Memorandum **JE Fuller/ Hydrology & Geomorphology, Inc.**

DATE: July 26, 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

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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 prepared this memorandum for the Flood Control District of Maricopa County (District) as the deliverable for Task Assignment #5 of On-Call Contract No. FCD 2002C009. Task 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. 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, grade control structures, flood channel/terrace transition, transition from non-channelized to channelized reaches, and tributary inflows.
- ***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. Where as-built plans did not provide adequate information, HEC-RAS models were based HEC-RAS or HEC-2 models prepared for floodplain delineation studies that had been previously 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 channel 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 0.060 was run, which resulted in water surface elevations that averaged 1.53 feet higher than the subcritical profile model, with a maximum increase of 2.34 feet within the channelization reach.

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. Therefore, further hydraulic analysis of these drop structures is recommended prior to final design.

Mixed Profile. The HEC-RAS model indicates that supercritical flow occurs over the face of most of the 6:1 drop structures. A mixed profile HEC-RAS model was 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. A table 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 topwidth (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.

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. 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 1.

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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.

For reaches with levees, the minimum freeboard is usually dictated by FEMA standards, which range from three to four feet depending on the location relative to hydraulic structures.

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 topwidth (ft)
R_c = radius of curvature (ft)
g = gravitational constant, 32.2 ft/sec^2

For the AFR WCMP channel, superelevation averages 0.1 foot, with a maximum computed superelevation of 1.1 foot, based on the HEC-RAS subcritical profile results.

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 Table 1, as well as the recommended action to provide the required capacity. The channel depth/levee height would need to be increased an average of 1.78 feet higher if no channel maintenance activities will be conducted to prevent Manning's n values from exceeding 0.030 and 0.045 in the flood channel and terrace, respectively, and n values of 0.045 and 0.060 were applicable. Refer to HEC-RAS model results for more detailed information.

Table 1. Channel Capacity Including Freeboard & Superelevation from HEC-RAS Results

River Station	100-Year WSEL	Freeboard (ft)	Super-Elevation (ft)	Required Elevation	Design Top of Channel	Additional Capacity Needed	Recommended Action to Provide Capacity
At toe of drop structure upstream of Grand Avenue & Vulcan Mining Operation							
946+82	1126.56	2.6	0.0	1129.14	1129.01	0.13	Widen flood channel 100 ft. or raise levee 0.3 ft.
945+82	1126.46	2.6	0.0	1129.05	1129.85	0.20	
931+82	1124.04	2.6	0.2	1126.88	1126.61	0.27	
930+00	1123.76	2.6	0.2	1126.61	1126.32	0.29	
Downstream of drop structure adjacent to El Mirage WWTP between Peoria Ave & Olive Ave alignments							
759+88	1083.89	2.6	0.1	1086.55	1086.29	0.26	Widen flood channel 100 ft.
758+88	1083.80	2.6	0.1	1086.47	1086.13	0.34	
754+88	1082.78	2.6	0.1	1085.48	1085.49	0.00	
752+38	1082.40	2.6	0.1	1085.09	1085.09	0.00	
749+88	1082.03	2.6	0.1	1084.72	1084.69	0.03	
747+35	1081.75	2.6	0.1	1084.43	1084.29	0.14	
744+88	1081.21	2.6	0.1	1083.90	1083.89	0.01	
742+38	1080.82	2.6	0.1	1083.51	1083.49	0.02	
Downstream of Glendale Avenue drop structure							
612+58	1047.66	2.6	0.2	1050.40	1050.33	0.07	Widen channel

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

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

elevation during inundation of the terrace exceeds the flood channel/terrace bank 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 2. 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 2.

Table 2. 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 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 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.

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.

- 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 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.

- 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.

- 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:

- <u>Freeboard</u> – 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.

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

- <u>Scour and Toe-Down</u> – 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.

- <u>Erosion Protection</u> – 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.

- <u>Sedimentation</u> – sediment transport should be considered to ensure that excess erosion or deposition of sediment doesn't occur, thus reducing channel capacity.

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 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.

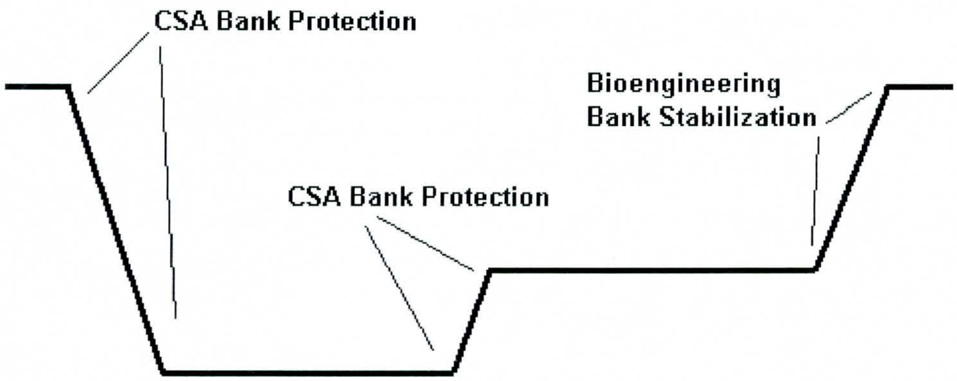

Figure 22. Sketch of proposed bank protection configuration.

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 3. 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 3. 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 3, 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 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. 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 understory and overstory 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 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. Long-term scour was estimated from the equilibrium slope analysis described below. Contraction and abutment scour were estimated using live-bed scour equations from the FWHA HEC-18 bridge scour manual. 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 4 to 7.

Table 4. General and Long-Term Scour Estimates for the 2-Year Event					
Reach	**River Stationing**	**Proposed Slope (ft/ft)**	**Avg. Equilibrium Slope (ft/ft)**	**COT 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
Note: A positive value for the long-term scour estimate indicates aggradation.					

Table 5. General and Long-Term Scour Estimates for the 10-Year Event

Reach	River Stationing	Proposed Slope (ft/ft)	Avg. Equilibrium Slope (ft/ft)	COT 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

Note: A positive value for the long-term scour estimate indicates aggradation.

Table 6. General and Long-Term Scour Estimates for the 100-Year Event					
Reach	**River Stationing**	**Proposed Slope (ft/ft)**	**Avg. Equilibrium Slope (ft/ft)**	**COT 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
Note: A positive value for the long-term scour estimate indicates aggradation.					

Table 7. 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. 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 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 on 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):

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

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

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

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):

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

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 4 to 6). 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.

Table 8: 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

[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.

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 8.

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.

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.

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.

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

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, the results of the AMAFCA, BUREC, Bray, and Henderson equations were used to predict an equilibrium slope.

As shown in Table 4, 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 5 and 6) 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 in Tables 4 to 6.

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

[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.

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 9, 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 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 9 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.

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

Table 9. Reach-Average Sediment Continuity Results
Showing Relative Difference in Sediment Transport Capacity

Reach	Upstream Station	Downstream Station	Landmarks	Sediment Continuity		
				Q10	Q100	SPF
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

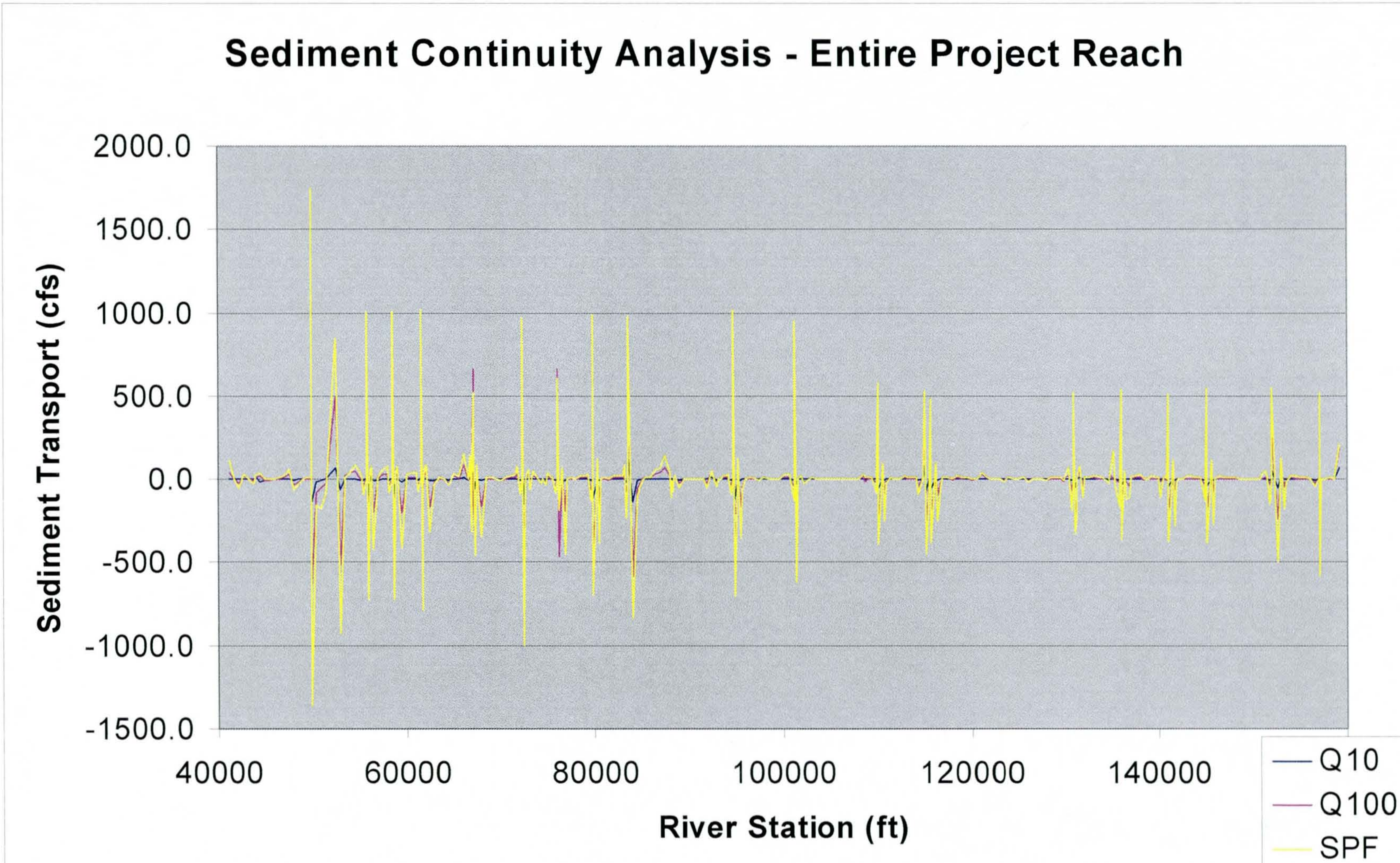


Figure 23. Relative sediment transport capacity computed from the Zeller-Fullerton equation, compared to adjacent cross section. Note discontinuities at drop structures and relative continuity in reaches between drop structures.

