)^{0.411}[1-(h/Y)]^{-0.118}$$

Submerged Drop Structure with a 1:1 Sloped Wall (SLA, 1986)[2]:

$$Z_{lss} = 0.54q^{0.667}(h/Y)^{0.158}[1-(h/Y)]^{-0.134}$$

Where:
Z_{lss} = Depth of local scour (ft.)
q = Discharge per unit width (cfs/ft.)
h = Drop height (ft.)
Y = Downstream depth of flow (ft.)

Unsubmerged Drop Structure with a Vertical Wall (SLA, 1989):

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[1] Simons, Li & Associates, 1989, Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. Prepared for the City of Tucson Department of Transportation Engineering Division.

[2] Simons, Li & Associates, 1986, "Hydraulic Model Study for Local Scour Downstream of Rigid Grade-Control Structures," Prepared for Pima County Department of Transportation and Flood Control District, Tucson, AZ.

$$Z_{lsf} = 1.32q^{0.54}H_t^{0.225} - TW$$

Where:
Zlsf = Depth of local scour (ft.)
q = Discharge per unit width (cfs/ft.)
H_t = Drop height (ft.)
TW = Downstream depth of flow (ft.)

The drop heights for the drop structures were taken from DEA's conceptual plans, while the drop heights for the grade control structures were evaluated as the long term scour depths (Tables 6 to 8). As suggested by HEC 23 (FHWA, 2001) the largest value obtained from the scour equations considered was used to determine the recommended scour depth for design. ADOT (1983) also provides an equation for determining the local scour over a 1V:4H sloping sill. However, the ADOT equation applies to clear water conditions (and assumes infinite flow duration and no armoring potential) and therefore produces unrealistically high values for the AFR channelization project conditions.[1] The recommended toe-down for bank protection adjacent to the drop structure is the sill scour depth, plus the expected long-term scour for that reach times a safety factor of 1.3.

The drop structure scour equations used were developed for significantly steeper drop structures (1:1, 0:1) than for the 6:1 sloped drop structures designed by DEA. Because the proposed drop structures are flatter than those considered in the equations used, the scour estimates are conservative with respect to depth, but may be non-conservative with respect to the length of the scour hole.[2] Additional modeling may be necessary during final design to determine the length of the scour hole below the drop structures. Given the lack of published data on 6:1 drop structures, physical modeling may be required.

The longitudinal extents of the scour holes downstream of the drop structures were determined to be 12 times the scour depth (SLA, 1989). The CSA bank protection at and downstream of the drop structures should be toed-down the recommended depth for at least the predicted length of the scour hole. Predicted scour depths and longitudinal extents for the 100-yr. storm event are shown in Table 10. Given the uncertainty in the available scour equations, lining the entire transition downstream of the 6:1 drops is recommended.

Note that the toe-down recommended in this memorandum relates to toe-down for scour protection only. Any additional toe down required to assure structural, geotechnical, or hydrodynamic stability, or other construction-related factors is outside the scope of the JEF analyses.

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[1] The ADOT equation predicted scour depths in excess of 50 ft, which are described in the ADOT manual itself as unrealistic for live-bed conditions.

[2] Personal communication from Larry Roberts, PE/Tetratech on October 28, 2004. Flume experiments conducted by Simons Li & Associates for Pima County used vertical drops and 1:1 drops. When the drop slope being modeled was flattened to less than 1:1, the scour hole extended beyond the length of the flume.

Table 10: 100-Year Scour Depth & Recommended Toe Down Depth at Proposed Grade Control and Drop Structures

River Stationing	Local Scour Depth (ft)	Longitudinal Extent of Scour Hole (ft)	Recommended Toe Down Depth (ft)
1570+00	12.1	145	17.8
1519+88	7.4	89	12.2
1449+82	8.9	107	13.1
1408+85	8.4	101	12.6
1358+73	10.2	123	16.2
1308+76	11.0	132	16.9
1254+88	8.3	100	13.1
1204+88	8.0	96	11.6
1156+85	9.8	117	12.8
1149+82	9.7	117	13.8
1100+07	8.9	107	9.7
1012+38	8.2	99	13.0
946+82	9.2	110	14.5
881+50	7.5	90	11.1
850+00	5.1	61	7.3
834+88	8.0	96	11.7
796+88	8.2	98	11.9
759+88	8.1	97	12.0
722+88	8.5	101	11.9
669+88	8.0	97	11.3
615+58.5	8.9	107	12.0
584+82	9.3	111	12.4
557+82	9.3	112	14.4
498+60.68	7	85	11.1

Bridge Piers. The proposed channelization design includes channel bed paving through every bridge section within the project reach, except at Grand Avenue. Given the narrow channel section, age of the Grand Avenue and ATSF railroad bridges, and scour status, it is strongly recommended that channel bed paving be provided through the Grand Avenue and ATSF railroad bridge sections. The proposed slope paving at bridge sections extends from the upstream face of the drop or grade control structures through the bridge section a distance of at least 12 feet. The slope paving will eliminate pier scour, as well as the contraction and abutment scour in the bridge sections.[1] Therefore, no new bridge pier scour analyses were conducted for the sedimentation engineering analyses.

[1] Contraction and abutment scour occur upstream of the bridge sections near the entrance to the drop structure approach reach.

Armoring. When the channel sediment transport capacity exceeds the upstream sediment supply, the balance of the sediment load may be eroded from the channel bed, causing the channel to degrade. Because fine sediments can be transported at more frequent lower discharges and velocities than coarse sediments, which may require large floods to be moved, fine sediment tends to be preferentially removed from the channel bed. Selective removal of fine sediments causes channel bed material to become progressively coarser over time, as long as the upstream sediment supply is limited. If this process continues over a long period, it ultimately creates a surficial layer of coarse channel sediments, called an armor layer, that the stream is incapable of transporting.

Armoring is unlikely to prevent scour within the flood channel of the completed channelization corridor. Armoring analyses conducted for the *AFR WCMP Lateral Migration Study* (JEF, 2001) concluded that armoring was unlikely during the 100-year event in the natural channel. For the constructed channel, over-excavation of the channel corridor by aggregate miners is likely to selectively remove the coarsest fraction of sediment material and thus reduce the potential for armoring. In addition, flow velocities and depths in the channelized corridor will increase slightly above the natural values due to narrowing of the floodplain, increasing the sediment size required to form an armor layer. Therefore, armoring is unlikely to limit either short-term or long-term scour in the proposed channel corridor.

Equilibrium Slope Analyses. Equilibrium slope is defined as the slope which causes the channel's sediment transport capacity to equal the incoming sediment supply (ADWR, 1985). If the slope is too steep, channel velocities will be high and net erosion will occur. If the slope is too flat, channel velocities will be low and net deposition will occur. The equilibrium slope is the slope that the undisturbed, natural channel will tend towards over the long term. While there are philosophical and practical problems with applying equilibrium slope concepts to small ephemeral streams with variable channel geometry and high flash flood potential, equilibrium slope equations provide a useful order-of-magnitude assessment of the likelihood of vertical channel adjustments. Reach-averaged data required for application of equilibrium slope equations to the study area were derived from the HEC-RAS modeling and the proposed channelization profile prepared by DEA.

Most equilibrium slope equations are based on the mean annual flood, the "channel-forming," or "bankfull" discharge. On many alluvial streams, the mean annual flood and the channel-forming and bankfull discharges are nearly equivalent. However, on ungauged ephemeral streams where flow events are rare, the average annual discharge is difficult to determine, particularly given upstream storage of most low flow events in Lake Pleasant. To account for the discrepancies in what flow rate is appropriate for equilibrium slope analyses, and to assess the trend of expected slope adjustments during floods, the 2-, 10-, and 100-year peaks were used in the equilibrium slope equations to assess the expected slope adjustment over a range of discharges. The 2-year event approximates the mean annual flood calculated on a weighted probability basis. The 10-year event better approximates bankfull conditions on the streams in the study area. The

100-year event represents possible channel responses during extreme flooding. The following equilibrium slope equations were applied to the study reach:

- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) Equations
- BUREC Equation
- Bray Equation
- Henderson Equation
- Schoklitsch Equation
- Meyer-Peter Muller Equation
- Shield's Diagram Method
- Lane's Tractive Force Method

The AMAFCA, BUREC, Bray, and Henderson equations predict equilibrium slopes for channel with active sediment transport. The Schoklitsch, Meyer-Peter Muller, Shield, and Lane equations are stable slope equations intended for application in reaches with no sediment inflow, and thus represent minimum potential slope values. Because sediment will be supplied to the channelization reach from undisturbed reaches upstream of the proposed project, as demonstrated in the Agua Fria River WCMP analyses, the results of the AMAFCA, BUREC, Bray, and Henderson equations were used to predict an equilibrium slope.

As shown in Table 6, the predicted equilibrium slope for the 2-year event is slightly steeper than the proposed constructed channel. Therefore, some deposition of sediment should be expected during the most frequent events, particularly near the mouths of tributaries. The estimated equilibrium slopes based on the 10- and 100-year peak discharges (Tables 7 and 8) are slightly flatter than the proposed constructed channel, and thus will tend to scour during large floods. This dichotomy between frequent event deposition and flood scour is analogous to natural processes documented for other ephemeral systems in Arizona.[1] However, because the predicted tendency for deposition during the most frequent events, regular maintenance and inspection should occur to assure that adequate conveyance capacity is maintained in the corridor.

Long-Term Scour Depths. Long-term scour depths were estimated from the results of the equilibrium slope analyses. The proposed drop structures function as grade controls, limiting potential long-term scour to the reaches between drop structures. Therefore, the maximum predicted long-term scour depth is simply the difference between the predicted equilibrium slope and the constructed channel slope times the distance from the drop structure/grade control located downstream. Long-term scour depth estimates are shown Tables 6 to 8 are, therefore, maximum values for the reach, and represent the predicted change at the point within the reach most distal from the downstream grade control.

Sediment Continuity Analysis. The Zeller-Fullerton equation (ADWR, 1985) was used to evaluate sediment continuity between adjacent cross sections and reaches of the

[1] Pearthree, M.S., and Baker, V.R., 1987, Channel change along the Rillito Creek system of southeastern Arizona 1941 through 1983, Implications for Flood-Plain Management: Arizona Bureau of Geology and Mineral Technology, Geological Survey Branch, Special Paper 6, 58 p.

channelization corridor. Hydraulic data required to apply the Zeller-Fullerton equation were obtained from the HEC-RAS models.

The Zeller-Fullerton Equation is a total bed-material discharge equation developed for sand-bed channels, and is formulated as follows:

$$Q_s = 0.0064\, n^{1.77}\, V^{4.32}\, G^{0.45}\, Y_h^{-0.30}\, D_{50}^{-0.61}$$

Where:
Q_s = sediment discharge rate (cfs)
n = Manning's roughness coefficient, channel
V = mean channel velocity (ft/s)
G = gradation coefficient
Y_h = hydraulic depth, channel (ft)
D_{50} = median bed sediment size (mm)

The Zeller-Fullerton equation was applied using the HEC-RAS data for the 10-year, 100-year, and SPF discharges on a section-by-section basis. The change in sediment transport capacity between adjacent cross sections was estimated by subtracting the sediment inflow rate from the sediment outflow rate (i.e., continuity) to determine if a net sediment deficit or net sediment surplus was likely. A sediment deficit (i.e., more sediment leaving a reach than entering a reach) translates to potential scour and degradation. A sediment surplus (i.e., more sediment entering a reach than leaving a reach) translates to potential deposition and aggradation. The sediment continuity analysis was also applied using reach-average hydraulic data to compare sediment continuity between adjacent channelization reaches.[1]

As shown in Figure 23, the sediment continuity analysis predicts relative sediment balance between adjacent cross sections, except for the channel sections located near the proposed drop structures. Discontinuity in sediment transport capacity is expected given the change in channel width (narrow floodplain, wider flood channel), unit discharge (eliminate terrace flow, constrict corridor width), and slope breaks (6:1 drop structure, change in reach slope). The fluctuation in the sediment transport capacity is illustrated in Figure 24, which shows the computed sediment transport capacity for one reach of the corridor in conjunction with the channel bed elevation at, and between, the drop structures. Given the uniform channel section in the reaches between drop structures, sediment continuity is expected in those reaches.

Sediment continuity was also compared on a reach basis to evaluate the impact of slight adjustments in the proposed design channel slope. As shown in Figure 25 and Table 11, the sediment continuity results predict that most of the corridor will experience a sediment deficit during floods and thus will have tendency toward net scour and degradation, a prediction which is consistent with the equilibrium slope analysis. Sediment deficits in the 10-year event generally are not significant. Not surprisingly, the largest sediment deficits occur in the narrow channelized reach downstream of Grand

[1] A channelization reach is defined as the area between grade control or drop structures.

Avenue, providing support for the conclusion of the original AFR WCMP that this reach may require full lining to prevent erosion of the El Mirage Landfill. Net sediment surplus is predicted for the reaches between the Cactus Road alignment and Olive Road, the reach upstream of Camelback Road, and the USACE levee reach. Regular inspection for sediment deposition and loss of conveyance capacity should be conducted in these potentially aggrading reaches.

The magnitude of potential scour or deposition can be estimated by multiplying the relative sediment transport capacity difference shown in Table 11 by a flow duration to obtain a sediment volume and dividing by the channel area in the reach. For example, for a 100-year peak discharge over a six hour duration in Reach 15 would result in net scour of about 1.9 feet, one of the more extreme results within the project reach.

Table 11. Reach-Average Sediment Continuity Results Showing Relative Difference in Sediment Transport Capacity

Reach	Upstream Station	Downstream Station	Landmarks	Sediment Continuity		
				Q10 (cfs)	Q100 (cfs)	SPF (cfs)
1	1591+66	1571+04	Upstream of CAP			
2	1570+00	1520+12	CAP to Future SR303	7	4	-18
3	1519+88	1450+18	SR303 to Dixeleta Dr	1	1	-5
4	1449+82	1409+15	Dixeleta Dr to Lone Mtn Pkwy	0	-3	-4
5	1408+85	1358+73	Lone Mtn Pkwy to Jomax Rd	1	3	6
6	1358+73	1309+24	Jomax Rd to Agua Fria Blvd	-3	-7	7
7	1308+76	1210+00	Agua Fria Blvd to Rose Garden Ln	-1	-19	-24
8	1205+00	1157+15	Rose Garden Ln to Walker Pit	-1	4	20
9	1156+85	1150+18	Walker Pit	-11	-56	-108
10	1149+82	1100+43	Walker Pit to McMicken Outfall	14	73	134
11	1100+43	1012+62	McMicken Outfall to Bell Rd	-1	-1	-1
12	1012+38	947+18	Bell Rd to Vulcan Pit	-8	-49	-76
13	946+82	881+70	Vulcan Pit to Grand Ave	9	39	57
14	881+50	855+00	Grand Ave to Grade Control	-6	-57	-108
15	850+00	835+12	Grade Control to Cactus Rd	-26	-85	-100
16	834+88	797+12	Cactus Rd to Peoria Ave	18	68	94
17	796+88	760+12	Peoria Ave to El Mirage WWTP	2	15	44
18	759+88	723+12	El Mirage WWTP to Olive Rd.	4	29	35
19	722+88	670+12	Olive Rd to Northern Ave	-3	-35	-28
20	669+88	615+91.5	Northern Ave to Drop Structure	0	4	-25
21	615+58.5	585+18	Drop Structure to Bethany Home Rd	-3	-23	-47
22	584+82	558+18	Bethany Home Rd to Drop Structure	-2	-7	-9
23	557+82	500+00	Drop Structure to Camelback Rd	14	55	63
24	498+84.8	475+00	Camelback Rd to Indian Bend Rd	-10	-29	-121
25	470+00	412+80.8	USACE Levee Reach	18	108	257

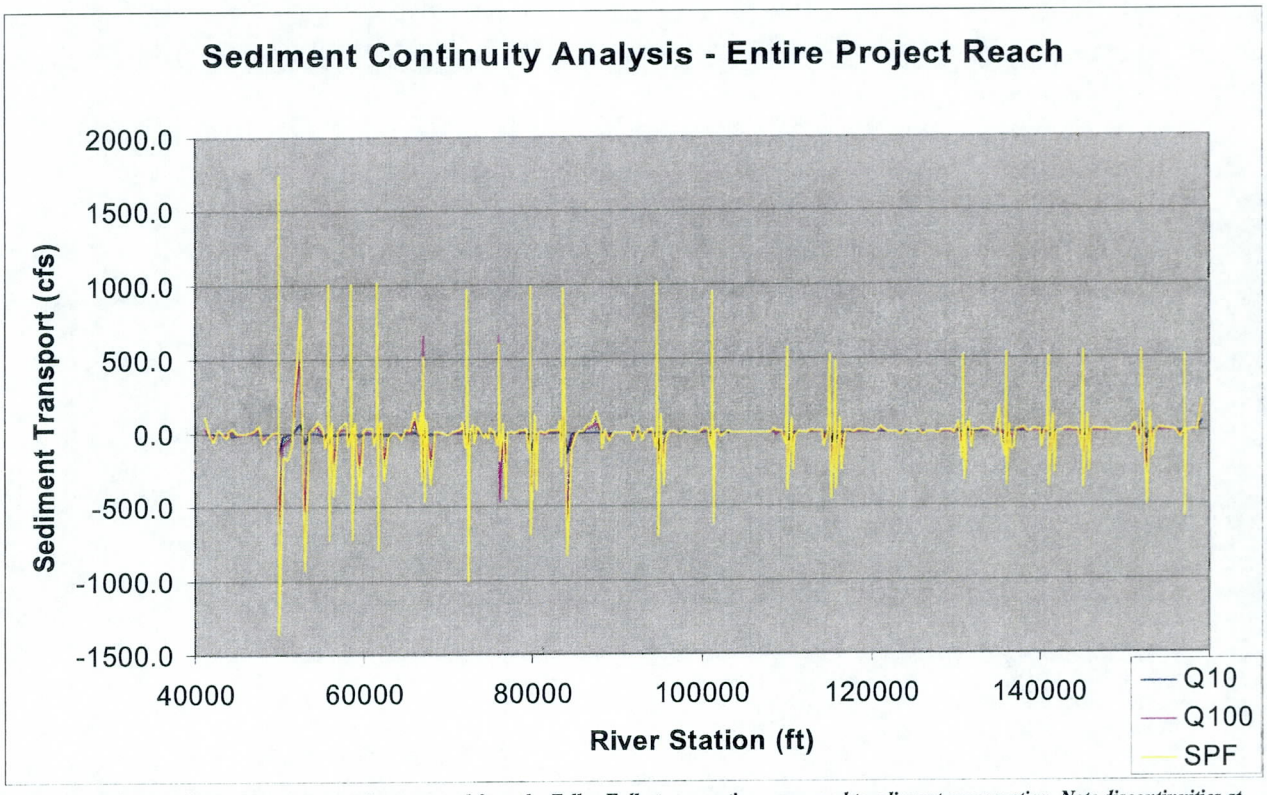


Figure 23. Relative sediment transport capacity computed from the Zeller-Fullerton equation, compared to adjacent cross section. Note discontinuities at drop structures and relative continuity in reaches between drop structures.

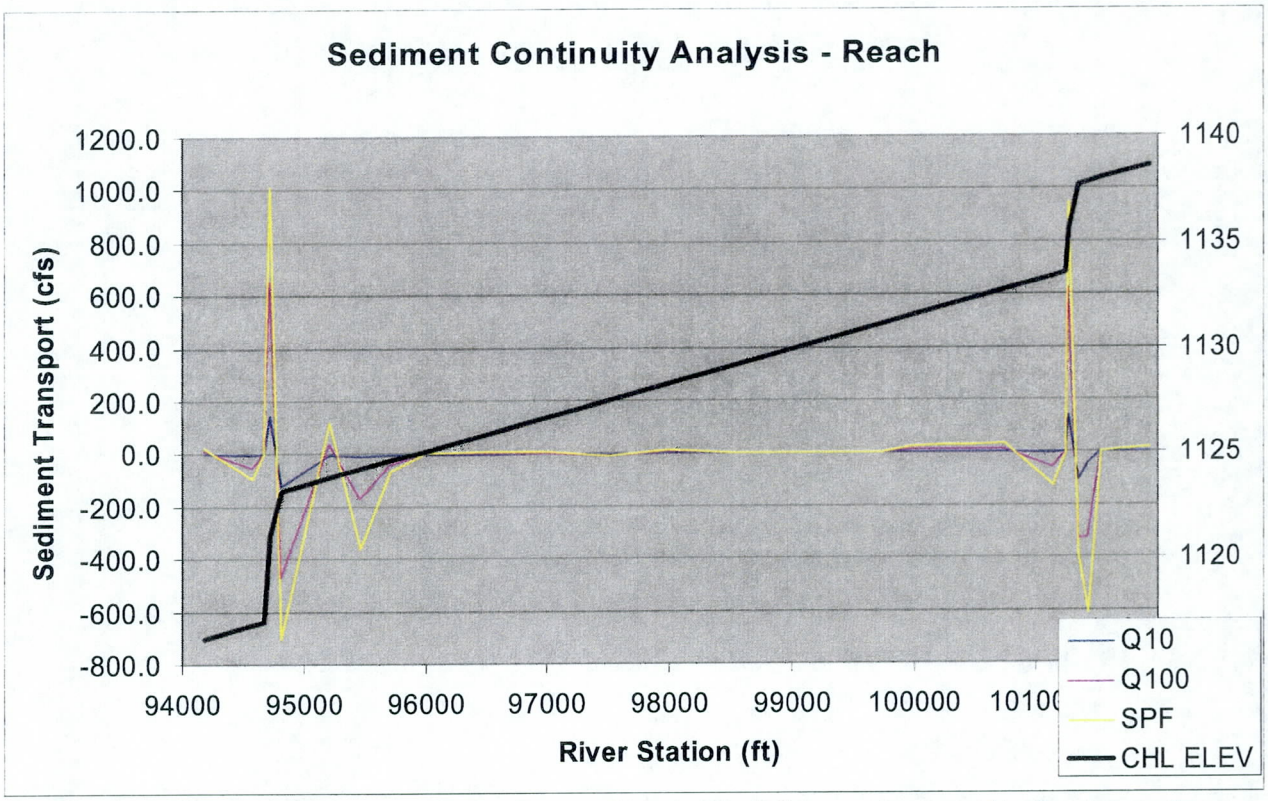


Figure 24. Relative sediment transport capacity for a single reach computed from the Zeller-Fullerton equation, compared to adjacent cross section.

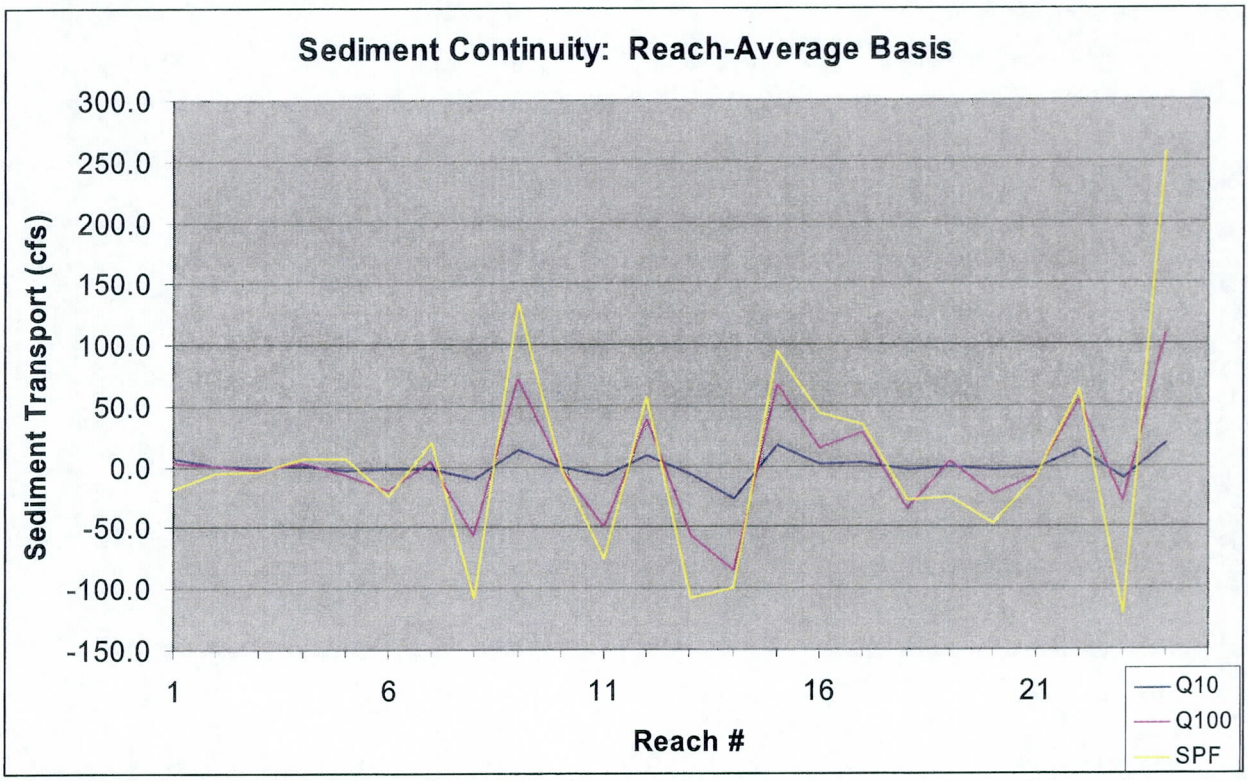


Figure 25. Reach-averaged sediment continuity results.