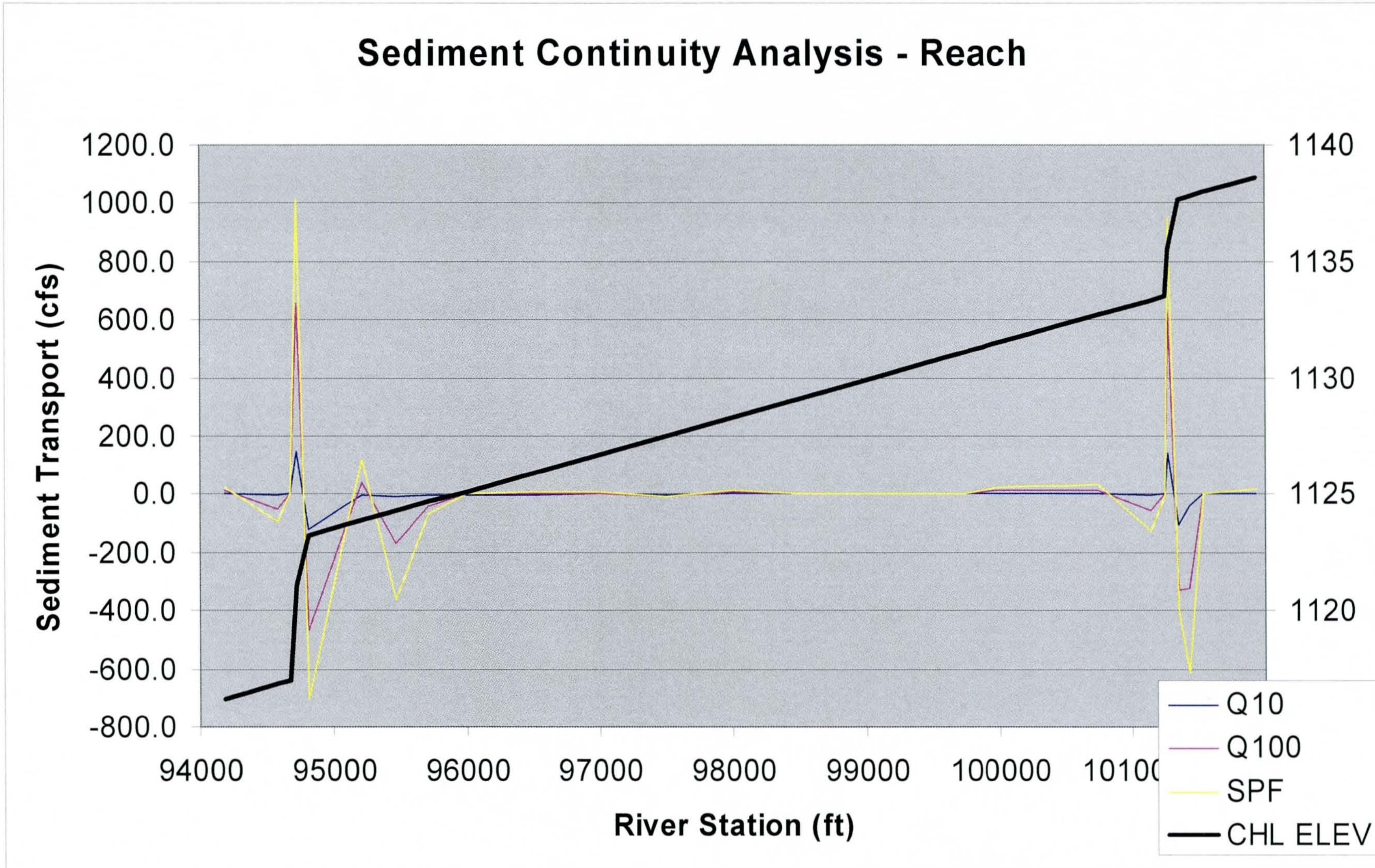


Figure 24. Relative sediment transport capacity for a single reach computed from the Zeller-Fullerton equation, compared to adjacent cross section.

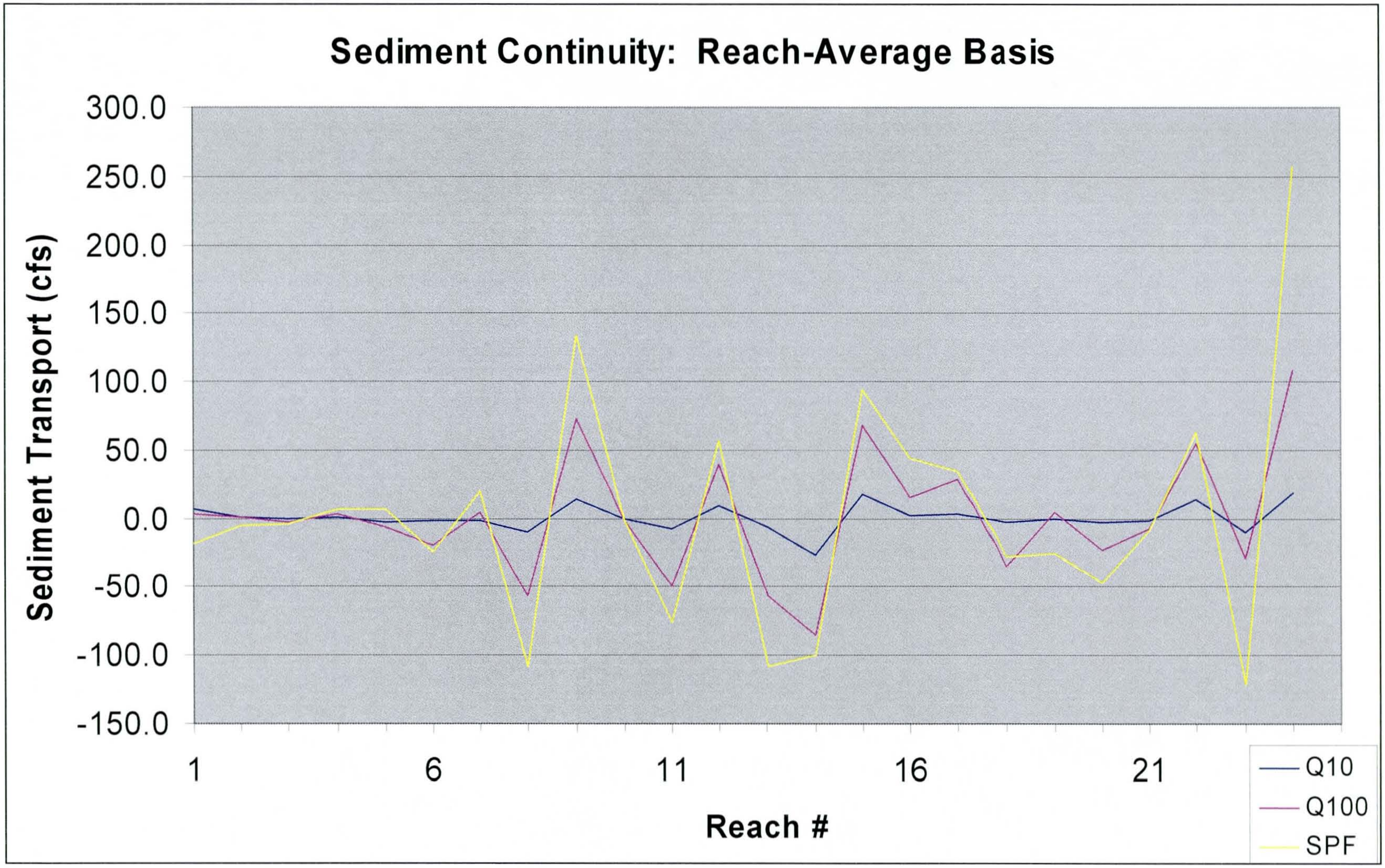


Figure 25. Reach-averaged sediment continuity results.

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.

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.

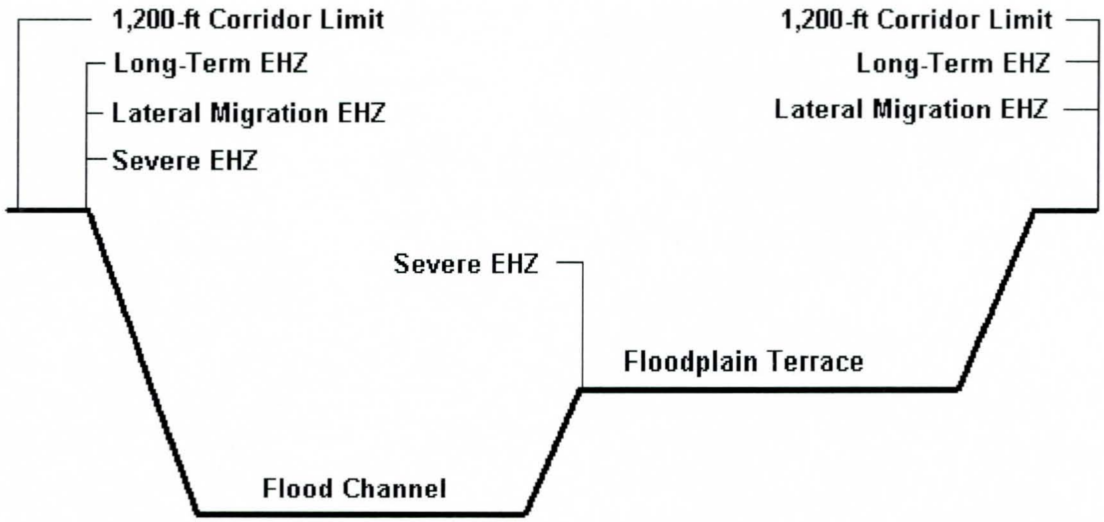


Figure 26. Placement of revised erosion hazard zones upon full implementation of channelization plan.

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.

- 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.

- 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.

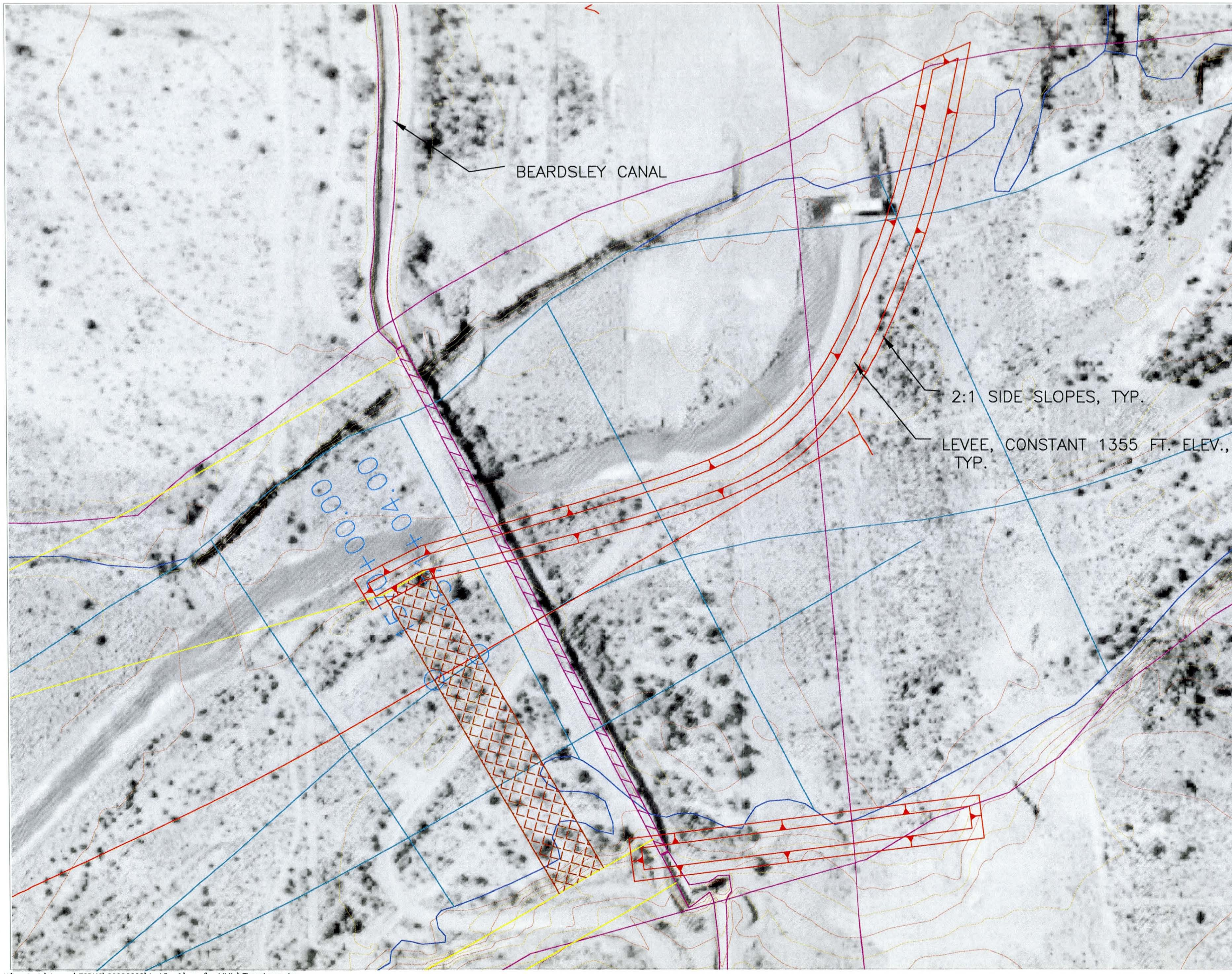
Appendix

Table A-1. Comparison of Computed 100-Year Water Surface Elevation and Channel Velocity Results for Subcritical vs. Mixed Profile HEC-RAS Model								
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
144232	0	0	108007	0	0	68000	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

Station	WSEL	Velocity	Station	WSEL	Velocity	Station	WSEL	Velocity
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
133000	-0.06	0.09	93182	0	0	55818	0	0
132500	-0.14	0.2	93000	0	0	55782	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								
Station	**WSEL**	**Velocity**	**Station**	**WSEL**	**Velocity**	**Station**	**WSEL**	**Velocity**
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			



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

JE FULLER
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FIGURE 1: PROPOSED TRANSITION INTO CHANNELIZED AFR

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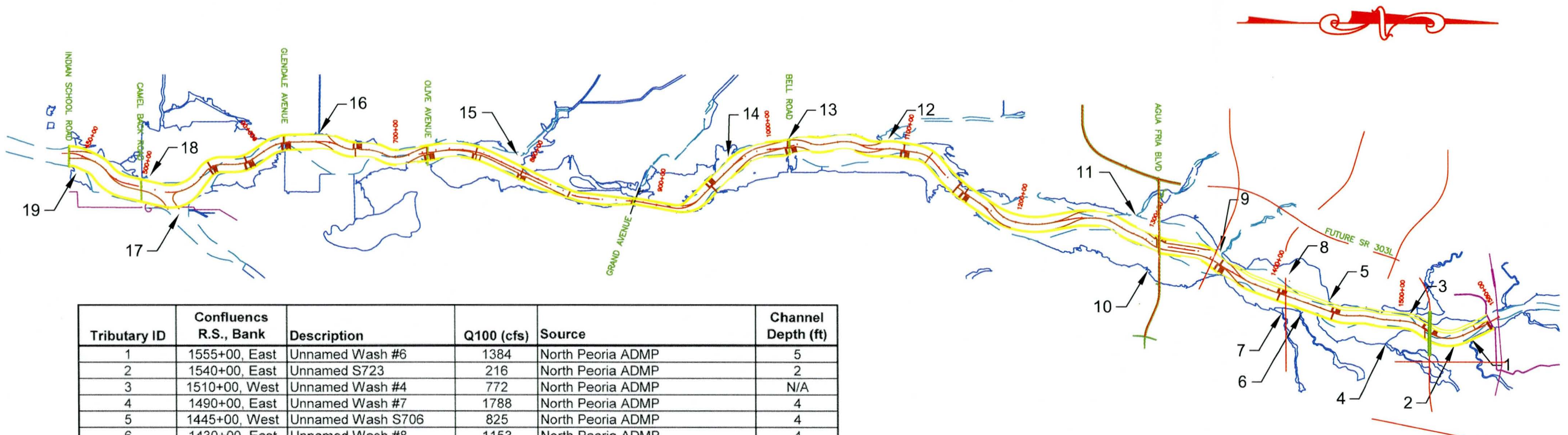
JULY, 2004



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.

MARICOPA COUNTY FLOOD CONTROL DISTRICT

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FIGURE 2: TRIBUTARY OUTFALL LOCATIONS

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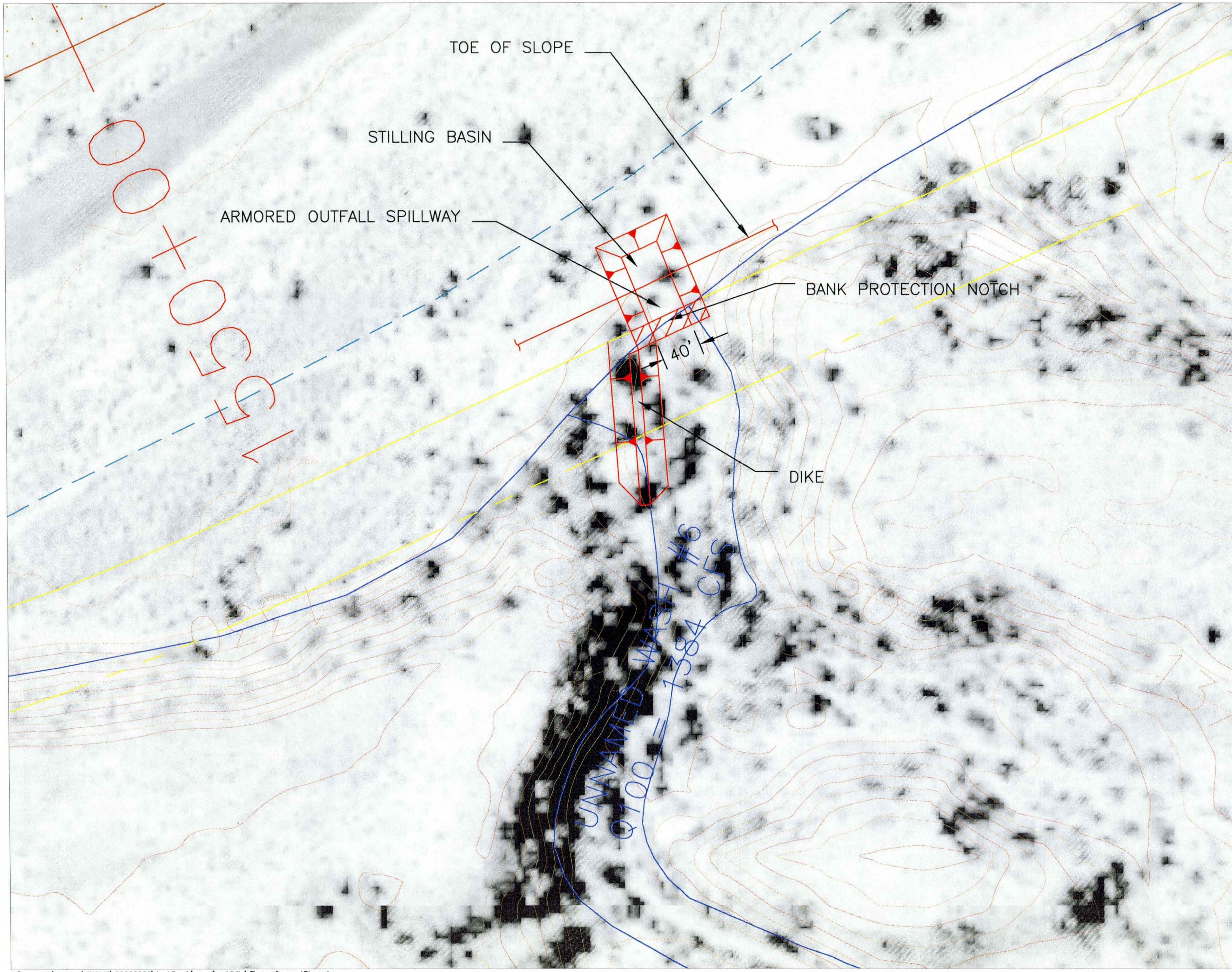

LEGEND

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	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 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.

MARICOPA COUNTY
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FIGURE 3: UNNAMED WASH #6, ID1