Sediment Deposition Zones. Zones of likely sediment deposition were identified from the results of the equilibrium slope and sediment continuity analyses. Overall, the channel is expected to experience net scour over the long-term, with short-term term deposition locally during periods with no large floods. Therefore, periodic inspection for unacceptable levels of sediment deposition should be performed, particularly in the reach between the Cactus Road alignment and Olive Road, the reach upstream of Camelback Road, and the USACE levee reach.

Sediment Impacts on Adjacent Reaches. Channelization of the Agua Fria River could alter the sediment supply to Gila River and/or reaches downstream of the channelization. A cursory analysis was completed to estimate potential sediment impacts on adjacent stream reaches, with the following results:

- Sediment transport modeling of the Gila River conducted by Stantec Consulting, Inc. (2004) for the El Rio Watercourse Master Plan concluded that the sediment inflow from the Agua Fria River to the Gila River was "negligible."
- Stantec's conclusion is supported by comparison of watershed size, which is directly proportional to sediment yield. The watershed of the Agua Fria River at the Gila River confluence is 1,944 mi2, compared to the Gila River's drainage area of 51,000 mi2, or about 4% of the total.
- A high percentage of the Agua Fria River watershed is either urbanized, controlled by dams that intercept nearly all the sediment yield, or channelized, effectively reducing the natural sediment supplied to downstream reaches.
- The Agua Fria WCMP Lateral Migration Report (JEF, 2001) concluded that the Agua Fria was characterized by historically decreased sediment supply induced by aggregate mining, urbanization, and storage in Lake Pleasant.
- The USACE levee reach provides a buffer between the proposed channelization and the Gila River. Since the geometry of the USACE levee reach will not change, and since a large percentage of the flow into the USACE levee reach is derived from the New River, upstream channelization will have minimal impact downstream of the USACE levees.
- Field observations made during the WCMP geomorphic analysis indicated that the USACE levee reach was experiencing net long-term degradation. Therefore, limited sediment is supplied to downstream reaches during existing conditions.
- Regardless of the projected impact, any stream response to changes induced by upstream channelization would be extremely slow due to the infrequency of surface flow in the Agua Fria River, particularly where the channel is currently intercepted by in-stream aggregate mines.
- Sediment continuity was compared using the Zeller-Fullerton equation for the 2-, 10-, and 100-year peak discharge, and hydraulic parameters derived from the FDS HEC-2 model and the channelization HEC-RAS models prepared for this study. The reach between the New River confluence and Glendale Avenue was chosen to simulate potential differences in supply to the USACE levee reach. The results indicate that the 2-year transport rate would be decreased by about 50%, and the

10- and 100-year transport rate would be increased. That is, less sediment is transported downstream during small floods (probably due to widening of the low flow channel), and more sediment is transported into the reach.

Based on this cursory analysis, it is concluded that the proposed channelization will not significantly impact downstream reaches.

Erosion Hazard Zone Delineation

Erosion hazard zones were delineated for the Agua Fria River as part of the AFR WCMP, as documented in the *Agua Fria River Lateral Stability Report* (JEF, 2001). The following three erosion hazard zones were defined:

- ***Severe Erosion Hazard Zone***. The Severe Erosion Hazard Zone encompasses the active channel, and the area next to the active channel that could reasonably be expected to erode during a large flood.
- ***Lateral Migration Erosion Hazard Zone***. The Lateral Migration Erosion Hazard Zone includes the portion of the floodplain that could reasonably be expected to erode during a series of floods. This is the minimum area required to maintain the processes of natural channel movement. The Lateral Migration Erosion Hazard Zone is also the minimum area required for preservation of the natural form and function of the stream.
- ***Long-Term Erosion Hazard Zone***. The Long-Term Erosion Hazard Zone includes the area within and adjacent to the floodplain that could be subject to erosion and lateral migration as indicated by geologic and historic evidence. The Lateral Migration Erosion Hazard Zone is also the area necessary to implement nonstructural flood management.

Full Implementation. Implementation of the proposed channelization concept would significantly alter the erosion hazards along the river corridor. Upon full implementation of the proposed channelization, the previously delineated erosion hazard zones would be modified as follows and as shown in Figure 26:

- ***Severe Erosion Hazard Zone***. The severe erosion hazard zone is moved to the top of bank of the flood channel.
- ***Lateral Migration Erosion Hazard Zone***. The lateral migration erosion hazard zone is moved to the top of the flood channel bank opposite the floodplain terrace, and to the outside of the 1,200 foot corridor limit adjacent to the top of the floodplain terrace bank.
- ***Long-Term Erosion Hazard Zone***. Engineered bank protection removes the long-term erosion hazard, making the long-term erosion hazard zone coincident with the lateral migration erosion hazard zone.

A revised erosion hazard zone delineation has been completed and was delivered digitally to the District.

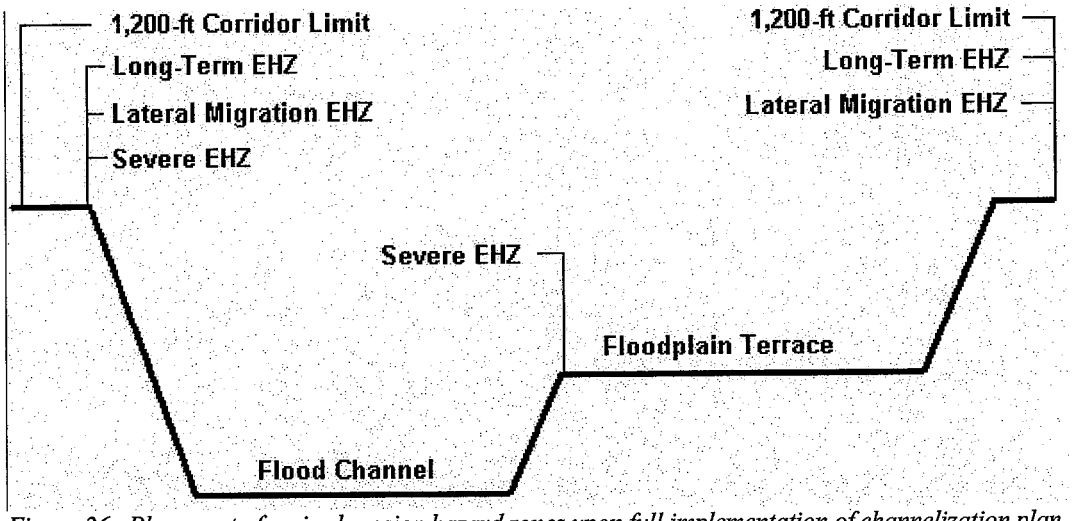


Figure 26. Placement of revised erosion hazard zones upon full implementation of channelization plan.

Piecemeal Implementation. If the proposed channelization plan is implemented in phases or as discrete reaches, then erosion hazard zones should be redelineated to reflect the specific conditions of the constructed portions of the channel. Because of the infinite number of possible piecemeal implementation scenarios, it is not possible to provide estimates of the probable impacts on the existing erosion hazard zones. However, if the piecemeal implementation effectively contains the 100-year flood within engineered channelization measures, then it is likely that the erosion zones will be modified as indicated under the "Full Implementation" discussion above.

Recommendations for Further Analysis

Based on the results of the technical analyses performed by JEF, we offer the following recommendations for further analyses during the final design process:

- Survey bridge structures to determine horizontal position of piers and abutments relative to the proposed channel alignment. Revise the HEC-RAS model to reflect corrected bridge data, as needed.

- Explore flood detention concepts utilizing existing, abandoned, and proposed aggregate mines.

- Revise channel dimensions to assure adequate conveyance capacity, pending decisions by District staff regarding freeboard and design flow rates.

- Increase the slope of the reach between Bell Road and the McMicken Outfall Channel to 0.0018 ft/ft to reduce the potential for significant long-term aggradation.

- Prepare a sediment maintenance and inspection plan.

- Fully line the reach adjacent to the El Mirage Landfill to prevent erosion and long-term scour.

- Perform detailed hydraulic analyses of the channel transitions proposed at drop structures. Specifically, consider the potential impacts of cross waves and hydraulic jumps on channel capacity and scour.

- Perform detailed hydraulic analyses of the skewed drop structures to estimate impacts on hydraulic jumps and scour depths at varying flow rates.

- Perform physical modeling of 6:1 drop structures to estimate the length of scour holes at drop structures.

- Explore natural channel design concepts for low flow channel to accommodate the natural braided pattern likely to develop in the flood channel.

- Explore natural channel design concepts for the tributary inflows that could be integrated into multiple-use concepts proposed for the floodplain terraces.

Appendix

Table A-1. Comparison of Computed 100-Year Water Surface Elevation and Channel Velocity Results for Subcritical vs. Mixed Profile HEC-RAS Model

Column 1			Column 2			Column 3		
Station	WSEL	Velocity	Station	WSEL	Velocity	Station	WSEL	Velocity
159166	0	0	117000	0	0	79712	0	0
158641	0	0	116715	0	0	79688	0	0
158196	0	0	116465	0	0	79588	0	0
157736	0	0	116215	0	0	79188	0	0
157241	0	0	115815	0	0	78938	0	0
157171	0	0	115715	0	0	78688	0	0
157104	0	0	115685	-4.32	13.17	78438	0	0
157000	-5.13	24.63	115585	0	0	78188	0	0
156500	0	0	115185	0	0	77938	0	0
156000	0	0	115018	0	0	77688	0	0
155500	0	0	114982	-5.27	15.43	77500	0	0
155000	0	0	114882	0	0	77000	0	0
154500	0	0	114482	0	0	76872	0	0
154000	0	0	114185	0	0	76512	0	0
153500	0	0	113935	0	0	76112	0	0
153281	0	0	113685	0	0	76012	0	0
153031	0	0	113435	0	0	75988	0	0
152631	0	0	113185	0	0	75888	0	0
152012	0	0	113000	0	0	75488	0	0
151988	0	0	112500	0	0	75238	0	0
151838	0	0	112400	0	0	74988	0	0
151438	0	0	112300	0	0	74738	0	0
151188	0	0	112200	0	0	74488	0	0
150938	0	0	112100	0	0	74238	0	0
150688	0	0	112000	0	0	74000	0	0
150438	0	0	111900	0	0	73500	0	0
150000	0	0	111800	0	0	73000	0	0
149500	0	0	111700	0	0	72892	0	0
149000	0	0	111600	0	0	72692	0	0
148500	0	0	111500	0	0	72492	0	0
148000	0	0	111043	0	0	72392	0	0
147500	0	0	110793	0	0	72312	-1.41	4.61
147000	0	0	110543	0	0	72288	0	0
146500	0	0	110143	0	0	72188	0	0
146018	0	0	110043	0	0	71988	0	0
145918	0	0	110007	0	0	71788	0	0
145768	0	0	109907	0	0	71500	0	0
145518	0	0	109507	0	0	71000	0	0
145118	0	0	109257	0	0	70500	0	0
145018	0	0	109007	0	0	70000	0	0
144982	-4.89	15.15	108757	0	0.01	69500	0	0
144882	0	0	108507	-0.01	0	69000	0	0
144482	0	0	108257	0	0.01	68500	0	0

Table A-1. Comparison of Computed 100-Year Water Surface Elevation and Channel Velocity Results for Subcritical vs. Mixed Profile HEC-RAS Model

Column 1			Column 2			Column 3		
Station	WSEL	Velocity	Station	WSEL	Velocity	Station	WSEL	Velocity
144232	0	0	108007	0	0	68000	0	0
143982	0	0	107507	-0.01	0	67512	0	0
143732	0	0	107000	-0.01	0.01	67312	0	0
143482	0	0	106500	0	0.01	67112	0	0
143232	0	0	106000	-0.02	0.01	67012	0	0
142982	0	0	105500	-0.02	0.03	66988	0	0
142732	0	0	105000	-0.03	0.04	66888	0	0
142482	0	0	104500	-0.05	0.07	66688	0	0
142000	0	0	104000	-0.09	0.11	66488	0	0
141915	0	0	103500	-0.13	0.16	66000	0	0
141815	0	0	103000	-0.2	0.24	65500	0	0
141665	0	0	102500	-0.28	0.29	65000	0	0
141415	0	0	102180	-0.37	0.4	64500	0	0
141015	0	0.01	101930	-0.53	0.59	64000	0	0
140915	0	0	101530	-0.67	0.6	63500	0	0
140885	-4.47	13.7	101430	-0.7	0.62	63000	0	0
140785	0	0	101344	-2.21	6.51	62500	0	0
140385	0	0	101262	0	0	62187	0	0
140135	0	0	101238	0	0	61987	0	0
139885	0	0	101138	0	0	61787	0	0
139635	0	0	100738	0	0	61687	0	0
139385	0	0	100488	0	0	61591.5	0	0
139135	0	0	100238	0	0	61558.5	0	0
138885	0	0	99988	0	0	61458.5	0	0
138635	0	0	99738	0	0	61258.5	0	0
138385	0	0	99488	0	0	61058.5	0	0
138000	0	0	99000	0	0	61000	0	0
137500	0	0	98500	0	0	60500	0	0
137000	0	0	98000	0	0	60000	0	0
136923	0	0	97500	0	0	59500	0	0
136500	0	0	97000	0	0	59018	0	0
136427	0	0	96500	0	0	58818	0	0
136027	0	0	96000	0	0	58618	0	0
135927	0.01	-0.01	95718	0	0	58518	0	0
135873	-5.25	18.22	95468	0	0	58482	0	0
135773	0	0	95218	0	0	58382	0	0
135373	0	0	94818	0	0	58182	0	0
135123	0	0	94718	0	0	57982	0	0
134873	0	0.01	94682	0	0	57500	0	0
134623	0	0	94582	0	0	57000	0	0
134373	0	0.01	94182	0	0	56500	0	0
134123	-0.01	0.01	93932	0	0	56318	0	0
134000	-0.01	0.01	93682	0	0	56118	0	0
133500	-0.02	0.03	93432	0	0	55918	0	0

Table A-1. Comparison of Computed 100-Year Water Surface Elevation and Channel Velocity Results for Subcritical vs. Mixed Profile HEC-RAS Model

Column 1			Column 2			Column 3		
Station	WSEL	Velocity	Station	WSEL	Velocity	Station	WSEL	Velocity
133000	-0.06	0.09	93182	0	0	55818	0	0
132500	-0.14	0.2	93000	0	0	55782	0	0
132126	-0.21	0.29	92500	0	0	55682	0	0
131876	-0.27	0.35	92000	0	0	55482	0	0
131626	-0.56	0.76	91600	0	0	55282	0	0
131226	-0.85	0.91	91500	0	0	55000	0	0
131126	-0.94	1.01	91000	0	0	54500	0	0
130984	-1.97	5.45	90500	0	0	54000	0	0
130924	0	0	90000	0	0	53500	0	0
130876	-4.82	16.55	89500	0	0	53000	0	0
130664	0	0	89000	0	0	52500	0	0
130264	0	0	88500	0	0	51500	0	0
130014	0	0	88356	0	0	51000	0	0
129764	0	0	88331	0	0	50500	0	0
129500	0	0	88295	0	0	50000	0	0
129000	0	0	88170	0	0	49884.8	0	0
128500	0	0	88150	0	0	49860.68	0	0
128000	0	0	88000	0	0	49709.6	0	0
127500	0	0	87500	0	0	49500	0	0
127000	0	0	87000	0	0	49000	0	0
126500	0	0	86500	0	0	48500	0	0
126000	0	0	86000	0	0	48000	0	0
125500	0	0	85500	0	0	47500	0	0
125000	0	0	85000	0	0	47000	0	0
124500	0	0	84500	0	0	46500	0	0
124000	0	0	84000	0	0	46000	0	0
123500	0	0	83512	0	0	45500	0	0
123000	0	0	83488	0	0	45000	0	0
122500	0	0	83380	0	0	44500	0	0
122000	0	0	83280	0	0	44026.4	0	0
121500	0	0	83000	0	0	43973.6	0	0
121000	0	0	82500	0	0	43920.8	0	0
120500	0	0	82000	0	0	43762.4	0	0
120000	0	0	81500	0	0	43287.2	0	0
119500	0	0	81000	0	0	42759.2	0	0
119000	0	0	80592	0	0	42231.2	0	0
118500	0	0	80462	0	0	41756	0	0
118000	0	0	80212	0	0	41280.8	0	0
117500	0	0	79812	0	0			

Notes:

1. WSEL indicates the difference in subcritical and mixed profile HEC-RAS model results. A negative value means the subcritical profile had a higher water surface elevation.
2. Velocity indicates the difference in the subcritical and mixed profile HEC-RAS channel velocity. A positive value means the mixed profile had higher velocities.

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
157104								
157000					0.24	1.62	2.56	0.65
156500					0.69	0.69	2.86	2.86
156000					0.72	0.72	3.00	3.00
155500					0.77	0.77	3.04	3.04
155000					0.87	0.87	3.10	3.10
154500					1.06	1.06	3.23	3.23
154000					1.40	1.40	3.42	3.42
153500					1.93	1.93	3.69	3.69
153281					1.99	1.99	3.65	3.65
153031					0.78	0.78	2.64	2.64
152631					0.62	0.62	2.02	2.02
152012								
151988					1.73	1.73	3.90	3.90
151838					1.90	1.90	3.99	3.99
151438					0.98	0.98	3.34	3.34
151188					0.80	0.80	3.11	3.11
150938					0.71	0.71	2.99	2.99
150688					0.69	0.69	2.94	2.94
150438					0.67	0.67	2.92	2.92
150000					0.68	0.68	2.94	2.94
149500					0.69	0.69	2.94	2.94
149000					0.69	0.69	2.95	2.95
148500					0.72	0.72	2.97	2.97
148000					0.78	0.78	3.01	3.01
147500					0.86	0.86	3.04	3.04
147000					1.03	1.03	3.10	3.10
146500					1.34	1.34	3.19	3.19
146018					1.78	1.78	3.31	3.31
145918					1.89	1.89	3.34	3.34
145768					1.86	1.86	3.22	3.22
145518					0.19	0.19	1.85	1.85
145118							0.32	0.32
145018								
144982					1.85	1.85	4.09	4.09
144882					1.96	1.96	4.16	4.16
144482					1.20	1.20	3.63	3.63
144232					1.07	1.07	3.43	3.43
143982					0.95	0.95	3.25	3.25
143732					0.85	0.85	3.12	3.12
143482					0.80	0.80	3.00	3.00

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	**Q100 HEC-RAS Profiles**				**SPF HEC-RAS Profiles**			
	Low N	**Low N Mixed**	**High N**	**High N Mixed**	**Low N**	**Low N Mixed**	**High N**	**High N Mixed**
143232					0.82	0.82	2.97	2.97
142982					0.90	0.90	2.98	2.98
142732					1.06	1.06	3.05	3.05
142482					1.29	1.29	3.17	3.17
142000					1.68	1.68	3.28	3.28
141915					1.79	1.79	3.31	3.31
141815					1.89	1.89	3.34	3.34
141665					1.86	1.86	3.20	3.21
141415					0.18	0.18	1.85	1.85
141015							0.31	0.31
140915								
140885					1.01		3.40	3.40
140785					1.07	1.07	3.42	3.42
140385					1.45	1.45	3.59	3.59
140135					1.67	1.67	3.65	3.65
139885					0.90	0.90	3.21	3.21
139635					0.79	0.79	3.06	3.06
139385					0.70	0.70	2.94	2.94
139135					0.67	0.67	2.88	2.88
138885					0.68	0.68	2.86	2.86
138635					0.69	0.69	2.85	2.85
138385					0.71	0.71	2.85	2.85
138000					0.72	0.72	2.82	2.82
137500					0.94	0.94	2.85	2.85
137000					1.39	1.39	3.02	3.03
136923					1.49	1.49	3.06	3.06
136500					0.74	0.74	2.21	2.21
136427					0.06	0.06	1.69	1.70
136027							0.30	0.31
135927								
135873					1.74		3.73	3.72
135773					1.86	1.86	3.78	3.75
135373					0.82	0.82	2.98	2.95
135123					0.33	0.32	2.55	2.50
134873					0.22	0.20	2.36	2.30
134623					0.39	0.37	2.45	2.38
134373					0.39	0.36	2.56	2.47
134123					0.42	0.38	2.61	2.49
134000					0.44	0.39	2.63	2.51
133500					0.60	0.52	2.79	2.57
133000					0.93	0.77	3.01	2.70
132500					1.43	1.18	3.34	2.88
132126			0.12		1.94	1.62	3.58	3.07
131876			0.27		2.32	1.96	3.80	3.23

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
131626					1.45	0.75	2.95	2.05
131226					1.73	0.74	2.68	1.14
131126					1.85	0.78	2.65	0.79
131055								
130984								
130924								
130876			0.35	0.35	1.42		3.68	3.68
130664			0.47	0.47	1.62	1.62	3.76	3.76
130264			0.20	0.20	0.84	0.84	3.21	3.21
130014			0.11	0.11	0.78	0.78	3.08	3.08
129764			0.09	0.09	0.76	0.76	3.01	3.01
129500			0.05	0.05	0.72	0.72	2.94	2.94
129000					0.66	0.66	2.84	2.84
128500					0.67	0.67	2.79	2.79
128000					0.77	0.77	2.83	2.83
127500					1.01	1.01	2.93	2.93
127000			0.04	0.04	1.45	1.45	3.16	3.16
126500			0.19	0.19	2.09	2.09	3.47	3.47
126012					0.21	0.21	1.77	1.77
125612								
125512								
125488			0.31	0.31	1.34	1.34	3.51	3.51
125388			0.34	0.34	1.41	1.41	3.52	3.52
124988			0.06	0.06	0.70	0.70	3.01	3.01
124500					0.51	0.51	2.72	2.72
124000					0.52	0.52	2.70	2.70
123500					0.64	0.64	2.80	2.80
123000					0.84	0.84	2.93	2.93
122500			0.00	0.00	1.09	1.09	3.04	3.04
122000			0.08	0.08	1.52	1.52	3.24	3.24
121500			0.22	0.22	2.12	2.12	3.52	3.52
121012					0.23	0.23	1.84	1.84
120612							0.19	0.19
120512								
120488			0.96	0.96	2.12	2.12	4.41	4.41
120388			1.02	1.02	2.21	2.21	4.46	4.46
119988			0.70	0.70	1.38	1.38	3.87	3.87
119500			0.35	0.35	1.05	1.05	3.41	3.41
119000			0.10	0.10	0.82	0.82	3.10	3.10
118500			0.06	0.06	0.88	0.88	3.08	3.08
118000			0.15	0.15	1.12	1.12	3.23	3.23
117500			0.24	0.24	1.45	1.45	3.43	3.43
117000			0.30	0.30	1.87	1.87	3.55	3.55
116715			0.36	0.36	2.17	2.17	3.67	3.67

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
116465			0.06	0.06	1.97	1.97	3.34	3.34
116215					0.31	0.31	1.96	1.96
115815							0.35	0.35
115715								
115685					0.51		2.31	2.31
115585					0.47	0.47	2.15	2.15
115185					0.11	0.11	1.15	1.15
115018								
114982			0.63	0.63	1.56		3.95	3.95
114882			0.67	0.67	1.61	1.61	3.97	3.97
114482			0.85	0.85	1.91	1.91	4.09	4.09
114185			0.62	0.62	1.28	1.28	3.71	3.71
113935			0.49	0.49	1.16	1.16	3.52	3.52
113685			0.38	0.38	1.04	1.04	3.35	3.35
113435			0.29	0.29	0.96	0.96	3.23	3.23
113185			0.21	0.21	0.91	0.91	3.15	3.15
113000			0.16	0.16	0.90	0.90	3.10	3.10
112500			0.12	0.12	1.00	1.00	3.09	3.09
112400			0.12	0.12	1.04	1.04	3.11	3.11
112300			0.10	0.10	1.07	1.07	3.11	3.11
112200			0.13	0.13	1.13	1.13	3.16	3.16
112100			0.14	0.14	1.18	1.18	3.19	3.19
112000			0.16	0.16	1.25	1.25	3.23	3.23
111900			0.17	0.17	1.32	1.32	3.26	3.26
111800			0.18	0.18	1.38	1.38	3.30	3.30
111700			0.19	0.19	1.45	1.45	3.33	3.33
111600			0.21	0.21	1.53	1.53	3.36	3.36
111500			0.22	0.22	1.61	1.61	3.40	3.40
111043			0.30	0.30	2.06	2.06	3.59	3.59
110793			0.07	0.07	1.97	1.97	3.36	3.36
110543					0.32	0.32	1.97	1.97
110143							0.37	0.37
110043								
110007			1.93	1.92	2.71	2.71	5.32	5.31
109907			1.97	1.97	2.77	2.77	5.34	5.34
109507			1.71	1.71	2.00	1.99	4.78	4.77
109257			1.60	1.60	1.82	1.83	4.58	4.57
109007			1.52	1.52	1.67	1.66	4.41	4.40
108757			1.47	1.46	1.54	1.53	4.25	4.24
108507			1.33	1.32	1.41	1.40	4.13	4.12
108257			1.27	1.27	1.36	1.34	4.09	4.07
108007			1.28	1.27	1.37	1.36	4.10	4.08
107507			1.31	1.29	1.41	1.39	4.12	4.08
107000			1.36	1.34	1.50	1.45	4.14	4.10