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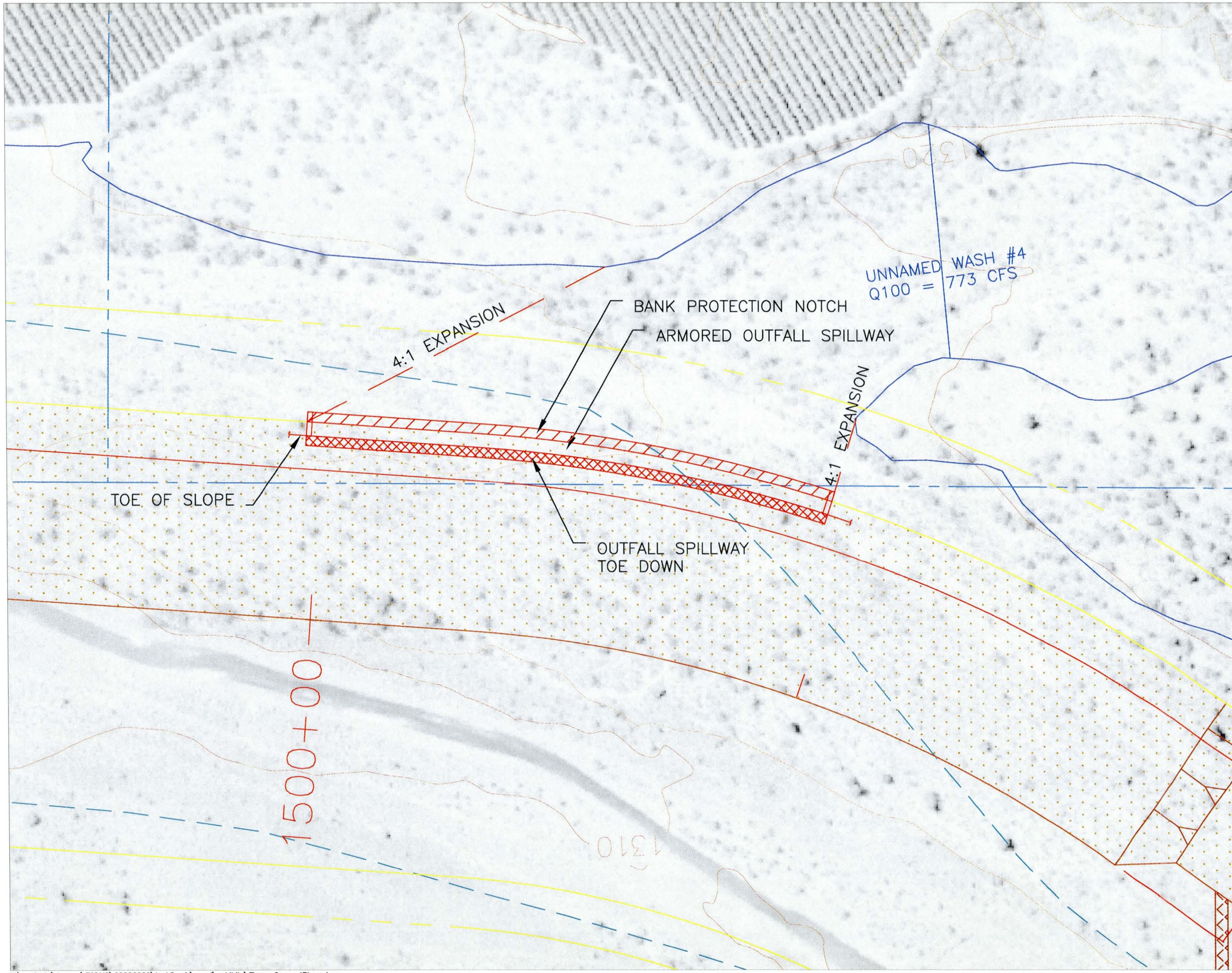

LEGEND

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. 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.

MARICOPA COUNTY
FLOOD CONTROL DISTRICT

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FIGURE 5: UNNAMED WASH #4, ID3

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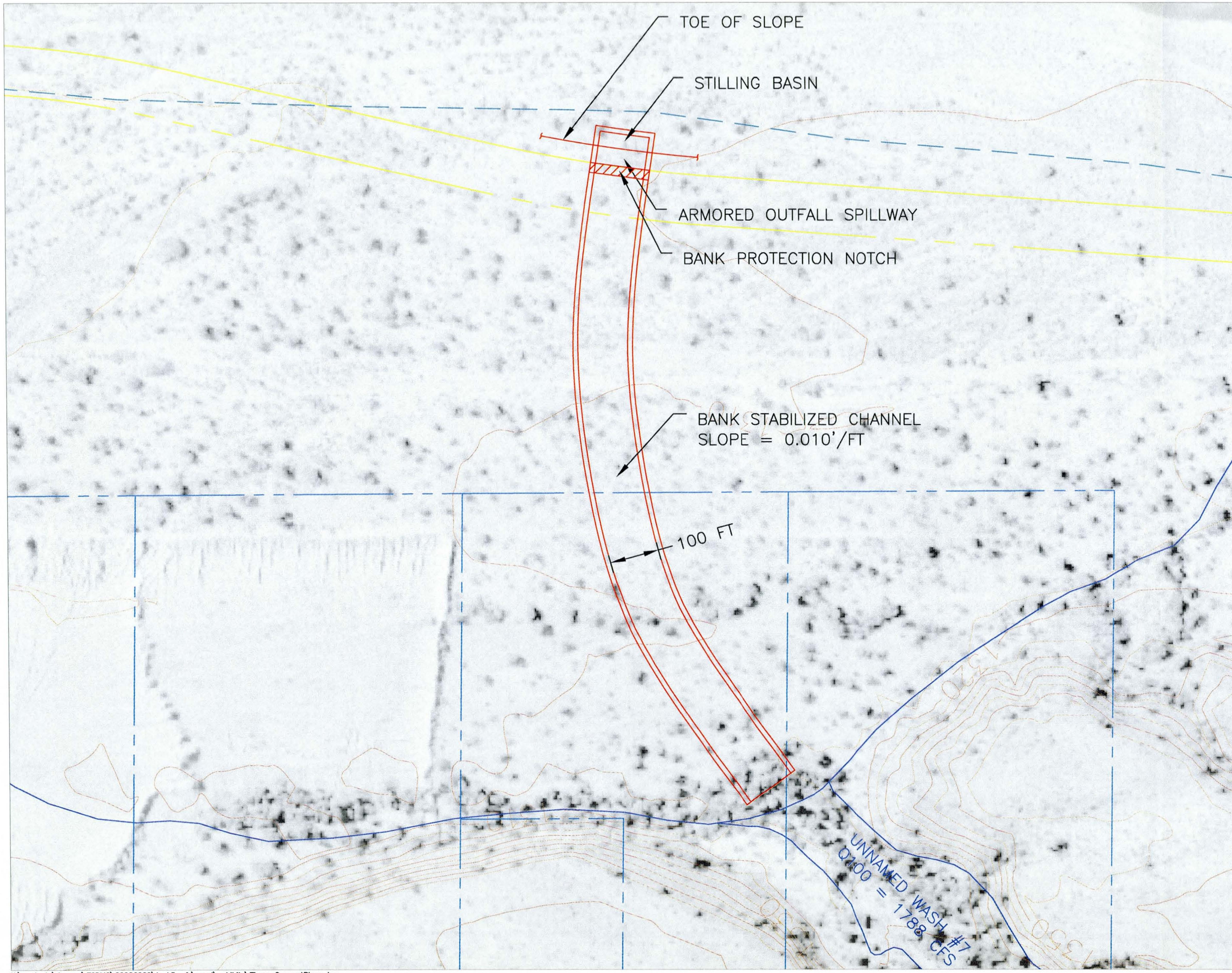

LEGEND

- 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.

MARICOPA COUNTY
FLOOD CONTROL DISTRICT

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FIGURE 6: UNNAMED WASH #7, ID4

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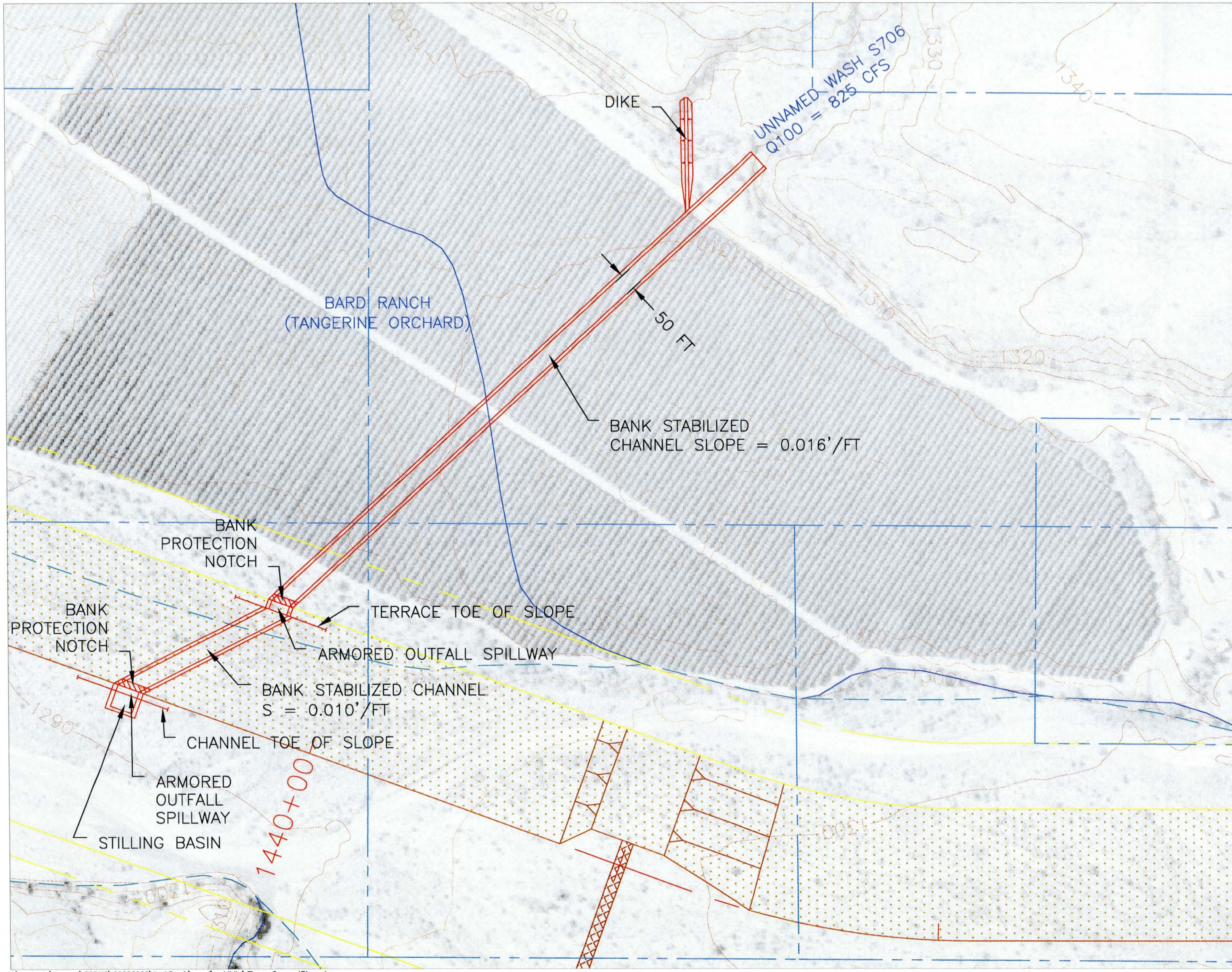
JULY, 2004