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
106500			1.41	1.38	1.58	1.53	4.20	4.14
106000			1.40	1.36	1.62	1.54	4.20	4.12
105500			1.38	1.34	1.68	1.56	4.21	4.10
105000			1.36	1.29	1.76	1.59	4.21	4.05
104500			1.35	1.26	1.89	1.66	4.25	4.03
104000			1.34	1.21	2.07	1.74	4.28	3.98
103500			1.34	1.15	2.34	1.89	4.35	3.95
103000			1.35	1.07	2.67	2.10	4.45	3.91
102500			1.39	0.99	3.08	2.37	4.59	3.88
102180			1.26	0.74	3.20	2.35	4.50	3.58
101930			0.89	0.12	3.00	1.92	4.05	2.87
101530			0.53		3.19	1.88	3.77	2.14
101430			0.46		3.27	1.91	3.72	1.91
101347								
101344							0.29	
101262							0.41	0.41
101238			1.87	1.87	2.26	2.26	4.90	4.90
101138			1.87	1.87	2.29	2.29	4.91	4.91
100738			1.53	1.53	1.61	1.61	4.39	4.39
100488			1.38	1.38	1.47	1.47	4.22	4.22
100238			1.28	1.28	1.38	1.38	4.10	4.10
99988			1.23	1.23	1.32	1.32	4.04	4.04
99738			1.21	1.21	1.33	1.33	4.03	4.03
99488			1.20	1.20	1.34	1.34	4.02	4.02
99000			1.21	1.21	1.37	1.37	4.01	4.01
98500			1.21	1.21	1.42	1.42	3.99	3.99
98000			1.22	1.22	1.48	1.48	3.96	3.96
97500			1.27	1.27	1.62	1.62	3.98	3.98
97000			1.26	1.26	1.74	1.74	3.96	3.96
96500			1.19	1.19	1.85	1.85	3.88	3.88
96000			1.09	1.09	2.02	2.02	3.78	3.78
95718			1.03	1.03	2.14	2.14	3.73	3.73
95468			0.68	0.68	1.84	1.84	3.34	3.34
95218					0.54	0.54	2.05	2.05
94818							0.11	0.11
94718								
94682	0.39	0.39	2.84	2.84	3.23	3.23	6.13	6.13
94582	0.46	0.46	2.88	2.88	3.30	3.30	6.19	6.19
94182	0.19	0.19	2.72	2.72	2.83	2.83	5.89	5.89
93932	0.20	0.20	2.69	2.69	2.84	2.84	5.85	5.85
93682	0.24	0.24	2.68	2.68	2.87	2.87	5.86	5.86
93432	0.31	0.31	2.73	2.73	2.96	2.96	5.92	5.92
93182	0.39	0.39	2.79	2.79	3.06	3.06	6.00	6.00
93000	0.42	0.42	2.82	2.82	3.13	3.13	6.06	6.06

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
92500	0.12	0.12	2.71	2.71	2.92	2.92	6.00	6.00
92000			0.74	0.74	0.98	0.98	4.06	4.06
91600			0.67	0.67	0.78	0.78	3.94	3.94
91500			0.67	0.67	0.80	0.80	3.93	3.93
91000			0.64	0.64	0.81	0.81	3.87	3.87
90500			0.59	0.59	0.83	0.83	3.80	3.80
90000			0.53	0.53	0.86	0.86	3.70	3.70
89500			0.45	0.45	0.89	0.89	3.58	3.58
89000			0.34	0.34	0.95	0.95	3.42	3.42
88500			0.10	0.10	0.80	0.80	3.08	3.08
88356								
88347								
88331								
88295								
88174								
88170								
88150							2.02	2.02
88000							1.69	1.69
87500							0.87	0.87
87000							2.39	2.39
86500							2.20	2.20
86000							2.23	2.23
85500					0.05	0.05	2.36	2.36
85000					0.38	0.38	2.51	2.51
84500					0.55	0.55	2.45	2.45
84000					0.14	0.14	1.85	1.85
83512							0.64	0.64
83488	0.50	0.50	2.66	2.66	3.32	3.32	5.83	5.83
83380	0.63	0.63	2.75	2.75	3.44	3.44	5.92	5.92
83280			2.35	2.35	2.49	2.49	5.37	5.37
83000			2.20	2.20	2.39	2.39	5.19	5.19
82500			1.97	1.97	2.26	2.26	4.92	4.92
82000			1.75	1.75	2.18	2.18	4.67	4.67
81500			1.56	1.56	2.18	2.18	4.46	4.46
81000			1.26	1.26	2.17	2.17	4.14	4.14
80592			0.95	0.95	2.18	2.18	3.81	3.81
80462			0.80	0.80	2.18	2.18	3.70	3.70
80212					0.53	0.53	2.35	2.35
79812							0.87	0.87
79712							0.91	0.91
79688	0.12	0.12	2.34	2.34	2.88	2.88	5.42	5.42
79588	0.18	0.18	2.37	2.37	2.94	2.94	5.45	5.45
79188			2.03	2.03	2.29	2.29	4.94	4.94
78938			1.88	1.88	2.18	2.18	4.77	4.77

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	**Q100 HEC-RAS Profiles**				**SPF HEC-RAS Profiles**			
	Low N	**Low N Mixed**	**High N**	**High N Mixed**	**Low N**	**Low N Mixed**	**High N**	**High N Mixed**
78688			1.76	1.76	2.12	2.12	4.63	4.63
78438			1.65	1.65	2.09	2.09	4.53	4.53
78188			1.58	1.58	2.10	2.10	4.47	4.47
77938			1.51	1.51	2.13	2.13	4.43	4.43
77688			1.44	1.44	2.17	2.17	4.36	4.36
77500			1.37	1.37	2.20	2.20	4.29	4.29
77000			1.12	1.12	2.31	2.31	4.10	4.10
76872			1.05	1.05	2.34	2.34	4.06	4.06
76512					0.39	0.39	2.63	2.63
76112							1.42	1.42
76012							1.61	1.61
75988	0.45	0.45	2.84	2.84	3.48	3.48	6.13	6.13
75888	0.53	0.53	2.88	2.88	3.55	3.56	6.19	6.19
75488	0.29	0.29	2.72	2.72	3.20	3.20	5.91	5.91
75238	0.30	0.30	2.67	2.67	3.24	3.24	5.85	5.85
74988	0.32	0.32	2.62	2.62	3.28	3.29	5.81	5.81
74738	0.40	0.40	2.63	2.63	3.40	3.40	5.84	5.84
74488	0.29	0.29	2.50	2.50	3.22	3.22	5.64	5.64
74238	0.31	0.31	2.41	2.41	3.26	3.26	5.56	5.56
74000	0.20	0.20	2.22	2.22	3.10	3.10	5.30	5.30
73500	0.14	0.14	1.81	1.81	3.07	3.08	4.85	4.85
73000	0.17	0.17	1.41	1.41	3.19	3.19	4.49	4.49
72892			1.06	1.06	2.78	2.78	4.00	4.00
72692			0.80	0.80	2.79	2.79	3.74	3.74
72492			0.62	0.62	2.88	2.88	3.56	3.56
72392			0.45	0.45	2.84	2.84	3.37	3.37
72352								
72312							0.11	
72288			1.78	1.78	2.10		4.58	4.58
72188			1.75	1.75	2.09	1.90	4.54	4.54
71988			1.74	1.74	2.13	1.91	4.52	4.52
71788			1.42	1.42	1.56	1.61	4.11	4.11
71500			1.35	1.35	1.53	1.53	4.04	4.04
71000			1.28	1.28	1.53	1.53	3.97	3.97
70500			1.31	1.31	1.62	1.62	4.03	4.03
70000			1.41	1.41	1.83	1.83	4.13	4.13
69500			1.47	1.47	2.03	2.02	4.22	4.22
69000			1.49	1.49	2.23	2.23	4.30	4.30
68500			1.44	1.44	2.44	2.44	4.31	4.31
68000			1.35	1.35	2.68	2.68	4.31	4.31
67512					0.92	0.92	2.69	2.69
67312					0.54	0.52	2.08	2.08
67112							1.19	1.19
67012							1.30	1.30

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	Q100 HEC-RAS Profiles				SPF HEC-RAS Profiles			
	Low N	Low N Mixed	High N	High N Mixed	Low N	Low N Mixed	High N	High N Mixed
66988	0.71	0.77	2.68	2.68	3.64	3.69	5.80	5.80
66888	0.77	0.84	2.70	2.70	3.72	3.76	5.82	5.82
66688			2.04	2.04	2.21	2.32	4.91	4.91
66488			1.77	1.77	1.93	2.07	4.58	4.58
66000			1.26	1.26	1.46	1.68	4.01	4.01
65500			1.28	1.28	1.56	1.47	3.99	3.99
65000			1.16	1.16	1.54	1.53	3.87	3.87
64500			1.38	1.38	1.89	1.88	4.09	4.09
64000			1.48	1.48	2.06	2.04	4.19	4.19
63500			1.44	1.44	2.24	2.20	4.21	4.21
63000			1.41	1.41	2.46	2.42	4.22	4.22
62500			1.10	1.10	2.46	2.40	3.95	3.95
62187					0.89	0.65	2.45	2.45
61987					0.49	0.17	1.74	1.74
61787					0.17		1.03	1.03
61687							0.39	0.39
61637								
61591.5								
61558.5	0.27	0.27	2.44	2.44	3.08	3.08	5.49	5.49
61458.5	0.32	0.32	2.46	2.46	3.12	3.12	5.49	5.49
61258.5	0.23	0.23	2.35	2.35	2.98	2.98	5.34	5.34
61058.5			2.09	2.09	2.56	2.56	4.97	4.97
61000			2.06	2.06	2.54	2.54	4.91	4.91
60500			1.73	1.73	2.43	2.43	4.54	4.54
60000			1.48	1.48	2.44	2.44	4.26	4.26
59500			1.27	1.27	2.58	2.58	4.11	4.11
59018					0.57	0.57	2.18	2.18
58818					0.05	0.05	1.36	1.36
58618							0.46	0.46
58518								
58482	0.14	0.14	2.20	2.20	2.92	2.92	5.15	5.15
58382	0.18	0.18	2.19	2.19	2.97	2.97	5.15	5.15
58182	0.06	0.06	2.05	2.05	2.79	2.79	4.93	4.93
57982			1.72	1.72	2.29	2.29	4.48	4.48
57500			1.32	1.32	2.16	2.16	4.02	4.02
57000			1.08	1.08	2.26	2.26	3.81	3.81
56500			0.85	0.85	2.47	2.47	3.70	3.70
56318					0.59	0.59	2.18	2.18
56118					0.05	0.05	1.36	1.36
55918							0.45	0.45
55818								
55782	0.04	0.04	2.13	2.13	2.88	2.88	5.34	5.34
55682	0.09	0.09	2.12	2.12	2.91	2.91	5.34	5.34
55482			1.98	1.98	2.73	2.73	5.15	5.15

Table A-2. Additional Bank Height Needed to Contain WSEL + Freeboard + Superelevation (ft)								
RM	**Q100 HEC-RAS Profiles**				**SPF HEC-RAS Profiles**			
	Low N	**Low N Mixed**	**High N**	**High N Mixed**	**Low N**	**Low N Mixed**	**High N**	**High N Mixed**
55282			1.63	1.63	2.21	2.21	4.75	4.75
55000			1.30	1.30	2.02	2.02	4.45	4.45
54500			0.68	0.68	1.77	1.77	4.00	4.00
54000			0.15	0.15	1.79	1.79	3.84	3.84
53500					2.15	2.15	4.04	4.04
53000					2.73	2.73	4.48	4.48
52500					1.11	1.11	3.79	3.79
51500					0.95	0.95	3.35	3.35
51000					0.90	0.90	3.25	3.25
50500					0.38	0.38	2.71	2.71
50000							1.83	1.83
49930								
49884.8							1.80	1.80
49860.68							2.42	2.42
49709.6			0.78	0.78	2.89	2.89	6.37	6.37
49500			0.76	0.76	2.89	2.88	6.36	6.36
49000			0.74	0.74	2.90	2.88	6.34	6.34
48500			0.70	0.70	2.90	2.89	6.34	6.34
48000			0.58	0.58	2.79	2.76	6.27	6.27
47500			0.29	0.29	2.42	2.39	6.07	6.07
47000			0.05	0.05	2.22	2.19	5.93	5.93
46500					2.13	2.10	5.90	5.90
46000					2.10	2.06	5.93	5.93
45500					2.07	2.03	5.95	5.95
45000					2.06	1.99	5.98	5.98

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
157104	1339.92	3.0	0.3	1343.2	1345.88		
157000	1319.08	9.3	1.7	1330.2	1328.55	1.6	
156500	1324.92	3.0	0.1	1328.0	1327.35	0.7	
156000	1323.52	3.2	0.2	1326.9	1326.15	0.7	
155500	1322.37	3.2	0.2	1325.7	1324.95	0.8	
155000	1321.27	3.2	0.2	1324.6	1323.75	0.9	
154500	1320.27	3.2	0.2	1323.6	1322.55	1.1	
154000	1319.40	3.2	0.1	1322.7	1321.35	1.4	
153500	1318.70	3.3	0.1	1322.1	1320.15	1.9	
153281	1318.17	3.3	0.1	1321.6	1319.63	2.0	
153031	1316.12	3.5	0.2	1319.8	1319.03	0.8	
152631	1315.41	3.1	0.2	1318.7	1318.07	0.6	
152012	1310.95	3.7	0.3	1315.0	1316.58		
151988	1310.96	3.2	0.1	1314.3	1312.58	1.7	
151838	1310.75	3.2	0.1	1314.1	1312.22	1.9	
151438	1308.56	3.5	0.2	1312.2	1311.26	1.0	
151188	1307.89	3.4	0.2	1311.5	1310.66	0.8	
150938	1307.34	3.2	0.2	1310.8	1310.06	0.7	
150688	1306.80	3.2	0.2	1310.1	1309.46	0.7	
150438	1306.18	3.2	0.2	1309.5	1308.86	0.7	
150000	1305.13	3.2	0.2	1308.5	1307.80	0.7	
149500	1303.94	3.2	0.2	1307.3	1306.60	0.7	
149000	1302.74	3.2	0.2	1306.1	1305.40	0.7	
148500	1301.57	3.2	0.2	1304.9	1304.20	0.7	
148000	1300.43	3.2	0.2	1303.8	1303.00	0.8	
147500	1299.32	3.2	0.2	1302.7	1301.80	0.9	
147000	1298.29	3.2	0.2	1301.6	1300.60	1.0	
146500	1297.39	3.2	0.1	1300.7	1299.40	1.3	
146018	1296.66	3.2	0.1	1300.0	1298.25	1.8	
145918	1296.53	3.3	0.1	1299.9	1298.01	1.9	
145768	1296.06	3.3	0.1	1299.5	1297.65	1.9	
145518	1293.47	3.5	0.3	1297.2	1297.05	0.2	
145118	1292.18	3.1	0.2	1295.5	1296.09		
145018	1290.22	3.7	0.3	1294.3	1295.85		
144982	1288.34	3.2	0.1	1291.7	1289.85	1.9	
144882	1288.20	3.3	0.1	1291.6	1289.61	2.0	
144482	1286.18	3.5	0.2	1289.9	1288.65	1.2	
144232	1285.51	3.4	0.2	1289.1	1288.05	1.1	
143982	1284.85	3.3	0.2	1288.4	1287.45	0.9	
143732	1284.21	3.3	0.2	1287.7	1286.85	0.9	
143482	1283.61	3.3	0.2	1287.1	1286.25	0.8	
143232	1283.08	3.2	0.2	1286.5	1285.65	0.8	
142982	1282.61	3.2	0.2	1286.0	1285.05	0.9	
142732	1282.21	3.2	0.1	1285.5	1284.45	1.1	
142482	1281.85	3.2	0.1	1285.1	1283.85	1.3	
142000	1281.02	3.2	0.1	1284.4	1282.70	1.7	
141915	1280.91	3.2	0.1	1284.3	1282.49	1.8	

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
141815	1280.77	3.3	0.1	1284.1	1282.25	1.9	
141665	1280.30	3.3	0.1	1283.8	1281.89	1.9	
141415	1277.70	3.5	0.3	1281.5	1281.29	0.2	
141015	1276.41	3.1	0.2	1279.7	1280.33		
140915	1274.48	3.7	0.3	1278.5	1280.09		
140885	1267.20	5.3	0.9	1273.3	1275.09		
140785	1272.62	3.2	0.1	1275.9	1274.85	1.1	
140385	1271.99	3.2	0.1	1275.3	1273.89	1.5	
140135	1271.60	3.2	0.1	1275.0	1273.29	1.7	
139885	1270.04	3.3	0.2	1273.6	1272.69	0.9	
139635	1269.38	3.3	0.2	1272.9	1272.09	0.8	
139385	1268.74	3.3	0.2	1272.2	1271.49	0.7	
139135	1268.16	3.2	0.2	1271.6	1270.89	0.7	
138885	1267.62	3.2	0.2	1271.0	1270.29	0.7	
138635	1267.03	3.2	0.2	1270.4	1269.69	0.7	
138385	1266.45	3.2	0.2	1269.8	1269.09	0.7	
138000	1265.54	3.2	0.2	1268.9	1268.16	0.7	
137500	1264.60	3.1	0.1	1267.9	1266.96	0.9	
137000	1263.86	3.2	0.1	1267.2	1265.76	1.4	
136923	1263.79	3.2	0.1	1267.1	1265.58	1.5	
136500	1261.79	3.3	0.2	1265.3	1264.56	0.7	
136427	1260.84	3.4	0.3	1264.5	1264.39	0.1	
136027	1259.51	3.1	0.2	1262.8	1263.43		
135927	1257.58	3.7	0.3	1261.6	1263.19		
135873	1245.71	6.7	1.2	1253.6	1254.19		
135773	1252.44	3.2	0.1	1255.8	1253.95	1.9	
135373	1250.12	3.5	0.2	1253.8	1252.99	0.8	
135123	1249.04	3.4	0.3	1252.7	1252.39	0.3	
134873	1248.82	3.0	0.2	1252.0	1251.79	0.2	
134623	1248.49	2.9	0.1	1251.6	1251.19	0.4	
134373	1247.73	3.1	0.2	1250.9	1250.59	0.4	
134123	1247.12	3.1	0.2	1250.4	1249.99	0.4	
134000	1246.84	3.1	0.2	1250.1	1249.70	0.4	
133500	1245.77	3.1	0.2	1249.0	1248.50	0.5	
133000	1244.82	3.1	0.1	1248.1	1247.30	0.8	
132500	1244.02	3.1	0.1	1247.3	1246.10	1.2	
132126	1243.54	3.2	0.1	1246.8	1245.20	1.6	
131876	1243.25	3.2	0.1	1246.6	1244.60	2.0	
131626	1241.16	3.4	0.2	1244.8	1244.00	0.8	
131226	1240.52	3.1	0.1	1243.8	1243.04	0.7	
131126	1240.31	3.1	0.1	1243.6	1242.80	0.8	
131055					Bridge		
130984	1235.58	3.9	0.5	1240.0	1242.45		
130924	1236.55	3.7	0.3	1240.5	1242.31		
130876	1225.85	6.3	1.1	1233.3	1234.31		
130664	1232.12	3.2	0.1	1235.4	1233.82	1.6	
130264	1230.13	3.4	0.2	1233.7	1232.88	0.8	

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
130014	1229.63	3.3	0.2	1233.1	1232.30	0.8	
129764	1229.07	3.2	0.2	1232.5	1231.71	0.8	
129500	1228.43	3.2	0.2	1231.8	1231.10	0.7	
129000	1227.24	3.2	0.2	1230.6	1229.93	0.7	
128500	1226.12	3.1	0.2	1229.4	1228.76	0.7	
128000	1225.08	3.1	0.2	1228.4	1227.59	0.8	
127500	1224.15	3.1	0.1	1227.4	1226.42	1.0	
127000	1223.41	3.2	0.1	1226.7	1225.25	1.5	
126500	1222.83	3.2	0.1	1226.2	1224.08	2.1	
126012	1219.37	3.5	0.3	1223.1	1222.93	0.2	
125612	1218.07	3.1	0.2	1221.4	1222.00		
125512	1216.15	3.7	0.3	1220.2	1221.76		
125488	1215.78	3.2	0.1	1219.1	1217.76	1.3	
125388	1215.61	3.2	0.1	1218.9	1217.53	1.4	
124988	1213.76	3.3	0.2	1217.3	1216.61	0.7	
124500	1212.64	3.2	0.2	1216.0	1215.49	0.5	
124000	1211.61	3.1	0.2	1214.9	1214.34	0.5	
123500	1210.59	3.1	0.1	1213.8	1213.19	0.6	
123000	1209.60	3.1	0.1	1212.9	1212.04	0.8	
122500	1208.70	3.1	0.1	1212.0	1210.89	1.1	
122000	1207.96	3.2	0.1	1211.3	1209.74	1.5	
121500	1207.37	3.2	0.1	1210.7	1208.59	2.1	
121012	1203.93	3.5	0.3	1207.7	1207.47	0.2	
120612	1202.64	3.1	0.2	1205.9	1206.55		
120512	1200.71	3.7	0.3	1204.7	1206.32		
120488	1201.05	3.3	0.1	1204.4	1202.32	2.1	
120388	1200.92	3.3	0.1	1204.3	1202.11	2.2	
119988	1198.99	3.5	0.2	1202.7	1201.27	1.4	
119500	1197.75	3.4	0.2	1201.3	1200.25	1.0	
119000	1196.62	3.2	0.2	1200.0	1199.20	0.8	
118500	1195.78	3.1	0.1	1199.0	1198.15	0.9	
118000	1195.00	3.1	0.1	1198.2	1197.10	1.1	
117500	1194.21	3.2	0.1	1197.5	1196.05	1.5	
117000	1193.55	3.2	0.1	1196.9	1195.00	1.9	
116715	1193.23	3.2	0.1	1196.6	1194.40	2.2	
116465	1192.39	3.3	0.1	1195.9	1193.88	2.0	
116215	1189.91	3.5	0.3	1193.7	1193.35	0.3	
115815	1188.65	3.1	0.2	1191.9	1192.51		
115715	1186.69	3.7	0.3	1190.7	1192.30		
115685	1179.41	5.3	0.9	1185.5	1187.30		
115585	1184.28	3.1	0.2	1187.6	1187.09	0.5	
115185	1183.09	3.1	0.2	1186.4	1186.25	0.1	
115018	1180.52	3.3	0.3	1184.1	1185.90		
114982	1171.79	5.7	1.0	1178.5	1179.90		
114882	1177.96	3.2	0.1	1181.3	1179.69	1.6	
114482	1177.41	3.2	0.1	1180.8	1178.85	1.9	
114185	1175.92	3.4	0.2	1179.5	1178.23	1.3	

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
113935	1175.33	3.3	0.2	1178.9	1177.70	1.2	
113685	1174.74	3.3	0.2	1178.2	1177.18	1.0	
113435	1174.18	3.3	0.2	1177.6	1176.65	1.0	
113185	1173.66	3.2	0.2	1177.0	1176.13	0.9	
113000	1173.30	3.2	0.2	1176.6	1175.75	0.9	
112500	1172.42	3.1	0.1	1175.7	1174.70	1.0	
112400	1172.25	3.1	0.1	1175.5	1174.49	1.0	
112300	1172.07	3.1	0.1	1175.4	1174.28	1.1	
112200	1171.92	3.1	0.1	1175.2	1174.07	1.1	
112100	1171.76	3.1	0.1	1175.0	1173.86	1.2	
112000	1171.61	3.2	0.1	1174.9	1173.65	1.2	
111900	1171.47	3.2	0.1	1174.8	1173.44	1.3	
111800	1171.32	3.2	0.1	1174.6	1173.23	1.4	
111700	1171.18	3.2	0.1	1174.5	1173.02	1.5	
111600	1171.04	3.2	0.1	1174.3	1172.81	1.5	
111500	1170.91	3.2	0.1	1174.2	1172.60	1.6	
111043	1170.36	3.2	0.1	1173.7	1171.63	2.1	
110793	1169.62	3.3	0.1	1173.1	1171.11	2.0	
110543	1167.15	3.5	0.3	1170.9	1170.58	0.3	
110143	1165.90	3.1	0.2	1169.2	1169.74		
110043	1163.90	3.7	0.3	1167.9	1169.53		
110007	1162.79	3.3	0.1	1166.2	1163.53	2.7	
109907	1162.68	3.4	0.1	1166.1	1163.37	2.8	
109507	1161.05	3.5	0.2	1164.7	1162.73	2.0	
109257	1160.55	3.4	0.2	1164.2	1162.33	1.8	
109007	1160.05	3.4	0.2	1163.6	1161.93	1.7	
108757	1159.57	3.3	0.2	1163.1	1161.53	1.5	
108507	1159.09	3.3	0.2	1162.5	1161.13	1.4	
108257	1158.76	3.2	0.1	1162.1	1160.73	1.3	
108007	1158.42	3.1	0.1	1161.7	1160.33	1.4	
107507	1157.65	3.1	0.1	1160.9	1159.53	1.4	
107000	1156.90	3.2	0.1	1160.2	1158.72	1.5	
106500	1156.15	3.2	0.1	1159.4	1157.92	1.5	
106000	1155.36	3.2	0.1	1158.7	1157.12	1.5	
105500	1154.58	3.2	0.1	1157.9	1156.32	1.6	
105000	1153.81	3.2	0.1	1157.1	1155.52	1.6	
104500	1153.07	3.2	0.1	1156.4	1154.72	1.7	
104000	1152.35	3.2	0.1	1155.7	1153.92	1.7	
103500	1151.69	3.2	0.1	1155.0	1153.12	1.9	
103000	1151.08	3.2	0.1	1154.4	1152.32	2.1	
102500	1150.55	3.2	0.1	1153.9	1151.52	2.4	
102180	1149.95	3.3	0.1	1153.4	1151.01	2.4	
101930	1149.03	3.4	0.1	1152.5	1150.61	1.9	
101530	1148.50	3.2	0.1	1151.9	1149.97	1.9	
101430	1148.37	3.2	0.1	1151.7	1149.81	1.9	
101347					Bridge		
101344	1142.53	4.2	0.6	1147.4	1149.67		

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
101262	1143.79	3.6	0.3	1147.7	1149.54		
101238	1144.36	3.3	0.1	1147.8	1145.50	2.3	
101138	1144.22	3.3	0.1	1147.6	1145.34	2.3	
100738	1142.73	3.4	0.2	1146.3	1144.70	1.6	
100488	1142.28	3.3	0.2	1145.8	1144.30	1.5	
100238	1141.87	3.3	0.2	1145.3	1143.90	1.4	
99988	1141.49	3.2	0.1	1144.8	1143.50	1.3	
99738	1141.15	3.1	0.1	1144.4	1143.10	1.3	
99488	1140.77	3.1	0.1	1144.0	1142.70	1.3	
99000	1140.02	3.1	0.1	1143.3	1141.92	1.4	
98500	1139.27	3.1	0.1	1142.5	1141.12	1.4	
98000	1138.53	3.2	0.1	1141.8	1140.32	1.5	
97500	1137.88	3.1	0.1	1141.1	1139.52	1.6	
97000	1137.15	3.2	0.1	1140.5	1138.72	1.7	
96500	1136.45	3.2	0.1	1139.8	1137.92	1.8	
96000	1135.81	3.2	0.1	1139.1	1137.12	2.0	
95718	1135.47	3.2	0.1	1138.8	1136.67	2.1	
95468	1134.66	3.3	0.1	1138.1	1136.27	1.8	
95218	1132.69	3.5	0.3	1136.4	1135.87	0.5	
94818	1131.49	3.1	0.2	1134.8	1135.23		
94718	1129.45	3.7	0.3	1133.5	1135.07		
94682	1128.72	3.4	0.1	1132.2	1129.01	3.2	
94582	1128.62	3.4	0.1	1132.1	1128.85	3.3	
94182	1127.33	3.6	0.2	1131.0	1128.21	2.8	
93932	1126.99	3.5	0.1	1130.6	1127.81	2.8	
93682	1126.67	3.5	0.1	1130.3	1127.41	2.9	
93432	1126.39	3.5	0.1	1130.0	1127.01	3.0	
93182	1126.06	3.5	0.1	1129.7	1126.61	3.1	
93000	1125.82	3.5	0.1	1129.5	1126.32	3.1	
92500	1124.72	3.6	0.2	1128.4	1125.52	2.9	
92000	1124.15	3.4	0.1	1127.7	1126.72	1.0	
91600	1123.22	3.5	0.1	1126.9	1126.08	0.8	
91500	1123.07	3.5	0.1	1126.7	1125.92	0.8	
91000	1122.28	3.5	0.1	1125.9	1125.12	0.8	
90500	1121.50	3.5	0.1	1125.1	1124.32	0.8	
90000	1120.73	3.5	0.1	1124.4	1123.52	0.9	
89500	1119.96	3.5	0.1	1123.6	1122.72	0.9	
89000	1119.21	3.5	0.1	1122.9	1121.92	0.9	
88500	1118.06	3.6	0.2	1121.8	1121.00	0.8	
88356	1117.87	3.5	0.2	1121.6	1140.04		
88347					Bridge		
88331	1117.65	3.5	0.2	1121.3	1140.00		
88295	1117.22	3.6	0.2	1121.0	1131.10		
88174					Bridge		
88170	1116.49	3.5	0.2	1120.2	1130.79		
88150	1116.43	3.5	0.2	1120.2	1120.56		
88000	1115.61	3.6	0.2	1119.4	1120.28		

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
87500	1114.12	3.3	0.2	1117.6	1119.35		
87000	1113.09	3.0	0.2	1116.2	1116.43		
86500	1112.20	2.8	0.1	1115.2	1115.50		
86000	1111.50	2.8	0.1	1114.4	1114.57		
85500	1110.79	2.8	0.1	1113.7	1113.64	0.1	
85000	1110.15	2.8	0.1	1113.1	1112.71	0.4	
84500	1109.47	2.9	0.1	1112.5	1111.91	0.5	
84000	1108.11	3.0	0.1	1111.3	1111.11	0.1	
83512	1104.64	3.1	0.3	1108.1	1110.33		
83488	1106.15	3.4	0.1	1109.6	1106.29	3.3	
83380	1106.10	3.4	0.1	1109.6	1106.12	3.4	
83280	1104.54	3.7	0.2	1108.5	1105.96	2.5	
83000	1104.23	3.5	0.2	1107.9	1105.51	2.4	
82500	1103.37	3.4	0.2	1107.0	1104.71	2.3	
82000	1102.55	3.4	0.1	1106.1	1103.91	2.2	
81500	1101.80	3.4	0.1	1105.3	1103.11	2.2	
81000	1100.99	3.4	0.1	1104.5	1102.31	2.2	
80592	1100.37	3.3	0.1	1103.8	1101.66	2.2	
80462	1100.15	3.3	0.1	1103.6	1101.45	2.2	
80212	1097.86	3.5	0.3	1101.6	1101.05	0.5	
79812	1096.66	3.1	0.2	1099.9	1100.41		
79712	1094.64	3.7	0.3	1098.7	1100.25		
79688	1095.62	3.4	0.1	1099.1	1096.21	2.9	
79588	1095.51	3.4	0.1	1099.0	1096.05	2.9	
79188	1094.02	3.5	0.2	1097.7	1095.41	2.3	
78938	1093.58	3.5	0.2	1097.2	1095.01	2.2	
78688	1093.18	3.4	0.2	1096.7	1094.61	2.1	
78438	1092.80	3.4	0.1	1096.3	1094.21	2.1	
78188	1092.45	3.3	0.1	1095.9	1093.81	2.1	
77938	1092.10	3.3	0.1	1095.5	1093.41	2.1	
77688	1091.74	3.3	0.1	1095.2	1093.01	2.2	
77500	1091.47	3.3	0.1	1094.9	1092.71	2.2	
77000	1090.78	3.3	0.1	1094.2	1091.91	2.3	
76872	1090.61	3.3	0.1	1094.0	1091.71	2.3	
76512	1087.78	3.5	0.3	1091.5	1091.13	0.4	
76112	1086.35	3.1	0.2	1089.7	1090.49		
76012	1085.43	3.6	0.3	1089.3	1090.33		
75988	1086.22	3.5	0.1	1089.8	1086.29	3.5	
75888	1086.13	3.5	0.1	1089.7	1086.13	3.6	
75488	1084.95	3.6	0.1	1088.7	1085.49	3.2	
75238	1084.61	3.6	0.1	1088.3	1085.09	3.2	
74988	1084.27	3.6	0.1	1088.0	1084.69	3.3	
74738	1084.05	3.5	0.1	1087.7	1084.29	3.4	
74488	1083.42	3.6	0.1	1087.1	1083.89	3.2	
74238	1083.05	3.6	0.1	1086.8	1083.49	3.3	
74000	1082.48	3.6	0.1	1086.2	1083.11	3.1	
73500	1081.66	3.6	0.1	1085.4	1082.31	3.1	

Table A-3. Channel Capacity Analysis
SPF Low N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
73000	1081.05	3.5	0.1	1084.7	1081.51	3.2	
72892	1080.41	3.6	0.2	1084.1	1081.34	2.8	
72692	1080.25	3.4	0.1	1083.8	1081.02	2.8	
72492	1080.11	3.4	0.1	1083.6	1080.70	2.9	
72392	1079.87	3.4	0.1	1083.4	1080.54	2.8	
72352					Bridge		
72312	1073.53	4.9	0.6	1079.1	1080.41		
72288	1068.42	5.9	1.0	1075.3	1076.37		
72188	1074.73	3.3	0.1	1078.1	1076.22	1.9	
71988	1074.44	3.3	0.1	1077.8	1075.92	1.9	
71788	1073.75	3.3	0.2	1077.2	1075.62	1.6	
71500	1073.36	3.2	0.1	1076.7	1075.19	1.5	
71000	1072.69	3.2	0.1	1076.0	1074.44	1.5	
70500	1072.09	3.1	0.1	1075.3	1073.69	1.6	
70000	1071.54	3.1	0.1	1074.8	1072.94	1.8	
69500	1070.91	3.2	0.1	1074.2	1072.19	2.0	
69000	1070.32	3.2	0.1	1073.7	1071.44	2.2	
68500	1069.76	3.3	0.1	1073.1	1070.69	2.4	
68000	1069.23	3.3	0.1	1072.6	1069.94	2.7	
67512	1066.44	3.5	0.2	1070.1	1069.21	0.9	
67312	1065.99	3.2	0.2	1069.4	1068.91	0.5	
67112	1064.14	3.3	0.3	1067.7	1068.61		
67012	1063.83	3.6	0.2	1067.6	1068.46		
66988	1064.52	3.5	0.1	1068.1	1064.42	3.7	0.1
66888	1064.43	3.5	0.1	1068.0	1064.27	3.8	0.16
66688	1062.31	3.7	0.2	1066.3	1063.97	2.3	
66488	1062.07	3.5	0.2	1065.7	1063.67	2.1	
66000	1061.06	3.4	0.2	1064.6	1062.94	1.7	
65500	1060.30	3.2	0.1	1063.7	1062.19	1.5	
65000	1059.79	3.1	0.1	1063.0	1061.44	1.5	
64500	1059.44	3.1	0.1	1062.6	1060.69	1.9	
64000	1058.65	3.2	0.1	1062.0	1059.94	2.0	
63500	1058.04	3.2	0.1	1061.4	1059.19	2.2	
63000	1057.48	3.3	0.1	1060.8	1058.43	2.4	
62500	1056.63	3.3	0.1	1060.1	1057.68	2.4	
62187	1054.16	3.5	0.2	1057.9	1057.21	0.7	
61987	1053.62	3.2	0.2	1057.1	1056.91	0.2	
61787	1053.08	3.1	0.2	1056.4	1056.61		
61687	1052.23	3.1	0.2	1055.6	1056.46		
61637					Bridge		
61591.5	1050.44	3.8	0.4	1054.6	1056.32		
61558.5	1050.34	3.4	0.1	1053.9	1050.78	3.1	
61458.5	1050.23	3.4	0.1	1053.8	1050.63	3.1	
61258.5	1049.72	3.5	0.1	1053.3	1050.33	3.0	
61058.5	1048.89	3.5	0.2	1052.6	1050.03	2.6	
61000	1048.80	3.5	0.2	1052.5	1049.94	2.5	
60500	1048.05	3.4	0.1	1051.6	1049.19	2.4	

Table A-3. Channel Capacity Analysis							
SPF Low N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
60000	1047.40	3.4	0.1	1050.9	1048.44	2.4	
59500	1046.85	3.3	0.1	1050.3	1047.69	2.6	
59018	1043.83	3.5	0.2	1047.5	1046.97	0.6	
58818	1043.25	3.2	0.2	1046.7	1046.67	0.0	
58618	1042.66	3.1	0.2	1045.9	1046.37		
58518	1040.59	3.7	0.3	1044.6	1046.22		
58482	1039.58	3.4	0.1	1043.1	1040.16	2.9	
58382	1039.47	3.4	0.1	1043.0	1040.01	3.0	
58182	1038.92	3.4	0.1	1042.5	1039.71	2.8	
57982	1038.02	3.5	0.2	1041.7	1039.41	2.3	
57500	1037.35	3.4	0.1	1040.8	1038.69	2.2	
57000	1036.85	3.2	0.1	1040.2	1037.94	2.3	
56500	1036.29	3.3	0.1	1039.7	1037.19	2.5	
56318	1033.79	3.5	0.2	1037.5	1036.92	0.6	
56118	1033.20	3.2	0.2	1036.7	1036.62	0.0	
55918	1032.61	3.1	0.2	1035.9	1036.32		
55818	1030.54	3.7	0.3	1034.6	1036.17		
55782	1029.49	3.4	0.1	1033.0	1030.11	2.9	
55682	1029.37	3.4	0.1	1032.9	1029.96	2.9	
55482	1028.82	3.4	0.1	1032.4	1029.66	2.7	
55282	1027.89	3.5	0.2	1031.6	1029.36	2.2	
55000	1027.35	3.4	0.2	1031.0	1028.94	2.0	
54500	1026.48	3.3	0.2	1030.0	1028.19	1.8	
54000	1025.99	3.1	0.1	1029.2	1027.44	1.8	
53500	1025.71	3.1	0.1	1028.8	1026.69	2.2	
53000	1025.50	3.1	0.0	1028.7	1025.94	2.7	
52500	1022.21	3.8	0.3	1026.3	1025.19	1.1	
51500	1021.67	2.9	0.1	1024.6	1023.68	0.9	
51000	1020.72	3.0	0.1	1023.8	1022.93	0.9	
50500	1019.34	3.1	0.2	1022.6	1022.18	0.4	
50000	1017.79	3.0	0.2	1021.0	1021.43		
49930					Bridge		
49884.8	1015.58	3.7	0.3	1019.6	1021.26		
49860.68	1016.70	3.4	0.1	1020.2	1021.26		
49709.6	1016.58	3.3	0.1	1020.0	1017.14	2.9	
49500	1016.41	3.3	0.1	1019.8	1016.95	2.9	
49000	1015.98	3.3	0.1	1019.4	1016.50	2.9	
48500	1015.54	3.3	0.1	1018.9	1016.05	2.9	
48000	1014.93	3.3	0.1	1018.4	1015.60	2.8	
47500	1014.06	3.4	0.1	1017.5	1015.15	2.4	
47000	1013.57	3.2	0.1	1016.9	1014.70	2.2	
46500	1013.10	3.2	0.1	1016.3	1014.25	2.1	
46000	1012.65	3.1	0.1	1015.9	1013.80	2.1	
45500	1012.18	3.1	0.1	1015.4	1013.35	2.0	
45000	1011.70	3.1	0.1	1014.9	1012.90	2.0	

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
157000	1319.23	8.4	1.5	1329.2	1328.55	0.6	
156500	1326.85	3.3	0.1	1330.2	1327.35	2.9	
156000	1325.68	3.4	0.1	1329.1	1326.15	3.0	
155500	1324.52	3.4	0.1	1328.0	1324.95	3.0	
155000	1323.37	3.4	0.1	1326.8	1323.75	3.1	
154500	1322.29	3.4	0.1	1325.8	1322.55	3.2	
154000	1321.26	3.4	0.1	1324.8	1321.35	3.4	
153500	1320.29	3.5	0.1	1323.8	1320.15	3.7	0.14
153281	1319.68	3.5	0.1	1323.3	1319.63	3.6	0.05
153031	1317.97	3.5	0.2	1321.7	1319.03	2.6	
152631	1316.71	3.3	0.1	1320.1	1318.07	2.0	
152012	1312.30	3.5	0.2	1316.1	1316.58		
151988	1312.88	3.5	0.1	1316.5	1312.58	3.9	0.3
151838	1312.59	3.5	0.1	1316.2	1312.22	4.0	0.37
151438	1310.85	3.6	0.1	1314.6	1311.26	3.3	
151188	1310.13	3.5	0.1	1313.8	1310.66	3.1	
150938	1309.52	3.4	0.1	1313.0	1310.06	3.0	
150688	1308.94	3.4	0.1	1312.4	1309.46	2.9	
150438	1308.32	3.4	0.1	1311.8	1308.86	2.9	
150000	1307.28	3.4	0.1	1310.7	1307.80	2.9	
149500	1306.08	3.4	0.1	1309.5	1306.60	2.9	
149000	1304.89	3.4	0.1	1308.3	1305.40	2.9	
148500	1303.71	3.4	0.1	1307.2	1304.20	3.0	
148000	1302.54	3.4	0.1	1306.0	1303.00	3.0	
147500	1301.37	3.4	0.1	1304.8	1301.80	3.0	
147000	1300.22	3.4	0.1	1303.7	1300.60	3.1	
146500	1299.10	3.4	0.1	1302.6	1299.40	3.2	
146018	1298.06	3.4	0.1	1301.6	1298.25	3.3	
145918	1297.85	3.4	0.1	1301.4	1298.01	3.3	
145768	1297.31	3.4	0.1	1300.9	1297.65	3.2	
145518	1295.23	3.5	0.2	1298.9	1297.05	1.8	
145118	1293.14	3.1	0.2	1296.4	1296.09	0.3	
145018	1290.22	3.7	0.3	1294.3	1295.85		
144982	1290.31	3.5	0.1	1293.9	1289.85	4.1	0.46
144882	1290.13	3.6	0.1	1293.8	1289.61	4.2	0.52
144482	1288.50	3.6	0.1	1292.3	1288.65	3.6	
144232	1287.77	3.6	0.1	1291.5	1288.05	3.4	
143982	1287.06	3.5	0.1	1290.7	1287.45	3.3	
143732	1286.38	3.5	0.1	1290.0	1286.85	3.1	
143482	1285.72	3.4	0.1	1289.3	1286.25	3.0	
143232	1285.12	3.4	0.1	1288.6	1285.65	3.0	
142982	1284.57	3.4	0.1	1288.0	1285.05	3.0	
142732	1284.06	3.4	0.1	1287.5	1284.45	3.1	
142482	1283.58	3.4	0.1	1287.0	1283.85	3.2	
142000	1282.48	3.4	0.1	1286.0	1282.70	3.3	
141915	1282.30	3.4	0.1	1285.8	1282.49	3.3	

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
141815	1282.09	3.4	0.1	1285.6	1282.25	3.3	
141665	1281.54	3.4	0.1	1285.1	1281.89	3.2	
141415	1279.47	3.5	0.2	1283.1	1281.29	1.8	
141015	1277.37	3.1	0.2	1280.6	1280.33	0.3	
140915	1274.48	3.7	0.3	1278.5	1280.09		
140885	1274.95	3.4	0.1	1278.5	1275.09	3.4	
140785	1274.73	3.4	0.1	1278.3	1274.85	3.4	
140385	1273.90	3.5	0.1	1277.5	1273.89	3.6	0.01
140135	1273.36	3.5	0.1	1276.9	1273.29	3.7	0.07
139885	1272.26	3.5	0.1	1275.9	1272.69	3.2	
139635	1271.57	3.5	0.1	1275.2	1272.09	3.1	
139385	1270.90	3.4	0.1	1274.4	1271.49	2.9	
139135	1270.28	3.4	0.1	1273.8	1270.89	2.9	
138885	1269.70	3.4	0.1	1273.2	1270.29	2.9	
138635	1269.09	3.3	0.1	1272.5	1269.69	2.8	
138385	1268.49	3.4	0.1	1271.9	1269.09	2.9	
138000	1267.53	3.3	0.1	1271.0	1268.16	2.8	
137500	1266.38	3.3	0.1	1269.8	1266.96	2.8	
137000	1265.36	3.3	0.1	1268.8	1265.76	3.0	
136923	1265.22	3.3	0.1	1268.6	1265.58	3.1	
136500	1263.23	3.4	0.1	1266.8	1264.56	2.2	
136427	1262.53	3.4	0.2	1266.1	1264.39	1.7	
136027	1260.47	3.1	0.2	1263.7	1263.43	0.3	
135927	1257.58	3.7	0.3	1261.6	1263.19		
135873	1254.33	3.5	0.1	1257.9	1254.19	3.7	0.14
135773	1254.12	3.5	0.1	1257.7	1253.95	3.8	0.17
135373	1252.22	3.6	0.2	1255.9	1252.99	3.0	
135123	1251.26	3.5	0.2	1254.9	1252.39	2.5	
134873	1250.76	3.2	0.1	1254.1	1251.79	2.3	
134623	1250.30	3.2	0.1	1253.6	1251.19	2.4	
134373	1249.72	3.2	0.1	1253.1	1250.59	2.5	
134123	1249.13	3.3	0.1	1252.5	1249.99	2.5	
134000	1248.85	3.3	0.1	1252.2	1249.70	2.5	
133500	1247.71	3.3	0.1	1251.1	1248.50	2.6	
133000	1246.62	3.3	0.1	1250.0	1247.30	2.7	
132500	1245.58	3.3	0.1	1249.0	1246.10	2.9	
132126	1244.85	3.3	0.1	1248.3	1245.20	3.1	
131876	1244.39	3.4	0.1	1247.8	1244.60	3.2	
131626	1242.44	3.4	0.2	1246.0	1244.00	2.0	
131226	1240.90	3.1	0.1	1244.2	1243.04	1.1	
131126	1240.32	3.1	0.1	1243.6	1242.80	0.8	
131055					Bridge		
130984	1235.58	3.9	0.5	1240.0	1242.45		
130924	1236.55	3.7	0.3	1240.5	1242.31		
130876	1234.43	3.5	0.1	1238.0	1234.31	3.7	0.12
130664	1234.01	3.5	0.1	1237.6	1233.82	3.8	0.19

Table A-4. Channel Capacity Analysis
SPF High N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
130264	1232.39	3.6	0.1	1236.1	1232.88	3.2	
130014	1231.81	3.5	0.1	1235.4	1232.30	3.1	
129764	1231.20	3.4	0.1	1234.7	1231.71	3.0	
129500	1230.55	3.4	0.1	1234.0	1231.10	2.9	
129000	1229.32	3.3	0.1	1232.8	1229.93	2.8	
128500	1228.14	3.3	0.1	1231.6	1228.76	2.8	
128000	1227.02	3.3	0.1	1230.4	1227.59	2.8	
127500	1225.94	3.3	0.1	1229.4	1226.42	2.9	
127000	1224.97	3.4	0.1	1228.4	1225.25	3.2	
126500	1224.07	3.4	0.1	1227.5	1224.08	3.5	
126012	1221.03	3.5	0.2	1224.7	1222.93	1.8	
125612	1218.71	3.1	0.2	1222.0	1222.00		
125512	1217.05	3.6	0.2	1220.9	1221.76		
125488	1217.72	3.5	0.1	1221.3	1217.76	3.5	
125388	1217.50	3.5	0.1	1221.1	1217.53	3.5	
124988	1215.98	3.5	0.1	1219.6	1216.61	3.0	
124500	1214.75	3.4	0.1	1218.2	1215.49	2.7	
124000	1213.66	3.3	0.1	1217.0	1214.34	2.7	
123500	1212.60	3.3	0.1	1216.0	1213.19	2.8	
123000	1211.56	3.3	0.1	1215.0	1212.04	2.9	
122500	1210.51	3.3	0.1	1213.9	1210.89	3.0	
122000	1209.53	3.4	0.1	1213.0	1209.74	3.2	
121500	1208.63	3.4	0.1	1212.1	1208.59	3.5	0.04
121012	1205.64	3.5	0.2	1209.3	1207.47	1.8	
120612	1203.47	3.1	0.2	1206.7	1206.55	0.2	
120512	1202.57	3.5	0.2	1206.3	1206.32		
120488	1203.06	3.6	0.1	1206.7	1202.32	4.4	0.74
120388	1202.89	3.6	0.1	1206.6	1202.11	4.5	0.78
119988	1201.34	3.7	0.1	1205.1	1201.27	3.9	0.07
119500	1200.00	3.5	0.1	1203.7	1200.25	3.4	
119000	1198.77	3.4	0.1	1202.3	1199.20	3.1	
118500	1197.81	3.3	0.1	1201.2	1198.15	3.1	
118000	1196.91	3.3	0.1	1200.3	1197.10	3.2	
117500	1196.01	3.4	0.1	1199.5	1196.05	3.4	
117000	1195.04	3.4	0.1	1198.5	1195.00	3.5	0.04
116715	1194.57	3.4	0.1	1198.1	1194.40	3.7	0.17
116465	1193.65	3.5	0.1	1197.2	1193.88	3.3	
116215	1191.64	3.5	0.2	1195.3	1193.35	2.0	
115815	1189.59	3.1	0.2	1192.9	1192.51	0.4	
115715	1186.69	3.7	0.3	1190.7	1192.30		
115685	1186.20	3.3	0.1	1189.6	1187.30	2.3	
115585	1185.85	3.3	0.1	1189.2	1187.09	2.2	
115185	1184.09	3.2	0.1	1187.4	1186.25	1.1	
115018	1180.52	3.3	0.3	1184.1	1185.90		
114982	1180.24	3.5	0.1	1183.9	1179.90	4.0	0.34
114882	1180.05	3.5	0.1	1183.7	1179.69	4.0	0.36