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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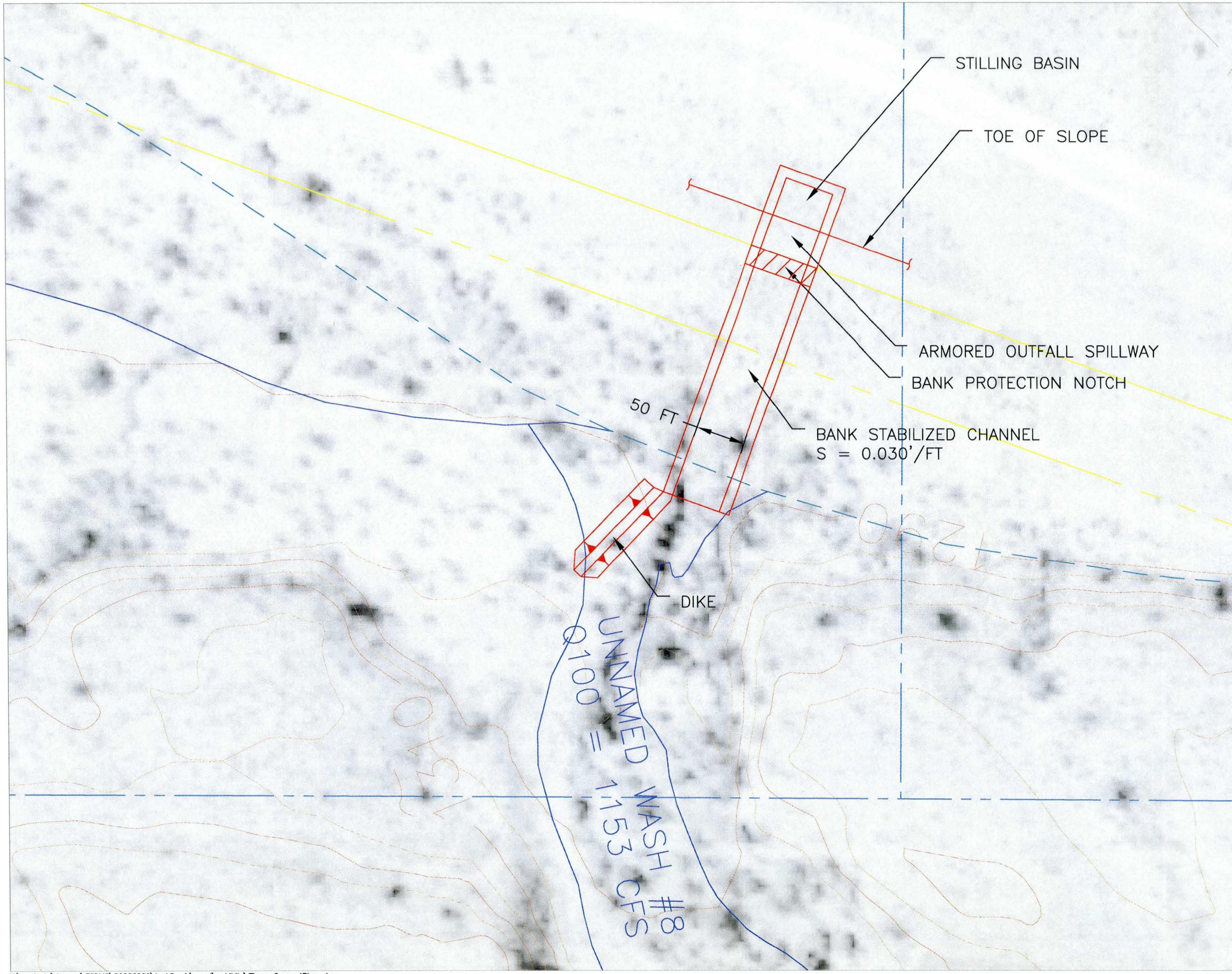

LEGEND

- 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 — 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 8: UNNAMED WASH #8, ID6

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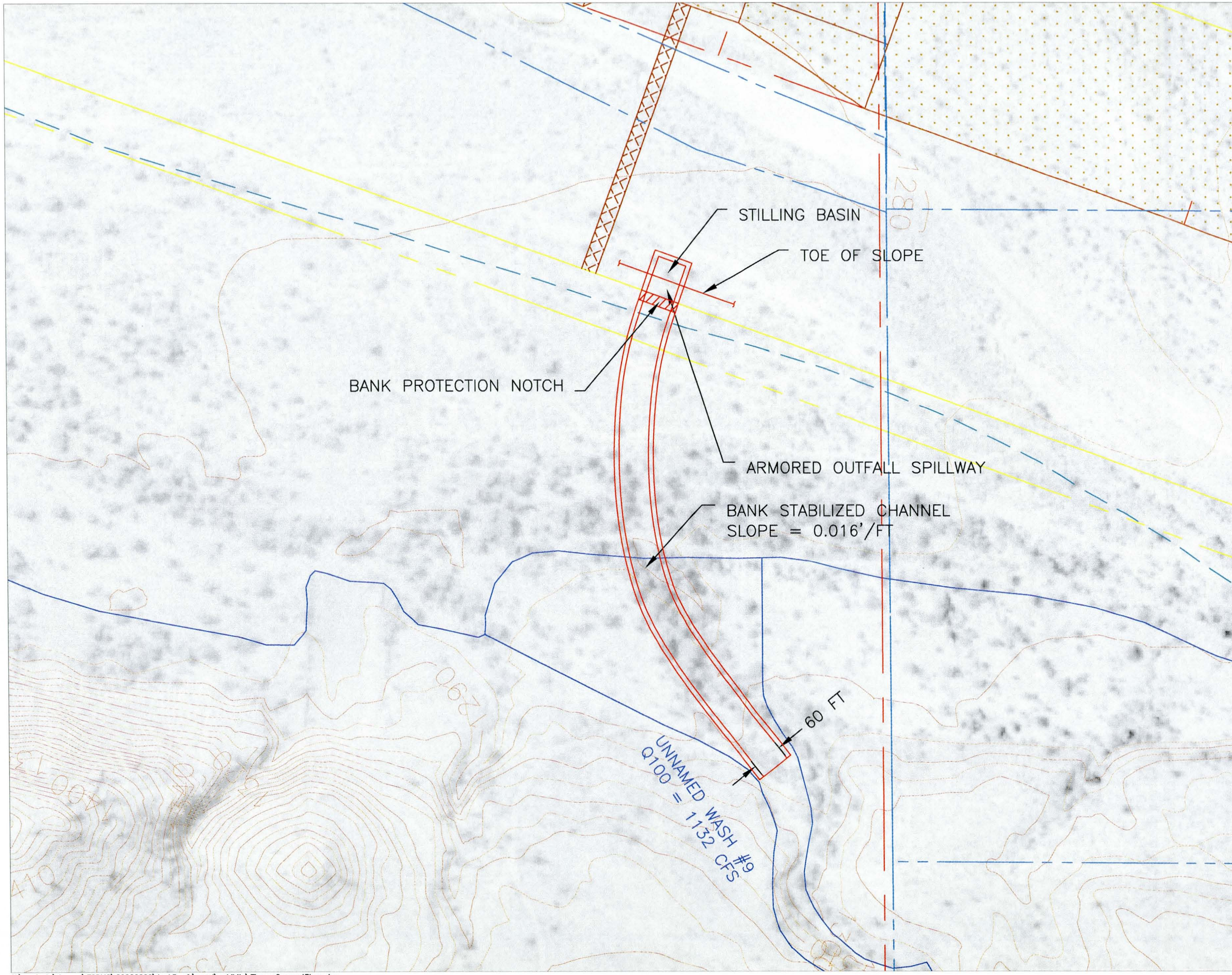

LEGEND

- 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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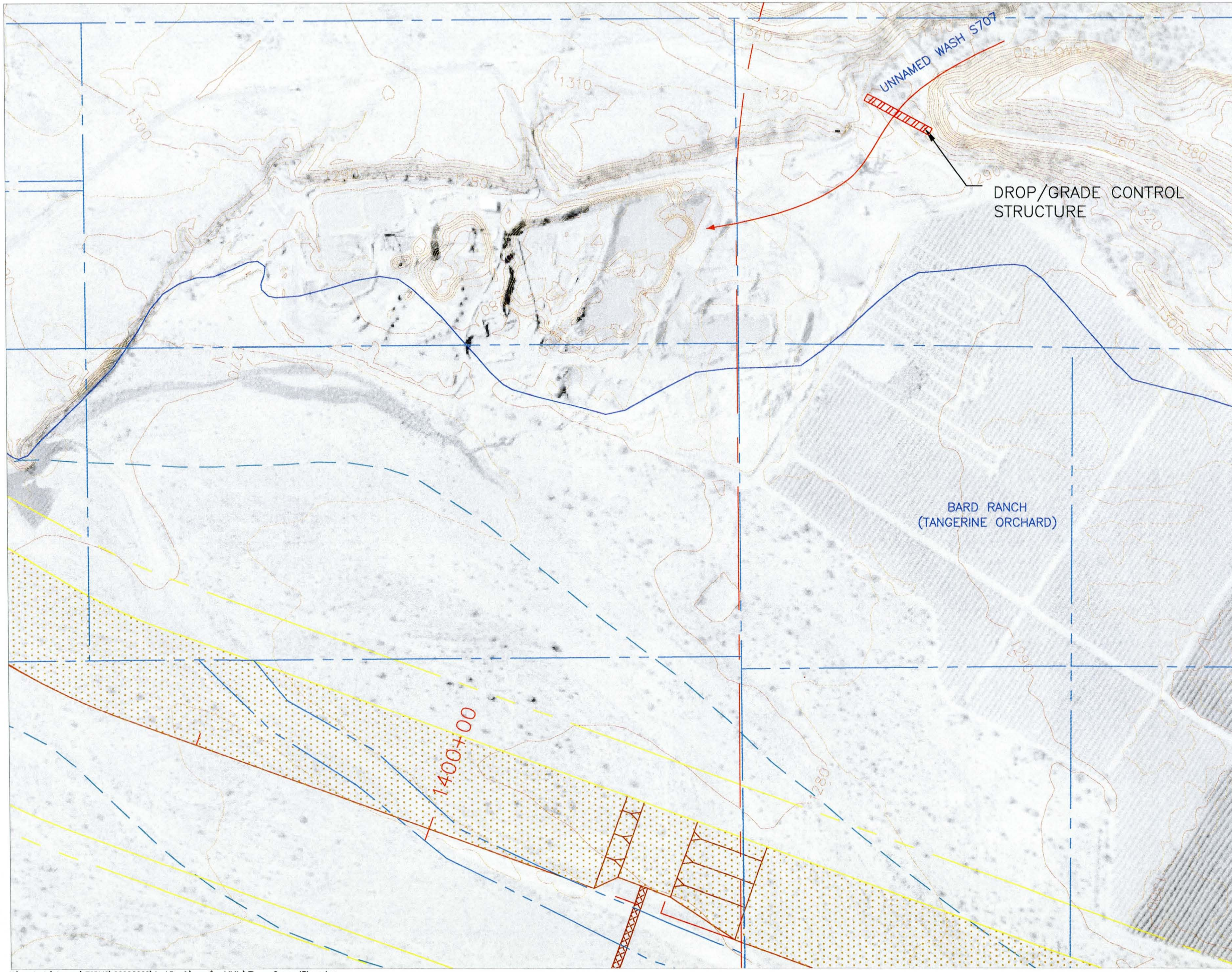