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
114482	1179.32	3.5	0.1	1182.9	1178.85	4.1	0.47
114185	1178.21	3.6	0.1	1181.9	1178.23	3.7	
113935	1177.56	3.5	0.1	1181.2	1177.70	3.5	
113685	1176.93	3.5	0.1	1180.5	1177.18	3.4	
113435	1176.33	3.4	0.1	1179.9	1176.65	3.2	
113185	1175.77	3.4	0.1	1179.3	1176.13	3.1	
113000	1175.37	3.4	0.1	1178.9	1175.75	3.1	
112500	1174.36	3.3	0.1	1177.8	1174.70	3.1	
112400	1174.17	3.3	0.1	1177.6	1174.49	3.1	
112300	1173.96	3.3	0.1	1177.4	1174.28	3.1	
112200	1173.79	3.4	0.1	1177.2	1174.07	3.2	
112100	1173.61	3.4	0.1	1177.1	1173.86	3.2	
112000	1173.43	3.4	0.1	1176.9	1173.65	3.2	
111900	1173.25	3.4	0.1	1176.7	1173.44	3.3	
111800	1173.07	3.4	0.1	1176.5	1173.23	3.3	
111700	1172.89	3.4	0.1	1176.3	1173.02	3.3	
111600	1172.71	3.4	0.1	1176.2	1172.81	3.4	
111500	1172.53	3.4	0.1	1176.0	1172.60	3.4	
111043	1171.73	3.4	0.1	1175.2	1171.63	3.6	0.1
110793	1170.89	3.5	0.1	1174.5	1171.11	3.4	
110543	1168.88	3.5	0.2	1172.5	1170.58	2.0	
110143	1166.84	3.1	0.2	1170.1	1169.74	0.4	
110043	1163.90	3.7	0.3	1167.9	1169.53		
110007	1165.04	3.7	0.1	1168.8	1163.53	5.3	1.51
109907	1164.91	3.7	0.1	1168.7	1163.37	5.3	1.54
109507	1163.60	3.8	0.1	1167.5	1162.73	4.8	0.87
109257	1163.07	3.7	0.1	1166.9	1162.33	4.6	0.74
109007	1162.56	3.7	0.1	1166.3	1161.93	4.4	0.63
108757	1162.06	3.6	0.1	1165.8	1161.53	4.2	0.53
108507	1161.59	3.6	0.1	1165.3	1161.13	4.1	0.46
108257	1161.21	3.5	0.1	1164.8	1160.73	4.1	0.48
108007	1160.83	3.5	0.1	1164.4	1160.33	4.1	0.5
107507	1160.03	3.5	0.1	1163.6	1159.53	4.1	0.5
107000	1159.24	3.5	0.1	1162.8	1158.72	4.1	0.52
106500	1158.48	3.5	0.1	1162.1	1157.92	4.1	0.56
106000	1157.66	3.5	0.1	1161.2	1157.12	4.1	0.54
105500	1156.84	3.5	0.1	1160.4	1156.32	4.1	0.52
105000	1156.00	3.5	0.1	1159.6	1155.52	4.1	0.48
104500	1155.18	3.5	0.1	1158.7	1154.72	4.0	0.46
104000	1154.34	3.5	0.1	1157.9	1153.92	4.0	0.42
103500	1153.51	3.5	0.1	1157.1	1153.12	3.9	0.39
103000	1152.68	3.5	0.1	1156.2	1152.32	3.9	0.36
102500	1151.86	3.5	0.1	1155.4	1151.52	3.9	0.34
102180	1151.03	3.5	0.1	1154.6	1151.01	3.6	0.02
101930	1149.91	3.4	0.1	1153.5	1150.61	2.9	
101530	1148.73	3.3	0.1	1152.1	1149.97	2.1	

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
101430	1148.37	3.2	0.1	1151.7	1149.81	1.9	
101347					Bridge		
101344	1142.53	4.2	0.6	1147.4	1149.67		
101262	1146.30	3.5	0.2	1149.9	1149.54	0.4	
101238	1146.66	3.7	0.1	1150.4	1145.50	4.9	1.16
101138	1146.51	3.7	0.1	1150.3	1145.34	4.9	1.17
100738	1145.27	3.7	0.1	1149.1	1144.70	4.4	0.57
100488	1144.78	3.6	0.1	1148.5	1144.30	4.2	0.48
100238	1144.33	3.6	0.1	1148.0	1143.90	4.1	0.43
99988	1143.93	3.5	0.1	1147.5	1143.50	4.0	0.43
99738	1143.56	3.5	0.1	1147.1	1143.10	4.0	0.46
99488	1143.15	3.5	0.1	1146.7	1142.70	4.0	0.45
99000	1142.36	3.5	0.1	1145.9	1141.92	4.0	0.44
98500	1141.54	3.5	0.1	1145.1	1141.12	4.0	0.42
98000	1140.72	3.5	0.1	1144.3	1140.32	4.0	0.4
97500	1139.96	3.5	0.1	1143.5	1139.52	4.0	0.44
97000	1139.12	3.5	0.1	1142.7	1138.72	4.0	0.4
96500	1138.25	3.5	0.1	1141.8	1137.92	3.9	0.33
96000	1137.36	3.5	0.1	1140.9	1137.12	3.8	0.24
95718	1136.89	3.4	0.1	1140.4	1136.67	3.7	0.22
95468	1136.04	3.5	0.1	1139.6	1136.27	3.3	
95218	1134.25	3.5	0.2	1137.9	1135.87	2.1	
94818	1132.07	3.1	0.2	1135.3	1135.23	0.1	
94718	1130.37	3.6	0.2	1134.2	1135.07		
94682	1131.22	3.9	0.1	1135.1	1129.01	6.1	2.21
94582	1131.11	3.9	0.1	1135.0	1128.85	6.2	2.26
94182	1130.05	3.9	0.1	1134.1	1128.21	5.9	1.84
93932	1129.66	3.9	0.1	1133.7	1127.81	5.8	1.85
93682	1129.31	3.9	0.1	1133.3	1127.41	5.9	1.9
93432	1128.99	3.9	0.1	1132.9	1127.01	5.9	1.98
93182	1128.65	3.9	0.1	1132.6	1126.61	6.0	2.04
93000	1128.41	3.9	0.1	1132.4	1126.32	6.1	2.09
92500	1127.49	3.9	0.1	1131.5	1125.52	6.0	1.97
92000	1126.83	3.9	0.1	1130.8	1126.72	4.1	0.11
91600	1126.00	3.9	0.1	1130.0	1126.08	3.9	
91500	1125.83	3.9	0.1	1129.8	1125.92	3.9	
91000	1124.98	3.9	0.1	1129.0	1125.12	3.9	
90500	1124.12	3.9	0.1	1128.1	1124.32	3.8	
90000	1123.24	3.9	0.1	1127.2	1123.52	3.7	
89500	1122.33	3.9	0.1	1126.3	1122.72	3.6	
89000	1121.40	3.8	0.1	1125.3	1121.92	3.4	
88500	1120.13	3.8	0.1	1124.1	1121.00	3.1	
88356	1119.83	3.8	0.1	1123.7	1140.04		
88347					Bridge		
88331	1119.66	3.8	0.1	1123.5	1140.00		
88295	1119.38	3.8	0.1	1123.3	1131.10		

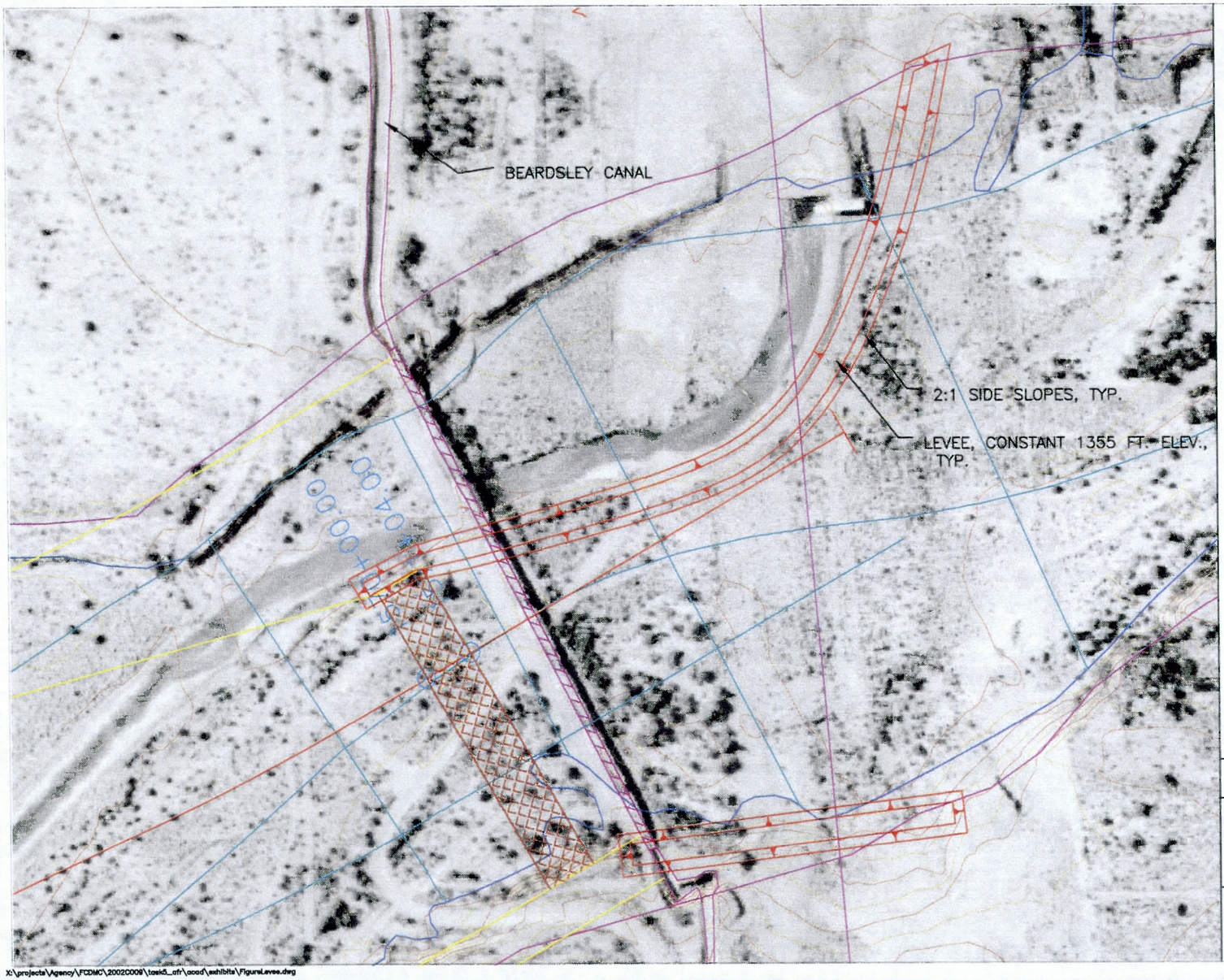
Table A-4. Channel Capacity Analysis
SPF High N HEC-RAS Model

River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
88174					Bridge		
88170	1118.76	3.7	0.1	1122.6	1130.79		
88150	1118.71	3.7	0.1	1122.6	1120.56	2.0	
88000	1118.11	3.7	0.2	1122.0	1120.28	1.7	
87500	1116.62	3.5	0.1	1120.2	1119.35	0.9	
87000	1115.46	3.3	0.1	1118.8	1116.43	2.4	
86500	1114.46	3.2	0.1	1117.7	1115.50	2.2	
86000	1113.62	3.1	0.1	1116.8	1114.57	2.2	
85500	1112.80	3.1	0.1	1116.0	1113.64	2.4	
85000	1112.00	3.2	0.1	1115.2	1112.71	2.5	
84500	1111.13	3.2	0.1	1114.4	1111.91	2.5	
84000	1109.68	3.2	0.1	1113.0	1111.11	1.9	
83512	1107.81	3.0	0.1	1111.0	1110.33	0.6	
83488	1108.27	3.8	0.1	1112.1	1106.29	5.8	1.98
83380	1108.19	3.8	0.1	1112.0	1106.12	5.9	2.07
83280	1107.22	4.0	0.1	1111.3	1105.96	5.4	1.26
83000	1106.76	3.8	0.1	1110.7	1105.51	5.2	1.25
82500	1105.76	3.8	0.1	1109.6	1104.71	4.9	1.05
82000	1104.78	3.7	0.1	1108.6	1103.91	4.7	0.87
81500	1103.83	3.6	0.1	1107.6	1103.11	4.5	0.72
81000	1102.75	3.6	0.1	1106.4	1102.31	4.1	0.44
80592	1101.82	3.5	0.1	1105.5	1101.66	3.8	0.162
80462	1101.54	3.5	0.1	1105.2	1101.45	3.7	0.09
80212	1099.72	3.5	0.2	1103.4	1101.05	2.4	
79812	1097.99	3.1	0.1	1101.3	1100.41	0.9	
79712	1097.43	3.6	0.2	1101.2	1100.25	0.9	
79688	1097.81	3.7	0.1	1101.6	1096.21	5.4	1.6
79588	1097.68	3.8	0.1	1101.5	1096.05	5.5	1.63
79188	1096.43	3.8	0.1	1100.4	1095.41	4.9	1.02
78938	1095.93	3.8	0.1	1099.8	1095.01	4.8	0.92
78688	1095.45	3.7	0.1	1099.2	1094.61	4.6	0.84
78438	1095.00	3.6	0.1	1098.7	1094.21	4.5	0.79
78188	1094.58	3.6	0.1	1098.3	1093.81	4.5	0.77
77938	1094.16	3.6	0.1	1097.8	1093.41	4.4	0.75
77688	1093.69	3.6	0.1	1097.4	1093.01	4.4	0.68
77500	1093.33	3.6	0.1	1097.0	1092.71	4.3	0.62
77000	1092.36	3.6	0.1	1096.0	1091.91	4.1	0.45
76872	1092.15	3.5	0.1	1095.8	1091.71	4.1	0.44
76512	1090.06	3.5	0.2	1093.8	1091.13	2.6	
76112	1088.58	3.2	0.1	1091.9	1090.49	1.4	
76012	1088.17	3.6	0.1	1091.9	1090.33	1.6	
75988	1088.50	3.9	0.1	1092.4	1086.29	6.1	2.21
75888	1088.39	3.9	0.1	1092.3	1086.13	6.2	2.26
75488	1087.35	3.9	0.1	1091.4	1085.49	5.9	1.86
75238	1086.92	3.9	0.1	1090.9	1085.09	5.8	1.83
74988	1086.50	3.9	0.1	1090.5	1084.69	5.8	1.81

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
74738	1086.19	3.9	0.1	1090.1	1084.29	5.8	1.9
74488	1085.56	3.9	0.1	1089.5	1083.89	5.6	1.67
74238	1085.08	3.9	0.1	1089.0	1083.49	5.6	1.59
74000	1084.44	3.9	0.1	1088.4	1083.11	5.3	1.33
73500	1083.25	3.8	0.1	1087.2	1082.31	4.9	0.94
73000	1082.21	3.7	0.1	1086.0	1081.51	4.5	0.7
72892	1081.52	3.7	0.1	1085.3	1081.34	4.0	0.18
72692	1081.09	3.6	0.1	1084.8	1081.02	3.7	0.07
72492	1080.70	3.5	0.1	1084.3	1080.70	3.6	
72392	1080.34	3.5	0.1	1083.9	1080.54	3.4	
72352					Bridge		
72312	1073.54	4.9	0.6	1079.1	1080.41		
72288	1077.21	3.7	0.1	1081.0	1076.37	4.6	0.84
72188	1077.05	3.6	0.1	1080.8	1076.22	4.5	0.83
71988	1076.76	3.6	0.1	1080.4	1075.92	4.5	0.84
71788	1076.07	3.6	0.1	1079.7	1075.62	4.1	0.45
71500	1075.62	3.5	0.1	1079.2	1075.19	4.0	0.43
71000	1074.87	3.5	0.1	1078.4	1074.44	4.0	0.43
70500	1074.20	3.5	0.1	1077.7	1073.69	4.0	0.51
70000	1073.55	3.5	0.1	1077.1	1072.94	4.1	0.61
69500	1072.85	3.5	0.1	1076.4	1072.19	4.2	0.66
69000	1072.15	3.5	0.1	1075.7	1071.44	4.3	0.71
68500	1071.41	3.5	0.1	1075.0	1070.69	4.3	0.72
68000	1070.66	3.5	0.1	1074.2	1069.94	4.3	0.72
67512	1068.20	3.5	0.2	1071.9	1069.21	2.7	
67312	1067.49	3.4	0.1	1071.0	1068.91	2.1	
67112	1066.40	3.2	0.2	1069.8	1068.61	1.2	
67012	1066.01	3.6	0.1	1069.8	1068.46	1.3	
66988	1066.32	3.8	0.1	1070.2	1064.42	5.8	1.9
66888	1066.19	3.8	0.1	1070.1	1064.27	5.8	1.92
66688	1064.79	3.9	0.1	1068.9	1063.97	4.9	0.82
66488	1064.37	3.8	0.1	1068.3	1063.67	4.6	0.7
66000	1063.21	3.6	0.1	1066.9	1062.94	4.0	0.27
65500	1062.47	3.6	0.1	1066.2	1062.19	4.0	0.28
65000	1061.83	3.4	0.1	1065.3	1061.44	3.9	0.39
64500	1061.32	3.4	0.0	1064.8	1060.69	4.1	0.63
64000	1060.56	3.5	0.1	1064.1	1059.94	4.2	0.62
63500	1059.82	3.5	0.1	1063.4	1059.19	4.2	0.63
63000	1059.07	3.5	0.1	1062.6	1058.43	4.2	0.64
62500	1058.02	3.5	0.1	1061.6	1057.68	3.9	0.34
62187	1055.97	3.5	0.2	1059.7	1057.21	2.4	
61987	1055.17	3.3	0.2	1058.6	1056.91	1.7	
61787	1054.34	3.2	0.1	1057.6	1056.61	1.0	
61687	1053.58	3.1	0.2	1056.9	1056.46	0.4	
61637					Bridge		
61591.5	1051.67	3.6	0.2	1055.5	1056.32		

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
61558.5	1052.42	3.8	0.1	1056.3	1050.78	5.5	1.64
61458.5	1052.27	3.8	0.1	1056.1	1050.63	5.5	1.64
61258.5	1051.78	3.8	0.1	1055.7	1050.33	5.3	1.45
61058.5	1051.07	3.8	0.1	1055.0	1050.03	5.0	1.04
61000	1050.94	3.8	0.1	1054.9	1049.94	4.9	1
60500	1049.95	3.7	0.1	1053.7	1049.19	4.5	0.76
60000	1049.03	3.6	0.1	1052.7	1048.44	4.3	0.59
59500	1048.20	3.5	0.1	1051.8	1047.69	4.1	0.51
59018	1045.47	3.5	0.2	1049.1	1046.97	2.2	
58818	1044.57	3.3	0.2	1048.0	1046.67	1.4	
58618	1043.55	3.1	0.2	1046.8	1046.37	0.5	
58518	1040.59	3.7	0.3	1044.6	1046.22		
58482	1041.51	3.7	0.1	1045.3	1040.16	5.2	1.35
58382	1041.36	3.7	0.1	1045.2	1040.01	5.2	1.35
58182	1040.81	3.7	0.1	1044.6	1039.71	4.9	1.1
57982	1040.02	3.8	0.1	1043.9	1039.41	4.5	0.61
57500	1039.04	3.6	0.1	1042.7	1038.69	4.0	0.35
57000	1038.23	3.4	0.1	1041.8	1037.94	3.8	0.29
56500	1037.38	3.4	0.1	1040.9	1037.19	3.7	0.19
56318	1035.42	3.5	0.2	1039.1	1036.92	2.2	
56118	1034.52	3.3	0.2	1038.0	1036.62	1.4	
55918	1033.50	3.1	0.2	1036.8	1036.32	0.5	
55818	1030.54	3.7	0.3	1034.6	1036.17		
55782	1031.62	3.8	0.1	1035.5	1030.11	5.3	1.51
55682	1031.47	3.8	0.1	1035.3	1029.96	5.3	1.51
55482	1030.95	3.8	0.1	1034.8	1029.66	5.2	1.29
55282	1030.21	3.8	0.1	1034.1	1029.36	4.8	0.85
55000	1029.57	3.7	0.1	1033.4	1028.94	4.4	0.63
54500	1028.52	3.6	0.1	1032.2	1028.19	4.0	0.33
54000	1027.79	3.4	0.1	1031.3	1027.44	3.8	0.35
53500	1027.32	3.4	0.0	1030.7	1026.69	4.0	0.63
53000	1026.97	3.4	0.0	1030.4	1025.94	4.5	1.03
52500	1025.15	3.7	0.1	1029.0	1025.19	3.8	
51500	1023.71	3.3	0.1	1027.0	1023.68	3.4	0.03
51000	1022.77	3.3	0.1	1026.2	1022.93	3.2	
50500	1021.46	3.3	0.1	1024.9	1022.18	2.7	
50000	1019.95	3.2	0.1	1023.3	1021.43	1.8	
49930					Bridge		
49884.8	1019.29	3.6	0.1	1023.1	1021.26	1.8	
49860.68	1019.69	3.9	0.1	1023.7	1021.26	2.4	
49709.6	1019.54	3.9	0.1	1023.5	1017.14	6.4	2.4
49500	1019.36	3.9	0.1	1023.3	1016.95	6.4	2.41
49000	1018.91	3.9	0.1	1022.8	1016.50	6.3	2.41
48500	1018.47	3.9	0.1	1022.4	1016.05	6.3	2.42
48000	1017.94	3.9	0.1	1021.9	1015.60	6.3	2.34
47500	1017.28	3.9	0.1	1021.2	1015.15	6.1	2.13

Table A-4. Channel Capacity Analysis							
SPF High N HEC-RAS Model							
River Stn	WSEL (ft)	Freeboard (ft)	Super-Elevation (ft)	Required Bank Elev'n (ft)	DEA Plan Bank Elev'n (ft)	Additional Bank Ht Needed (ft)	WSEL Overtop Bank Elev'n
47000	1016.78	3.8	0.1	1020.6	1014.70	5.9	2.08
46500	1016.33	3.8	0.1	1020.2	1014.25	5.9	2.08
46000	1015.91	3.8	0.0	1019.7	1013.80	5.9	2.11
45500	1015.48	3.8	0.0	1019.3	1013.35	5.9	2.13
45000	1015.06	3.8	0.0	1018.9	1012.90	6.0	2.16



LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- PROPOSED DROP STRUCTURE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED LEVEES

DESIGN NOTES

1. LEVEE TO CONFORM TO NFIP AND FCDMC REGULATIONS AND DESIGN STANDARDS

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

MARICOPA COUNTY FLOOD CONTROL DISTRICT

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FIGURE 1: PROPOSED TRANSITION INTO CHANNELIZED AFR

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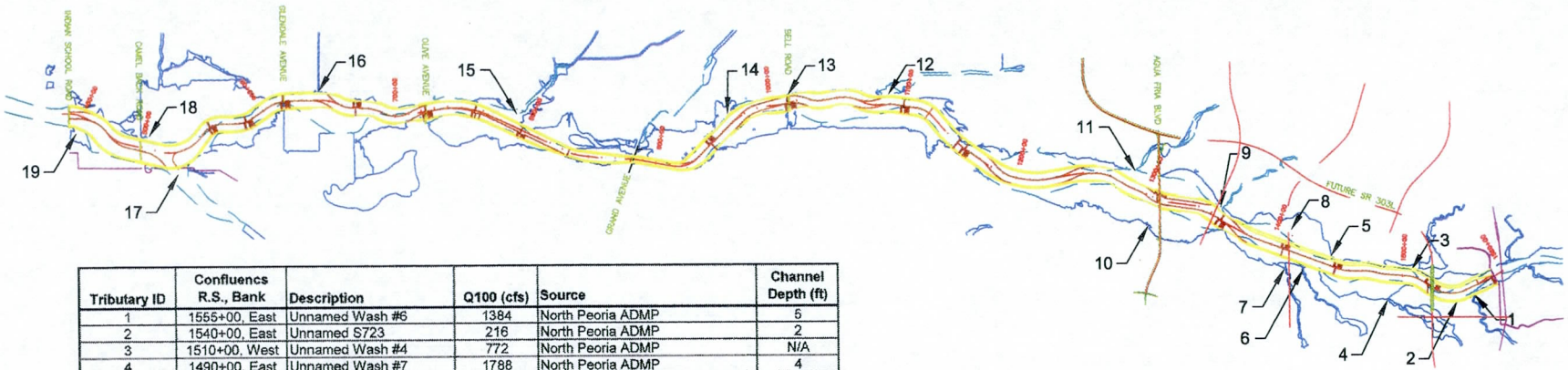

Tributary ID	Confluencs R.S., Bank	Description	Q100 (cfs)	Source	Channel Depth (ft)
1	1555+00, East	Unnamed Wash #6	1384	North Peoria ADMP	5
2	1540+00, East	Unnamed S723	216	North Peoria ADMP	2
3	1510+00, West	Unnamed Wash #4	772	North Peoria ADMP	N/A
4	1490+00, East	Unnamed Wash #7	1788	North Peoria ADMP	4
5	1445+00, West	Unnamed Wash S706	825	North Peoria ADMP	4
6	1430+00, East	Unnamed Wash #8	1153	North Peoria ADMP	4
7	1415+00, East	Unnamed Wash #9	1132	North Peoria ADMP	4
8	1410+00, West	Unnamed Wash S707	728	North Peoria ADMP	N/A
9	1350+00, West	Caterpillar Tank Wash	1556	North Peoria ADMP	5
10	1310+00, East	Unnamed (N of Hatfield Rd on East bank)	2820	Glendale Peoria ADMP Update Zone A F/P Delineation FCD 99-44	5
11	1285+00, West	Twin Buttes Wash	3775	North Peoria ADMP	5
12	1090+00, West	McMicken Outfall	6522	Wittman ADMS and FEMA FIS. Page 44	3
13	1015+00, West	Eastward Drainage along Bell Road	6500	No document was found with discharge identified for this location, discharge estimated based on channel capacity estimate	10, Match Existing
14	970+00, West	Lizard Acres Wash	2114	Flood Insurance Study Unincorporated Areas of Maricopa County, 1979	N/A
15	790+00, West	El Mirage Wash	1753	White Tanks/ Agua Fria River ADMS and LOMR	N/A
16	640+00, West	Dysart Drain	3978	CLOMR Application Package for Dysart Drain of White Tank/ Agua Fria ADMS	N/A
17	525+00, East	New River	39000	Phoenix, Arizona & Vicinity Hydrology Part 2, USACE (SPF = 69,000 cfs)	N/A
18	500+00, West	Colter Channel	2170	Hydrology of the Colter Channel	N/A
19	450+00, East	Westbound Drainage along Indian School Road	159	Drainage Study and Conceptual Design (Indian School Drain) FCD 84-32	N/A

GRAPHIC SCALE

8000 0 4000 8000 16000

(IN FEET)
1 inch = 8000 ft.

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FIGURE 2: TRIBUTARY OUTFALL LOCATIONS

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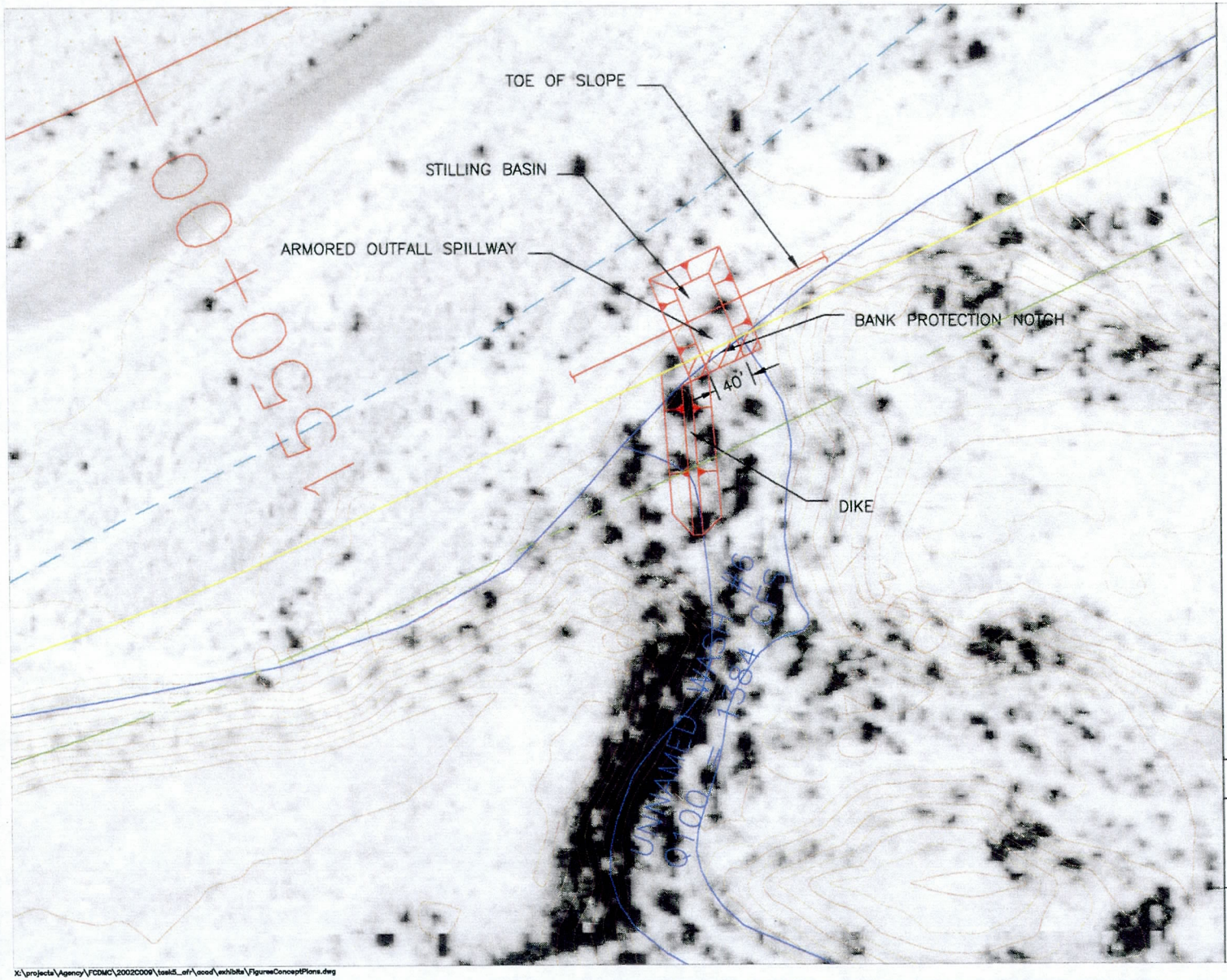

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1800)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK PROTECTION BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY OUTFALL SPILLWAY IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 3: UNNAMED WASH #6, ID1

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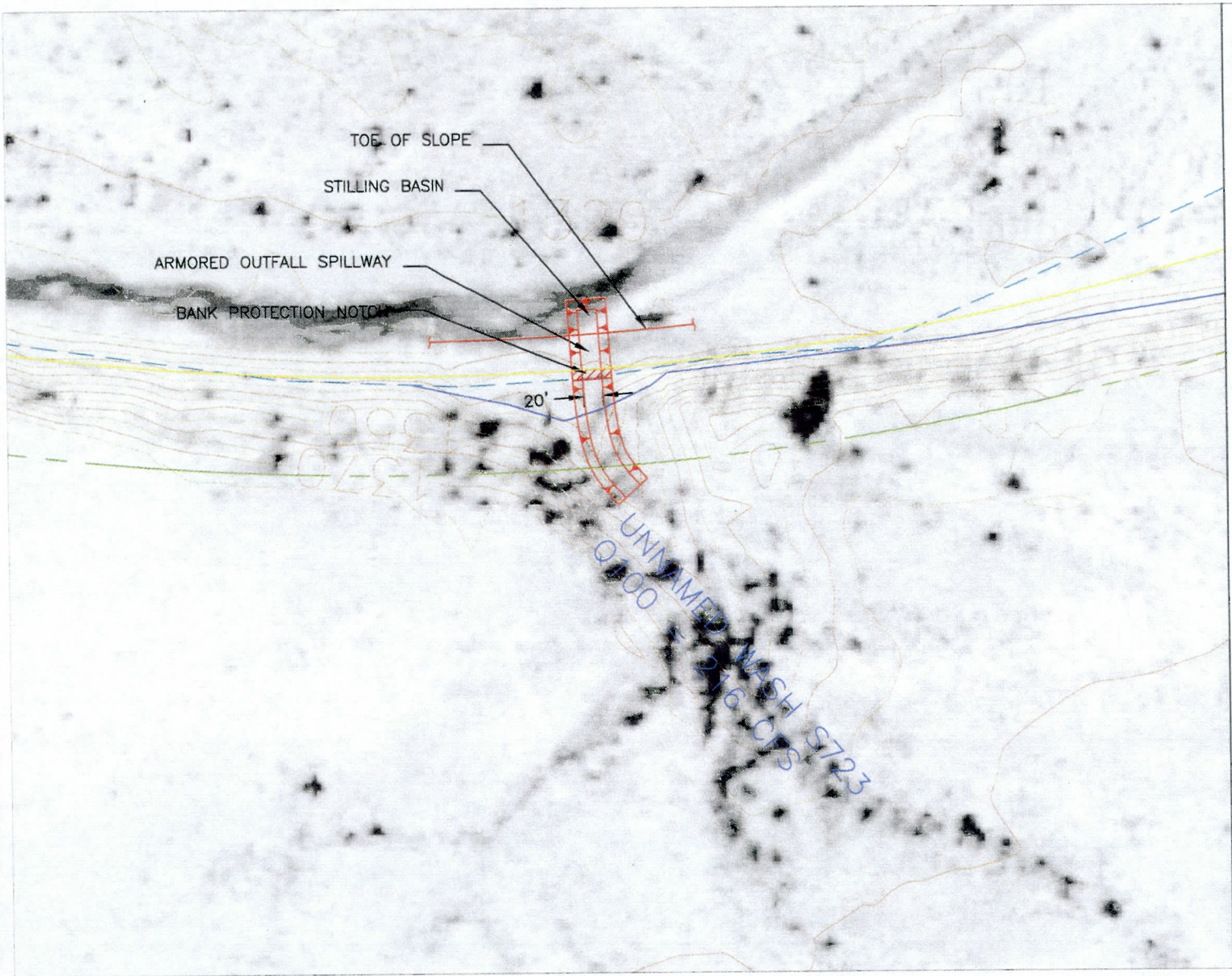

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES — TYPE
- EXISTING GROUND CONTOURS — 1000
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 4: UNNAMED WASH S723, ID2

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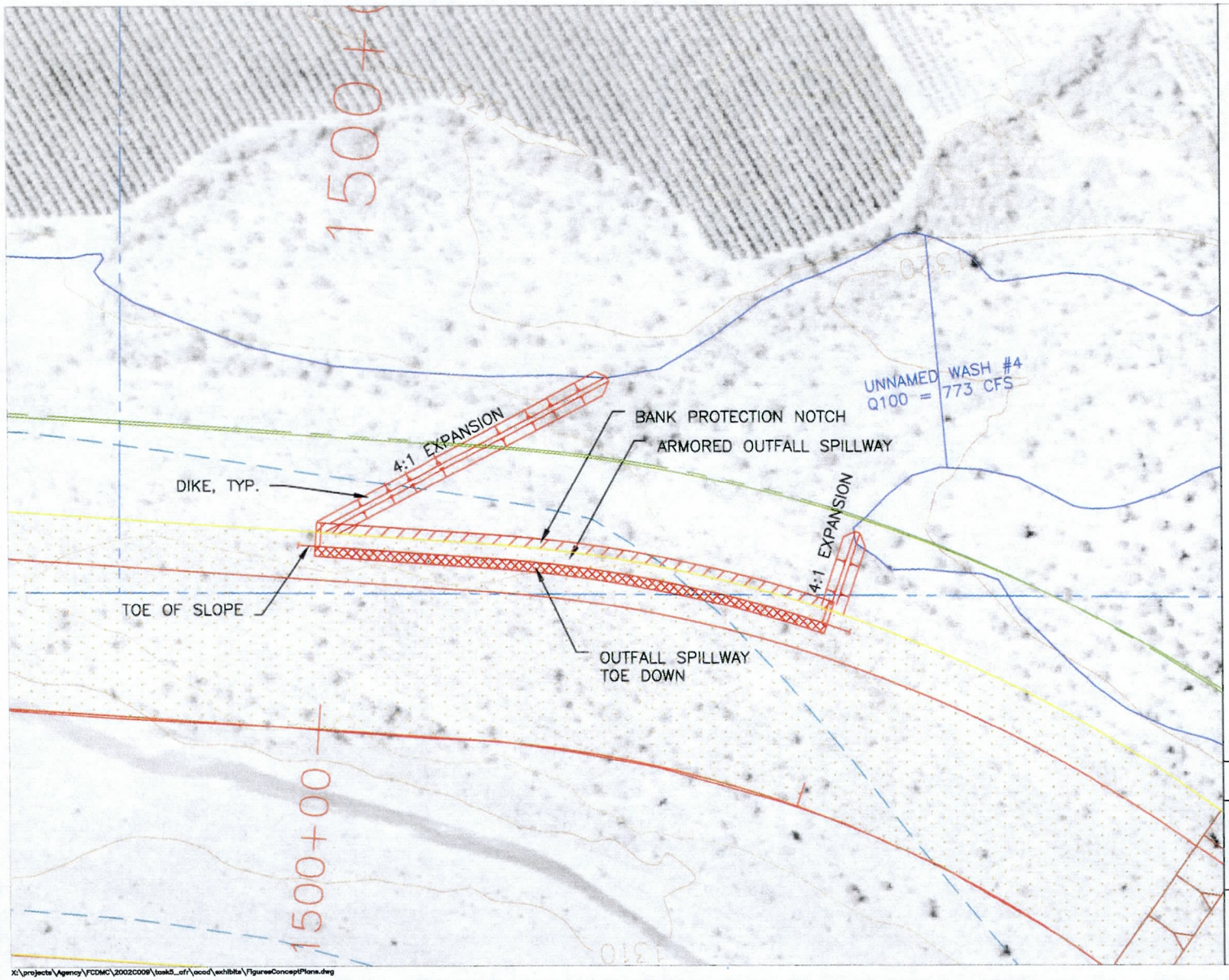

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. TOE-DOWN ARMORED SPILLWAY BASED ON LOCAL SCOUR FROM TRIBUTARY OUTFALL SPILLWAY.

2. DUE TO WIDE SHALLOW FLOW AND LOW VELOCITIES AT THIS OUTFALL, EROSION OVER THE TERRACE IS EXPECTED TO BE MINIMAL. THEREFORE A BASIC VEGETATED TERRACE IS RECOMMENED TO MINIMIZE FLOW VELOCITIES.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 5: UNNAMED WASH #4, ID3

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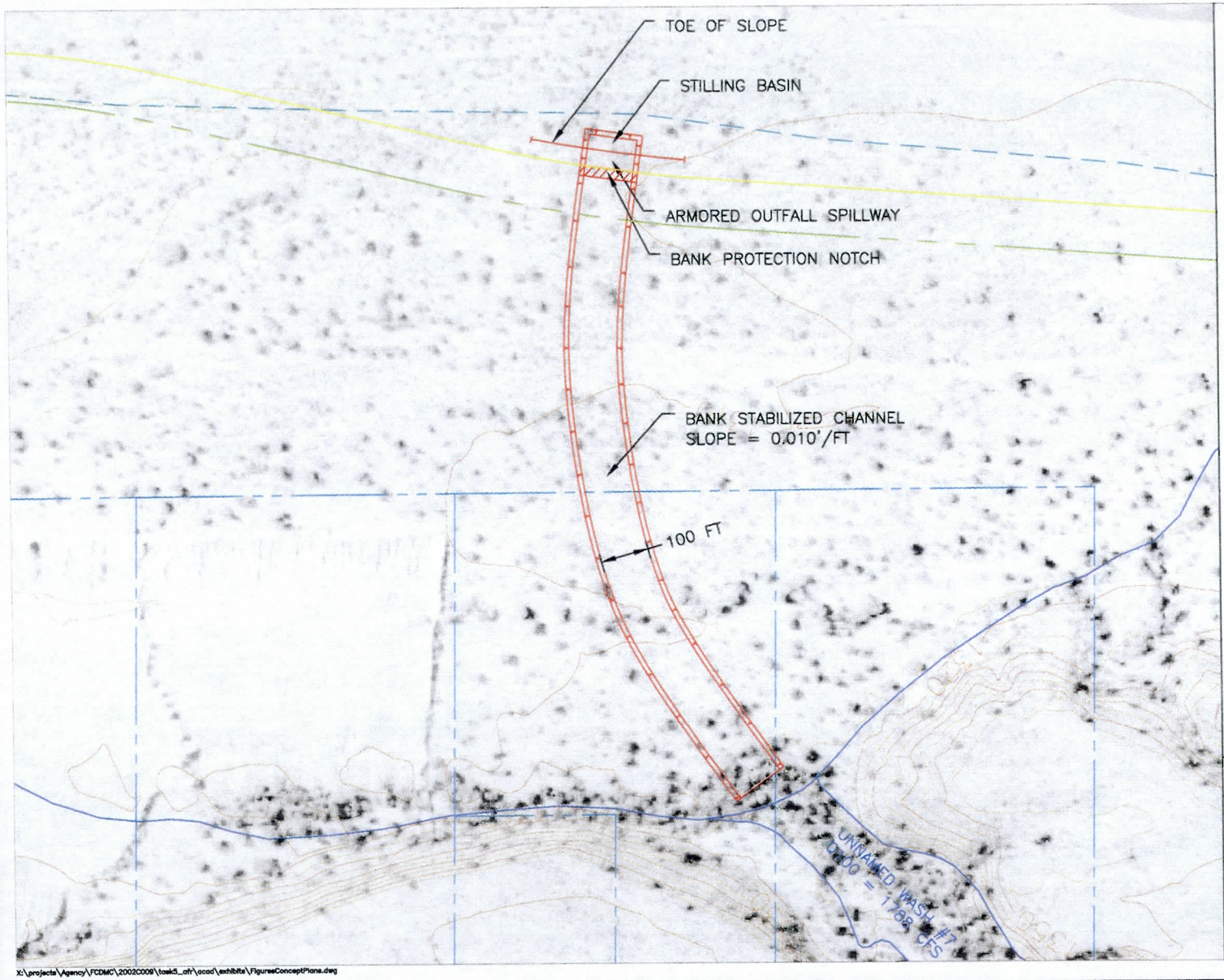

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 6: UNNAMED WASH #7, ID4

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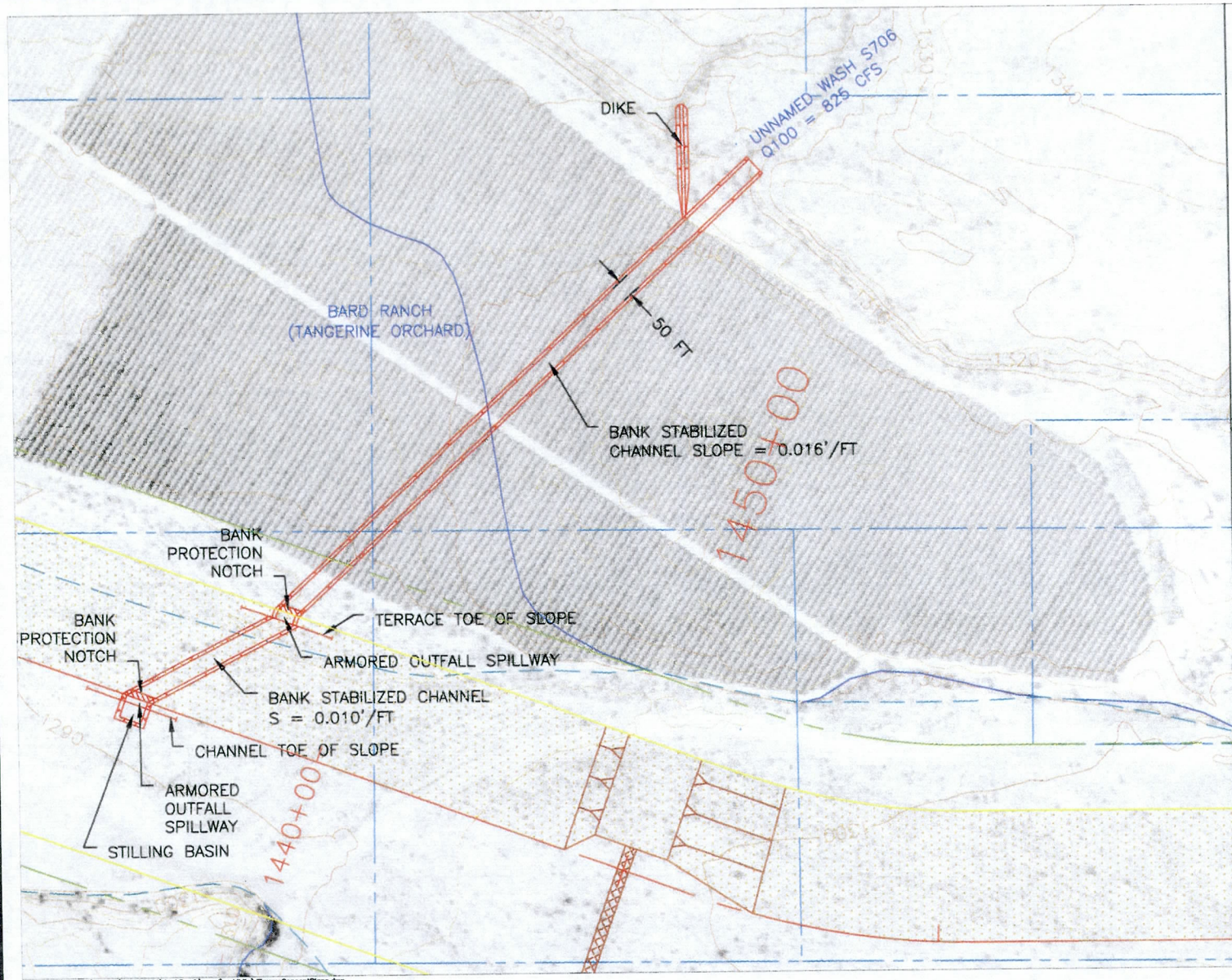

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF ADDING A TRADITIONAL ENGINEERED BANK PROTECTION TOE-DOWN BASED CHANNEL SCOUR AND ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO AGUA FRIA RIVER CHANNEL BANK PROTECTION NOTCH.

GRAPHIC SCALE

300 0 150 300

(IN FEET)
1 inch = 300 ft.

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FIGURE 7: UNNAMED WASH S706, ID5

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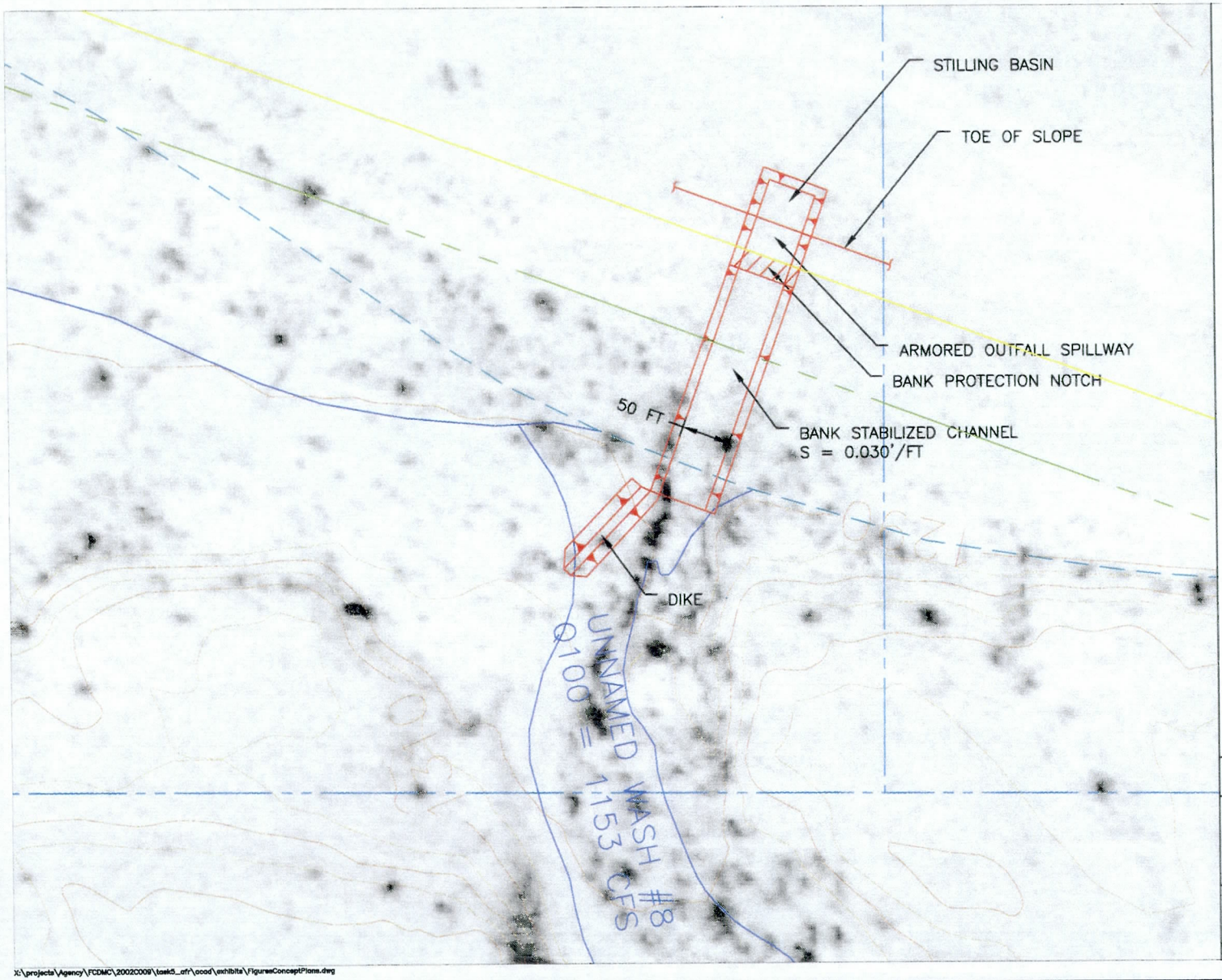

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES — TYPE
- EXISTING GROUND CONTOURS — 1900
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 8: UNNAMED WASH #8, ID6

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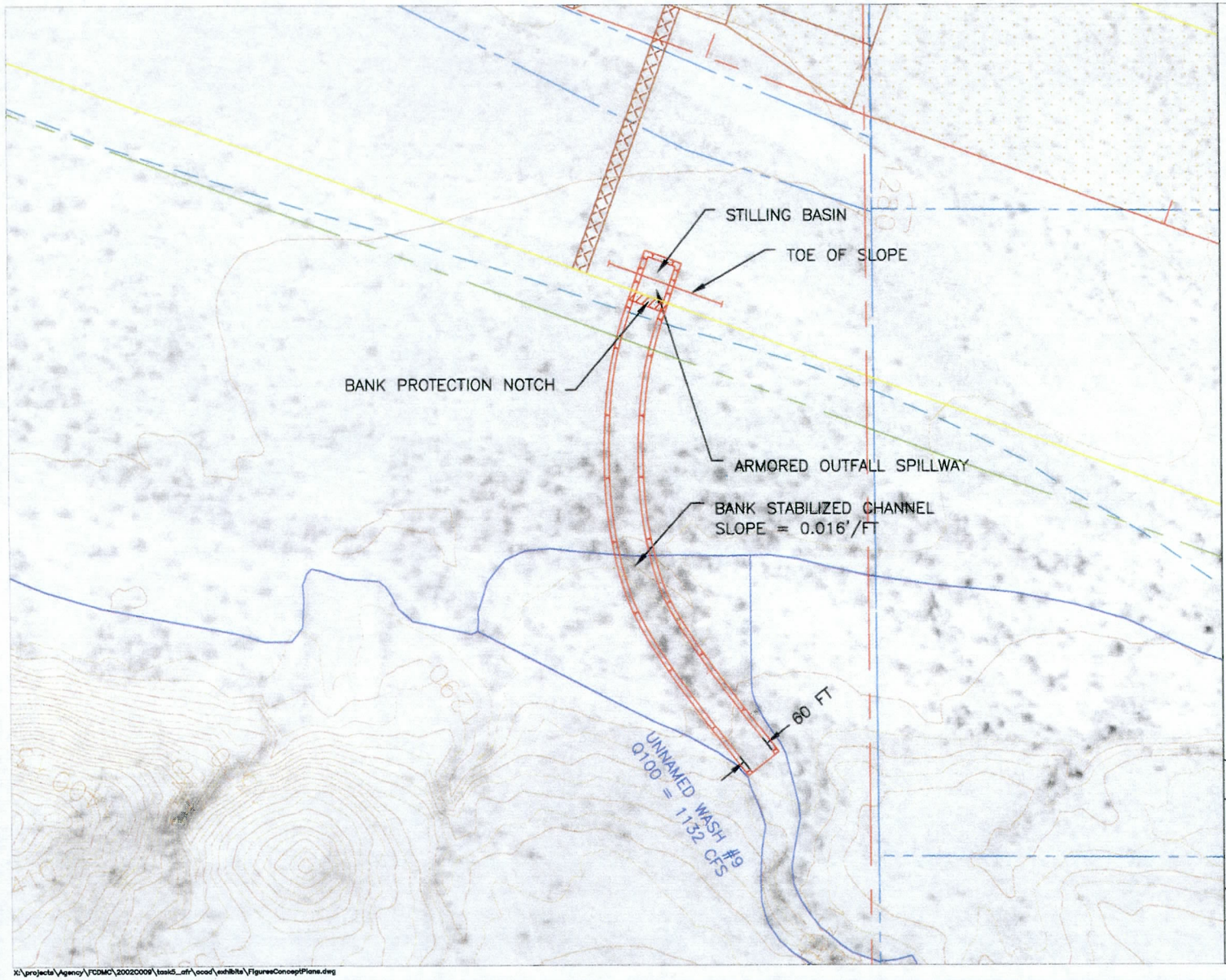