LEGEND

- 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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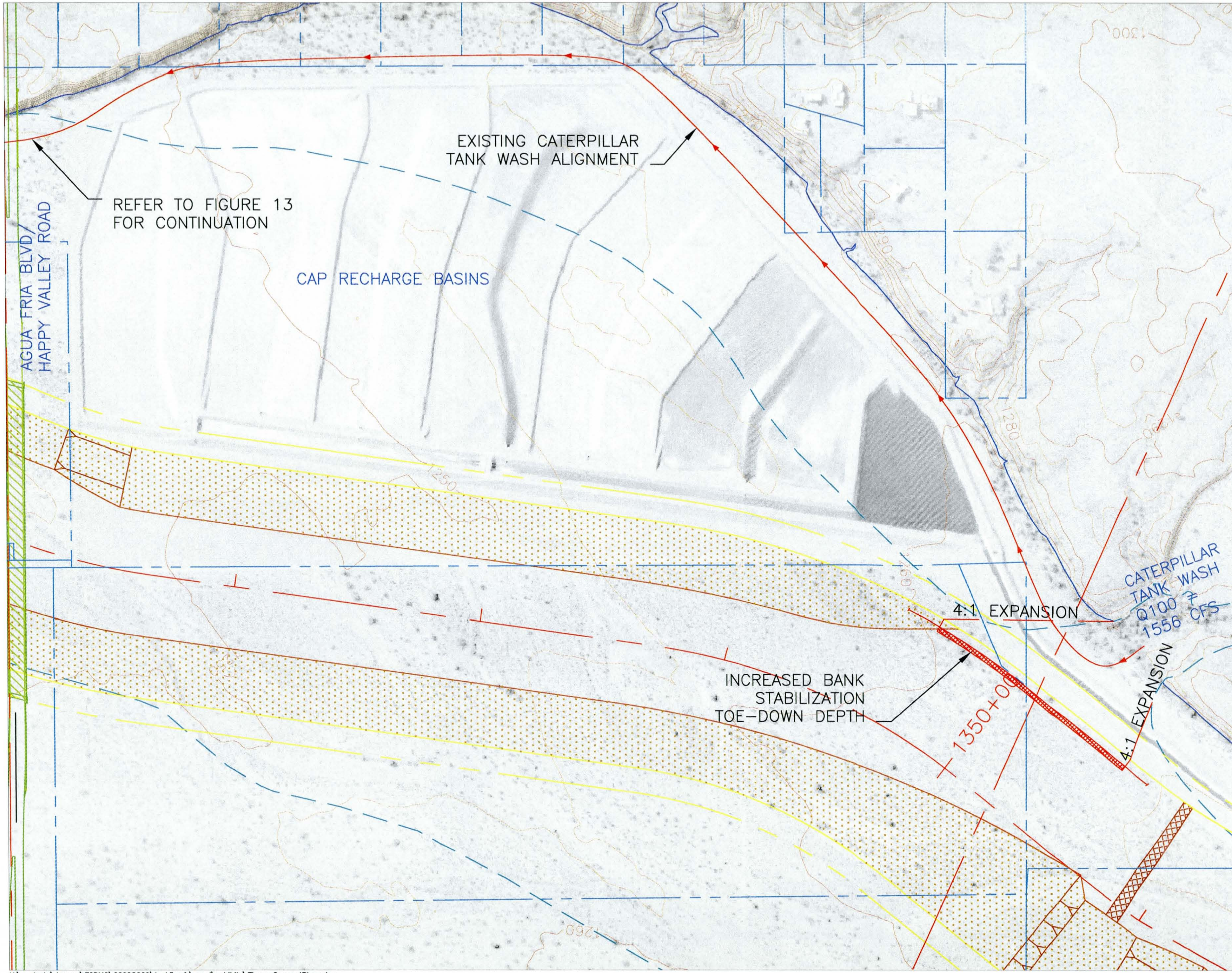

LEGEND

- 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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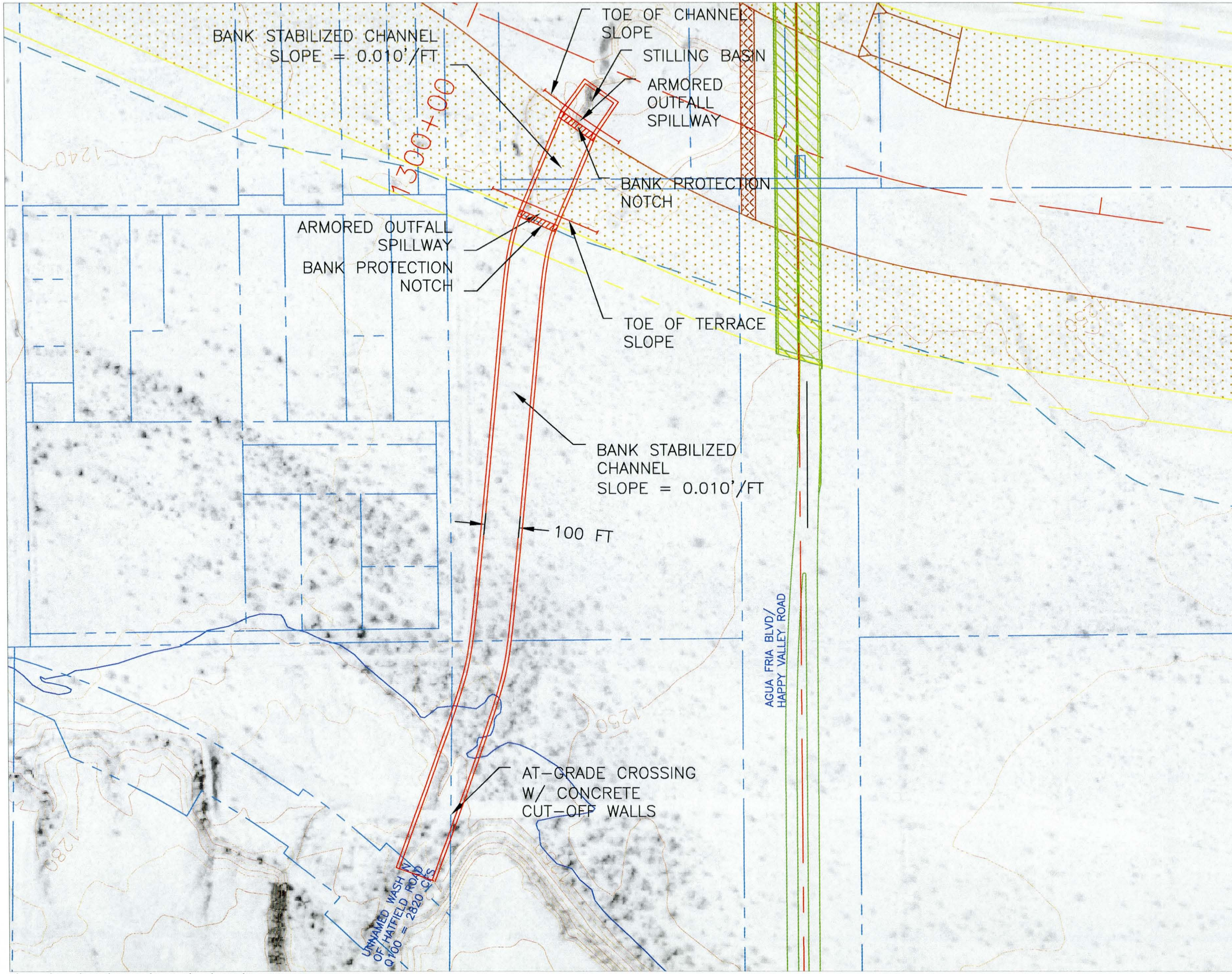

LEGEND

- 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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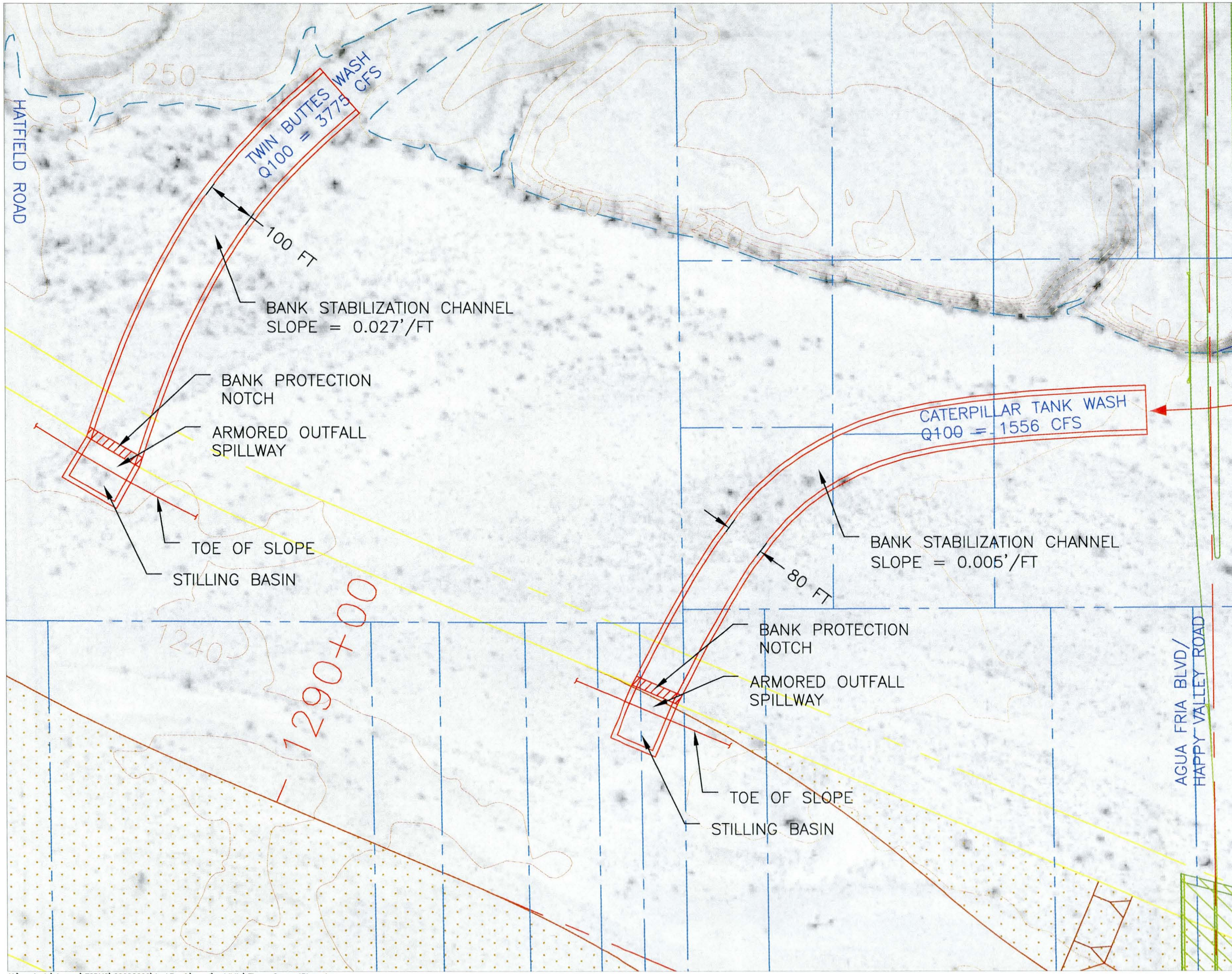