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 9: UNNAMED WASH #9, ID7

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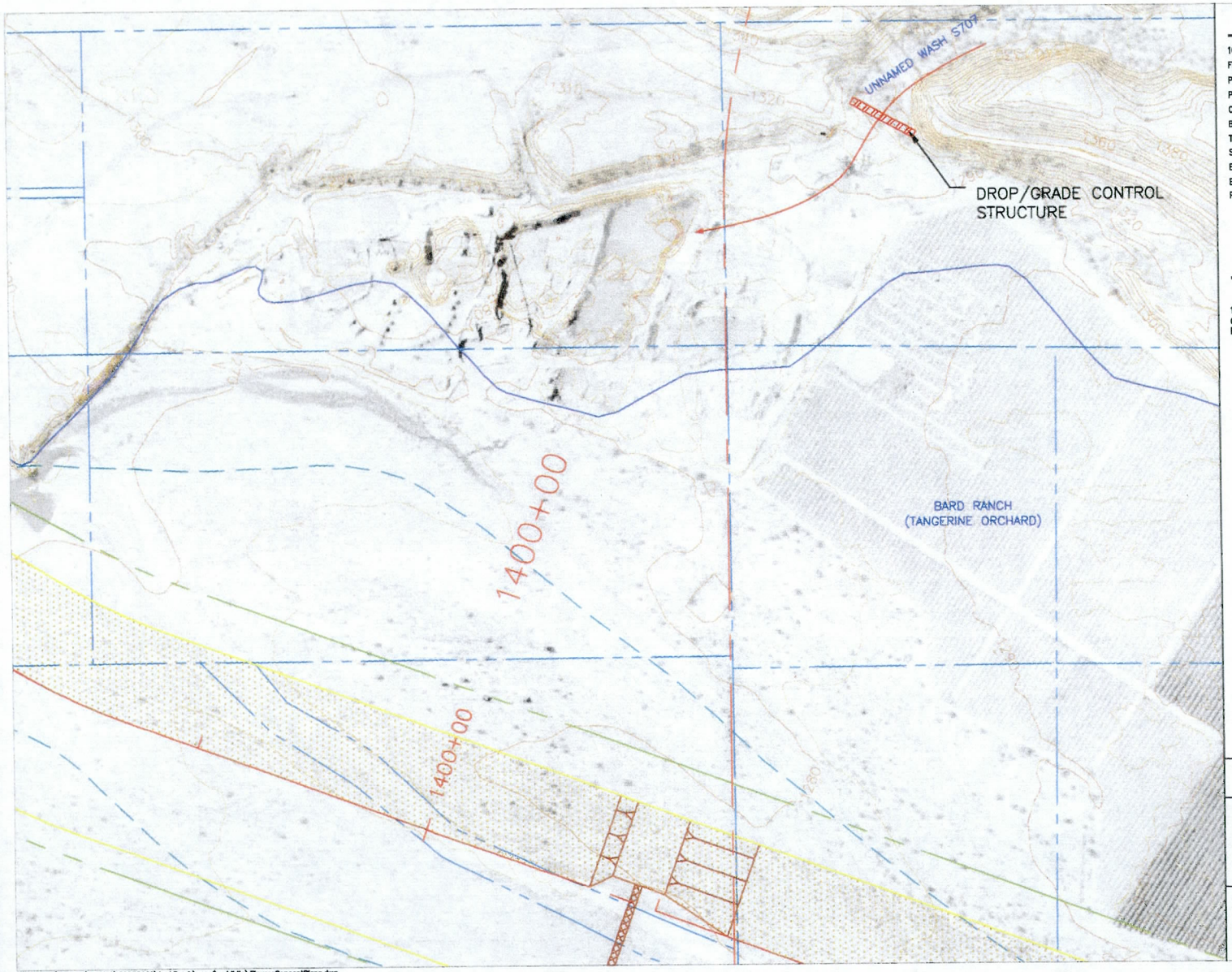

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. TRIBUTARY CAPUTURED BY SAND AND GRAVEL OPERATION. DROP/GRADE CONTROL STRUCTURE REQUIRED.

GRAPHIC SCALE

400 0 200 400

(IN FEET)
1 inch = 400 ft.

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FIGURE 10: UNNAMED WASH S707, ID8

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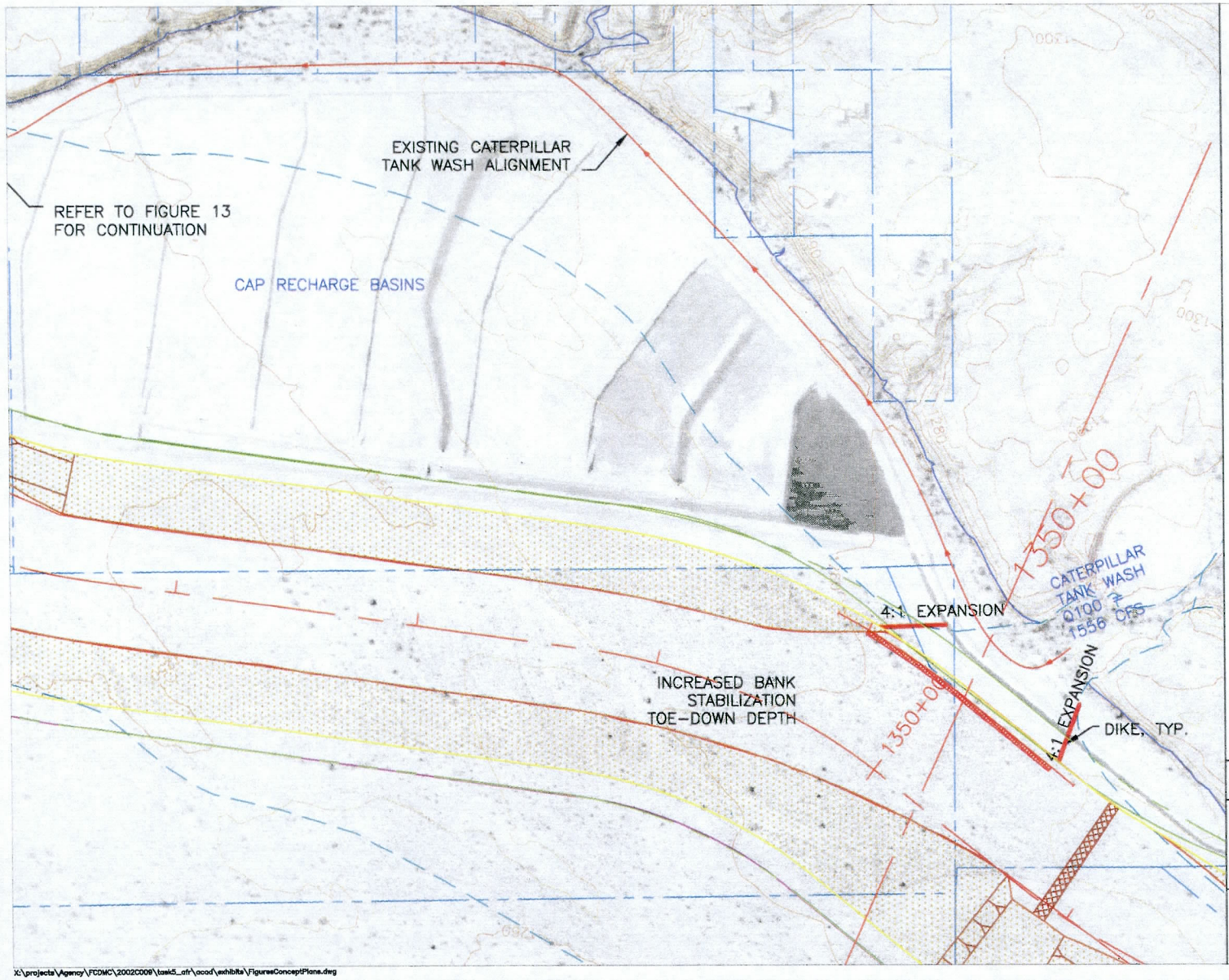

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. INCREASE THE BANK STABILIZATION TOE DOWN DEPTH TO ACCOUNT FOR LOCAL SCOUR DUE TO THE CATERPILLAR TANK WASH OUTFALL IN THE EVENT OF A CAP DITCH EMBANKMENT FAILURE. CONDUCT A STABILITY ANALYSIS TO DETERMINE IF THE CAP DITCH WILL REMAIN IN PLACE DURING LARGE FLOODING EVENTS AS AN ALTERNATIVE.

GRAPHIC SCALE

400 0 200 400

(IN FEET)
1 inch = 400 ft.

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FIGURE 11: CATERPILLAR TANK WASH, ID9

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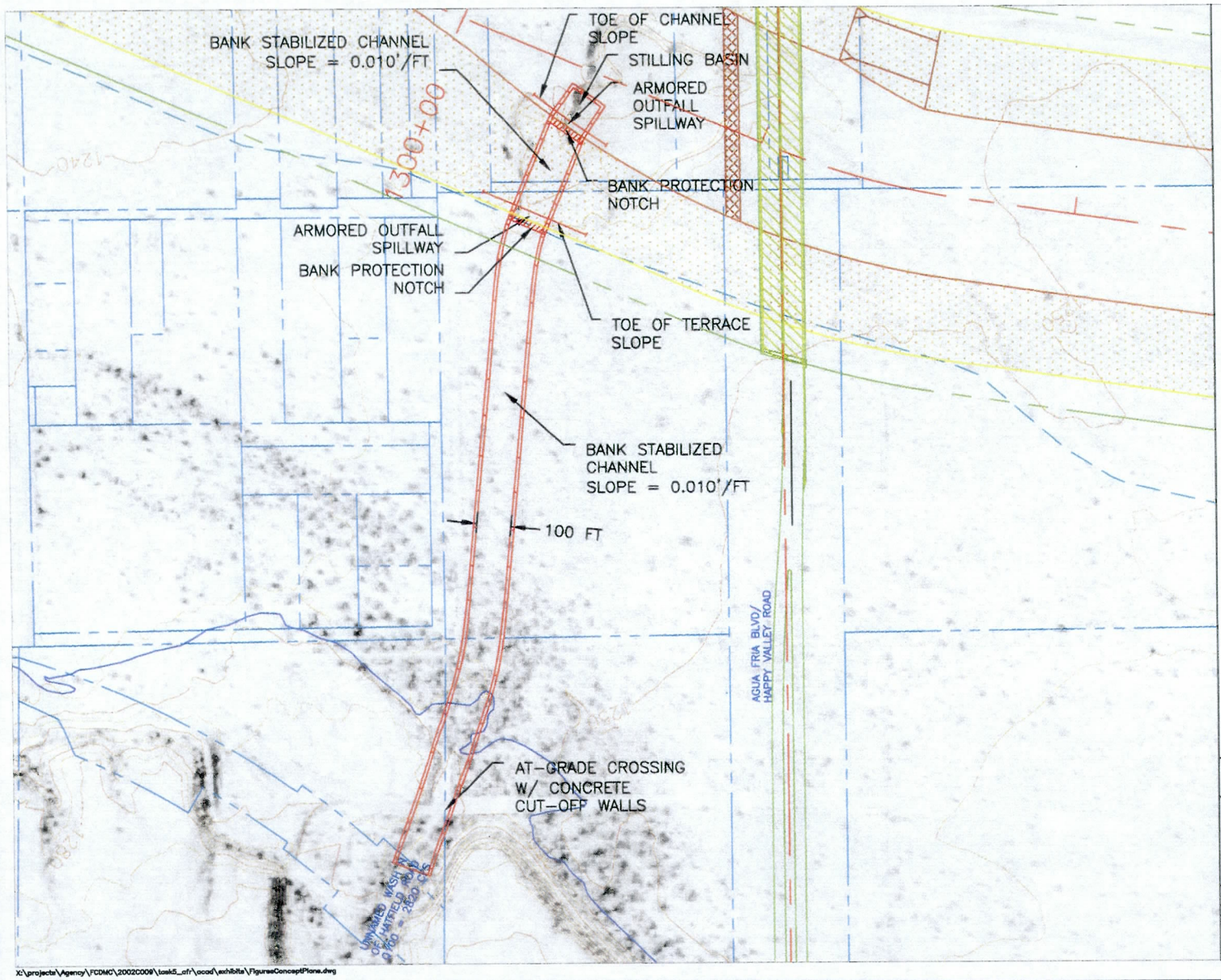

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

300 0 150 300

(IN FEET)
1 inch = 300 ft.

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FIGURE 12: UNNAMED WASH NORTH OF HATFIELD ROAD, ID10

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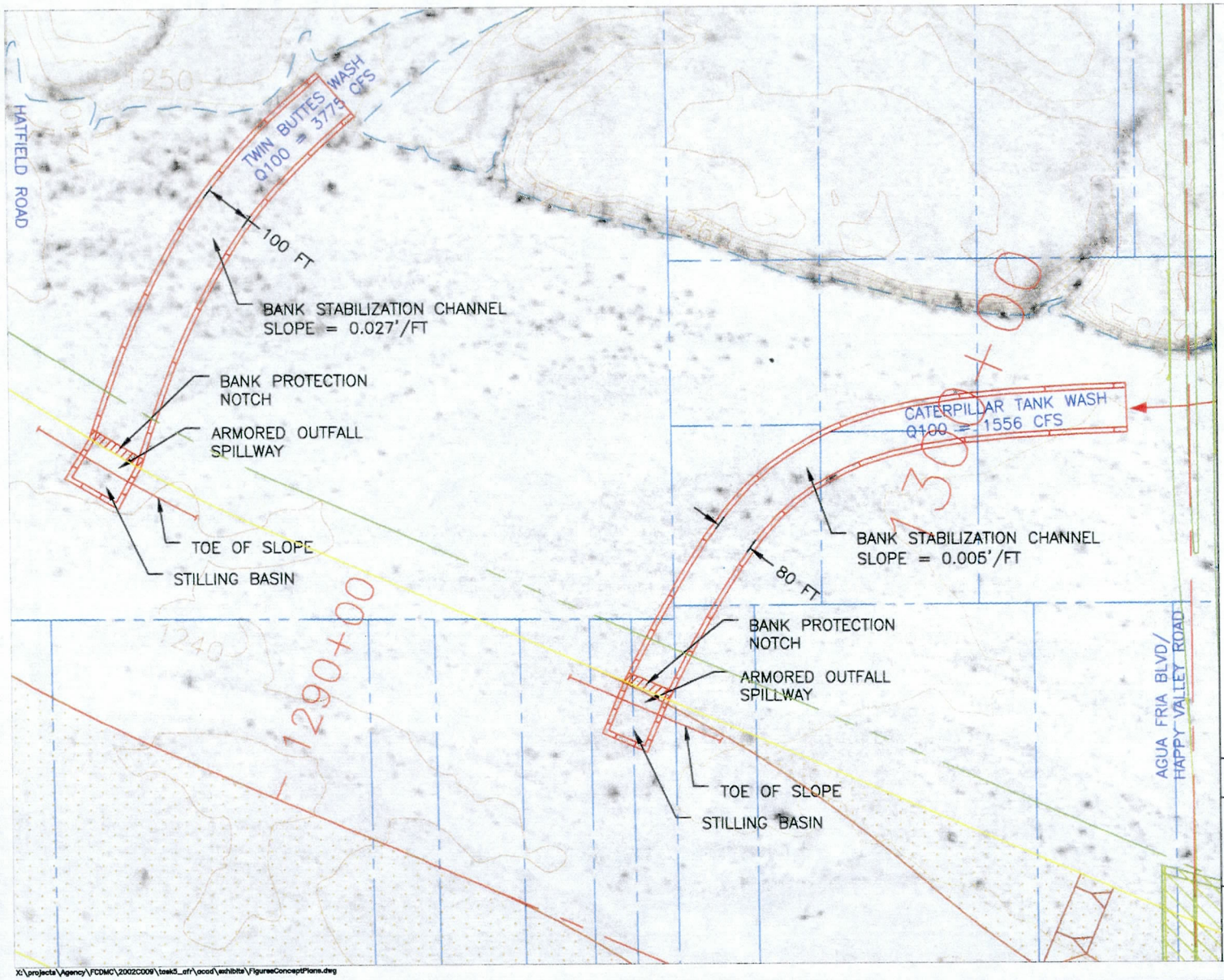

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.
2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.
3. CONSTRUCT CHANNEL FROM POINT OF CONTAINMENT TO BANK PROTECTION NOTCH.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 13: TWIN BUTTES WASH, ID 11, CATERPILLER TANK WASH, ID 9

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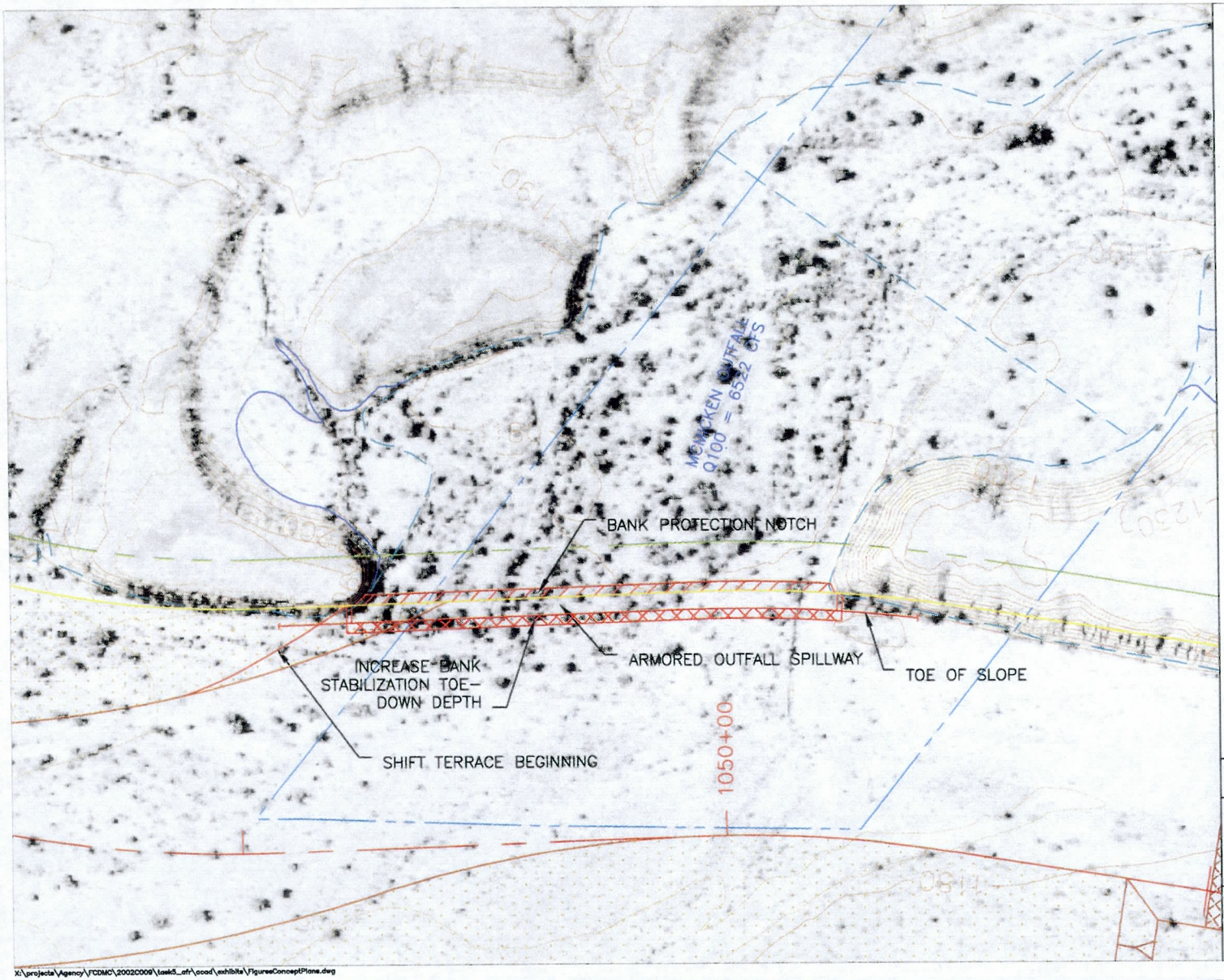

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. INCREASE TOE-DOWN DEPTH OF BANK BASED ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE.