LEGEND

- 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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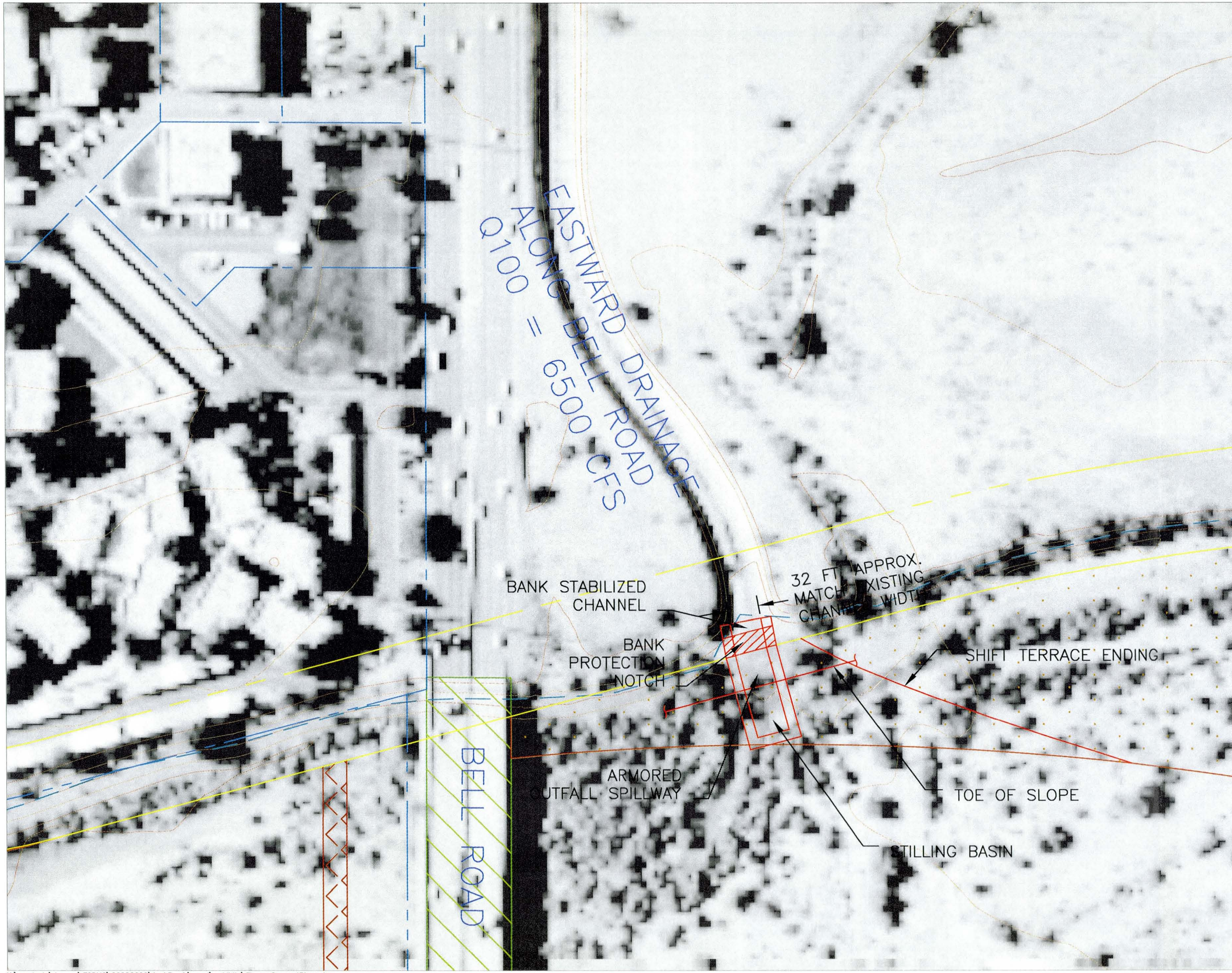

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LEGEND

- 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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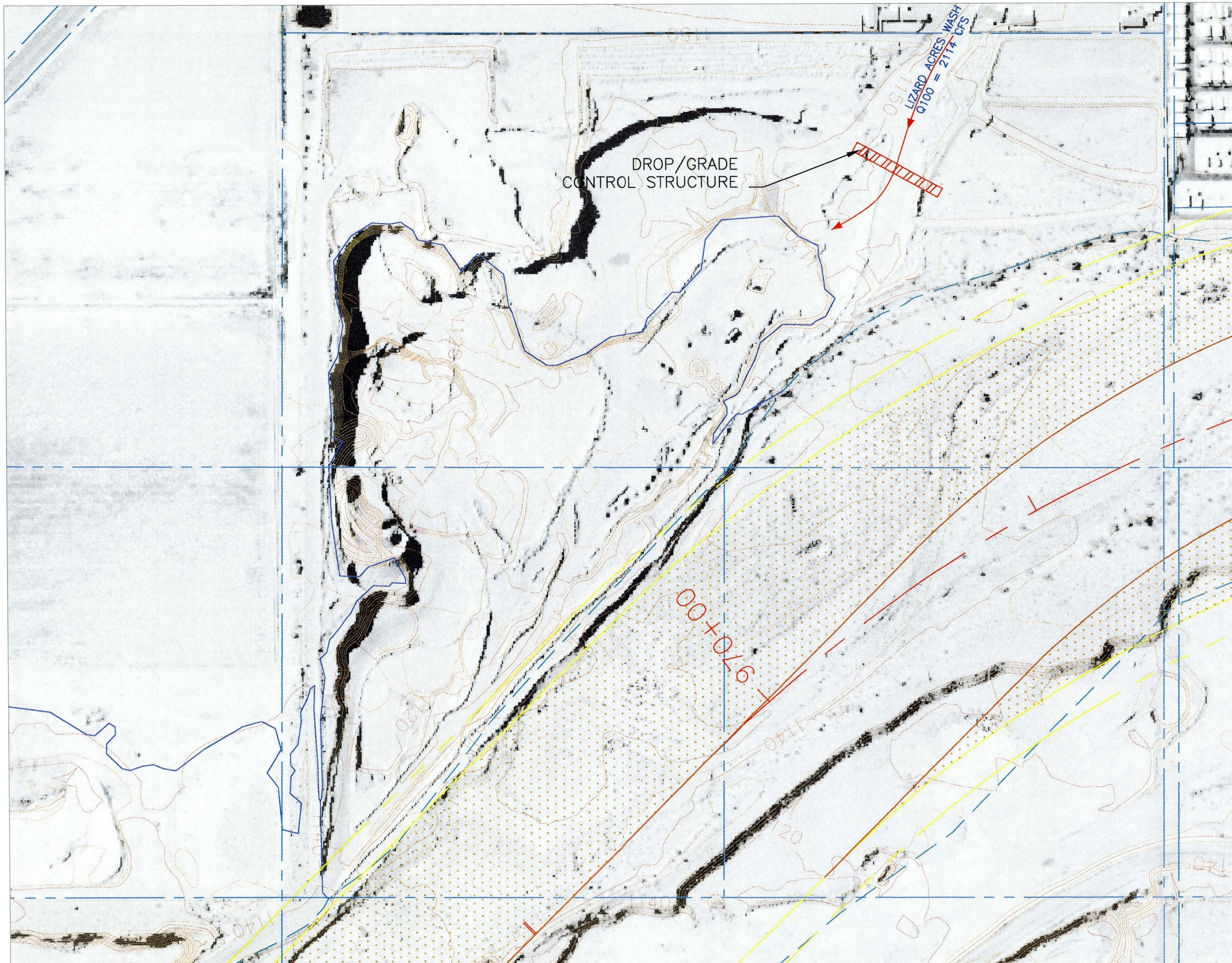

LEGEND

- 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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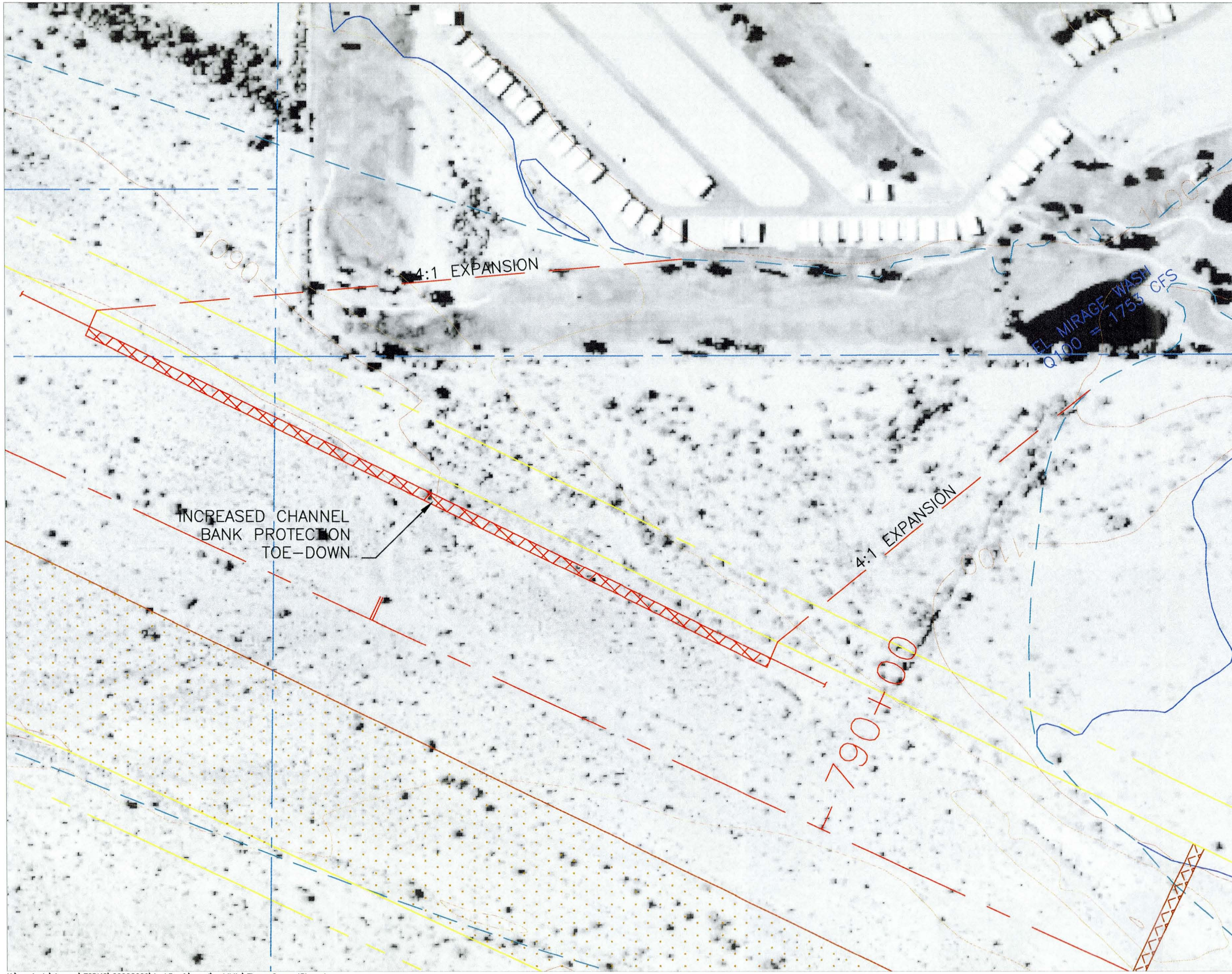