2. START WEST TERRACE AT THIS LOCATION 200 FEET FURTHER DOWNSTREAM.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 14: MCMICKEN OUTFALL, ID12

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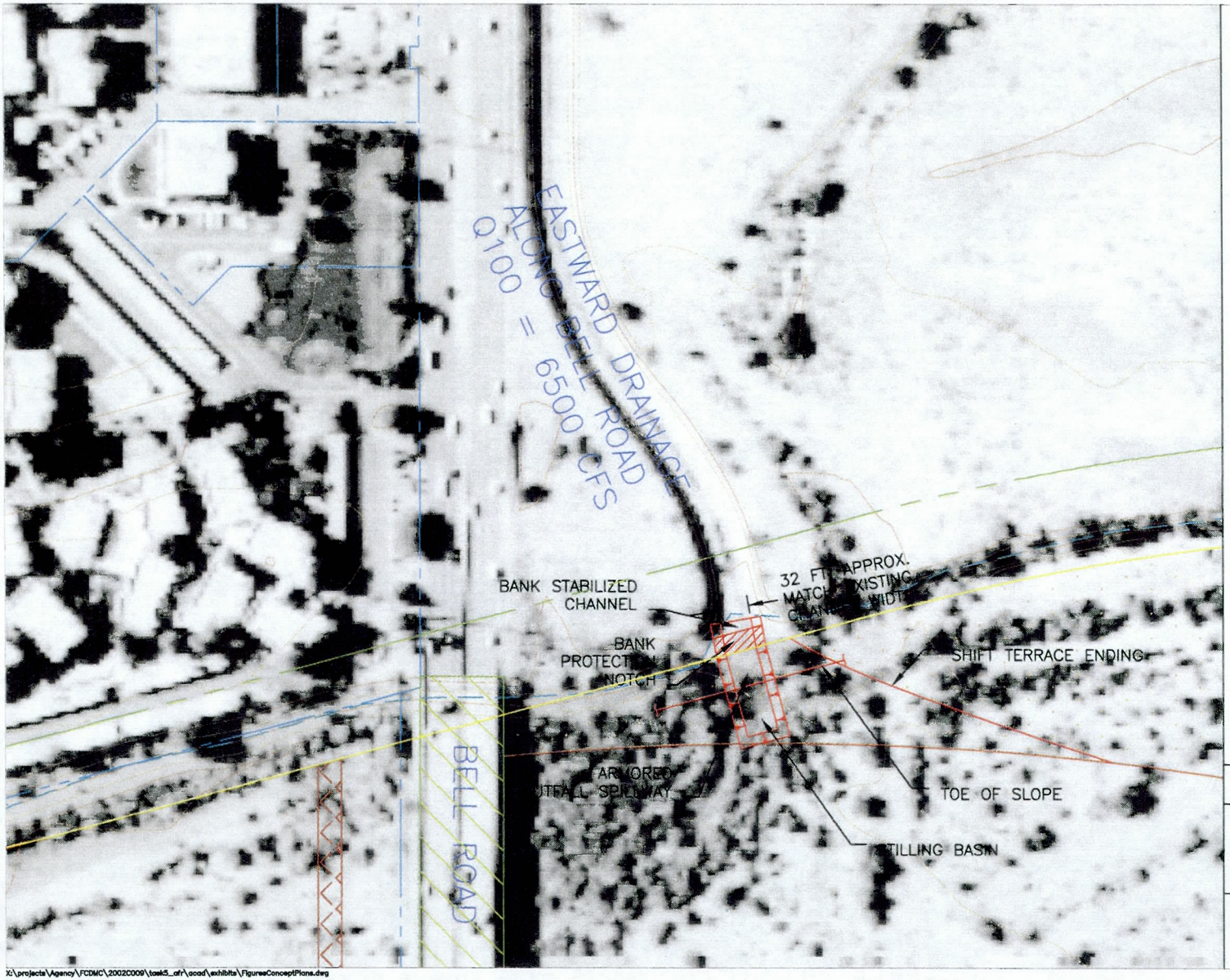

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

3. END WEST TERRACE 300 FEET FURTHER UPSTREAM.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 15: EASTWARD DRAINAGE ALONG BELL ROAD, ID 13

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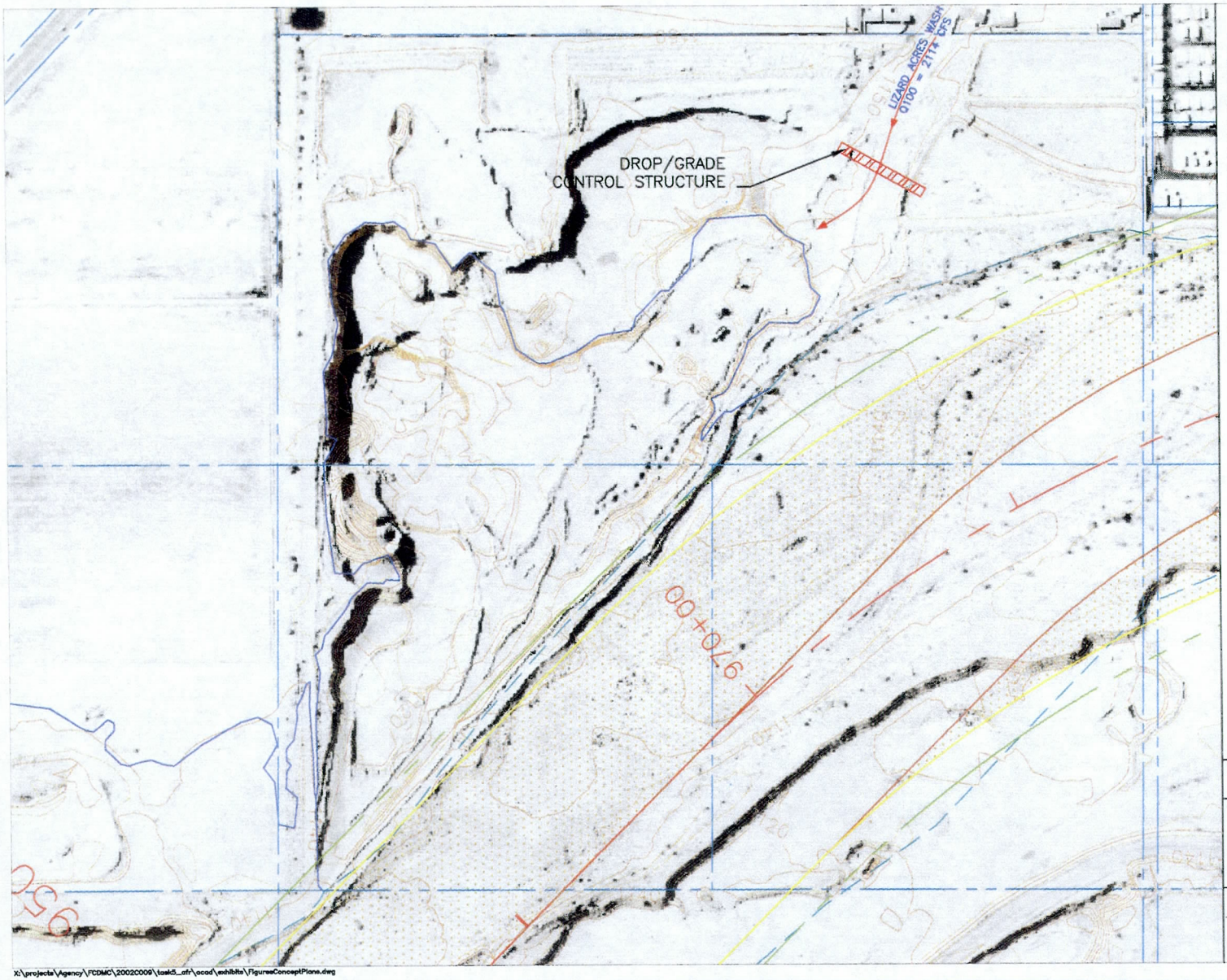

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. TRIBUTARY CAPUTURED BY SAND AND GRAVEL OPERATION. DROP/GRADE CONTROL STRUCTURE REQUIRED.

2. CHANNEL TO NORTH AROUND PIT AS AN ALTERNATIVE.

GRAPHIC SCALE

300 0 150 300

(IN FEET)
1 inch = 300 ft.

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FIGURE 16: LIZARD ACRES WASH, ID 14

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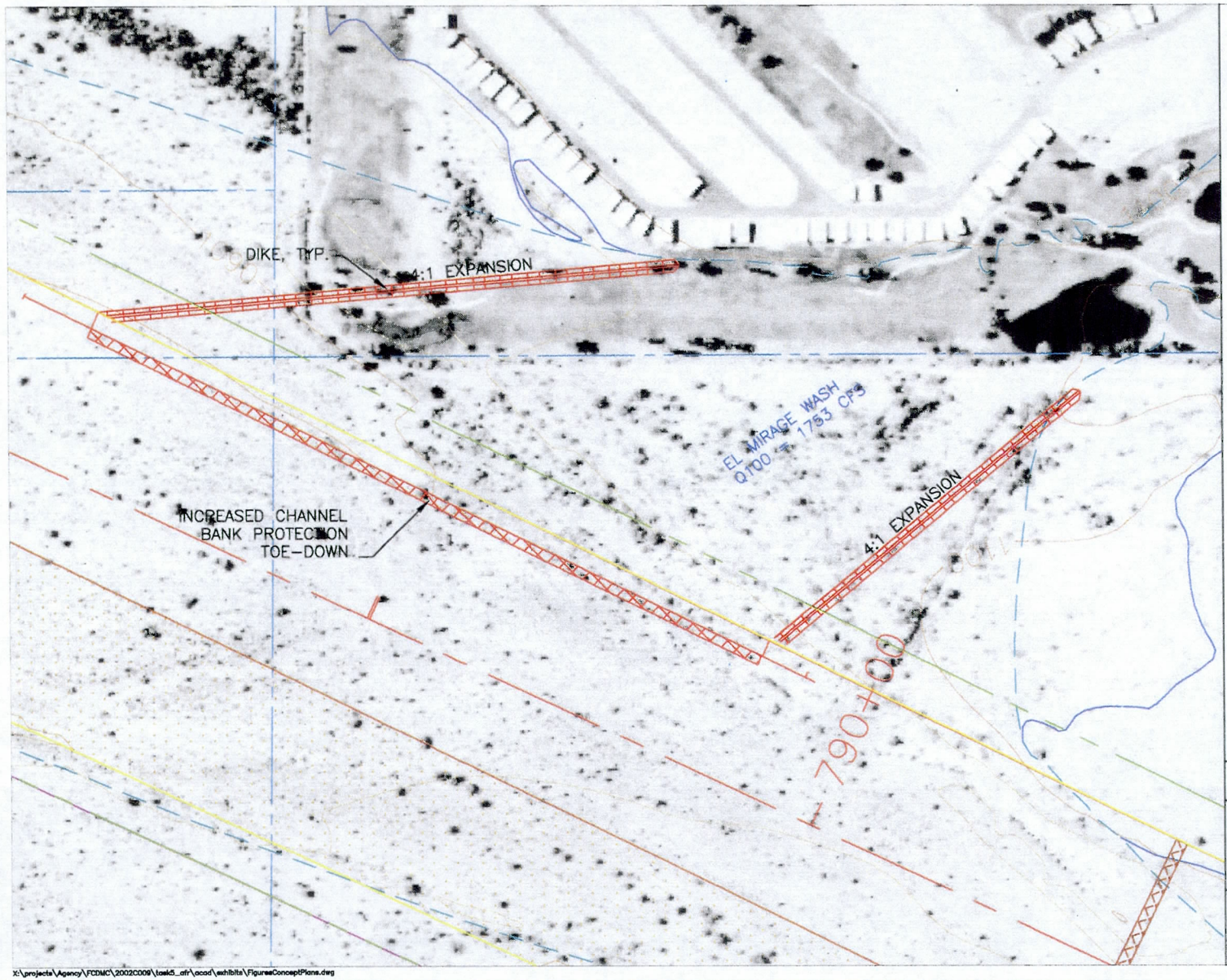

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. INCREASE TOE-DOWN DEPTH OF BANK PROTECTION BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY OUTFALL SPILLWAY.

2. DUE TO WIDE SHALLOW FLOW AND LOW VELOCITIES AT THIS OUTFALL, EROSION OVER THE BANK IS EXPECTED TO BE MINIMAL. THEREFORE IT IS ANTICIPATED THE MAIN CHANNEL BANK STABILIZATION MEASURES WILL BE SUFFICIENT TO MITIGATE EROSION ON THE BANK FROM THIS OUTFALL.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 17: EL MIRAGE WASH, ID 15

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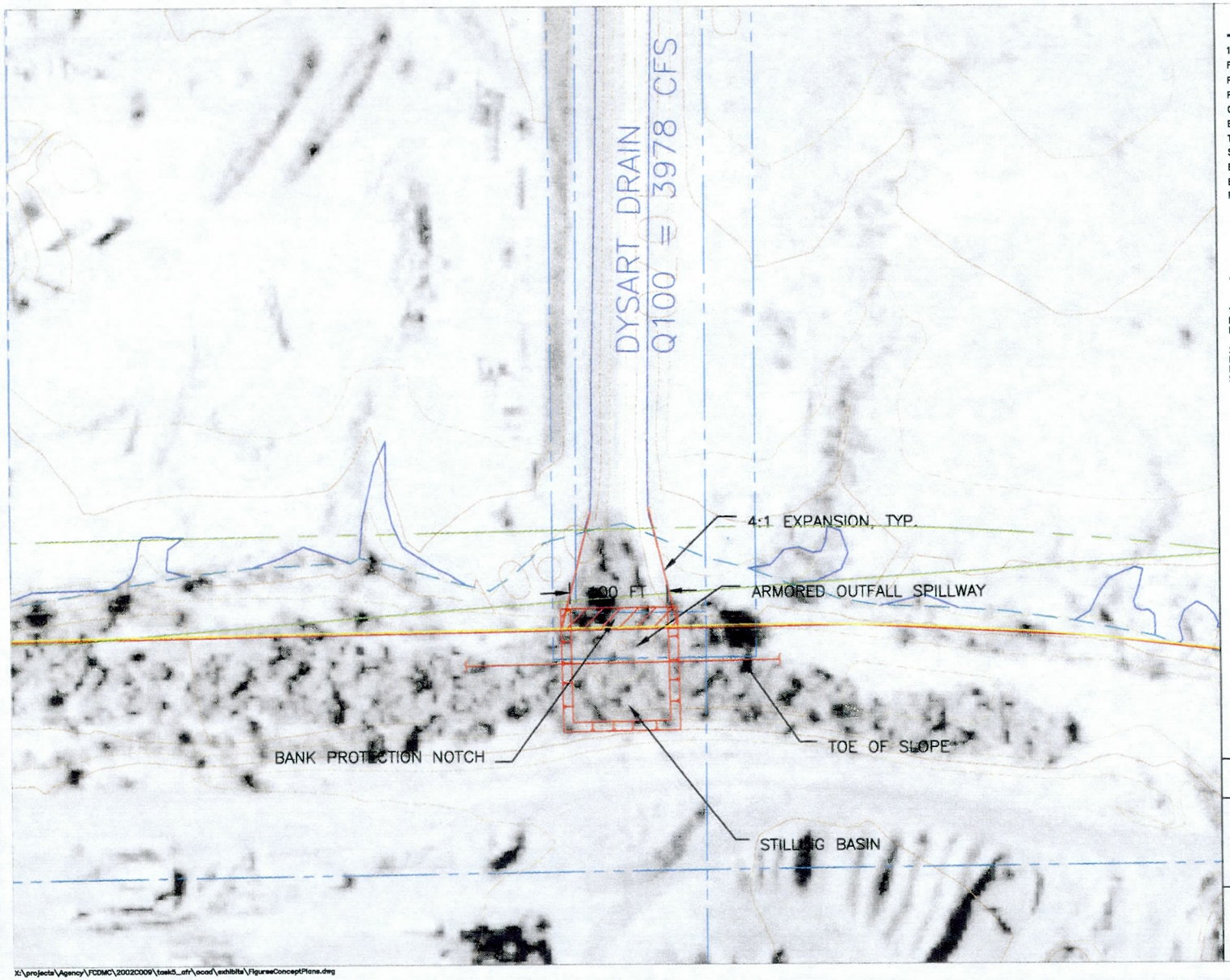

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.

2. CONSIDER ALTERNATIVE OF INCREASING TOE-DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 18: DYSART DRAIN, ID16

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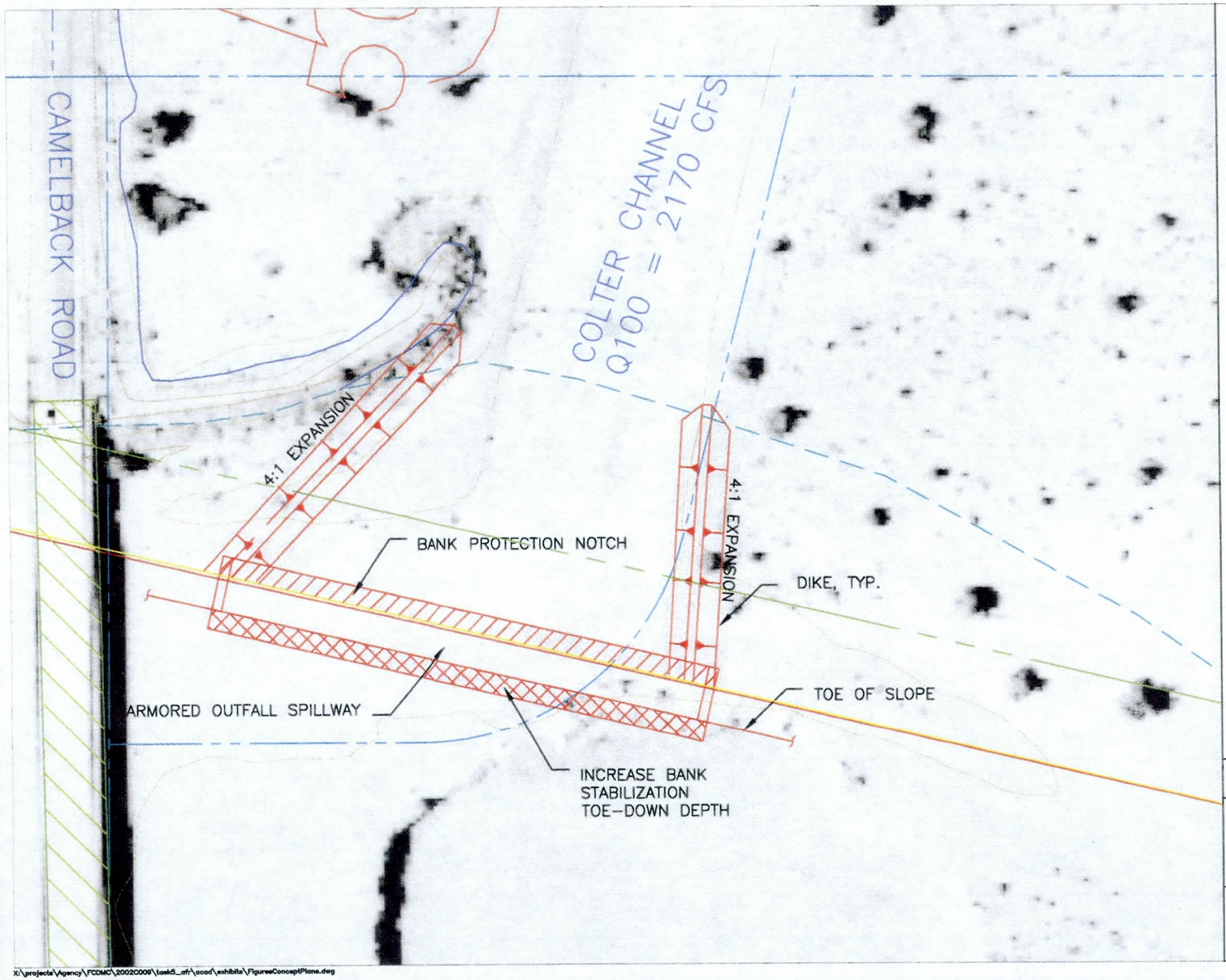

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. INCREASE TOE-DOWN DEPTH OF BANK PROTECTION BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY OUTFALL SPILLWAY.

GRAPHIC SCALE

100 0 50 100

(IN FEET)
1 inch = 100 ft.

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FIGURE 19: COLTER CHANNEL, ID18

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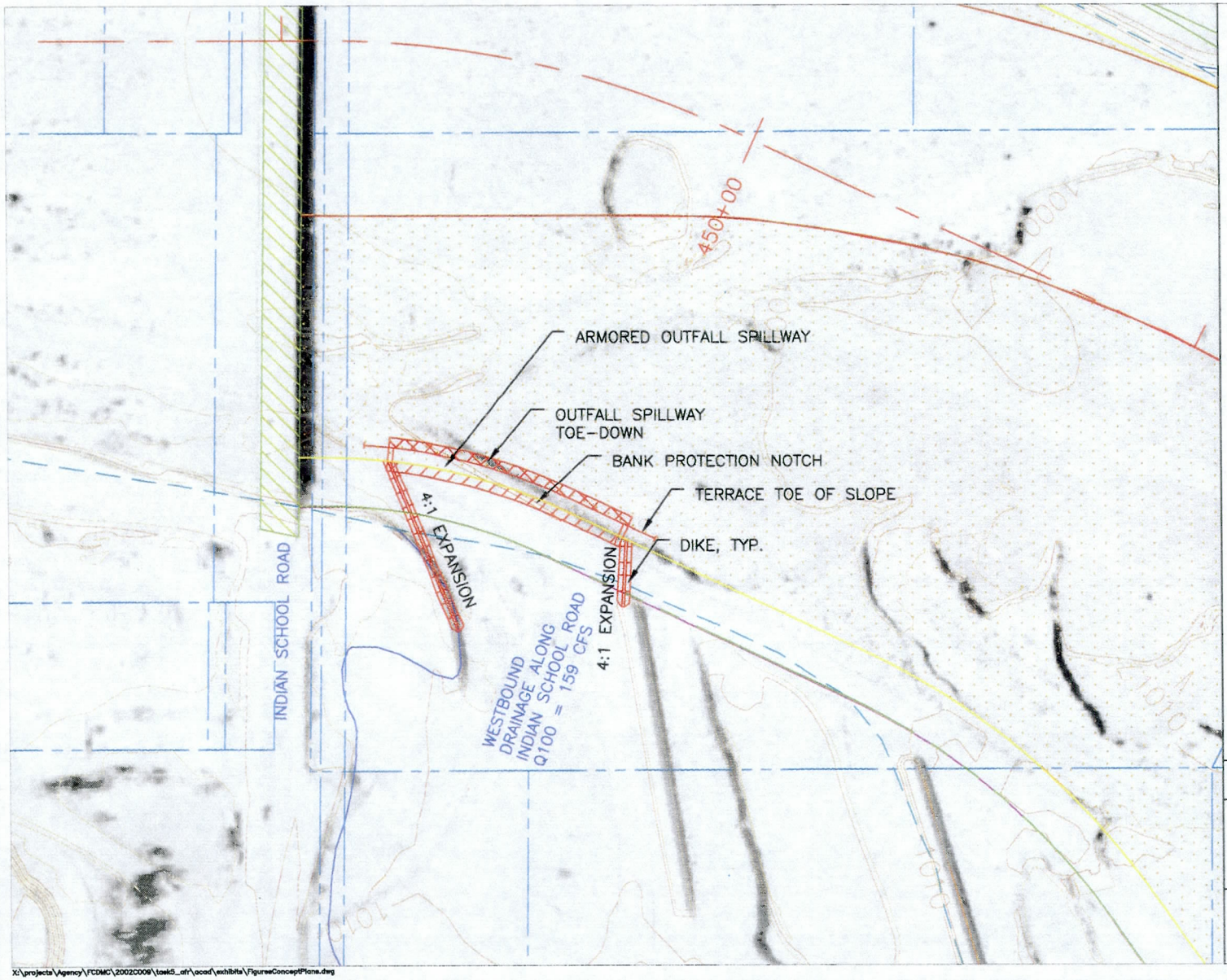

LEGEND

- 100-YR FLOODPLAIN BOUNDARY
- FLOODWAY BOUNDARY
- PROPOSED CHANNEL SETBACK
- PROPOSED CHANNEL TOP
- CHANNEL/ROADWAY CENTERLINE
- BRIDGE CROSSING
- TERRACE
- SECTION/PROPERTY LINE
- EXISTING UTILITIES (TYPE)
- EXISTING GROUND CONTOURS (1000)
- PROPOSED TRIBUTARY OUTFALLS

DESIGN NOTES

1. TOE-DOWN ARMORED OUTFALL SPILLWAY BASED ON LOCAL SCOUR FROM TRIBUTARY OUTFALL SPILLWAY.

2. DUE TO WIDE SHALLOW FLOW AND LOW VELOCITIES AT THIS OUTFALL, EROSION OVER THE TERRACE IS EXPECTED TO BE MINIMAL. THEREFORE A BASIC VEGETATED TERRACE IS RECOMMENED TO MINIMIZE FLOW VELOCITIES.

GRAPHIC SCALE

200 0 100 200

(IN FEET)
1 inch = 200 ft.

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FIGURE 20: WESTBOUND DRAINAGE ALONG INDIAN SCHOOL ROAD, ID19

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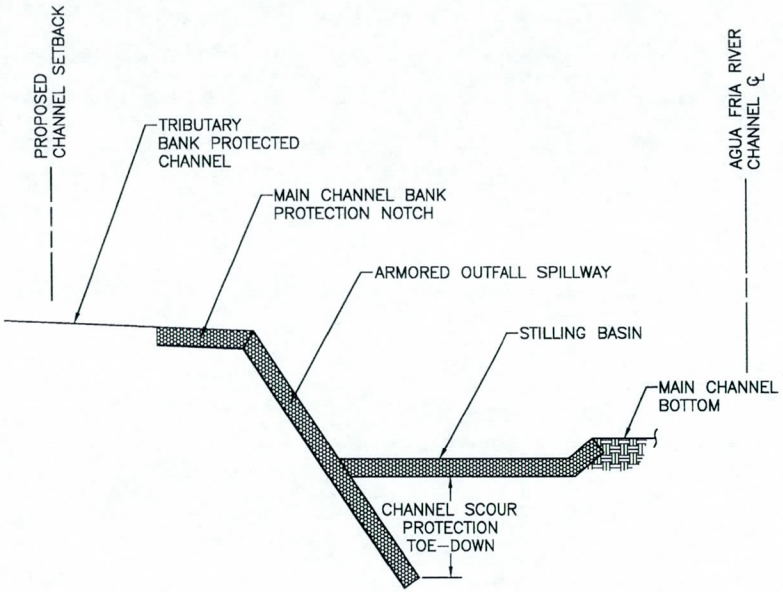
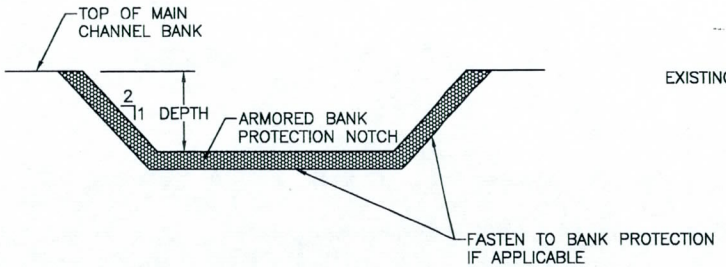


ARMORED OUTFALL SPILLWAY

N.T.S.

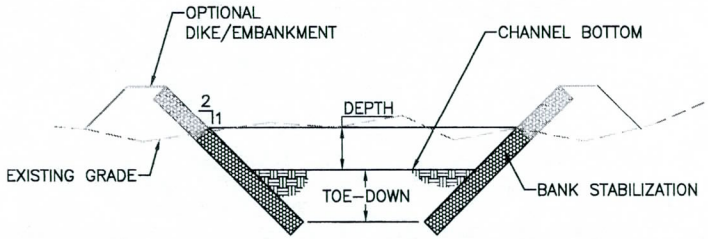


NOTES:

1. NOTCH DEPTH TO MATCH TRIBUTARY DEPTH.

BANK PROTECTION NOTCH

N.T.S.



NOTES:

1. CHANNEL DEPTH INCLUDES DESIGN WATER DEPTH AND REQUIRED FREEBOARD.
2. DESIGN CHANNEL FROM POINT OF CONTAINMENT TO MAIN CHANNEL BANK PROTECTION NOTCH

BANK STABILIZED CHANNEL

N.T.S.

LEGEND

DESIGN NOTES

1. EROSIVE FORCES DUE TO AGUA FRIA CHANNEL FLOODING SHOULD BE CONSIDERED IN STILLING BASIN DESIGN.
2. CONSIDER ALTERNATIVE OF INCREASING TOE–DOWN DEPTH OF BANK BASED ON ADDITIONAL LOCAL SCOUR FROM TRIBUTARY DROP STRUCTURE IN LIEU OF STILLING BASIN.
3. ARMORING SHOULD CONSIST OF TRADITIONAL ENGINEERED STABILIZATION SUCH AS RIP–RAP, GABION BASKETS OR SOIL CEMENT.

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FIGURE 21: CONCEPT DETAILS