LEGEND

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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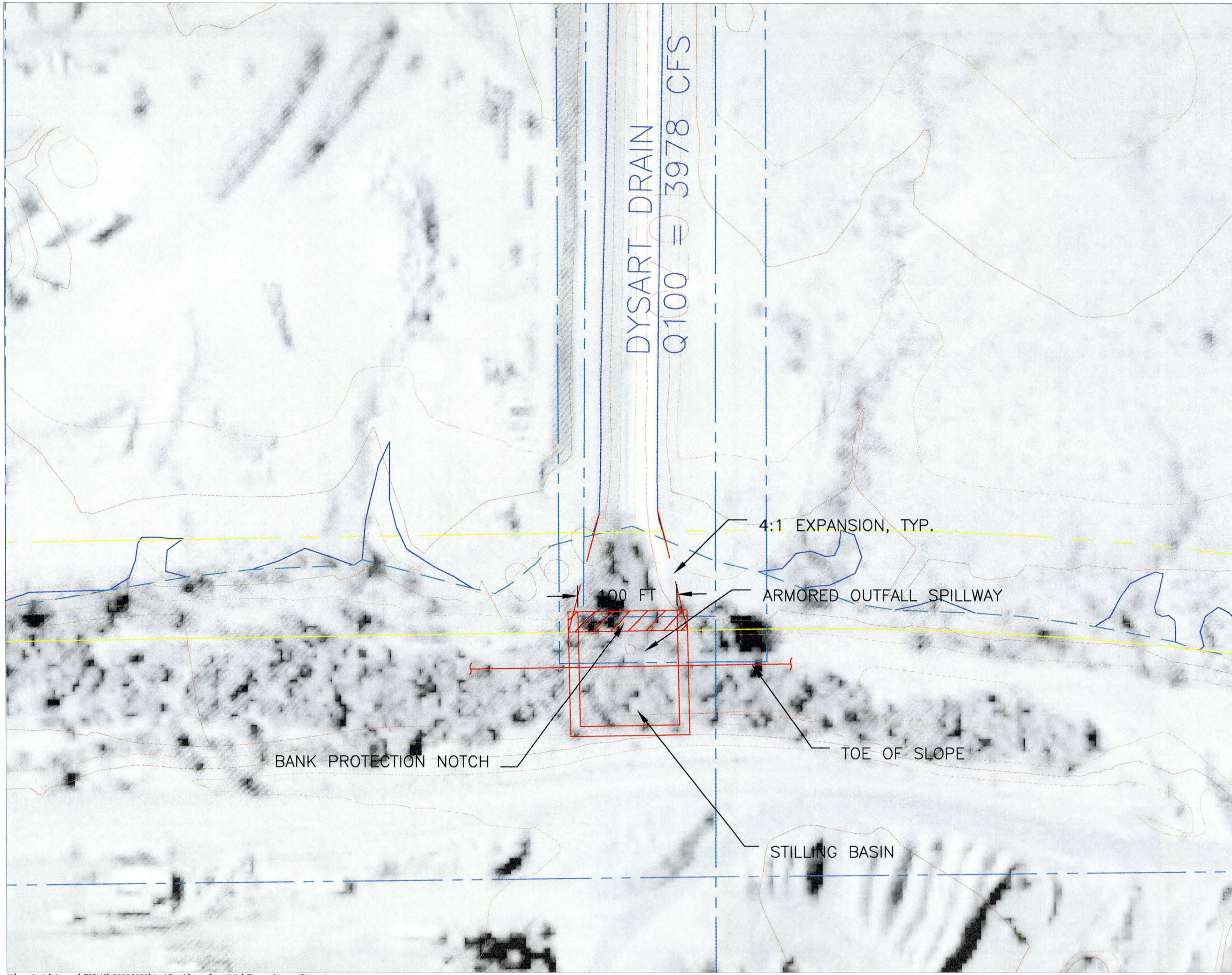

LEGEND

- 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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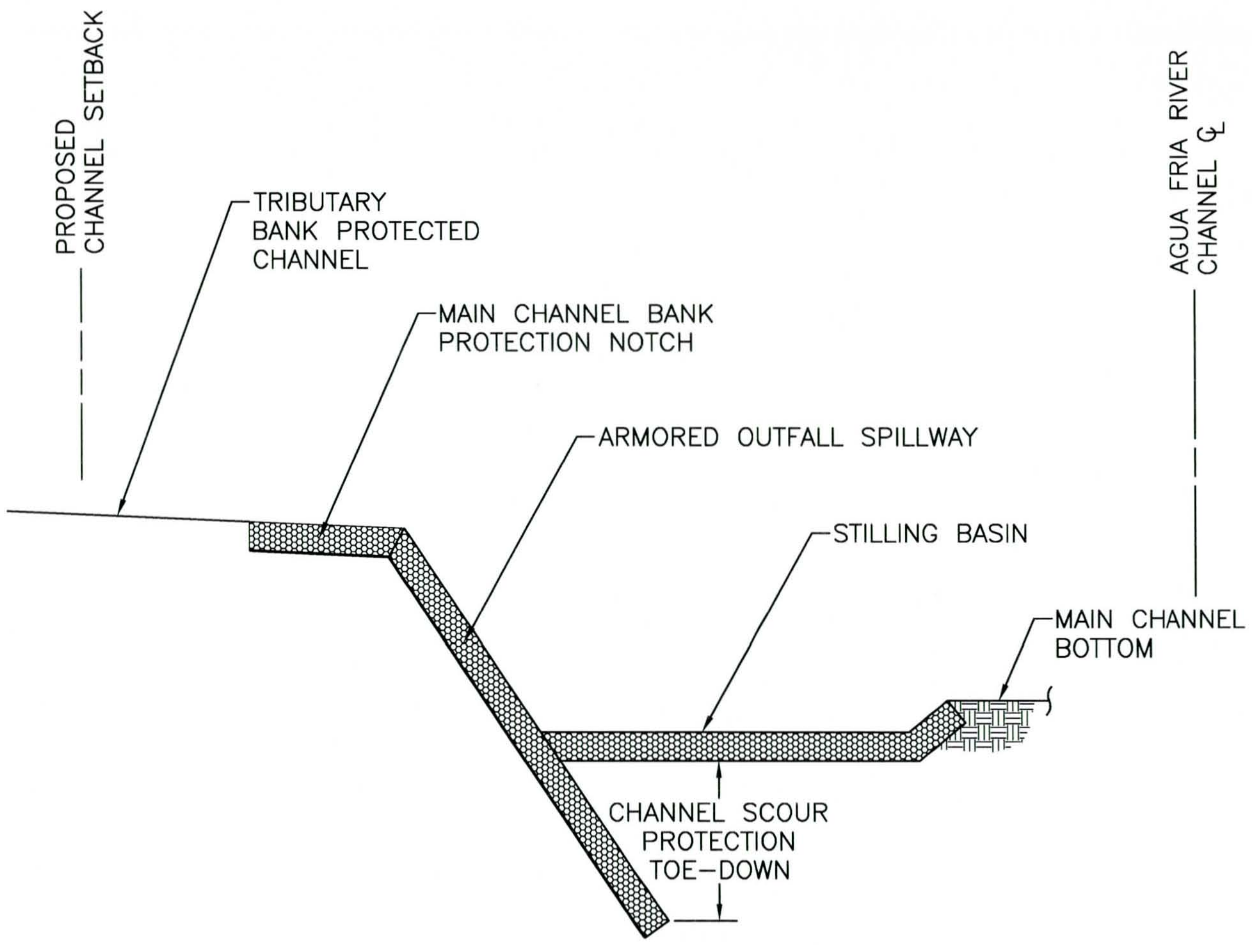
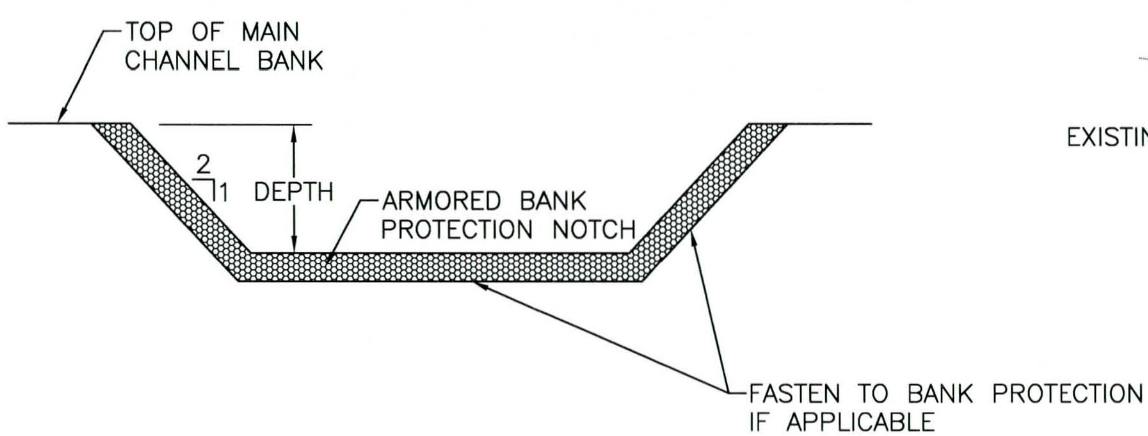


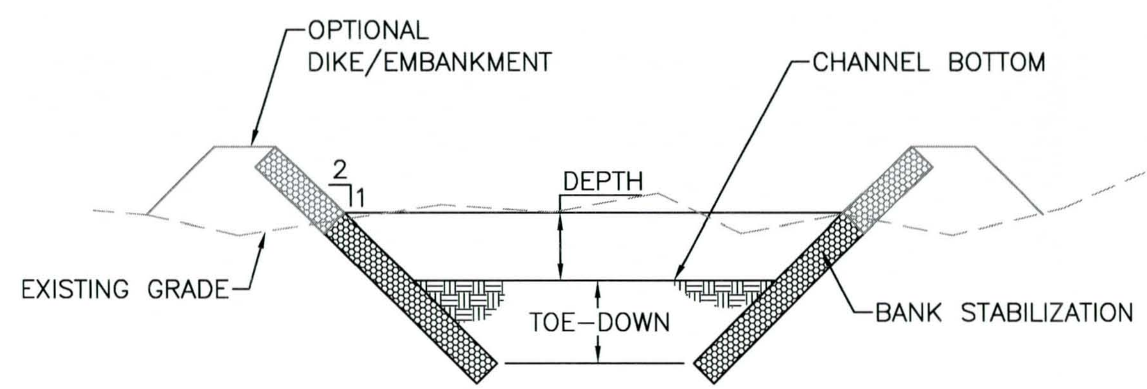
ARMORED OUTFALL SPILLWAY
N.T.S.



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

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